Appendix H

Flood Modelling Report
Part 1 of 2

Bell Road Limited Partnership

Wairakei South

Bell Road

Pāpāmoa

Flood Modelling Report

PROJECT INFORMATION

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1. Executive Summary

A flood modelling assessment was conducted for Bell Road Limited Partnership at the site at Bell Road, Pāpāmoa "Wairakei South". It is noted that this report should be read in conjunction with the corresponding Stormwater Management Plan, prepared by Maven (Appendix G of the AEE). Effort has been made not to repeat reporting already provided in the Stormwater Management Plan.

The purpose of this assessment was to evaluate flood risks associated with the Wairakei South development in order to determine the impacts of filling in the flood plain and how those effects were able to be avoided, remedied or mitigated effectively. The key 'design event' that the assessment focuses on is the 1 in 100 year, climate change adjusted storm. Other lesser and greater events have also been modelled and commentary is provided. A hydraulic model has been developed to quantify flooding extents and depths, as well as to support stormwater planning and engineering design outcomes for the site. The model has been applied to assess existing and future conditions, test the performance of proposed stormwater conveyance and attenuation areas, and assist in sizing new stormwater infrastructure to ensure appropriate levels of flood resilience across the development. This includes residential allotments, industrial land, stormwater corridors and management areas, commercial land, and a proposed school.

This report should be read in conjunction with the Stormwater Management Plan and Infrastructure Report (Appendix F of the AEE) prepared by Maven, along with the Hydrogeological Assessment Report (Appendix R of the AEE) prepared by ENGEO.

The study involved the development of hydrological and hydraulic models to assess flood risks under both pre-development and post-development scenarios for 5, 10, 50, 100, and 500-year ARI storm events, incorporating a current climate condition and 3.68°C climate change with 1.59m sea level adjustment in line with regional standards.

The post-development models incorporated significant terrain modifications, including the raising of road reserves and developable lots, realignment of existing culverts and swales, and the creation of new culverts and swales. These changes were designed to mitigate flood risks and ensure compliance with freeboard requirements for critical infrastructure and building platforms.

To determine groundwater effect, we have liaised with Geotechnical Engineers and Hydrogeologists (ENGEO) regarding groundwater inflow mechanisms and groundwater monitoring. ENGEO's Hydrogeological Assessment Report provides a comprehensive assessment of groundwater effects and has developed a detailed understanding of soil conditions and groundwater behaviour under both pre- and post-development scenarios.

While the report is not intended to directly inform hydraulic modelling, it has been reviewed to understand potential implications for surface water behaviour, such as infiltration assumptions, groundwater inflow locations or rates, or initial pond water levels that could be directly adopted within the HEC-RAS model. It is noted that in general, any changes to water inflow assumptions over time will be able to be managed by pump programming rather than changes to model and landform design.

The HEC-RAS model was verified against the previously approved BOPRC MIKE+ model. It is noted that differences exist in hydrologic inputs, boundary conditions, and event parameters between the two models. The pre-development scenarios produced comparable pre and post differences for the 100 year climate change event, indicating that the landform and pumping approach manages peak water level effects. Flow rates out of the catchment are consistent, with some flow rates within the catchment different due to some differences between how the models are constructed. This cross-verification provides confidence that the HEC-RAS results are robust for the broad assessment of the 100 year climate change scenario and that ongoing future work to develop the model for other current and future climate change scenarios will follow alongside the staged, detailed design process. Future work recommendations are set out in section 7 of this report.

Key findings from the assessment are presented below.

The site, pre-development has three (3) outflow points;

- Culvert flow beneath the Tauranga Eastern Link (TEL) towards the Bell Road Main Drain Pump Station
- Bell Road Pump Station A
- State Highway 2 underflow bridge in major events.

Within the existing drainage system there are three pump stations, as follows:

- Bell Road A Pump Station – Current capacity: 2.37m^3/s, Future Capacity 8.37 m^3/s
- Bell Road B Pump station – Current capacity: 0.57m^3/s
- Bell Road Main Drain Pump Station – Current capacity: 3.0m^3/s

Post-development scenarios showed a general increase in flood levels across all storm frequencies, with localised changes due to terrain modifications. This has been mitigated by adding a pump station with capacity of 12.8m^3/s in the South of the development that will feed into the Kopuaroa Canal and upgrading the Bell Road A Pump Station with an increased capacity of 8.37m^3/s (6m^3/s additional capacity).

