Raglan WWTP Concept Design

Conveyance & Discharge

Prepared for Watercare Services Ltd
Prepared by Beca Limited

21 August 2025

Contents

Appendices

Revision History

Revision Nº	Prepared By	Description	Date
A	Sarah Kennedy	Draft for Client Review	07/05/2025
B	Claire Scrimgeour	Final	21/05/2025
C	Garrett Hall	Revised Final	21/08/2025

Document Acceptance

Action	Name	Signed	Date
Prepared by	Sarah Kennedy		21/08/2025
Reviewed by	Claire Scrimgeour	CMScrimgeour	21/08/2025
Approved by	Garrett Hall		21/08/2025
on behalf of	Beca Limited		

Executive Summary

The Raglan/Whāingaroa Wastewater Treatment Plant (WWTP) is located to the south-west of Raglan/Whāingaroa and receives wastewater from the township, its surroundings, and the Whānga Coast reticulation system. The plant is currently undergoing an upgrade that will significantly improve the quality of treated wastewater as well as increase plant capacity to allow for future population growth.

Currently the treated wastewater discharges to the Raglan/Whāingaroa Harbour via an outfall pipeline. As there have been several instances of non-compliance with the discharge consent and ongoing opposition to a coastal discharge from Whāingaroa hapū and community representatives, work has been undertaken to arrive at the long-term solution where the WWTP discharges to land via a coastal gully system in the western area of Wainui Reserve.

Beca Limited (Beca) has been commissioned by Watercare Services Limited (Watercare) to prepare a concept report for the conveyance and discharge of treated wastewater to the coastal gully system with the ability to divert contingency wet weather discharges to the existing outfall in the Raglan/Whāingaroa Harbour.

The plant is to be upgraded to a modular MABR/MBR plant design with UV disinfection. Watercare advise that the plant capacity upgrade is based on two key projections:

- WWTP capacity **6,000 m^3/day**, based on 2030 peak outflow of 70 L/s
- WWTP capacity **7,500 m^3/day**, based on 2050 peak outflow of 90 L/s

During normal operation, fully treated wastewater will flow via gravity to the storage tanks where it will be pumped to the new land discharge point at the northern end of the western gully. The treated wastewater flowrate conveyed to the gully will match the treatment plant capacity – up to 70 L/s currently, with future flows up to 90 L/s. The discharge will flow down the gully in a rock channel on the steeper section at the head of the gully and via an overland flow path through the wetlands at the base of gully system.

During periods of abnormally high flows, when the inflow exceeds 70/90 L/s and inflow buffer storage is full, the excess flow above 70/90 L/s will be diverted to the existing discharge route to the outfall in the Raglan/Whāingaroa Harbour.

It is intended discharge to the gully is to be implemented and monitored in an adaptive way, where flows and discharge effects will be monitored over time. It is proposed to review the performance of the discharge to gully system 5-years post commissioning.

1 Introduction

The Raglan/Whāingaroa Wastewater Treatment Plant (WWTP) is located to the south-west of Raglan/Whāingaroa and receives wastewater from the township, its surroundings, and the Whānga Coast reticulation system. The plant is currently undergoing an upgrade to a Membrane Aerated Biofilm (MABR) and Membrane Bioreactor (MBR) treatment system that will significantly improve the quality of treated wastewater as well as increase plant capacity to allow for future population growth. The treatment plant upgrade is considered the first stage of the total WWTP upgrade, with the second stage focused on the location of the treated wastewater discharge.

The existing treated wastewater discharge consent enables the WWTP to discharge to the Raglan/Whāingaroa Harbour via an outfall pipeline. This consent expired in February 2020. An interim consent was lodged with the Waikato Regional Council (WRC) that allowed for the continuation to discharge to the Raglan/Whāingaroa Harbour whilst a preferred long-term alternative solution was confirmed.

There have been several instances of non-compliance with the discharge consent for treated wastewater to the Raglan Harbour during recent years. Whāingaroa hapū and community representatives have also consistently expressed the unequivocal need to remove the discharge from the harbour and utilise land disposal technologies. For this reason, signfiicant work has been undertaken to arrive at the long-term solution where the WWTP discharges to land via a coastal gully system in the western area of Wainui Reserve.

Beca Limited (Beca) has been commissioned by Watercare Services Limited (Watercare) to prepare a concept design report for the conveyance and discharge of treated wastewater to the coastal gully system with the ability to divert contingency wet weather discharges to the existing outfall in the Raglan/Whāingaroa Harbour. Watercare provide three waters operations and management services on behalf of Waikato District Council. The purpose of this report is to summarise the conveyance and discharge concept design for the Raglan/Whāingaroa WWTP discharge.

Figure 1 - Layout Overview.

2 WWTP Design

2.1 Treatment Plant

The WWTP upgrade is based on the *Raglan/Whāingaroa WWTP Upgrade Basis of Process Design Report* (Lutra, March 2023) where growth is predicted over a 35-year consent period until the year 2055. Following influent sampling taken over the 22/23 summer, a modular MABR/MBR plant design with UV disinfection was chosen.

The treatment plant has been designed to meet the following treated wastewater limits as agreed with Watercare:

Table 2-1 - Treated Water Quality Requirements.

Parameter	Median	90%ile
$cBOD_5$	5 mg/L	10 mg/L
TSS	5 mg/L	10 mg/L
Total N	10 mg/L	N/A
Ammoniacal Nitrogen	2 mg/L	4 mg/L
Total P	4 mg/L	N/A
pH	6.5 – 8.5	
UV validated dose	35 mWs/cm^2 99% of the time over each calendar month	

Watercare advise that the MABR/MBR design has been based on two key projections:

- WWTP capacity **6,000 m^3/day**, based on 2030 maximum flowrate of 70 L/s
- WWTP capacity **7,500 m^3/day**, based on 2050 median flowrate of 90 L/s

The ultimate design capacity of 7,500 m^3/day will be achieved by the construction of an additional train to the MABR/MBR system, alongside the two trains built to treat the 6,000 m^3/day capacity required. Wastewater will receive full treatment, (MABR/MBR followed by UV disinfection), before being discharged.

2.2 Pond Storage & Discharge

There are several pond facilities onsite. The large upper ponds will receive some screened wastewater and act as a storage buffer for peak flows before returning wastewater to the inlet works of the plant.

Storage tanks adjacent to the smaller day pond will receive MABR/MBR/UV treated wastewater. Treated wastewater from the storage tanks will then be pumped to the new land discharge point at the northern end of the western gully. The treated wastewater flowrate conveyed to the gully will match the treatment plant capacity – up to 70 L/s currently, with future flows up to 90 L/s.

During heavy rainfall events, when the inflow exceeds 70/90 L/s and storage is full, wastewater will be diverted via the day pond to the existing discharge route at the outfall in the Raglan/Whāingaroa Harbour.

The peak flow from the WWTP being conveyed to the gully is not expected to coincide with peak flow from rainfall due to the retention time in the wastewater network and WWTP. Typically, there is a minimum 2–4-hour delay before peak flows reach the WWTP.

It should be noted, discharge to the gully is to be implemented and monitored in an adaptive way, where flows and discharge effects will be monitored over time. It is proposed to review the performance of the discharge to gully system 5-years post commissioning.

2.3 Process Flow

A summary of the WWTP process flow has been provided below.

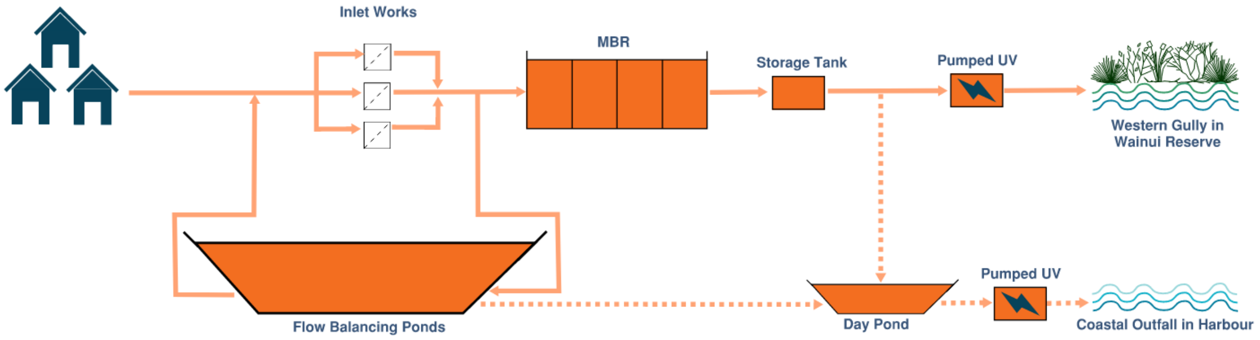

Figure 2 - Process Flow Diagram.

3 Conveyance

Following WWTP treatment, treated wastewater fed via gravity from the WWTP will collect in the storage tanks shown below in Figure 3. From here it will be pumped through the new rising main to the northern end of the gully in Wainui Reserve.

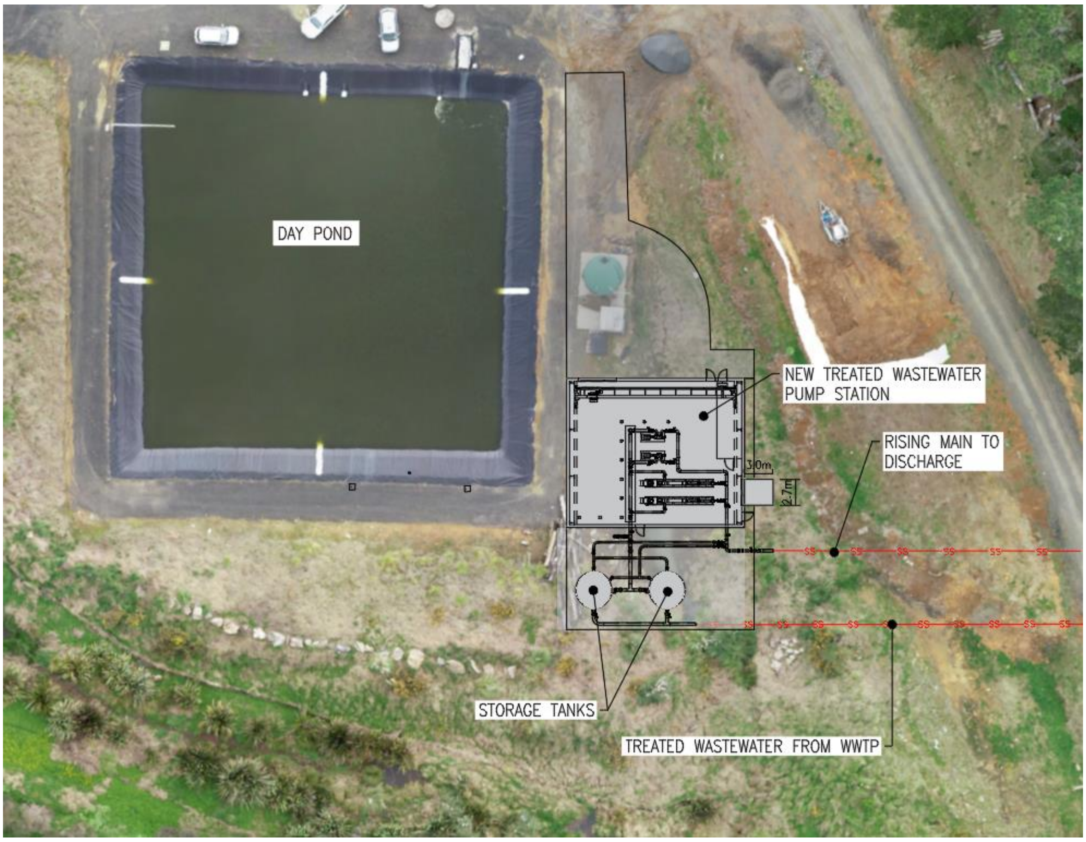

Figure 3 - Treated Wastewater Pump Station.

If flows exceed 70/90 L/s, as a contingency, wastewater will be diverted to the harbour outfall. Flows will be pumped to the existing outfall rising main via the existing pump station and UV unit. The conveyance route to the gully and pump stations have been outlined in the Figure below.

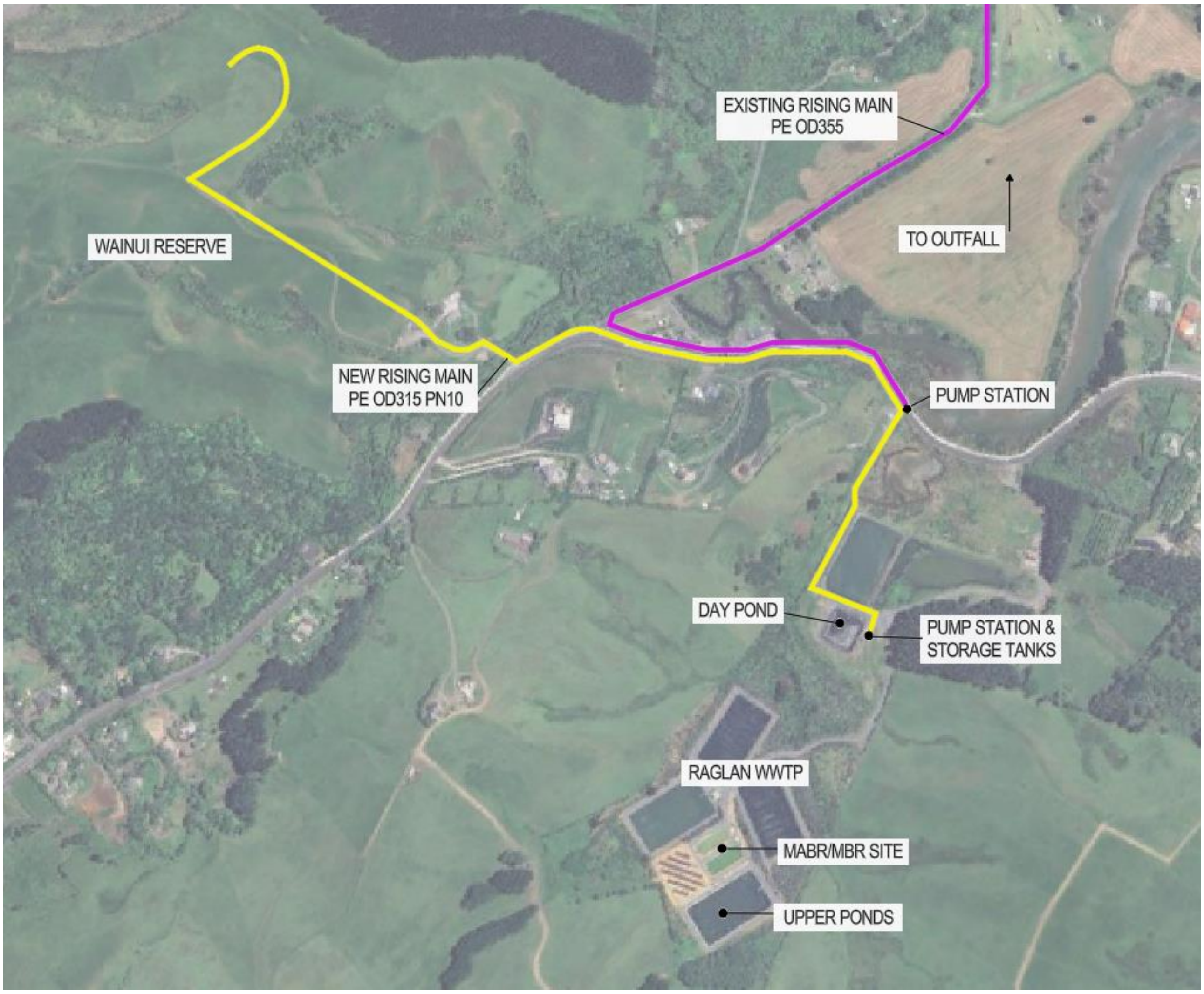

Figure 4 - Discharge Conveyance Route.

4 Discharge

Once the treated wastewater reaches the gully, it will be discharged through a headwall outlet at the head of the western gully (refer to Figure 5). It will flow down the head of the gully via a rock channel before splitting into two flow channels into the wetland to distribute the flow. An example of a rock channel down a slope is provided in Figure 6. The treated wastewater will then flow down the gully, with some infiltrating into the wetland and subsoils, before dissipating into the coastal area. The landscape design concept including an artist's impression of the gully system is provided in Appendix A.

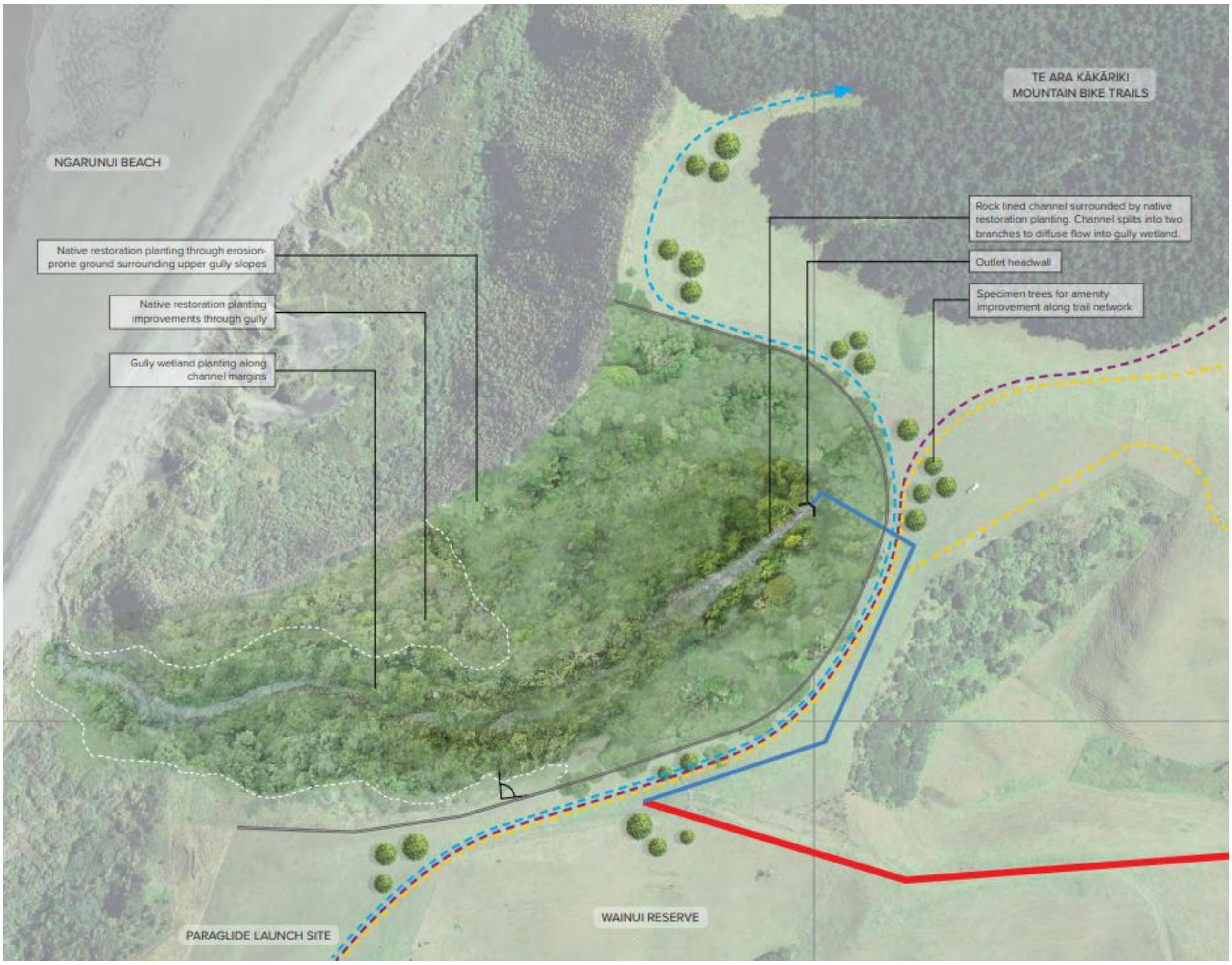

Figure 5 - Landscape design concept

A fence is proposed to restrict public access to the gully to protect the plantings and wastewater discharge infrastructure. The rock channel will be lined with geotextile or similar to minimise infiltration of the treated wastewater into soils in the upper gully section with steeper slopes and erodible soils.