The proposed pump stations will be provided with 24-hour backup electricity generation. Council has confirmed that the existing pump stations currently have no standby electricity. To test system resilience, the flood model was conservatively run assuming a complete loss of all pump stations for the duration of a 100-year storm event (3.68 °C climate change allowance plus 1.59 m sea level rise). Under this worst-case scenario, the predicted peak water level is RL 2.9 m. With an additional 500 mm freeboard (RL 3.4 m), flood levels remain below the proposed minimum development platform level of RL 3.5 m. Accordingly, the new development is not at risk of inundation, even during a total grid outage.

The post-development models demonstrated consistent flow patterns for the 100-year event and reduced the inundation time over the flood plain. Other lesser and greater events have also been modelled and commentary is provided.

Culvert upgrades and additions have been included in the post-development model to convey additional flow and minimise ponding caused by the new development.

Culverts to be upgraded in the future stages include;

- Bell Culvert 3 – Extend into the new Bell Road swale
- Bell Culvert 6 – Extend into the new Bell Road swale

A summary of the model results is shown in Section 4.4.3 - Flood Model Comparison.

The modelling was prepared in conjunction with a rolling peer review by Awa Environmental, along with inputs from BOPRC.

Overall, a comprehensive and conservative modelling approach has been undertaken to understand the current and future stormwater scenarios for the site and surrounding area, and the landform design and proposed asset upgrades respond to this to ensure the development is not at risk of flooding, and surrounding land is also not adversely impacted.

2. Introduction

2.1. Site Description

Refer to section 2.2 of Maven's Stormwater Management Plan (SMP) (Appendix G of the AEE) for details of the existing site and the proposed project.

2.2. Scope of Report

This assessment is undertaken for the Wairakei South development at Bell Road, Pāpāmoa to understand existing flood behaviour, assess potential development impacts, and confirm that the proposed development does not adversely affect surrounding land with respect to flooding. The assessment supports residential, commercial, industrial, stormwater corridor and school development planning.

The work includes reviewing existing information, including previous flood models, council data, ENGEO groundwater assessments, and stormwater design documentation. Hydrological and hydraulic models have been developed using HEC-RAS to represent both pre-development and post-development landform, infrastructure, culverts, swales and pump stations.

The modelling has been developed primarily to assess flood behaviour for the 100-year ARI design event including climate change and sea level rise allowances (3.68 °C temperature increase and 1.59 m SLR) in accordance with BOPRC guidance. Additional storm events including the 5-, 10-, 50- and 500-year ARI events have also been simulated as well as well as resilience testing under pump station failure scenarios to provide an indicative understanding of system performance across a range of storm events.

Model validation has been carried out by comparing results against the approved BOPRC MIKE+ flood model. Outputs include flood levels, extents, flow paths, velocities, and inundation duration. The outputs included vary with the scenario, the details are included in the flood risk assessment section. These results are presented either visually with figures of the model outputs or tabulated within Section 6 of this report.

The assessment reviews and confirm the effectiveness of proposed mitigation measures, including the new South Block pump station, Bell Road A pump upgrades, and culvert improvements. The report demonstrates that development levels and freeboard requirements are achieved and that downstream environments are not adversely affected.

The design of the development and associated modelling has been undertaken in order to determine flood risk and stormwater management to support a Fast Track Approval application of the development of the site. As further detailed design will be required as the development progresses over time, detailed modelling and validation will be an ongoing workstream over 10 plus years.

The 5-year ARI model results inform the evolving design rather than provide a definitive assessment at this stage. As noted in Awa's peer review, the 5-year event has not yet been fully validated against MIKE+, and some routing differences are present, including a tendency for excess flow to be directed south rather than toward the Bell Road Drain / TEL culvert. These differences are typical of early-stage modelling for more frequent events, which are highly sensitive to local channel geometry, smaller culverts, and grid resolution. Importantly, this assessment relates to a low-hazard storm event where risks to people and property are not anticipated. The focus is instead on drain-down performance and avoiding prolonged ponding that may affect agricultural productivity.