Figure 6 - Example of rock channel on slope

Planting is proposed to the gully side slopes and wetland areas. The slopes will be planted in native plantings to maintain slope stability and enhance the biodiversity of the gully. The base of the gully is to be revegetated where needed with wetland species including raupo. Planting will reduce flow velocities, encourage nutrient uptake by plants and maintain channel stability.

Stormwater runoff through the gully from the surrounding catchment (refer Figure 7) discharges via a shallow overland flow path (OLFP) to Ngarunui Beach through existing sand dunes. Observations from Beca's site visit on 28 August 2024, along with a review of aerial images, indicate the presence of a shallow erosional channel that has formed across the beach. Given the steep terrain of the gully, this channel is likely a result of high-velocity stormwater runoff to the beach during rainfall events.

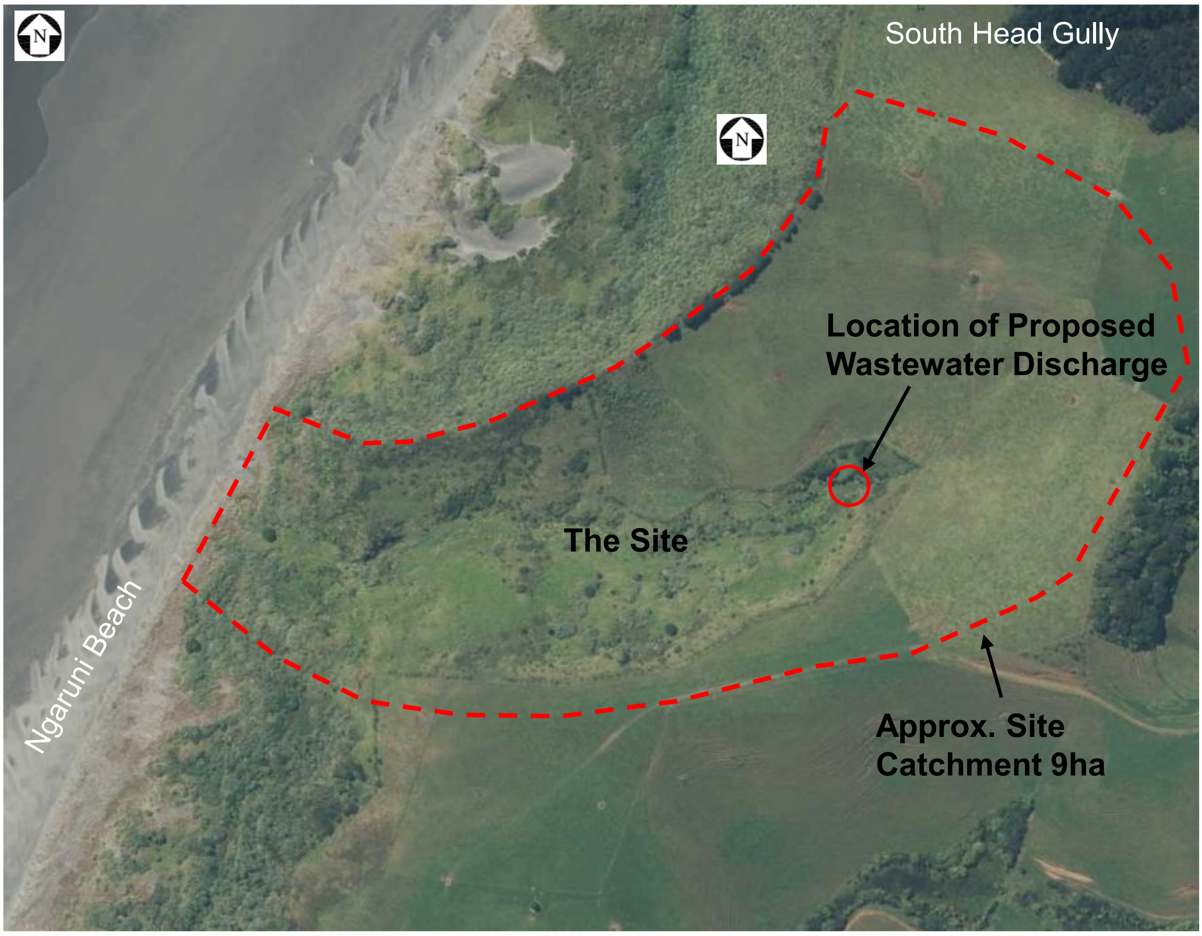

Figure 7: Stormwater catchment extent

Modelling was carried out to establish what changes were expected to the OLFP with the addition of the treated wastewater flows. The modelling report is included in Appendix B. The model results indicate that continuous flow from the proposed wastewater outlet may form a visible OLFP as shown in Figure 8, but this will not likely create a permanent channel or significantly alter the gully's morphology. The OLFP generated by wastewater flows will be noticeable in dry weather conditions. However, during rainfall events, stormwater runoff will dominate the flow pattern. The model results indicate minor increases in velocity and stream power from wastewater flows. This will not pose a risk to erosion initiation or continual degradation of the channel.

Figure 8: Wastewater flow-generated OLFP

5 Next Steps

Once resource consent has been obtained for the new discharge location, the following next steps are recommended:

- Detailed design of the conveyance and discharge infrastructure
- Planting restoration plan including on-going maintenance requirements for vegetation
- Implement a monitoring regime to inspect the gully conditions annually for the first couple of years, and then every two years thereafter, to capture any potential erosion development and, if required, implement appropriate measures.

A

Appendix A – Landscape Design Concept

RAGLAN WAINUI RESERVE WESTERN GULLY WASTEWATER DISCHARGE

Concept Landscape Design Package 21.05.2025

Revision History

Revision No.	Prepared By	Description	Date
A	Will Gumbley and Logan Bunn	Draft Issue for Client	12/04/2024
B	Will Gumbley and Logan Bunn	Final Issue to Client	16/05/2024
C	Will Gumbley	Revised Issue for Community Engagement	15/11/2024
D	Will Gumbley	Preferred Option	02/05/2025
E	Will Gumbley	Preferred Option Updated	21/05/2025

Document Acceptance

Action	Name	Signed	Date
Prepared by	Will Gumbley Logan Bunn	WGumbley	16/05/2024
Reviewed by	Tom Abbott		16/05/2024
Approved by	Garrett Hall		16/05/2024
on behalf of Beca Ltd.			

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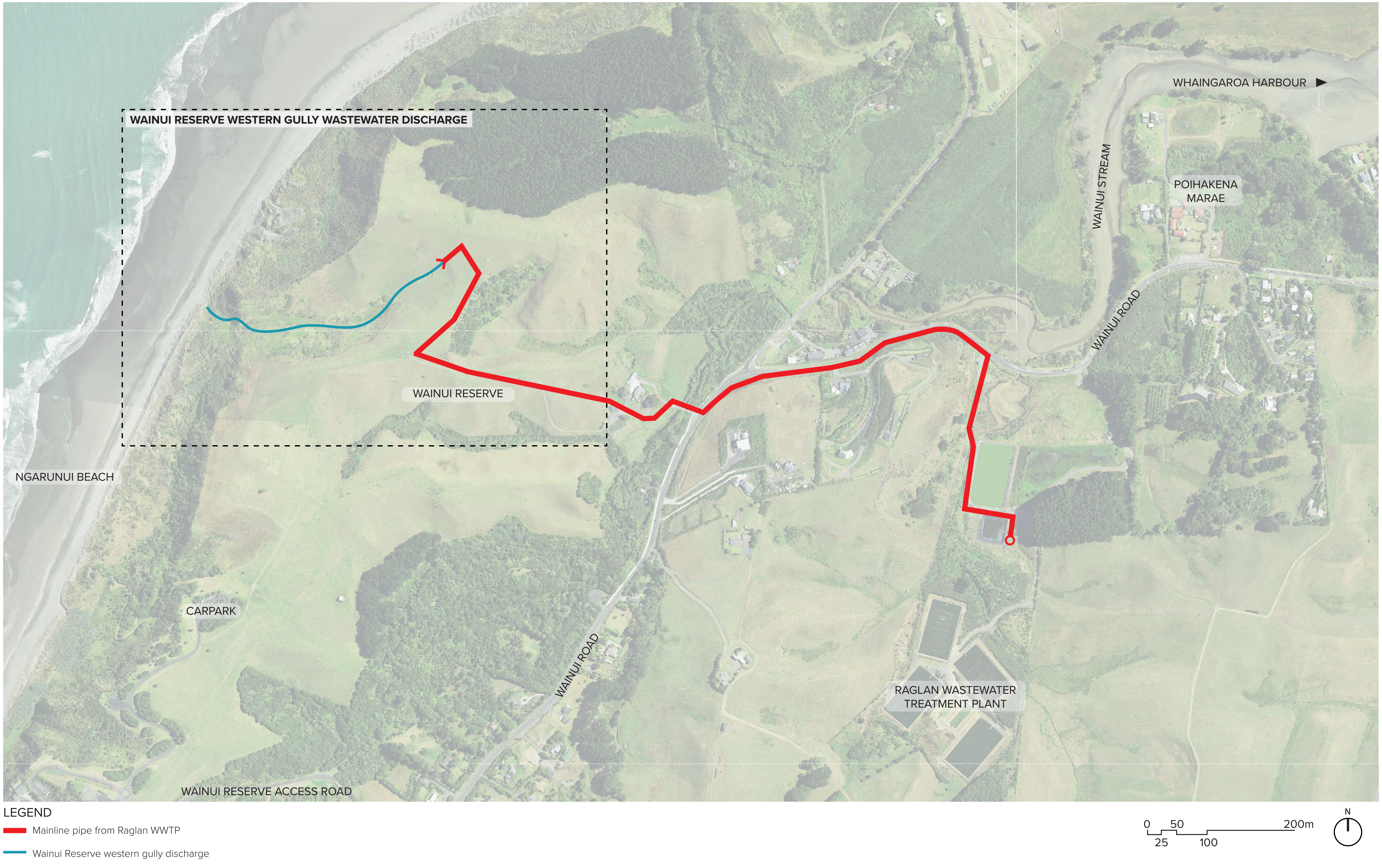
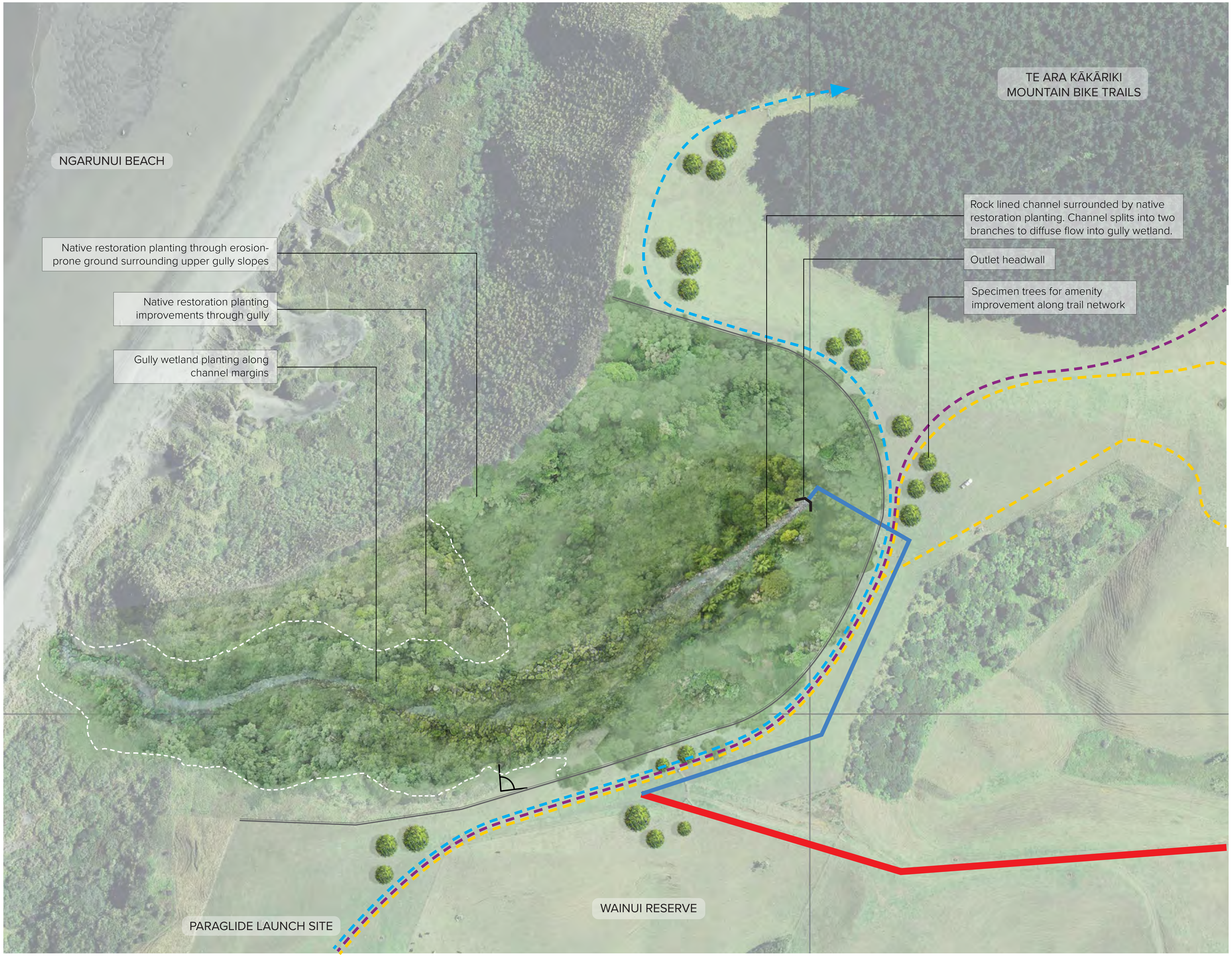
Images in this document: Unless otherwise noted, drawings, illustrations, photos and other images have been provided directly by Beca. In all other instances, best efforts have been made to reference the image to its original source.

LOCATION MAP



WAINUI RESERVE WESTERN GULLY DISCHARGE



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DESIGN PHILOSOPHY

Treated wastewater from the Raglan Wastewater treatment Plant is piped through the Wainui Reserve to a headwall outlet at the head of the western gully.

Treated wastewater is discharged to a rock lined channel diffuser before spilling into the gully, infiltrating into the stream and subsoils, before dissipating into the coastal groundwater table.

The proposed trail network shown is as per the Raglan Coastal Reserves Management Plan for Papahua, Manu Bay and Wainui Reserves (9 August 2021).

Proposed fencing restricts public access to treated wastewater discharge.

LEGEND

- Native Restoration Planting
- Gully Wetland Planting
- Rock-lined channel
- Treated Wastewater Feeder Lateral Line
- Treated Wastewater Mainline
- Outlet Headwall
- Mountain Bike Access Trail
- Walking Trail
- Horse Trek Trail
- Artist Visualisation Point of View
- Fence

N

0 10 50m
5 20

ARTISTS IMPRESSION

B

Appendix B – Overland Flow Path Modelling

Technical Memorandum

To:	Garrett Hall	**Date:**	16 April 2025
From:	Reza Shafiei	**Our Ref:**	4703642-1508349159-570
Copy:	Justin Kirkman	**:**	
Subject:	Raglan Wastewater Discharge - Hydraulic Assessment of Overland Flow Path		

1 Introduction

Stormwater runoff through South Head Gully (the gully), located west of Wainui Road in Raglan, discharges via a shallow overland flow path (OLFP) to Ngarunui Beach through existing sand dunes (see Figure 1). Observations from Beca's site visit on 28 August 2024, along with a review of aerial images, indicate the presence of a shallow erosional channel that has formed across the beach. Given the steep terrain of the gully, this channel is likely a result of high-velocity stormwater runoff to the beach during rainfall events.

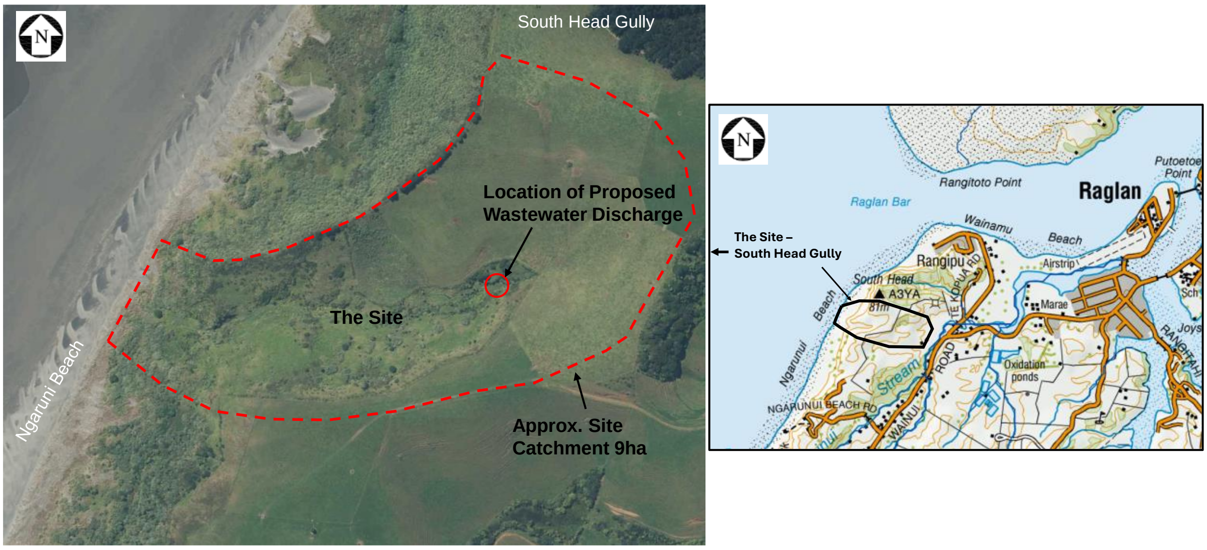

Figure 1: The site location (picture sourced from New Zealand River Flood Statistics database)

Watercare Waikato (Watercare) is planning to discharge treated wastewater from the Raglan/Whāingaroa Wastewater Treatment Plant into the gully as a new discharge point, located at the upper elevation of the gully. To support Watercare with the consenting of the discharge of treated wastewater into the gully, an assessment has been undertaken to evaluate its potential impact on erosion risk within the gully and the adjacent dune system.

This memorandum presents the methodology and results of hydraulic modelling conducted using HEC-RAS to analyse flow characteristics along the OLFP. The assessment focuses on how the proposed discharge may influence flow velocities, depths, and erosion potential along the gully and dune system.

2 Methodology

Given the elevation and distance of the proposed discharge point from the coastline, coastal conditions are not expected to directly impact it. The primary concern is how the additional flow from the proposed wastewater discharge may affect the future erosion of the gully and/or the stability of the dune system.

To assess this, an analysis was conducted by comparing changes in flow velocity and stream power within the gully due to the introduction of the new discharge. The following approach was undertaken:

- Catchment Delineation: A desktop assessment was performed to define the site-specific catchment area using available LiDAR-derived land contours.
- Rainfall Estimation: NIWA's HIRDS v4 dataset was used to determine current and future rainfall depths.
- Hydrology: Auckland Council's guidelines, as outlined in TP108, were adopted to model stormwater catchment runoff in HEC-HMS. The excess rainfall runoff depth was subsequently used in hydraulic modelling.
- Hydraulic Modelling: A rain-on-grid HEC-RAS 2D model was developed to simulate the OLFP and assess flow conditions within the gully.
- Scenario Simulation: The OLFP was simulated for present-day conditions as well as for the years 2030 and 2050, considering 5-, 10-, and 100-year Average Recurrence Interval (ARI) return period rainfall events. Each scenario was run for both existing conditions and the future condition including the new discharge point.
- Boundary Conditions: The worst-case coastal boundary for flow velocity was assumed to occur when there is no tidal backwater influence at the dune, as this would result in the highest flow velocities. A normal depth boundary condition was adopted as the downstream boundary for the HEC-RAS model.
- Flow Impact Analysis: Model outputs were processed to generate difference maps, highlighting changes in flow velocity and stream power[1] under different scenarios.
- Continuous Discharge Scenario: An additional scenario was simulated without rainfall, assuming a constant discharge from the new outfall. This was conducted to assess how the discharge behaves in dry conditions and its impact on flow dispersion toward the beach.

3 Site Hydrology

A HEC HMS[2] model was produced for the hydrological assessment using the methodology outlined in TP108 for the existing site catchment. The design parameters adopted for the assessment are summarised below.

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[1] Stream power is a measure of the energy of a stream channel including that which is related to the ability to transport sediment, and above a critical threshold, may indicate the initiation and erosive work done over time..

[2] Hydrologic Engineering Center (US). The Hydrologic Modelling System (HEC-HMS). US Army Corps of Engineers

3.1 Design Rainfall

Rainfall depths were derived from NIWA's High Intensity Rainfall Design System (HIRDS Version 4). Figure 3 shows the 5-, 10- and 100-year ARIs depth-duration-frequency data for the site. The 24-hour rainfall depths were adopted in this study.

Rainfall data were extracted from the Raglan Karioi station (Site ID: C74885 and Coordinates: 174.86788, -37.80211), as shown in Table 1. Present-day rainfall depths were assumed to align with historical data. The NIWA database provides various climate change (CC) rainfall depth scenarios based on IPCC5 scenarios. To account for climate change effects, rainfall data for the period 2031-2050 were used, where, for the year 2030, rainfall depths corresponding to RCP[3]-4.5 were adopted, and for the year 2050, rainfall depths corresponding to RCP-8.5 were adopted. This approach was taken based on the intermediate climate change scenario over the next 5 years (for the year 2030) and the higher uncertainty in climate change projections over the next 25 years (for the year 2050).

Table 1: Design rainfall depths

Return Period (year ARI)	Present-day Rainfall Depth (mm)	2030 Rainfall Depth – Period 2031-2051 with RCP4.5	2050 Rainfall Depth – Period 2031-2051 with RCP8.5
5	92.5	97.8	98.6
10	108	114	116
100	164	174	176

3.2 Time of Concentration

The time of concentration was calculated using the empirical lag equation given in TP108 (Equation 4.3 on page 12). A minimum time of concentration of 10 minutes was adopted, as per TP108.

3.3 Land Use

The site catchment is a grassy, hilly area, fully pervious, with an assumed curve number[4] of 74. An initial abstraction of 5 mm was adopted for pervious areas, in accordance with TP108.

[3] Representative Concentration Pathway (RCP) are climate change scenarios to project future greenhouse gas concentrations. The numerals after the RCP reflect the energy increase in the atmosphere in the units Watts/m²
caused by greenhouse gas concentrations that trap heat energy.

[4] The runoff curve number is an empirical parameter used in hydrology for predicting direct runoff or infiltration from rainfall excess.

Rainfall depths (mm) :: Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	9.20	12.4	14.8	19.9	26.5	41.0	52.7	66.5	81.8	91.1	97.7	103
2	0.500	10.0	13.5	16.1	21.6	28.9	44.6	57.4	72.3	89.0	99.1	106	112
5	0.200	12.8	17.3	20.6	27.7	36.9	57.0	73.4	92.5	114	127	136	143
10	0.100	14.9	20.1	24.0	32.2	43.0	66.3	85.4	108	132	147	158	166
20	0.050	17.2	23.1	27.5	37.0	49.3	76.1	98.0	123	152	169	181	190
30	0.033	18.5	24.9	29.7	39.9	53.2	82.1	106	133	164	182	195	205
40	0.025	19.5	26.3	31.3	42.0	56.0	86.4	111	140	172	192	205	216
50	0.020	20.3	27.3	32.5	43.7	58.3	89.9	116	146	179	199	214	225
60	0.017	20.9	28.2	33.5	45.1	60.1	92.7	119	150	185	206	220	232
80	0.013	22.0	29.6	35.2	47.3	63.1	97.3	125	158	194	216	231	243
100	0.010	22.8	30.7	36.5	49.0	65.4	101	130	164	201	224	240	252

Rainfall depths (mm) :: RCP4.5 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	10.0	13.5	16.1	21.6	28.7	43.8	55.9	70.0	85.3	94.5	101	106
2	0.500	10.9	14.7	17.5	23.6	31.4	47.8	61.0	76.2	93.0	103	110	116
5	0.200	14.0	18.9	22.5	30.3	40.3	61.4	78.4	97.8	119	132	141	148
10	0.100	16.4	22.1	26.3	35.3	47.0	71.6	91.4	114	139	154	165	173
20	0.050	18.8	25.4	30.2	40.6	54.0	82.4	105	131	160	177	189	199
30	0.033	20.3	27.4	32.6	43.8	58.3	88.9	113	141	172	191	204	214
40	0.025	21.4	28.9	34.4	46.2	61.4	93.7	119	149	181	201	215	226
50	0.020	22.3	30.0	35.8	48.0	63.9	97.4	124	155	189	209	224	235
60	0.017	23.0	31.0	36.9	49.6	65.9	101	128	160	195	216	231	242
80	0.013	24.2	32.6	38.7	52.0	69.2	106	134	168	205	227	242	254
100	0.010	25.1	33.8	40.2	54.0	71.8	109	140	174	212	235	251	263

Rainfall depths (mm) :: RCP8.5 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	10.1	13.7	16.3	21.9	29.0	44.2	56.3	70.5	85.8	95.0	102	107
2	0.500	11.1	14.9	17.7	23.9	31.7	48.3	61.5	76.8	93.6	104	111	116
5	0.200	14.2	19.2	22.8	30.7	40.8	62.1	79.1	98.6	120	133	142	149
10	0.100	16.6	22.4	26.6	35.8	47.6	72.4	92.3	115	140	155	166	174
20	0.050	19.1	25.7	30.6	41.1	54.7	83.3	106	132	161	178	191	200
30	0.033	20.6	27.8	33.1	44.4	59.0	89.9	114	143	174	192	206	215
40	0.025	21.7	29.3	34.8	46.8	62.2	94.7	121	150	183	203	216	227
50	0.020	22.6	30.4	36.2	48.7	64.7	98.5	125	156	190	211	225	236
60	0.017	23.3	31.4	37.4	50.2	66.8	102	129	161	196	218	232	243
80	0.013	24.5	33.0	39.3	52.8	70.1	107	136	169	206	228	244	256
100	0.010	25.4	34.2	40.7	54.7	72.7	111	141	176	214	237	253	265

Figure 2: Rainfall depths for Raglan Karioi station (source: NIWA HIRDS – Site ID: C74885)

4 Hydraulic Modelling

4.1 Model Description

For this assessment, a hydraulic model was developed using the HEC-RAS 6.3.1 modelling suite (Hydraulic Engineering Centre – River Analysis System), developed by the U.S. Army Corps of Engineers.

HEC-RAS 2D is capable of simulating two-dimensional unsteady flow across open channels, alluvial fans, and floodplains, and it can handle subcritical, supercritical, and mixed flow regimes. The model solves the full 2D Saint-Venant equations (shallow water equations) using an implicit finite difference method. Further details on the model's computational procedures can be found in the HEC-RAS 6.3.1 User's Manual.

To simulate surface water flow within the gully, a rain-on-grid OLFP simulation approach was used. This hydraulic modelling technique applies rainfall directly to the computational grid, allowing runoff to form dynamically based on terrain gradients and hydraulic conditions. This method is particularly effective for modelling distributed runoff, sheet flow, and flow concentration in areas with complex topography, such as gullies and dune systems.

4.2 Model Geometry

A digital terrain model (DTM) of the site catchment area was developed using the Land Information New Zealand (LINZ) LiDAR data for the Waikato region, captured between 5 January and 26 March 2021. The LiDAR data has a vertical accuracy of ±0.2m, horizontal accuracy of ±1m and contains the necessary physical description of the gully. Refer to Figure 3 for a hill-shaded relief DEM of the LiDAR data in HEC-RAS' inbuilt GIS platform RAS-Mapper.

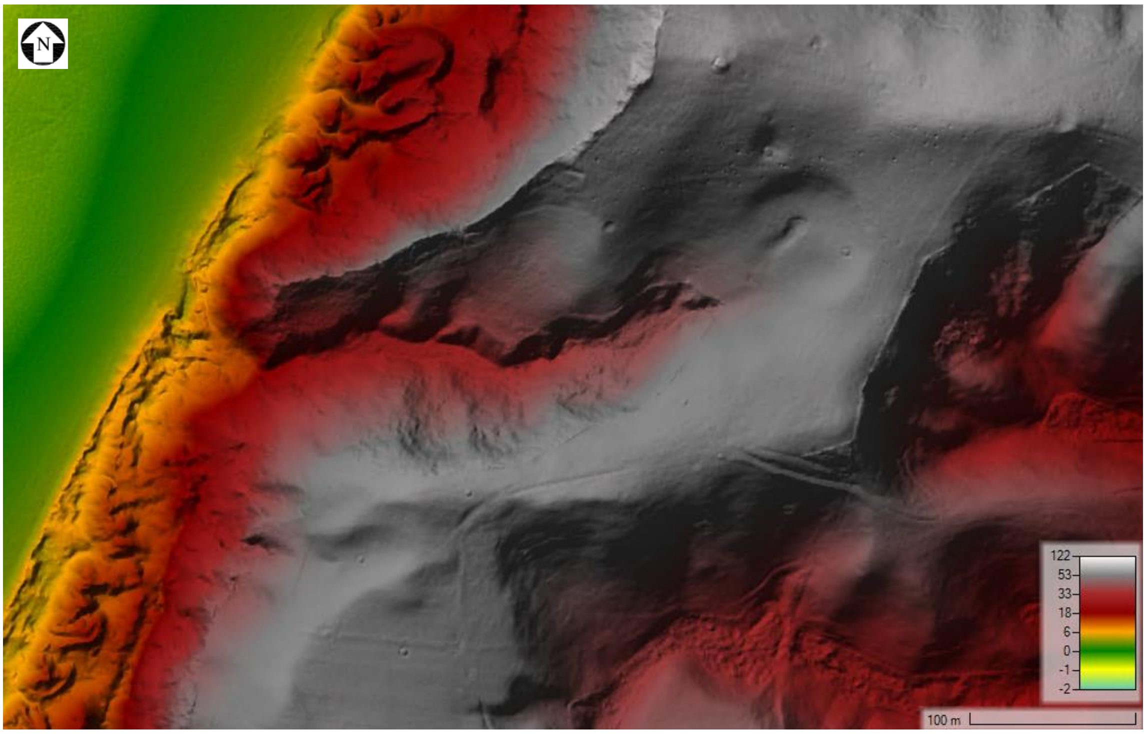


Figure 3: Model terrain (source LINZ LiDAR)

The model geometry was created using variable cell sizes to optimise the simulation runtime. The initial cell size was set to 3m x 3m, resulting in a predominantly rectangular mesh, except for cells adjacent to grid boundaries, which assumed irregular polygon shapes with not more than 8 sides. This initial cell size was applied in areas of high elevation. Further refinement to 2m x 2m was implemented around the OLFP area. A breakline was introduced approximately along the centreline of the OLFP to further reduce the cell size to 1m. The 2m x 2m cell size around the OLFP area aligns with Auckland Council's stormwater modelling specifications. The 1m x 1m refinement served as a precaution to account for potential model instability due to rapid velocity changes across the steep gully slope.

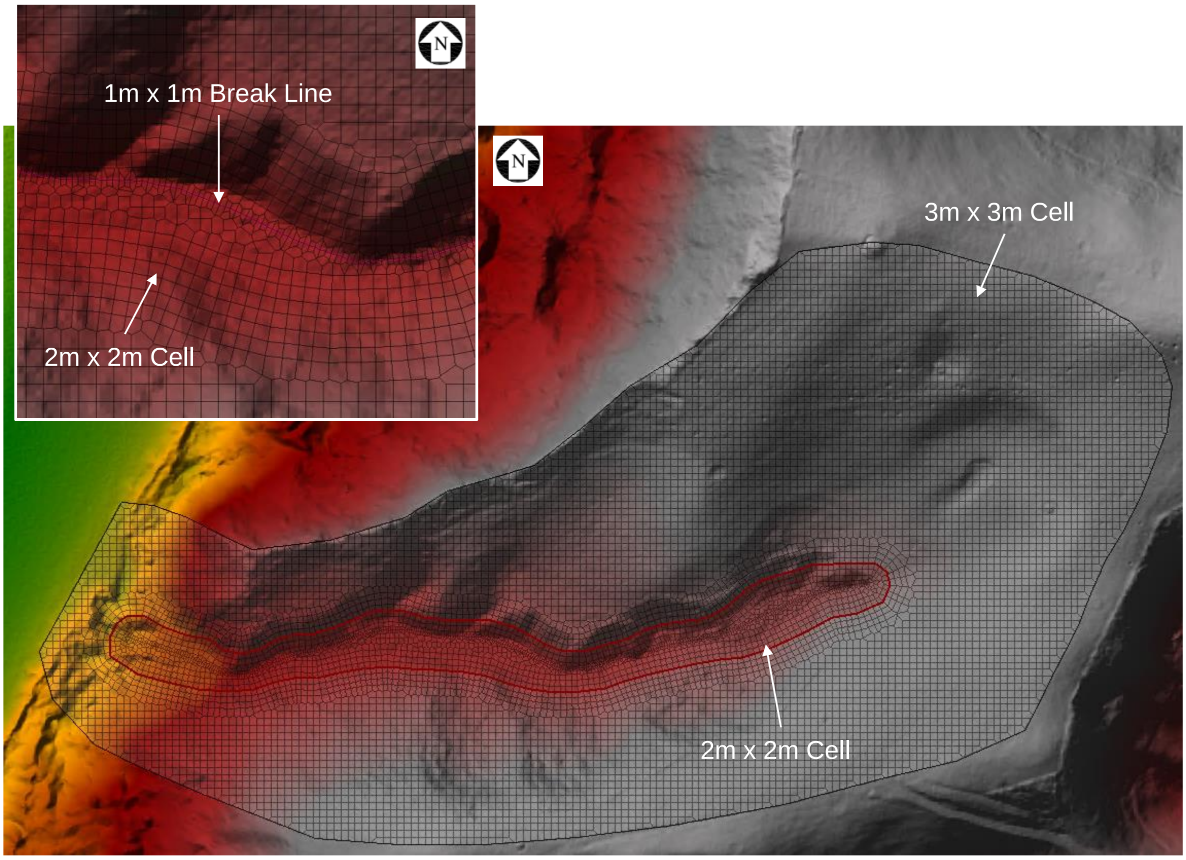
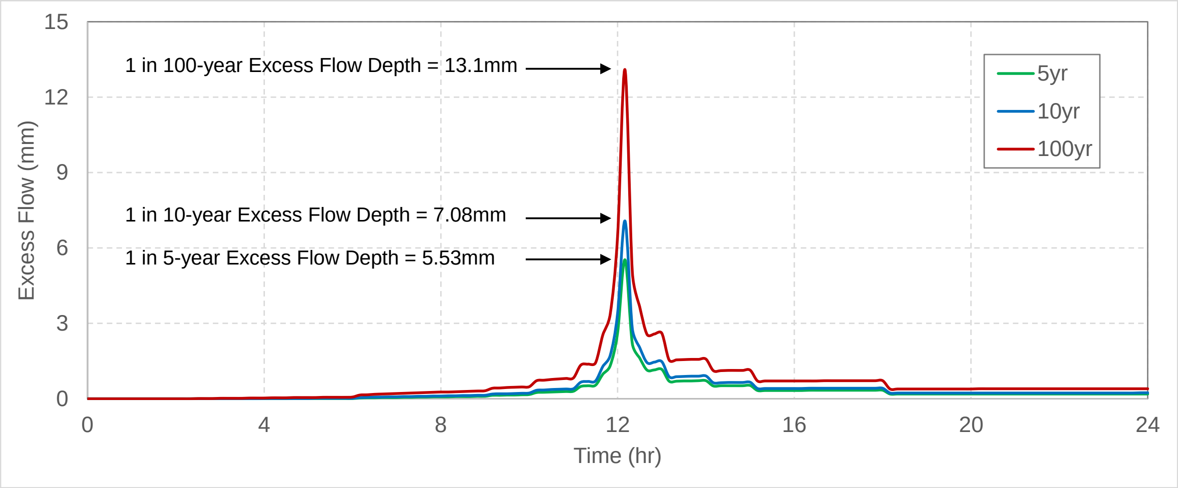