Ongoing detailed design will refine channel geometry, improve culvert representation, and optimise grid resolution. These refinements are not expected to materially alter the overall performance of the stormwater management approach. Draft conditions enable continued refinement as development progresses.

At a strategic level, no constraints have been identified that would prevent effective mitigation of 5-year storm effects. Preliminary first-principles assessment indicates that the proposed pump installations will enhance overall conveyance and reduce drawdown times for frequent events. Current limitations relate primarily to confidence in the distribution of flows, rather than the viability or performance of the proposed design.

Figures are included to provide spatial and visual context to the modelled scenarios, illustrate key elements of the model setup, and present supporting information relevant to the results. Not all figures are explicitly referenced in the text; however, they are included to provide completeness and transparency of the modelling approach and outputs.

2.3. Project

Wairakei South will be developed in stages to facilitate stormwater treatment, discharge and flood mitigation. A plan of the development is shown below, indicating proposed sequencing of infrastructure relating to stormwater.

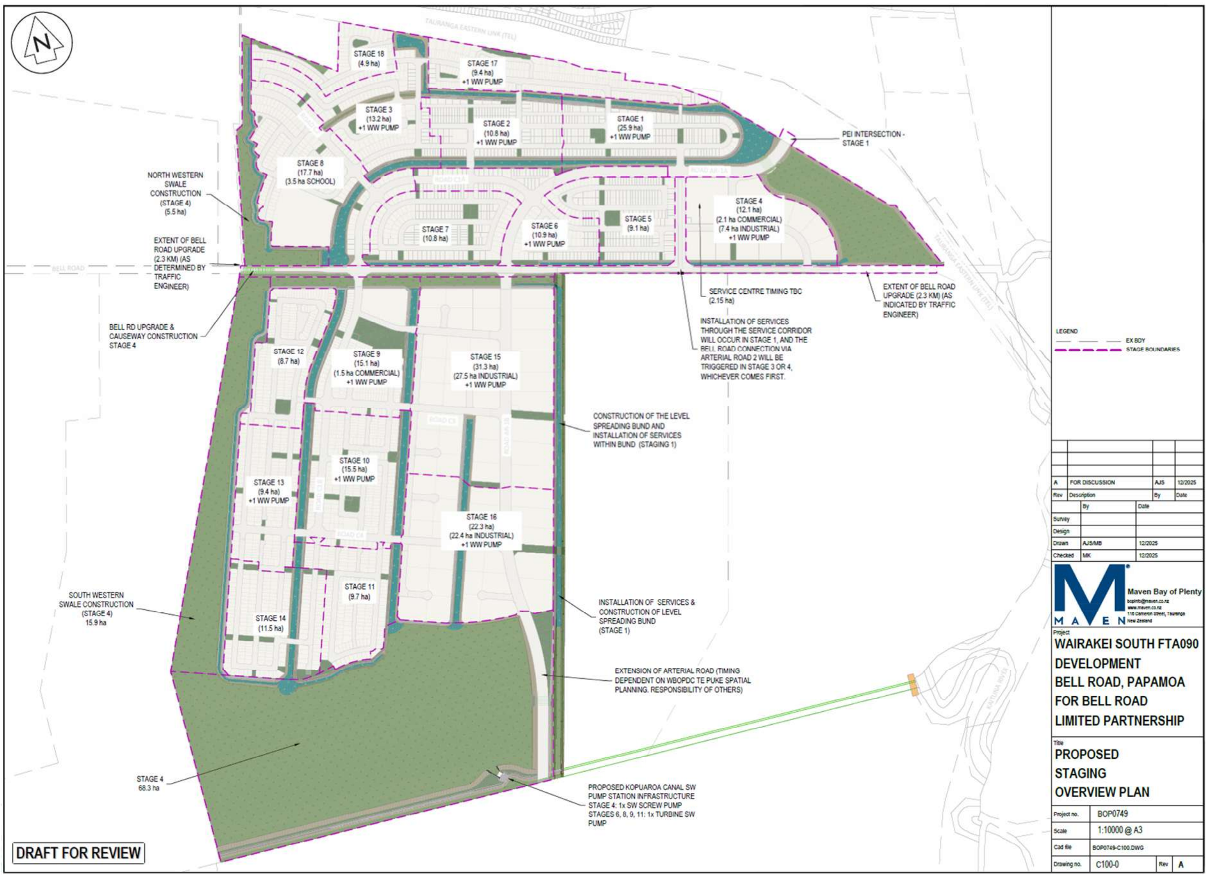

Figure 1 Project Staging (C150s – Appendix D of the AEE)

Staging of the project and a summary of the proposed stormwater management is set out in section 2.3-2.5 of the Maven SMP (Appendix G of the AEE).