Figure 4: Model geometry and typical cell structure

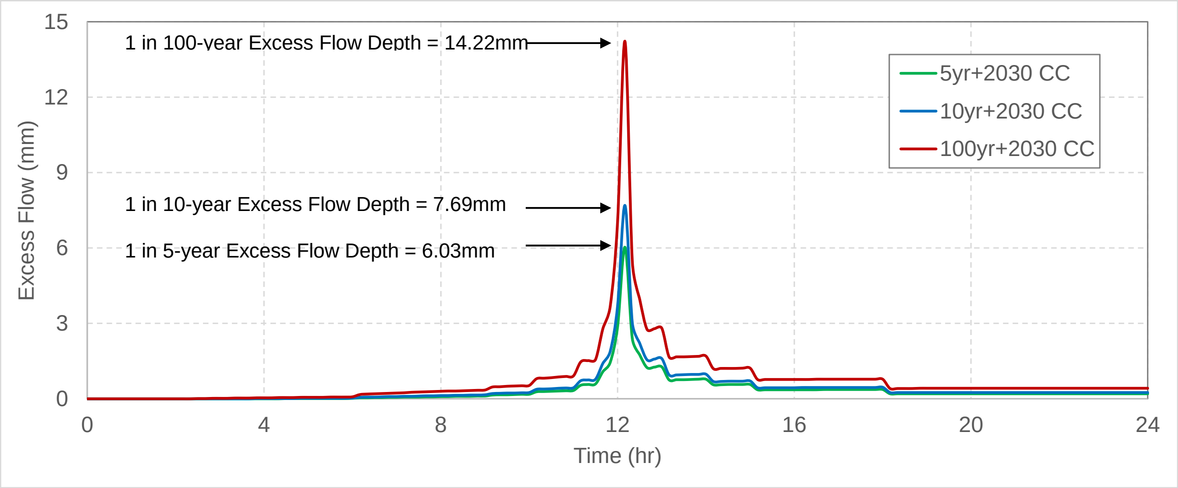
As prescribed in the Auckland Council stormwater modelling specifications, a Manning's "n" roughness coefficient of 0.05 was adopted as the default value for channel, bank and floodplain vegetation representing light brush and trees.

4.3 Model Forcing and Boundary Condition

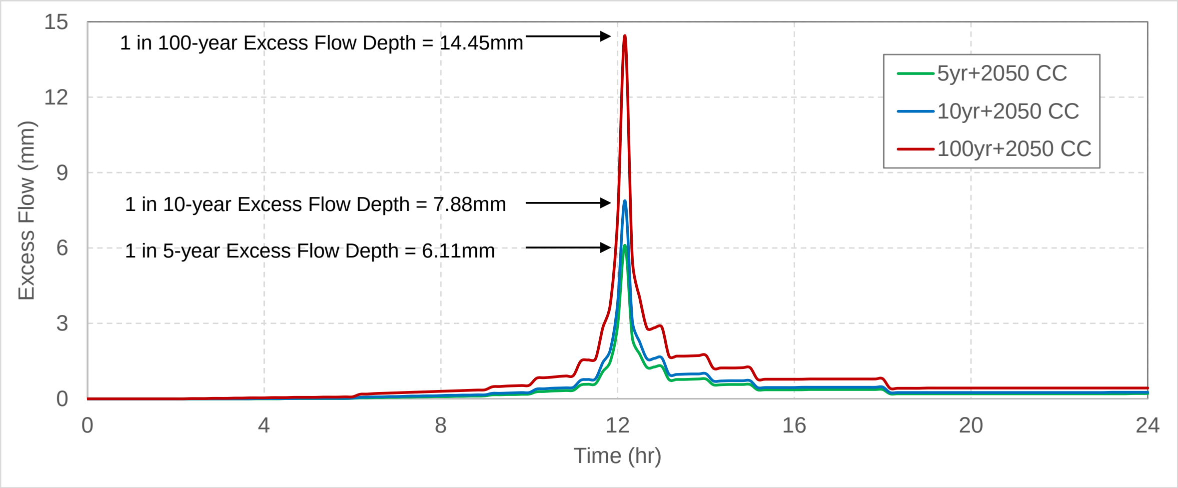
A rain-on-grid model was built in HEC-RAS 2D. For this purpose, infiltration was removed during the development of the nested rainfall hyetograph in HEC-HMS so that the precipitation time series includes rainfall excess only. The excess rainfall depths (i.e. 10-year and 100-year ARIs) for present-day and future scenarios were obtained in HEC-HMS 4.10 using Soil Conservation Service Runoff Curve Number method.



(a) Present-day



(b) 2030



(c) 2050

Figure 5: Excess rainfall runoff depth inputs to the rain-on-grid model for (a) present-day condition, (b) in 2030 and (c) in 2050.

The excess flows were further modelled in HEC-RAS to determine the channelized flow depths and velocities. Figure 4 shows the extent of the HEC-RAS 2D model. Normal depth was applied as downstream boundary condition. The normal depth condition assumes that OLFP flows under normal flow (uniform flow) conditions at the downstream boundary of the model. This allows to provide an energy slope, and then HEC-RAS will automatically back-calculate the depth using Manning's Equation.

The proposed wastewater discharge was implemented in the model as an internal boundary condition. Wastewater flowrate projections for present-day, 2030, and 2050 scenarios are detailed below:

- Present-day maximum wastewater flowrate 45L/s
- 2030 maximum wastewater flowrate 70L/s
- 2050 maximum wastewater flowrate 90L/s

It has been assumed that wastewater discharge is a constant flow, and the worst-case scenario occurs when peak stormwater runoff coincides with peak wastewater flow. However, it should be noted that this is a conservative modelling approach, as the likelihood of a fixed flow rate of 90L/s occurring simultaneously with peak stormwater runoff is low.

4.4 Model Runs

A total of 21 model scenarios, as outlined in Table 2, were designed to assess the impact of wastewater discharge on the gully's OLFP conditions and potential increase in erosion risk. The scenarios were structured as follows:

- Baseline Scenarios (Existing OLFP Conditions):
- These scenarios simulated the existing OLFP under present-day, 2030, and 2050 rainfall conditions (5-, 10-, and 100-year ARIs). The flow depth, velocity, and shear stress were evaluated to establish baseline conditions within the gully before introducing wastewater discharge.
- Combined Flow Scenarios (OLFP with Wastewater Discharge):
- These scenarios combined stormwater runoff (5-, 10-, and 100-year ARIs for present-day, 2030, and 2050) with the proposed wastewater discharge. The resulting changes in flow velocity and shear stress were quantified to assess potential increases in erosion risk within the gully.
- Wastewater-Only Scenarios (No Stormwater Runoff):
- These scenarios simulated wastewater discharge within the gully in the absence of stormwater runoff. This was designed to evaluate whether continuous wastewater discharge, combined with low soil permeability, could create a permanent, visible flow path and could cause additional erosion compared to OLFP.

Table 2: HEC-RAS 2D model runs

Run ID	Scenario	Comment
Run 1	5yr	Present-day OLFP conditions without proposed wastewater discharging into the gully
Run 2	10yr	
Run 3	100yr	
Run 4	5yr+2030CC	

Run ID	Scenario	Comment
Run 5	10yr+2030CC	OLFP conditions in 2030 without proposed wastewater discharging into the gully
Run 6	100yr+2030CC	
Run 7	5yr+2050CC	OLFP conditions in 2050 without proposed wastewater discharging into the gully
Run 8	10yr+2050CC	
Run 9	100yr+2050CC	
Run 10	5yr&45L/s	Present-day OLFP conditions with proposed wastewater discharging 40L/s into the gully
Run 11	10yr&45L/s	
Run 12	100yr&45L/s	
Run 13	5yr&70L/s+2030CC	OLFP conditions in 2030 with proposed wastewater discharging 70L/s into the gully
Run 14	10yr&70L/s +2030CC	
Run 15	100yr&70L/s +2030CC	
Run 16	5yr&90L/s+2050CC	OLFP conditions in 2050 with proposed wastewater discharging 90L/s into the gully
Run 17	10yr&90L/s +2050CC	
Run 18	100yr&90L/s +2050CC	
Run 19	WW 45L/s	Proposed wastewater discharging 45L/s into the gully in present-day and no stormwater runoff
Run 20	WW 70L/s	Proposed wastewater discharging 70L/s into the gully in 2030 and no stormwater runoff
Run 21	WW 90L/s	Proposed wastewater discharging 90L/s into the gully in 2050 and no stormwater runoff

4.5 Model Assumptions

The following assumptions were used:

- Catchment's time of concentration is 10 minutes.
- Initial abstraction of 5mm for previous areas.
- Curve number of 74 is assumed.
- Each grid cell at the OLFP location is 2m x 2m in size with a breakline reducing the cells to 1m x 1m along the centreline of the OLFP.
- Adjustable time step setting based on a maximum and minimum Courant number of 0.9 and 0.4, respectively.
- Manning's Coefficient (n) = 0.05 for the green field with light brush and trees.
- The model was run for a 24-hour simulation period.
- The effective rainfall depth was used in this study and the rainfall hyetograph was loaded directly onto the entire 2D model extent based on the specified rain-on-grid modelling approach.

- The proposed wastewater discharge was implemented as an internal boundary condition within the 2D model geometry, represented as a constant flow. This constant discharge assumption was employed to capture worst-case conditions, based on the premise that peak rainfall runoff coincides with peak wastewater discharge. This approach allows for the extraction of maximum values under combined peak flow scenarios.

5 Hydraulic Modelling Results

All model results were processed into raster mapper files and time series for maximum water level, flow velocity and depth. The presentation of these results aligns with the model run structure outlined in Section 4.4. Difference maps were then generated to highlight key changes in hydraulic parameters that could potentially affect gully conditions.

5.1 Baseline Scenarios Results

The extent of the OLFP for different model runs (refer to Table 2) was plotted to demonstrate the flow formation within the gully. Figure 6 shows the OLFP extents for present-day rainfall events (Runs 1, 2 and 3) and present-day wastewater discharge flow rate (Run 19). Figures for all scenario runs showing the modelled flow depth and flow velocity are provided in the Appendix.

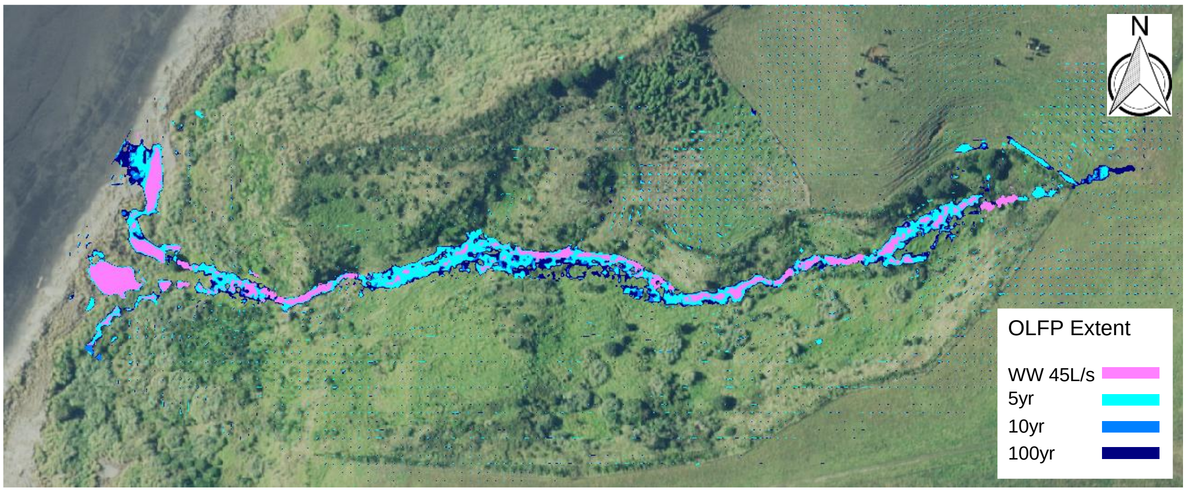


Figure 6: OLFP extent for different baseline present-day flow scenarios

As expected, the extent of the OLFP formed by wastewater discharge is insignificant compared to those generated by rainfall runoff events. The key observation for baseline flow scenarios is that the OLFP along the upper gully section forms a shallow flow path, creating localised ponding when discharging onto the flat foreshore.

Additionally, the sand dune topography contributes to flow retention, forming small water-trapping zones and ponding areas. The volume of water retained in these areas depends on:

- Local groundwater levels
- Sand permeability characteristics
- Infiltration rates into the subsurface

5.2 Combined Flow Scenarios Results

A further comparison was conducted to investigate the effect of the proposed wastewater discharge coinciding with rainfall runoff. A comprehensive set of model results for flow depths and velocities is provided in the Appendix. However, the results for the worst-case scenario–specifically, the 100-year ARI rainfall runoff incorporating climate change effects–are detailed below.

Figure 7 illustrates the present-day (Run 3) and future 100-year ARI OLFPs (Run 9), indicating that the impact of climate change on the extent of the OLFP is minimal. Moreover, the alignment of the OLFP remains reasonably consistent across both scenarios.

Additionally, a long section along the centreline of the OLFP was plotted to capture areas showing changes in water surface elevations. The long section profile indicates that the water surface elevation remains approximately unchanged. This minimal effect of additional flow due to climate change to the OLFP extent can be attributed to the steep gully profile, where flow conditions are more influenced by velocity head.

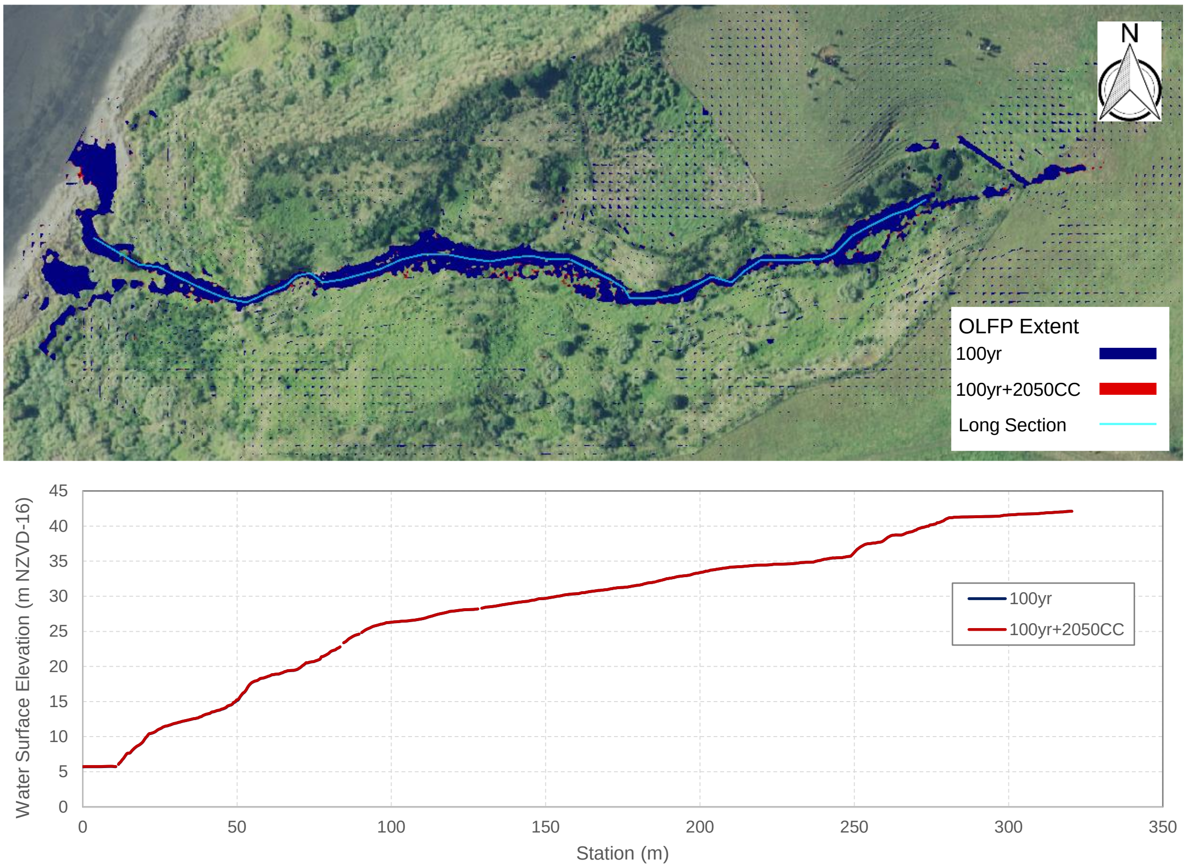
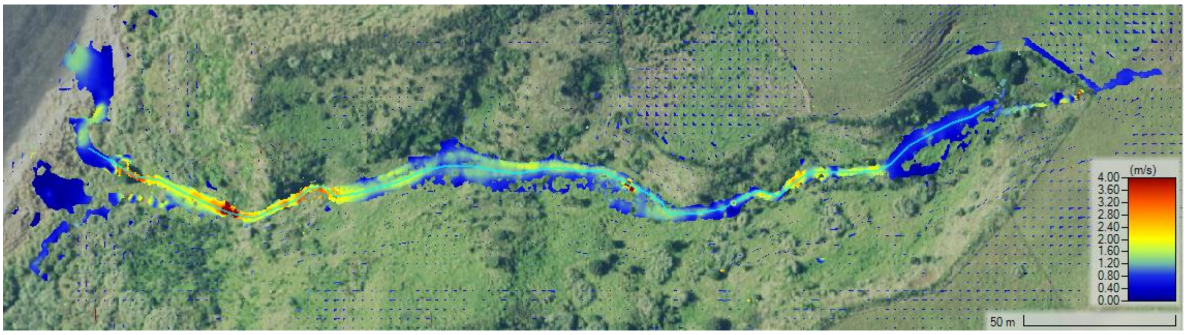
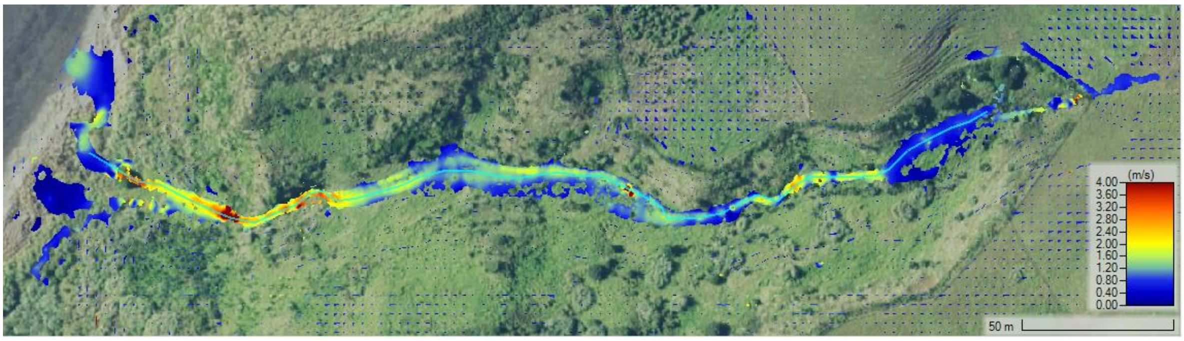


Figure 7: OLFP extents for the present-day 100-year ARI rainfall runoff and in 2050

Consistent with the OLFP extents shown in Figure 7, the velocity patterns and magnitudes within the gully also do not change significantly with the climate change effect, as shown in Figure 8. This suggests that the gully conditions over the next 25 years are not expected to be substantially impacted by climate change.