3. Summary of Data Sources and Regulatory Context

3.1. Summary Of Data Sources

A summary of data sources can be found in section 3.1 of the SMP (Appendix G of the AEE).

3.2. Regulatory Context

Relevant regulatory context information can be found in section 3.2 of the SMP (Appendix G of the AEE).

4. Catchment Context

General catchment context information can be found in section 4 of the Maven SMP (Appendix G of the AEE). The section below comments on the catchment specifics that are pertinent to the construction of the stormwater models.

4.1. Terrain Data

Terrain data contains a representation of the landform shape. Existing LiDAR data for the region was sourced from LINZ Data Service. Maven has performed additional surveying within the site and in surrounding areas using a combination of equipment including GPS, total station, and drone LiDAR. Post-processing of data was applied to create the terrain used in this analysis. Further information such as site observation and study of the Bell Road catchment model has been used to determine site conditions and establish reasonable hydrological assumptions.

Terrain data is typically imported as a Digital Elevation Model (DEM) into the models. Topographic surveys are processed into DEM formats, then exported. If required, these may undergo further processing through Geographic Information System (GIS) software before being imported into the model. Design surfaces are created in engineering software such as Civil 3D or 12D, then converted to a DEM, which may undergo similar post-processing through GIS. Surfaces are merged chronologically where appropriate, with older data being overwritten with newer data in each spatial cell. DEM grids of different resolutions are resampled during reprojection to align all data to the same resolution and coordinate system. The DEM of 1m grid size was used for pre- and post-development. Minor errors will occur due to limitations in the spatial resolution and alignment differences between the DEMs, especially in sloped and non-uniform areas.

4.1.1. Pre-Development Terrain

Two terrain datasets were used for the pre-development models. The development of the pre-development terrain occurred over several months using a combination of publicly available DEM data, Maven drone-topographical survey data, Bell Road catchment model from BOPRC, site observations and catchment study findings.

Results from this analysis were used to refine the catchment boundaries for direct rainfall runoff and upstream catchment boundary conditions, as well as 1D channel inflow conditions for rivers and channels. This enabled the detailed pre-development model to be built. The table below shows construction of the pre-development terrain (in order of subservience):Table 1 - Terrain Data sources

Name	Description	Organisation	Data Capture Date	Horizontal Resolution	Terrain reference height
Bay of Plenty-1m-dem-2021	Bay of Plenty LIDAR DEM from LINZ Data Service	LINZ	2021	1m	NZVD2016 (Vertical Datum)
Drone-Topo Survey 1m DEM.tiff	Maven/ALS Topographic DEM	Maven	2025	1m	
Culvert Export.dwg	Surveying of culverts	Maven	2025	N/A	

The DEMs were reprojected and merged into a combined surface with 1m resolution. The resultant surface was used as the base terrain for the combined hydrologic/hydraulic models.

Bases of the Kopuaroa Canal, Kaituna River, and the Bell Road Main Drain were adopted using the BOPRC model height.

4.1.2. Post-Development Terrain

The post-development terrain model incorporates significant changes to the site's topography, including:

- The raising of ground levels up to between RL3.5 – 5.0m, primarily for freeboard from flood levels and cover protection for underground services;
- The construction of conveyance channels labelled as 'greenways' and 'swales';
- The construction of stormwater wetland swales for treatment, conveyance, and attenuation;
- Causeway structures enabling conveyance across Bell Road and through new PEI link Road;
- Alteration to the Bell Road carriageway and road corridor.

These changes will significantly alter the flow characteristics of the site. This terrain has been used for the post-development model.