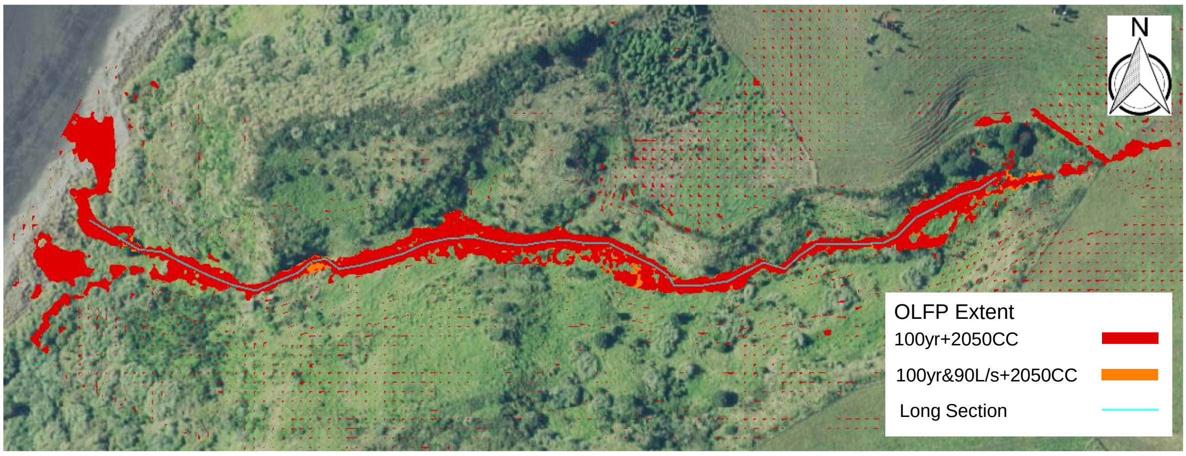
(a) 100-year ARI flow velocity



(b) 100-year ARI with climate change allowance to 2050

Figure 8: OLFP velocity maps for the 100-year ARI rainfall event with and without climate change effect

In addition to the climate change effect assessment, the anticipated 90L/s wastewater discharge rate was incorporated into the OLFP flow (Run 18). Figure 9 illustrates that the OLFP extent undergoes minor changes, estimated to be less than 3% in some very local areas. The long section along the centreline of the OLFP reveals no significant alterations in water surface elevation, leading to the conclusion that the additional 90L/s of wastewater discharge has a minimal impact on the OLFP conditions.



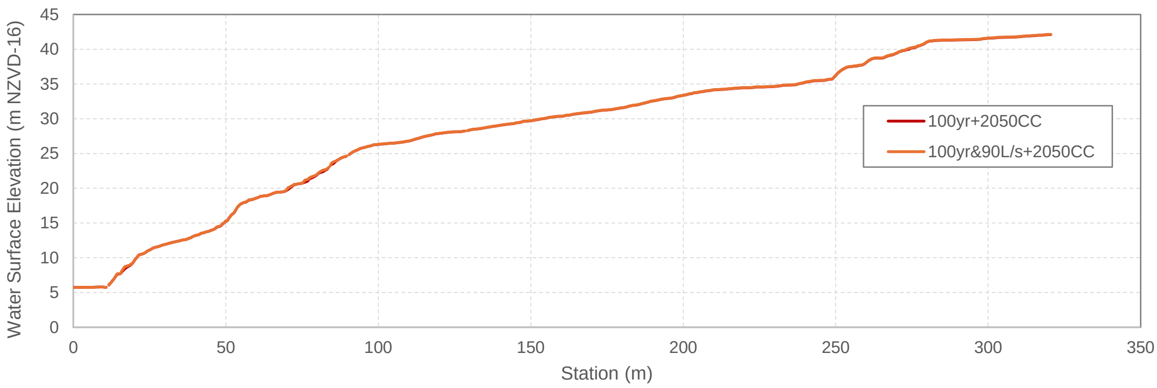


Figure 9: OLFP extents for the present-day 100-year ARI rainfall runoff and in 2050

The OLFP hydraulic conditions for present-day (Run 3), future (Run 9), and future with the addition of wastewater discharge (Run 18) indicate a high velocity zone near the lower section of the gully (refer to Figure 10). This area experiences localised flow acceleration due to water flowing over steep terrain.

A cross-section was plotted across this area to compare the flow rates, as shown in Figure 10. For the OLFP conditions with climate change allowance, the flow increases by 5.6% (from 1.60m³/s to 1.69m³/s) due to the discharge of 90L/s of wastewater into the gully. This increase is considered minimal and does not significantly change the OLFP effect in this area.

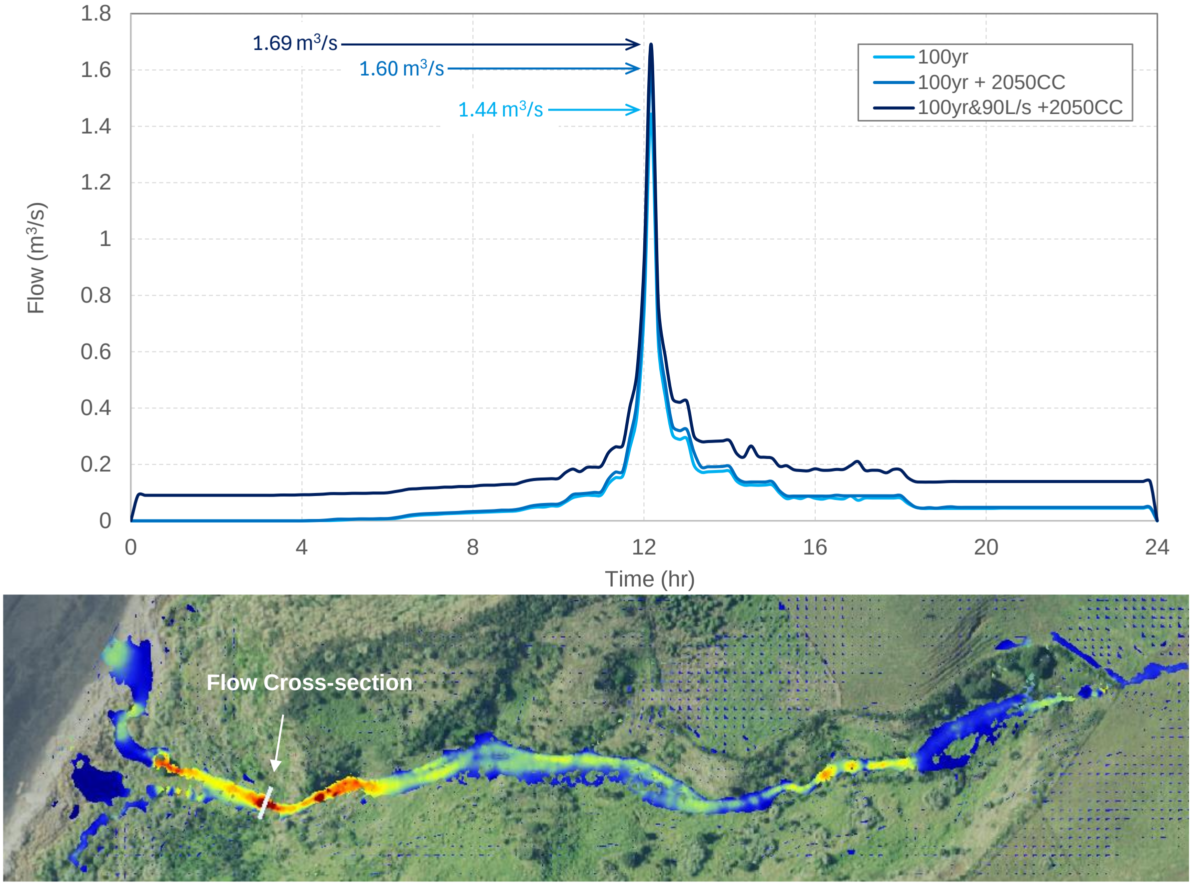


Figure 10: Flow rates at the location of high velocity

5.3 Wastewater-Only Scenarios Results

To investigate whether continuous wastewater discharge can create a visible and permanent flow through the gully and onto the beach, the model was run without rainfall events, solely using wastewater flow rates for present-day (Run 19), 2030 (Run 20), and 2050 (Run 21) scenarios. Figure 11 illustrates the OLFP generated by these wastewater flow rates, indicating that continuous wastewater discharge can leave a visible OLFP that can flow over the beach. However, due to the lower flow volumes compared to rainfall runoff, the duration of continuous discharge, combined with the volume of sand and ground permeability at the dune location, determines the volume that can infiltrate the ground and the volume that can form a permanent channel discharging into the sea.

Figure 11: Wastewater flow-generated OLFP

5.4 Difference Maps

To assess the impact of the proposed wastewater discharge on gully stability, a series of difference maps were created for various comparable modelled scenarios, focusing specifically on velocity changes. Utilising HEC-RAS post-processing capabilities, the stream power parameter was calculated for each model run as the product of average velocity and average shear stress. Subsequently, stream power difference maps were generated to evaluate the erosional potential of the OLFP due to the proposed wastewater discharge flows.

The difference maps for the worst-case scenario–combining the 100-year ARI rainfall runoff (with climate change allowances) and the 2050 wastewater discharge rate of 90 L/s–are presented in Figure 12 (velocity differences) and Figure 13 (stream power differences). These maps compare two critical scenarios:

1. Future Combined vs. Present-Day Rainfall: This scenario compares the 2050 combined scenario (100-year ARI + climate change + 90 L/s wastewater) with the present-day 100-year ARI rainfall runoff. It represents the maximum incremental impact due to both climate change and wastewater discharge.
2. Future Combined vs. Future Rainfall Alone: This scenario compares the 2050 combined scenario with the 2050 100-year ARI rainfall runoff (climate-adjusted). It isolates the specific contribution of wastewater discharge under future climate conditions.

Figure 12 illustrates that the velocity increases by approximately 0.2m/s along the OLFP for Scenario 1, with localised areas experiencing increases of up to 0.3m/s. These localised velocity increases are primarily attributed to abrupt changes in terrain elevation captured by existing LiDAR data.

In Scenario 2, when comparing the proposed wastewater discharge in isolation, the change in velocity relative to rainfall runoff is minor, up to 0.1m/s. The localised higher velocity increases observed in both scenarios occur at the same locations and are attributed to the terrain steepness causing flow acceleration. These modelled changes in velocity due to the proposed wastewater discharge are small and unlikely to impact gully stability compared to its condition under rainfall events.

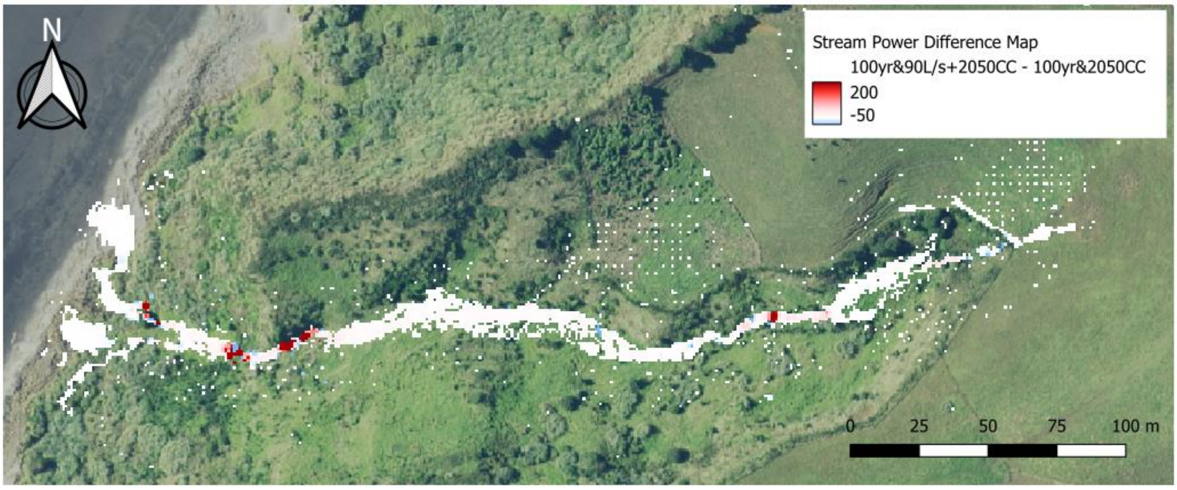
(a) Scenario 1

(b) Scenario 2

Figure 12: Velocity difference maps for (a) Scenario 1 and (b) Scenario 2

Stream power is a geomorphological metric commonly used to quantify the erosive power of water flow in landscapes. It is utilised to predict erosion and deposition patterns. To further assess the risk of erosion within the gully due to the minor velocity changes described above, stream power difference maps were created. Figure 13 illustrates negligible changes in stream power for both Scenarios 1 and 2, confirming that the velocity changes are minor and would not exacerbate gully conditions compared to its state under rainfall runoff events. The locations of the largest changes in stream power are consistent with the velocity changes and result from flow acceleration over abrupt elevation changes. Although these changes are minor, they could cause localised erosion in the long term. Erosion initiates when the stream power exceeds a critical shear threshold, meaning not all increases in stream power will necessarily cause more erosion.

(a) Scenario 1



(b) Scenario 2

Figure 13: Stream power difference maps for (a) Scenario 1 and (b) Scenario 2

6 Conclusions

A HEC-RAS 2D hydraulic model was developed to assess the potential effect of the new wastewater discharge on the gully's hydraulic conditions. The model results indicate that continuous flow from the proposed wastewater outlet may form a visible OLFP, but this will not likely create a permanent channel or significantly alter the gully's morphology. The OLFP generated by wastewater flows will be noticeable in dry weather conditions. However, during rainfall events, stormwater runoff will dominate the flow pattern.

Furthermore, the model results indicate minor increases in velocity and stream power from wastewater flows. This will not pose a risk to erosion initiation or continual degradation of the channel. However, it is recommended to implement a monitoring regime to inspect the gully conditions annually for the first couple of years, and then every two years thereafter, to capture any potential erosion development and, if required, implement appropriate measures.

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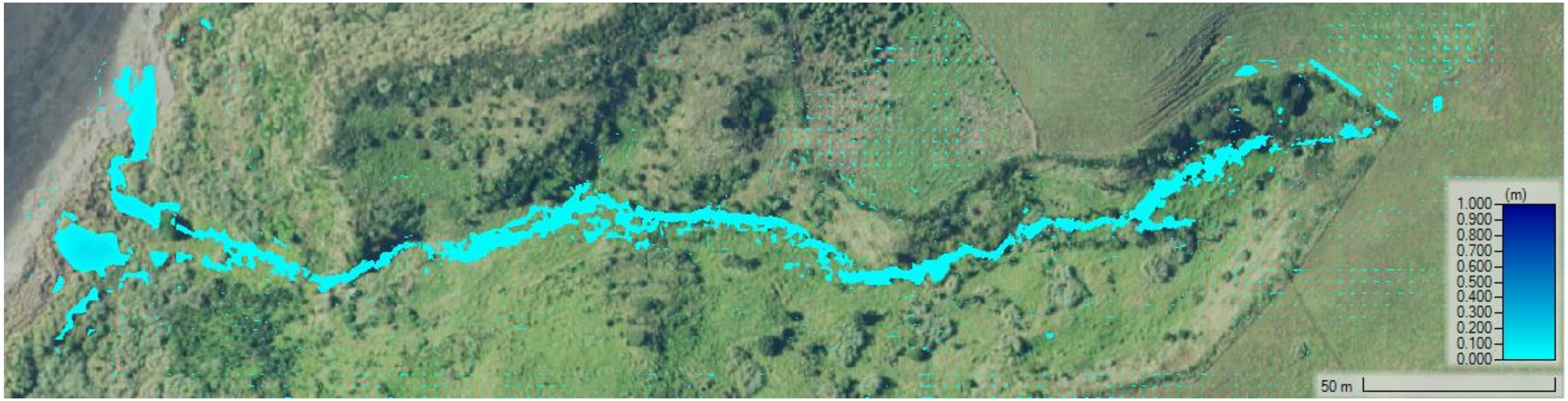
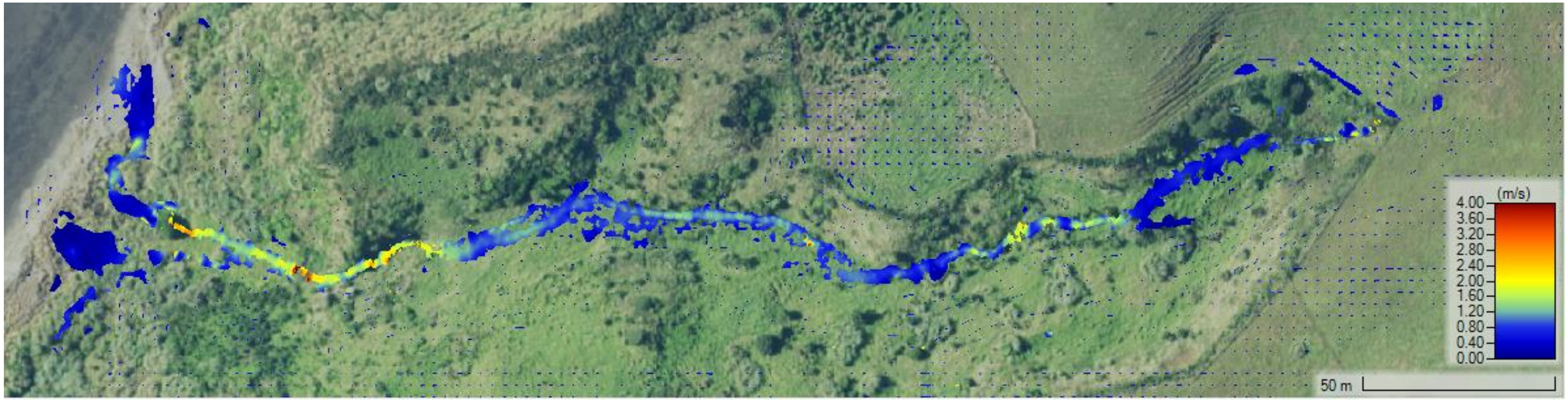
Appendix