4.1.3. Terrain Quality

The accuracy of the topographic information was provided alongside the data, with the root mean square error (RMSE) and maximum error detailed as follows:

- Hardstand surfaces: RMSE of 10mm, maximum error of 30mm;
- Soft surfaces: RMSE of 50mm, maximum error of 120mm;
- Bathymetry data: Riverbed level data were unavailable and could not be incorporated into the terrain model.

A sharp boundary with an abrupt height change was observed at the interface of the merged surfaces; however, given its limited extent, it is expected to have minimal impact on the model results.

The post-development terrain model was created using a combination of breaklines, modifications, and approximate culvert inverts. This allows for the incorporation of the design changes into the terrain model, enabling iterations to accommodate repeat changes to the layout. Hydraulic structures including culverts and drainage features have been incorporated into the model where they are considered to materially influence floodplain behaviour and catchment-scale flow conveyance. At the scale of modelling adopted for this assessment, not all minor drainage infrastructure has been explicitly represented. As the detailed design of the development progresses post consent, this model will continue to be refined.

4.1.4. Catchment Delineation

The extent of the preliminary model was established using a high-level assessment through River Environment Classification (REC2), assisted by initial model runs. The direct rainfall method was selected for the Bell Road catchment hydrologic modelling due to the nature of the catchment. Boundary conditions were placed upstream (west and north of the site) by using the BORPC Model.

Catchment delineation is relevant for capturing the catchment extents for direct rainfall models, but clear delineation is often not required. The catchment area for the pre-development model was based on the preliminary model, with catchment boundaries checked against HEC-HMS with a 5m DEM surface.

This is similar to the SmartGrowth Strategy 2024 – 2074 for the overland flow path, Lake Rotorua and Lake Rotoiti were not part of this catchment, as they have their own flood control measurements.

The catchment considered for the Wairakei South model is shown as follows. This catchment is part of a much larger Kaituna River control scheme catchment, also shown below.

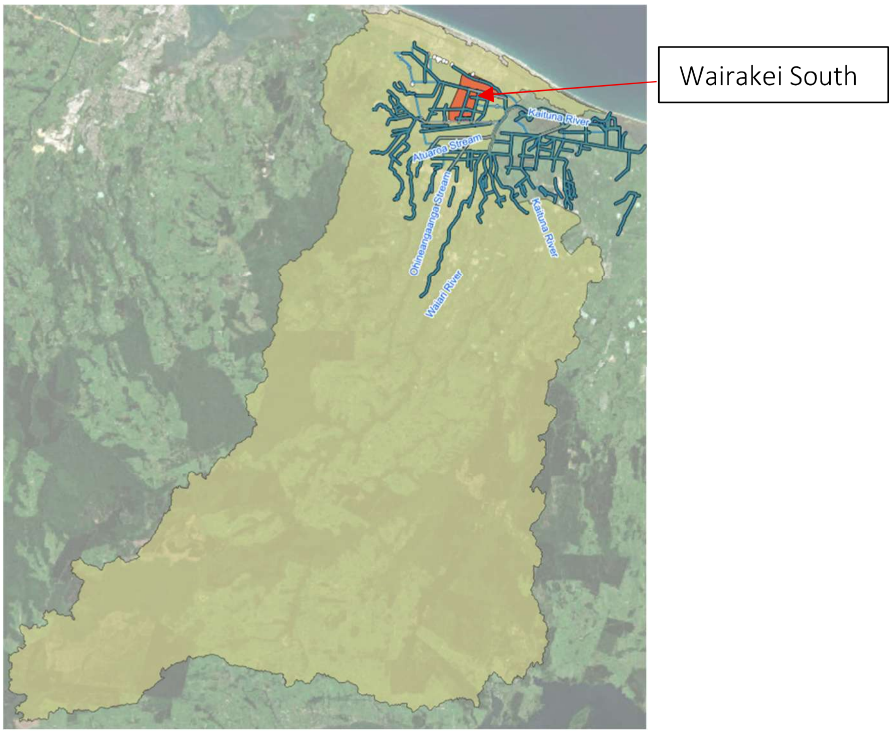

Figure 2: EC HMS Catchment (Source: QGIS combined with HEC-HMS)

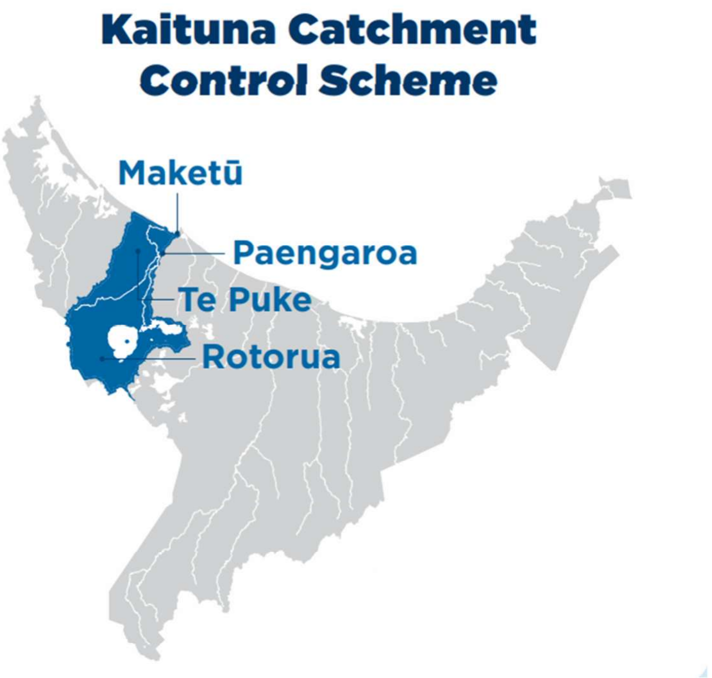

Figure 3: BOPRC Lower and Upper Kaituna River Catchment
(Source: BOPRC Rivers and Drainage Asset Management Plan 2024-2074)

4.2. Catchment Topography

To the south and west of State Highway 2 and Bell Road the landscape is an expansive and exceptionally flat agricultural plain. This low-lying area, which extends northwards beyond the Kopuaroa Canal, has almost no natural relief or undulation. Its topography is dominated by significant man-made features: the embankment of the highway on its eastern boundary, and the Kopuaroa Canal, which is incised into the level ground at the southern boundary, as well as Bell Road through the middle of the site and a series of drains and stop banks within the site. Aside from these structures, the entire region is a vast, uniform floodplain with a consistently flat profile, with elevations ranging from RL 0.55m to RL 2.20m across the development site.