Model Results

Run 1 – 5yr

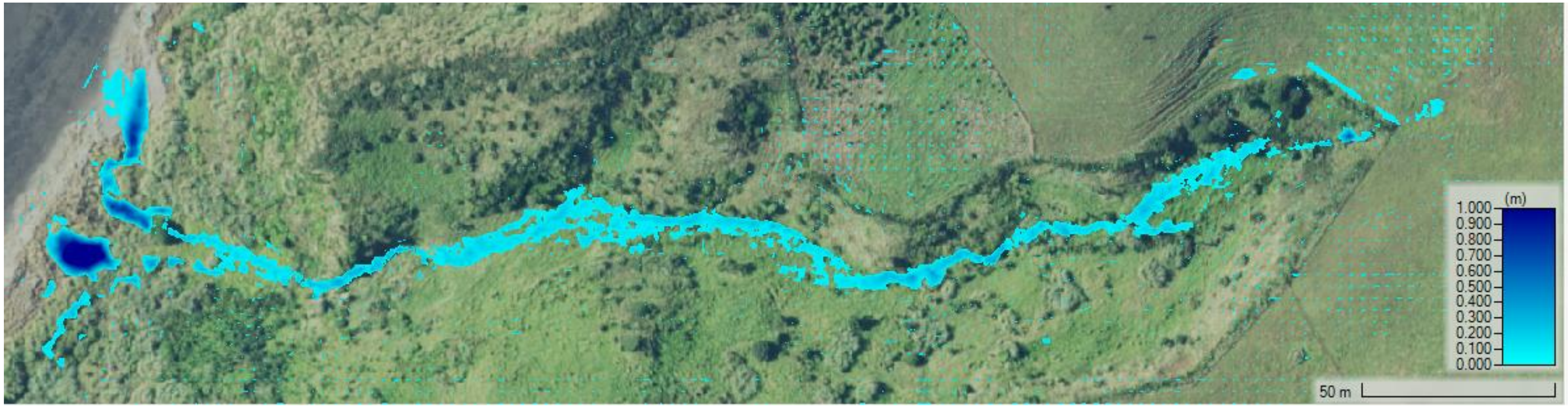


Maximum Flow Depth

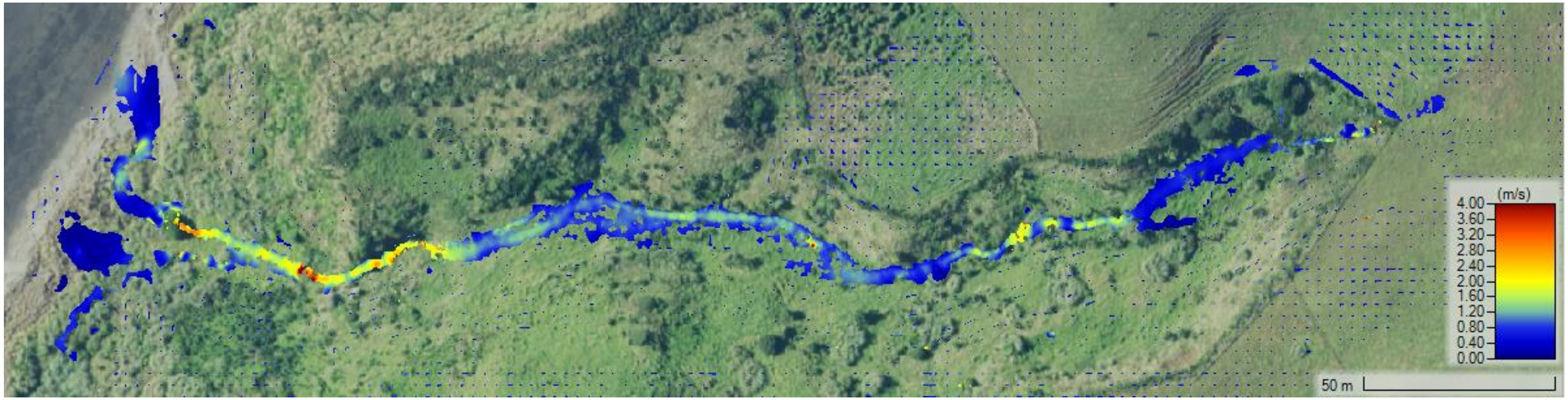


Maximum Flow Velocity

Run 2 – 10yr

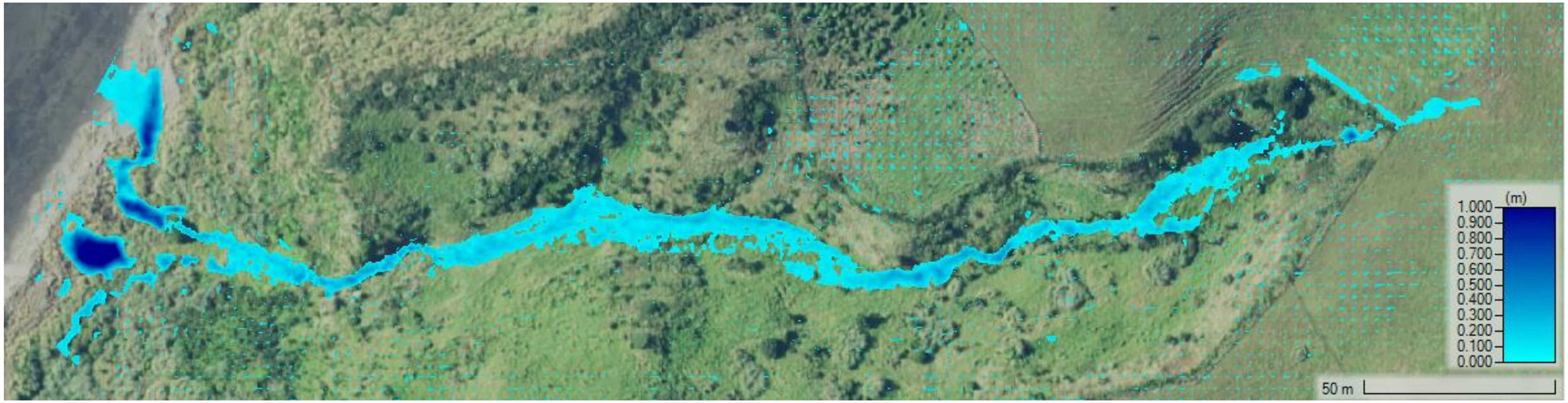


Maximum Flow Depth

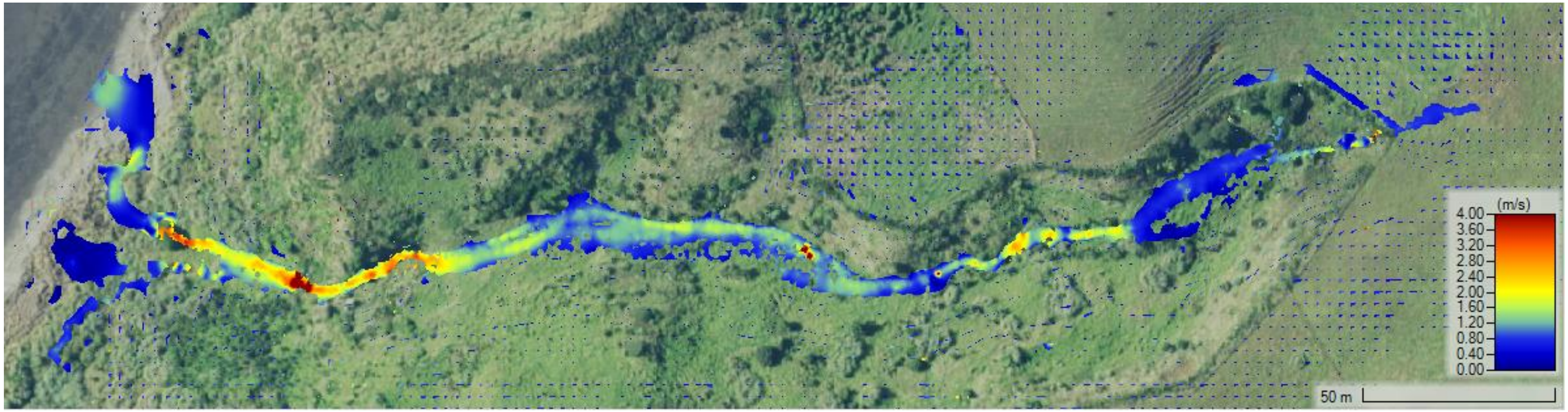


Maximum Flow Velocity

Run 3 – 100yr



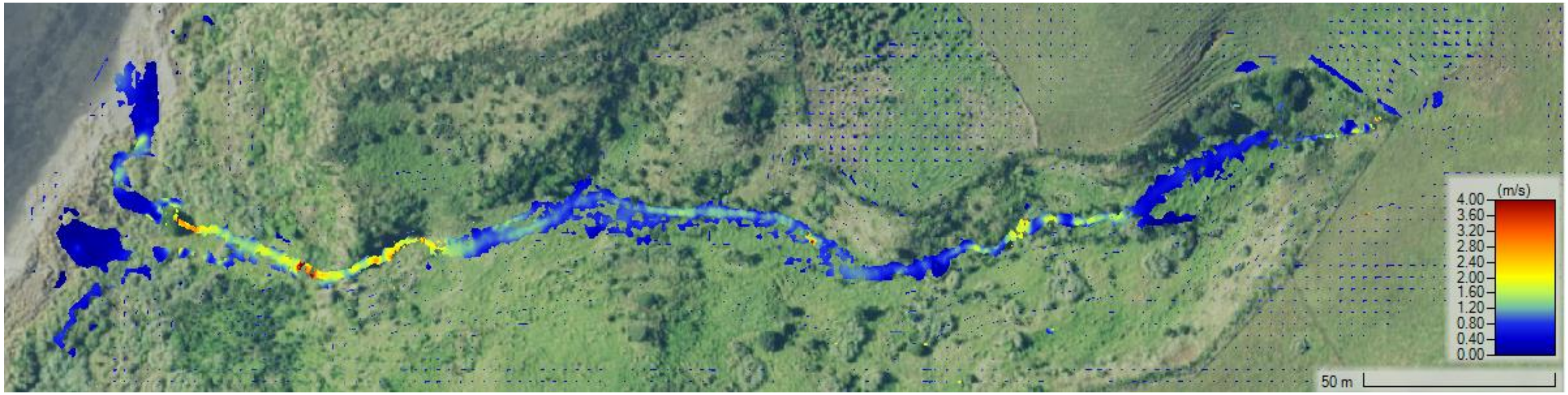
Maximum Flow Depth



Maximum Flow Velocity

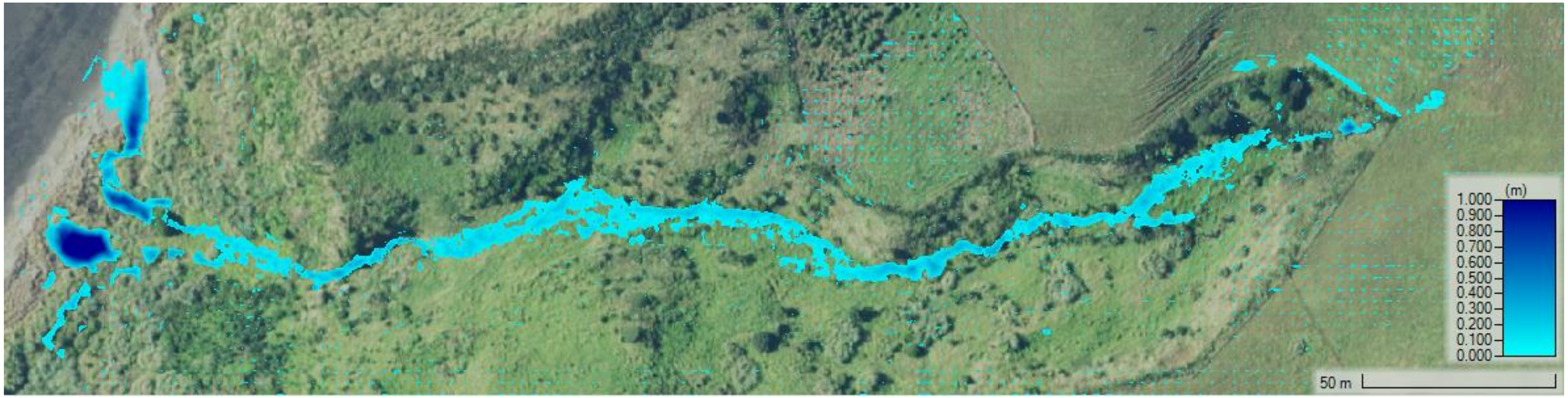
Run 4 – 5yr+2030CC

Maximum Flow Depth

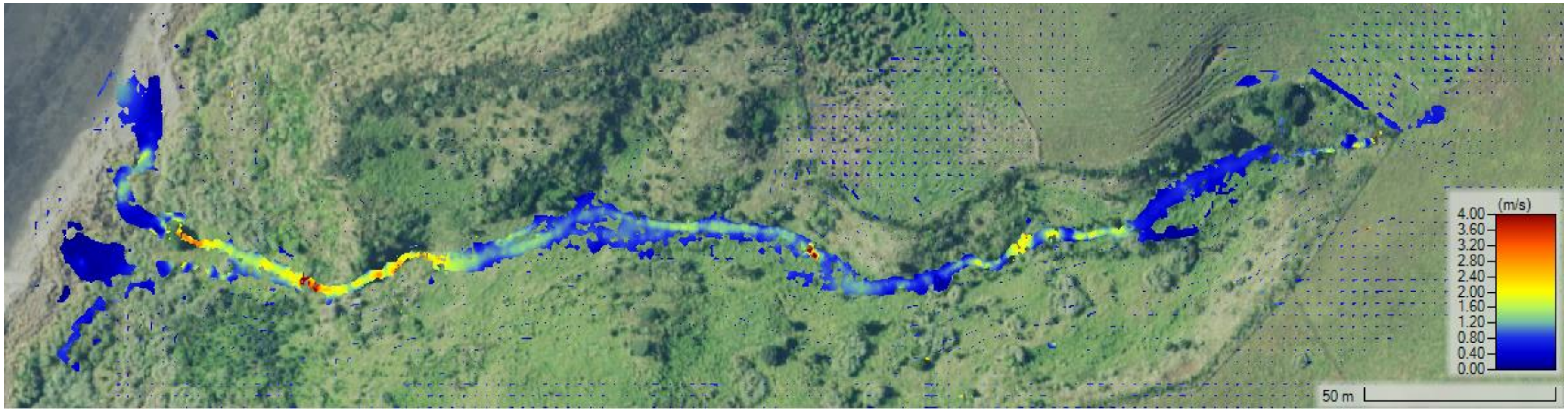


Maximum Flow Velocity

Run 5 – 10yr+2030CC

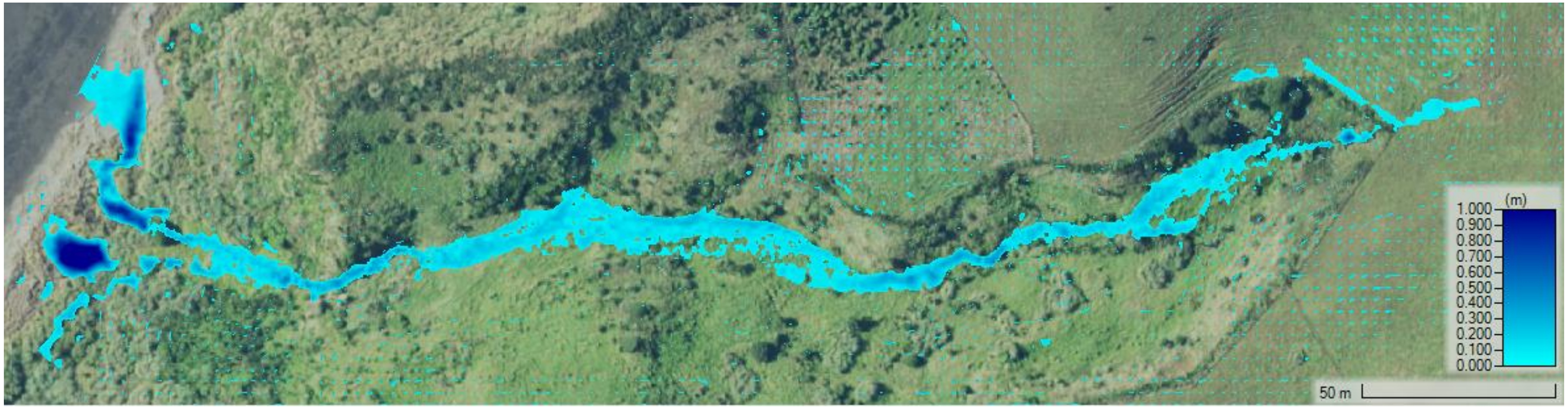


Maximum Flow Depth

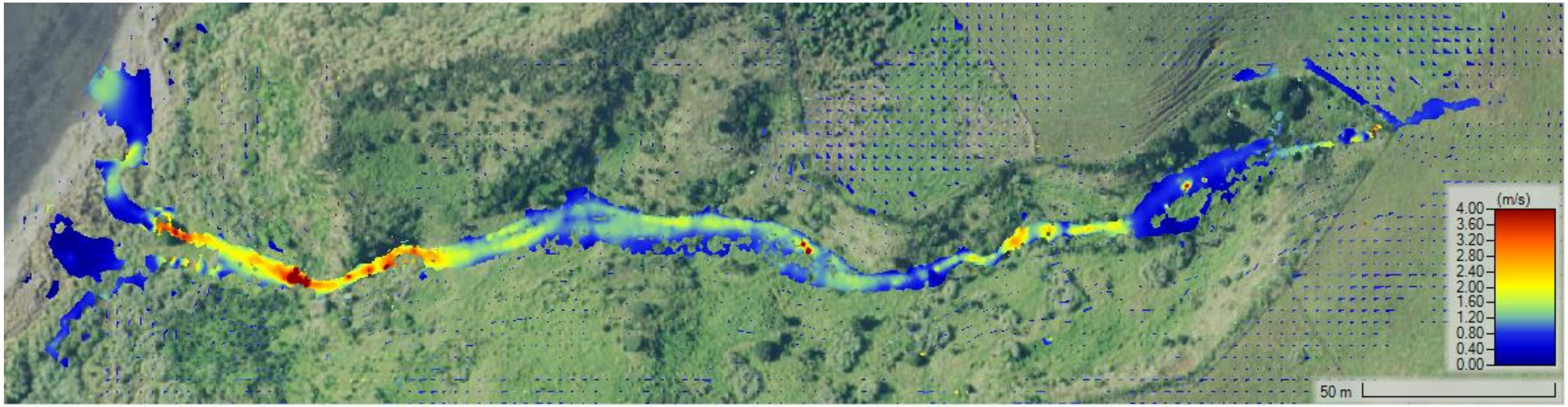


Maximum Flow Velocity

Run 6 – 100yr+2030CC



Maximum Flow Depth



Maximum Flow Velocity

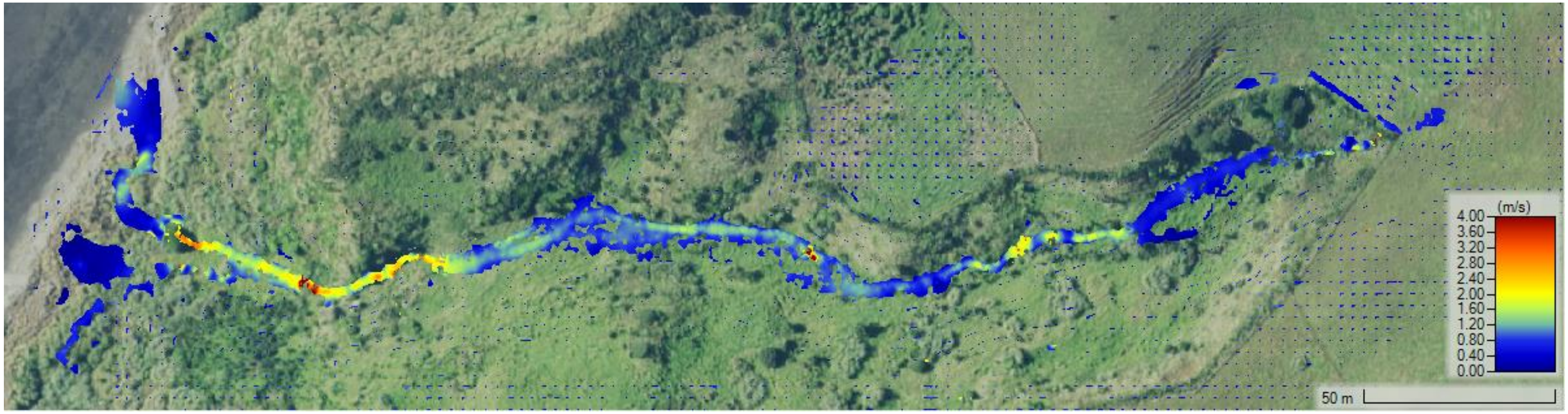
Run 7 – 5yr+2050CC

Maximum Flow Depth

Maximum Flow Velocity

Run 8 – 10yr+2050CC

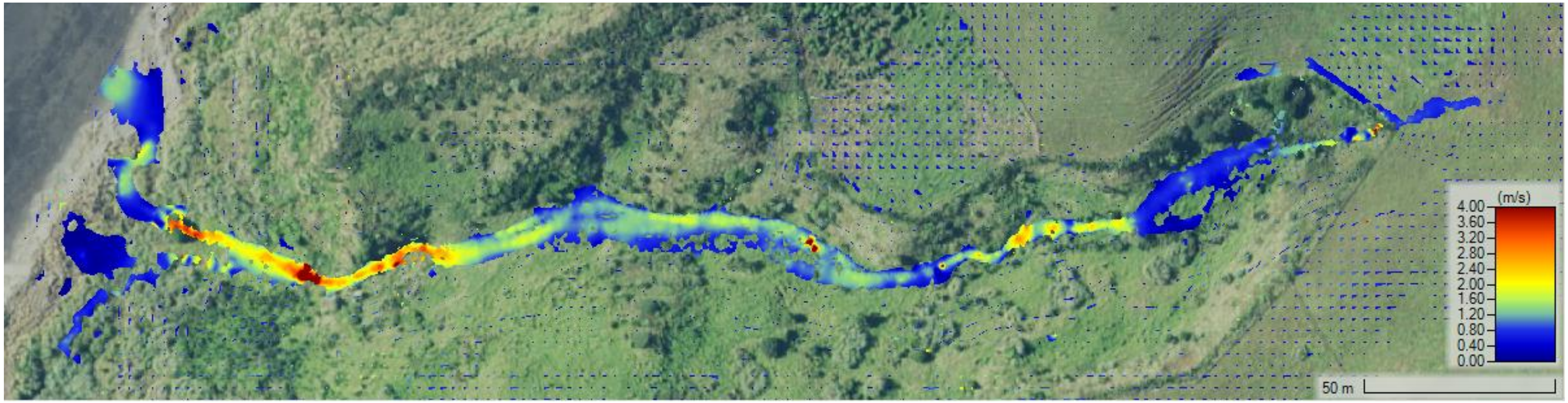
Maximum Flow Depth



Maximum Flow Velocity

Run 9 – 100yr+2050CC

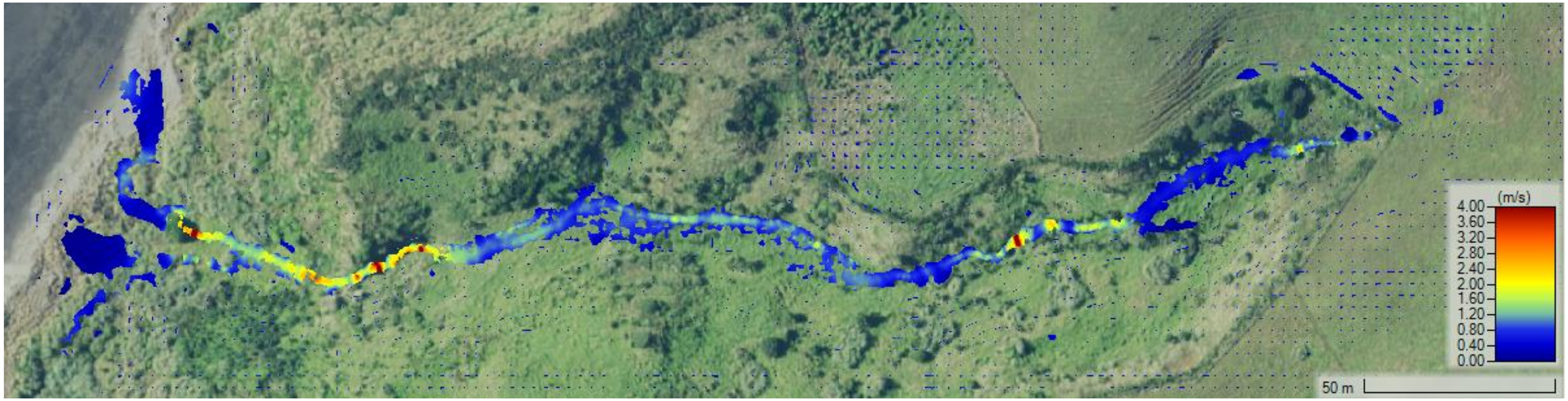
Maximum Flow Depth



Maximum Flow Velocity

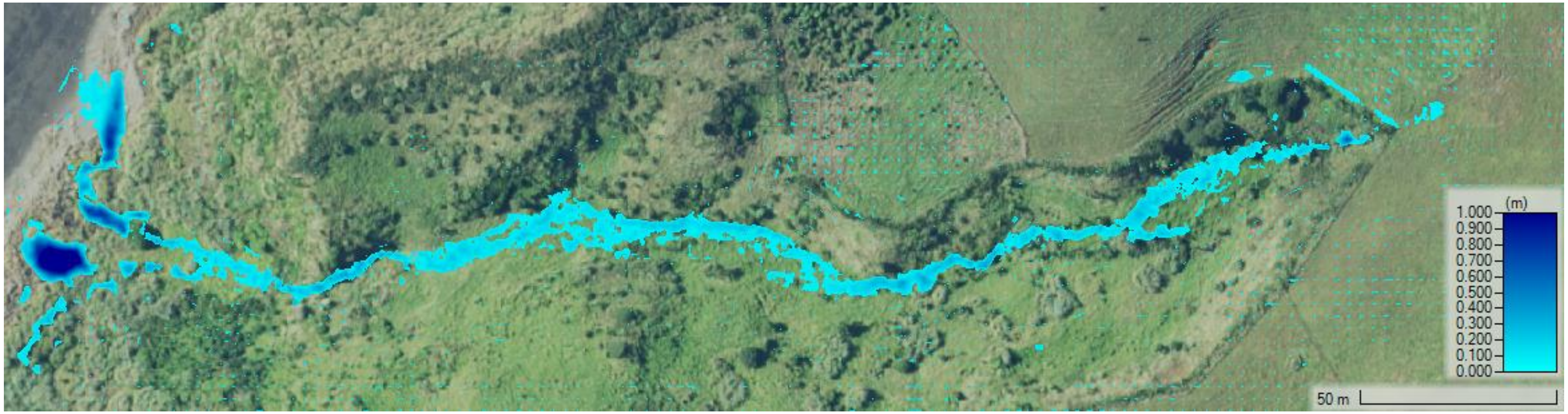
Run 10 – 5yr&45L/s

Maximum Flow Depth

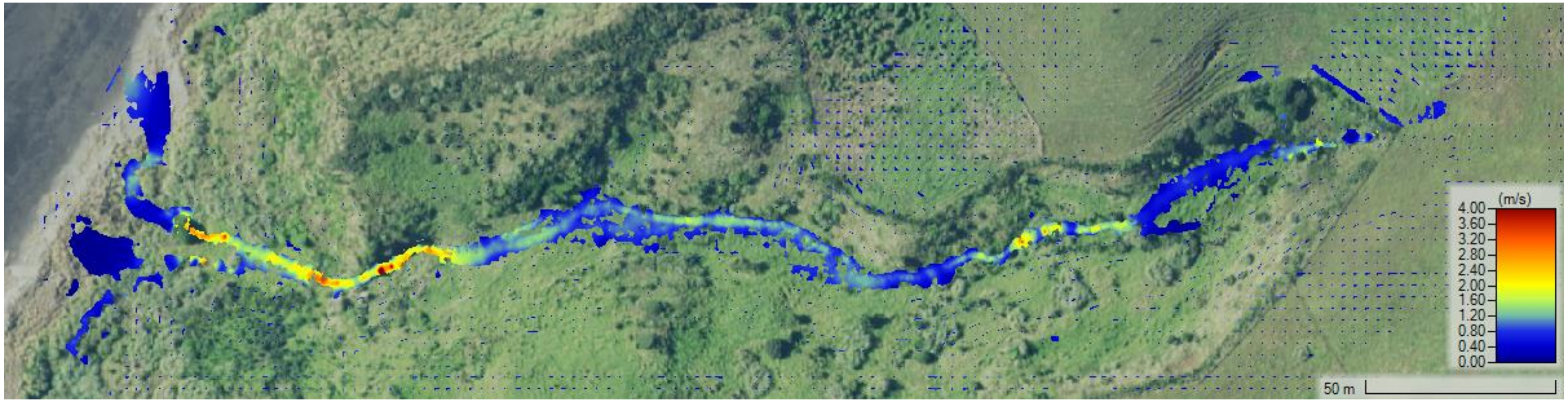


Maximum Flow Velocity

Run 11 – 10yr&45L/s

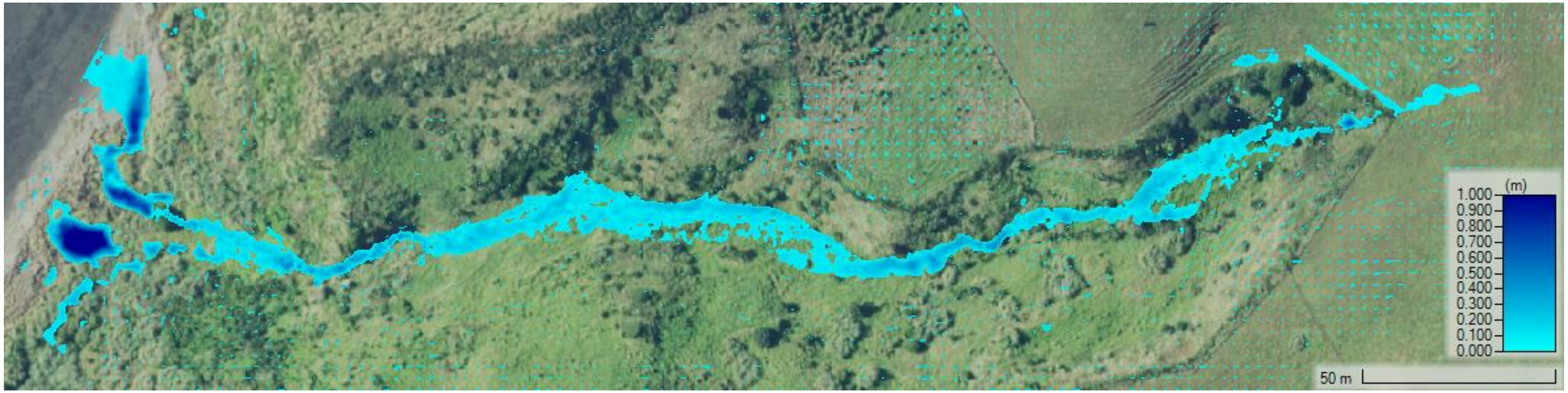


Maximum Flow Depth

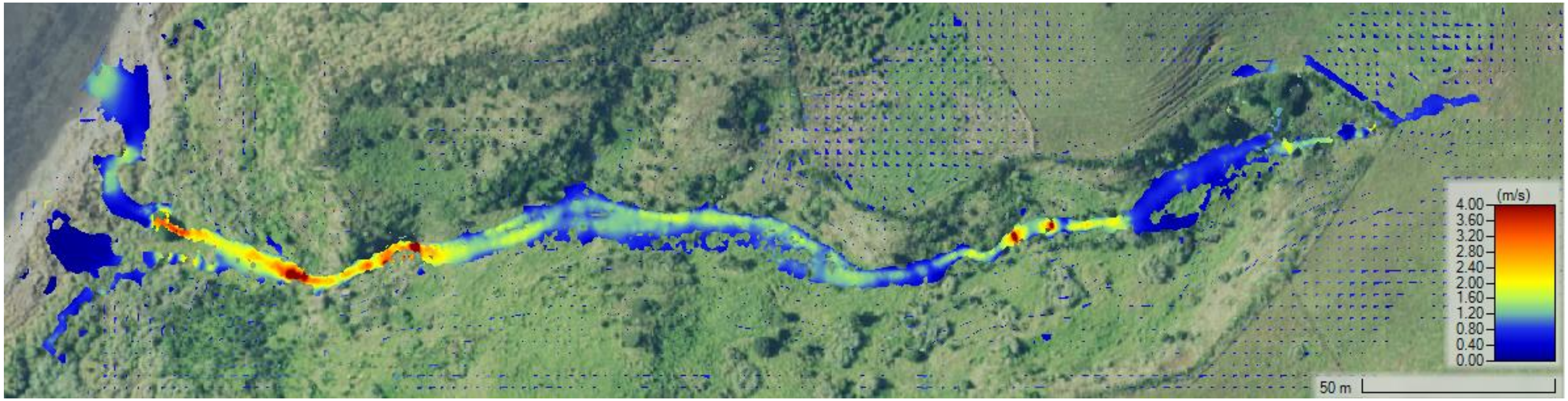


Maximum Flow Velocity

Run 12 – 100yr&45L/s



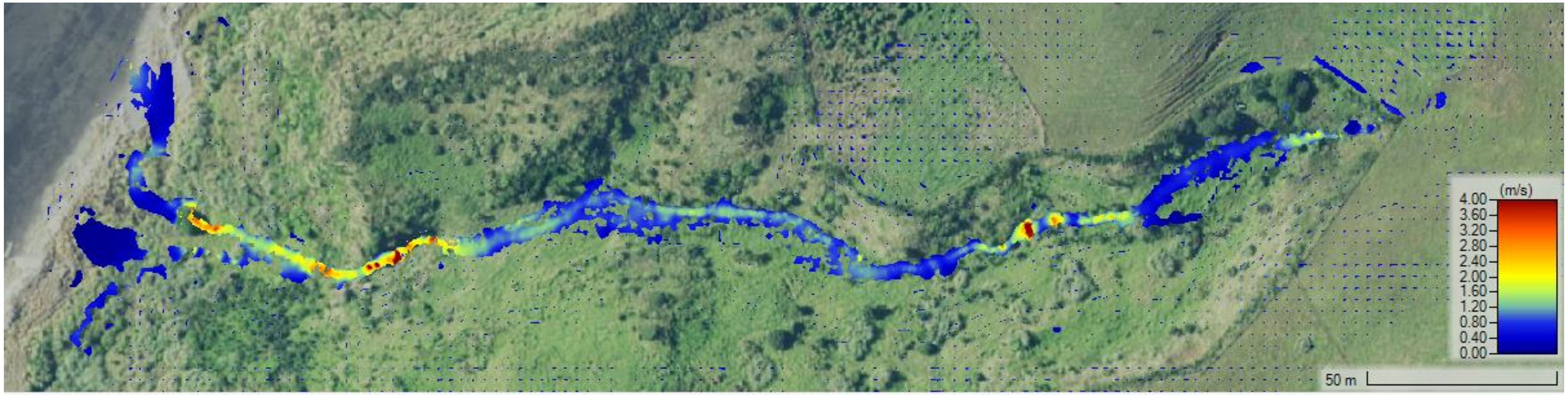
Maximum Flow Depth



Maximum Flow Velocity

Run 13 – 5yr&70L/s+2030CC

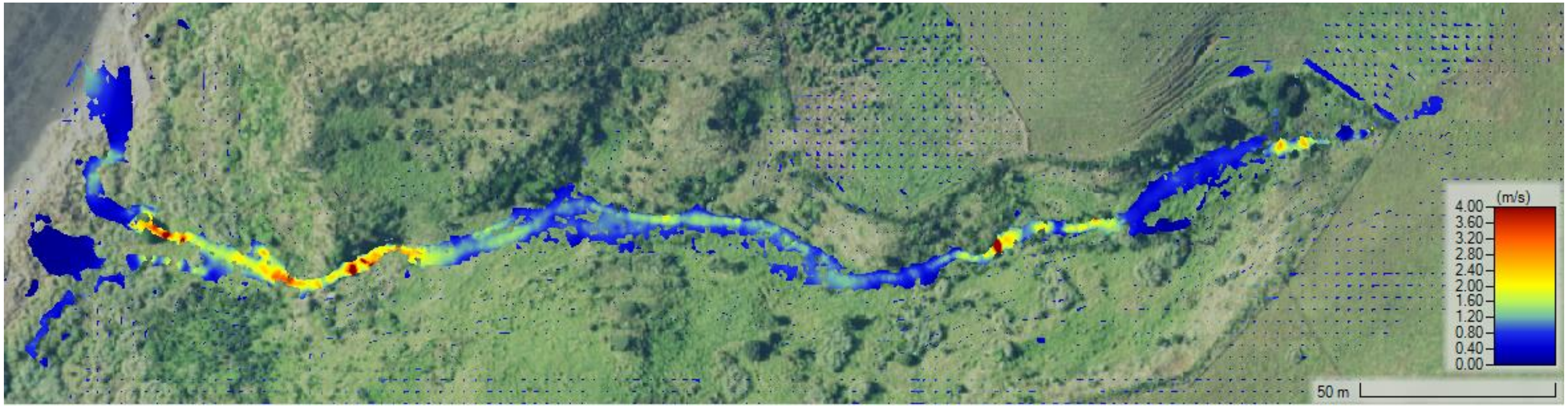
Maximum Flow Depth



Maximum Flow Velocity

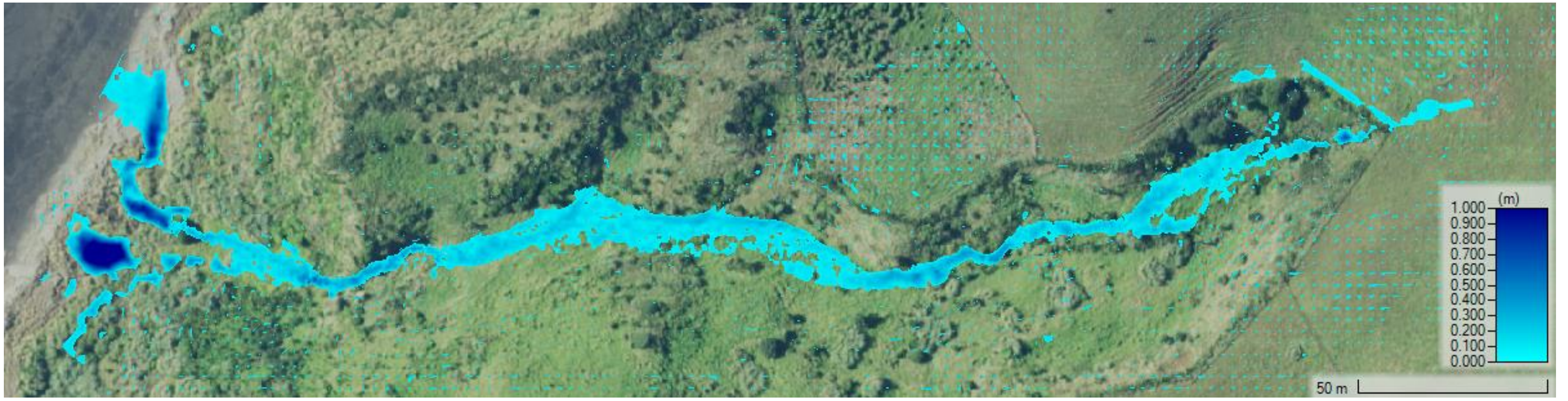
Run 14 – 10yr&70L/s +2030CC

Maximum Flow Depth

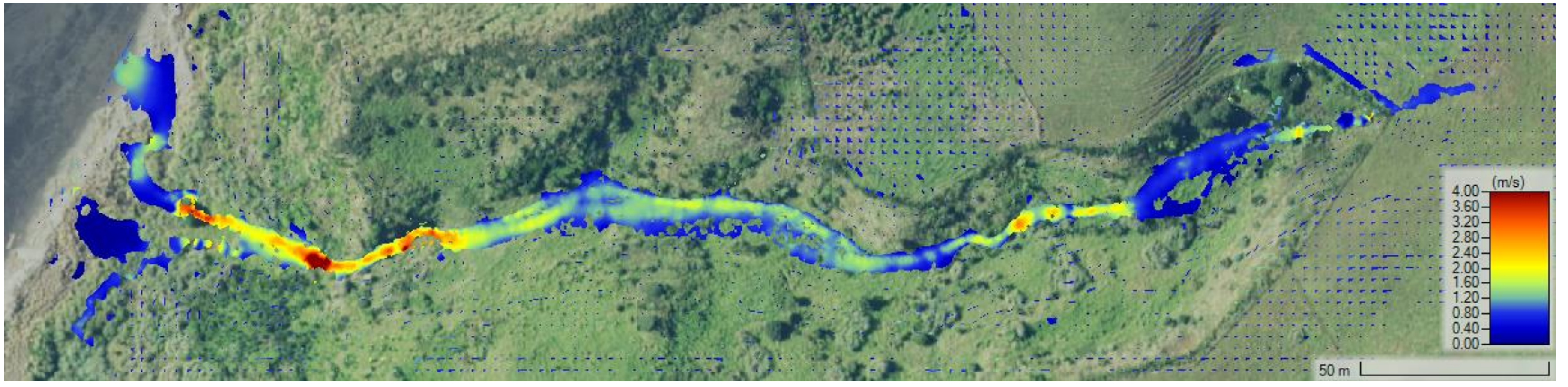


Maximum Flow Velocity

Run 15 – 100yr&70L/s +2030CC

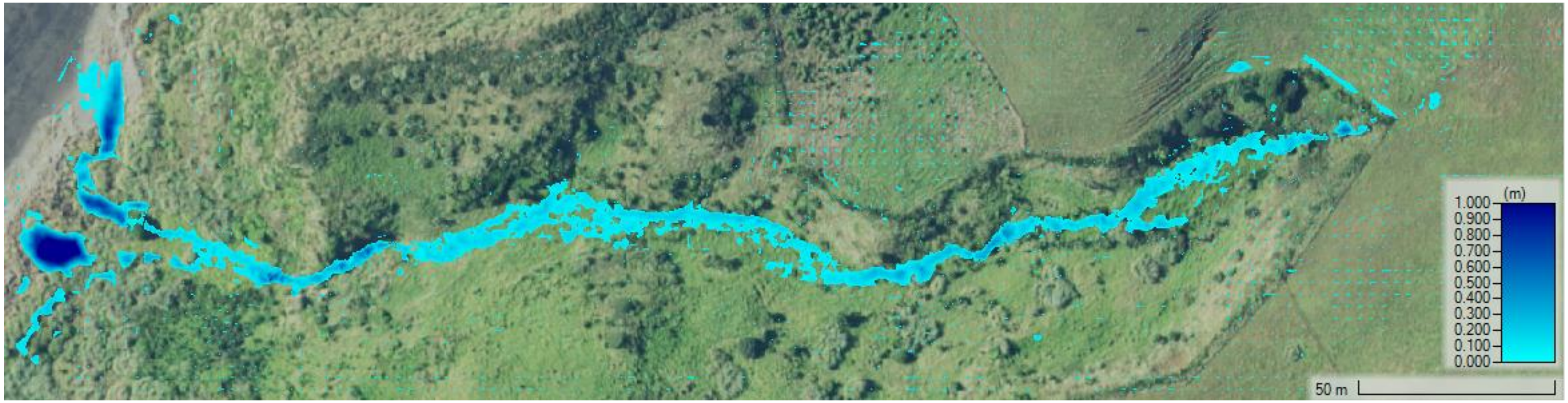


Maximum Flow Depth

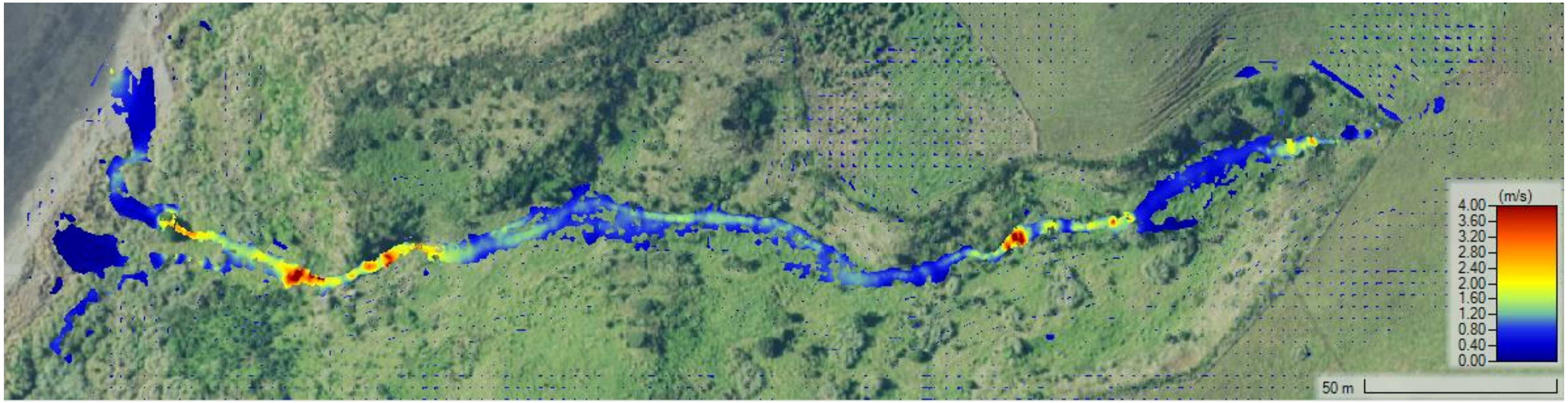


Maximum Flow Velocity

Run 16 – 5yr&90L/s+2050CC

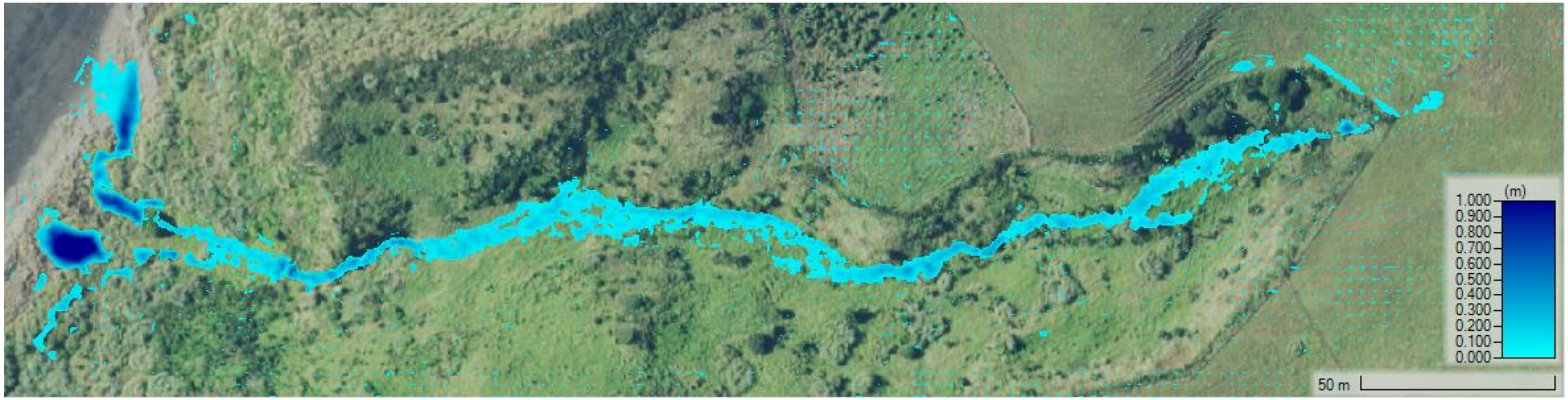


Maximum Flow Depth

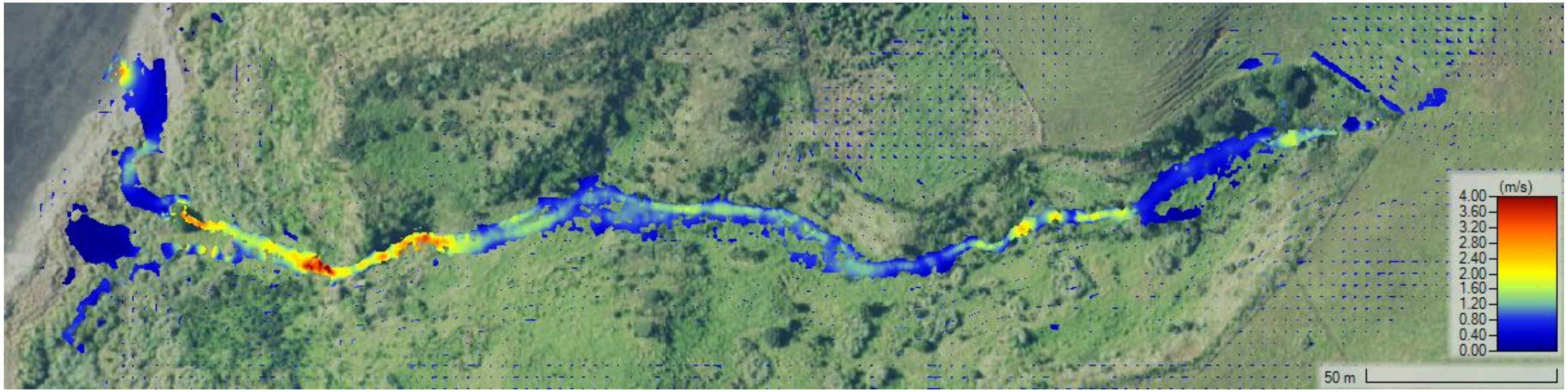


Maximum Flow Velocity

Run 17 – 10yr&90L/s +2050CC



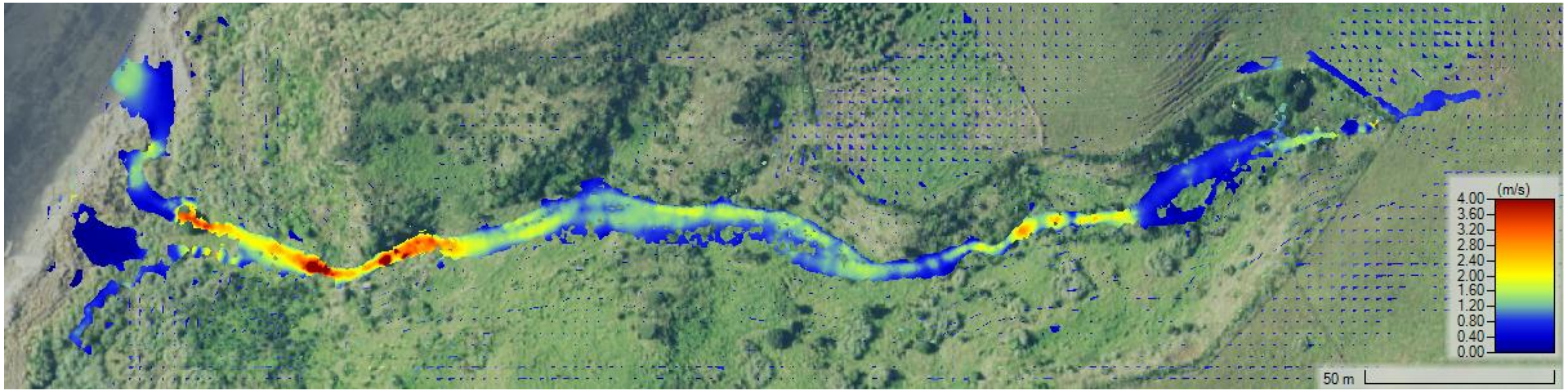
Maximum Flow Depth



Maximum Flow Velocity

Run 18 – 100yr&90L/s +2050CC

Maximum Flow Depth



Maximum Flow Velocity

Run 19 – WW 45L/s

Maximum Flow Depth

Maximum Flow Velocity

Run 20 – WW 70L/s

Maximum Flow Depth

Maximum Flow Velocity

Run 21 – WW 90L/s

Maximum Flow Depth

Maximum Flow Velocity