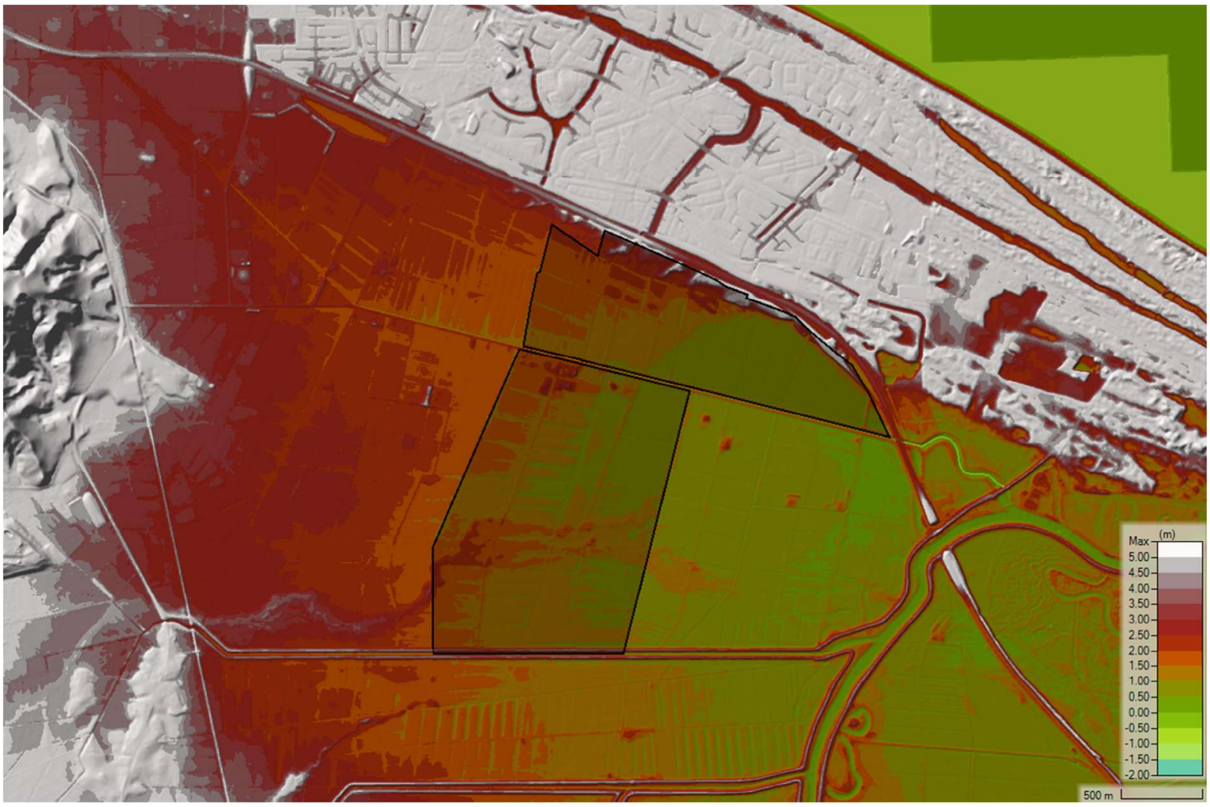

Figure 4: Bell Road Catchment Topography (0.5m colour gradation) (Source: HEC-RAS)

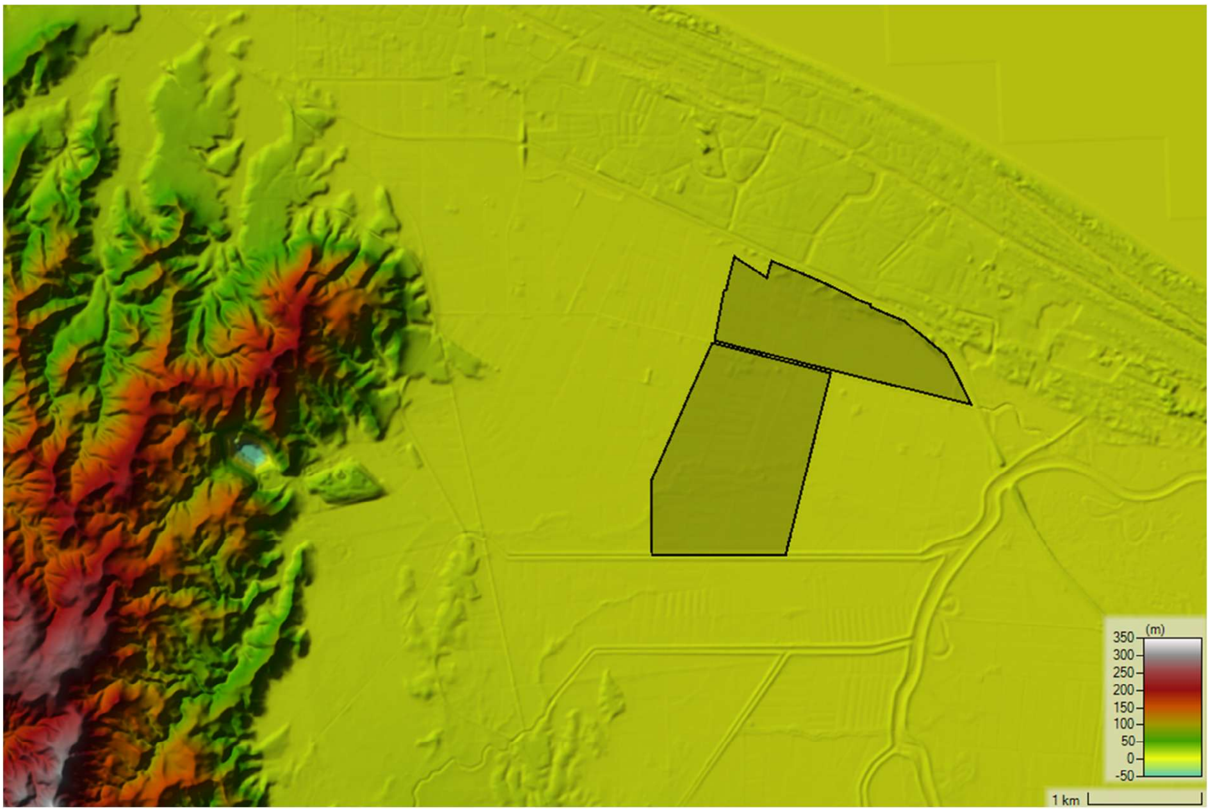

Figure 5: Wider Catchment Topography (50m colour gradation) (Source: HEC-RAS)

4.3. Catchment Description

This section should be read in context with section 4.1.2 of the Maven SMP (Appendix G of the AEE) which contains information on the various existing catchments that need to be considered.

The Bell Road drainage system is a critical component of the stormwater management strategy for the Pāpāmoa Catchment, particularly given the ongoing urban expansion in the Wairakei area and the construction of the Tauranga Eastern Link (TEL). The Bell Road Drain extends westward from Parton Road to the Kaituna River Oxbow, collecting water from numerous lateral drains.

Stormwater from the Wairakei North area's Western and Eastern Employment Zones flows into proposed treatment ponds (Ponds G and H) before being discharged into the Bell Road Drain. An important aspect of the drainage network is the Wairakei Stream diversion, which involves a proposed small pump station on the stream's banks to manage increased flows from residential development. This pump station is designed to pump up to 0.2 m^3/s of non-flood flow into Pond G, which then drains into the Bell Road Drain, aiming to restore the Wairakei Stream's water levels to their pre-development state, and notably, it is not intended for use during flood events. Consent conditions specify that stormwater management should prioritize ground soakage where practicable, especially for residential buildings, and encourage it for commercial ones.

Central to the flood protection in the Bell Road area is the Bell Road Pump Stations. The main Bell Road Pump Station is a crucial flood protection feature designed to manage discharge and water levels in the Bell Road Drain, especially during significant flood events. This station is proposed to house two x 1.5 cumec variable speed pumps, operating as a single unit, achieving a total pumping rate of up to 3.0

cubic metres per second. Its design capacity is to manage a 1 in 100-year 48-hour storm event (including climate change), ensuring that the 100-year flood level in the oxbow upstream of the pump station does not exceed RL 2.5m (Moturiki Datum). The Bell Road No.1 Drain discharges to the Kaituna River via twin gravity-operated $4m^2$ flap gates that close when the Kaituna River's water level is higher than the drain. The pump station is expected to operate approximately 2 to 3 times per year to lower water levels following heavy rainfall, ceasing when levels fall below RL-0.2m (Moturiki Datum).

The Kopuaroa Canal is a man-made arterial drain that flows eastward, collecting vast quantities of water. Its primary function is to intercept upstream runoff and convey it efficiently towards the Kaituna River, which is the ultimate discharge point for the entire area.

This leads to the most critical aspect: public flood protection. The land in this catchment is extremely low, in many places sitting at or even below the high tide level of the Kaituna River. Consequently, gravity alone cannot be relied upon to drain the land, especially during heavy rain or flood events in the river.

To overcome this, the catchment is protected by several key pieces of public infrastructure, managed as part of the wider Kaituna Catchment Control Scheme:

Bell Road Pump Station: Located at the end of the Bell Road Drain before it meets the Kaituna River, this is the most vital component. This large pump station actively lifts large volumes of water from the drain and pumps it up into the river. It operates when the river level is too high for the canal to drain by gravity, effectively forcing the catchment to drain and preventing widespread flooding.

Other smaller capacity pumpstations including Bell Road A Drain Pumpstation and Kopuaroa pumpstation, provide additional flood relief for the Bell Road catchment and land south of the Kopuaroa canal.

Stopbanks: Extensive stopbanks line the banks of the Kaituna River and Kopuaroa Canal in this area, as well as a series of minor stopbanks within the catchment, and stopbanks either side of the Bell Road Drain. These stopbanks contain flows within these natural and artificial watercourses providing protection to the farmland in the catchment.

Floodgates: Floodgates form part of pump station infrastructure. These automatically close to provide a physical barrier that stops the high water level of the Kaituna River and Kopuaroa from flowing backwards into the Bell Road catchment.

In summary, the existing Bell Road Stormwater Catchment is a highly engineered system where the public flood protection scheme, centred on the Bell Road Pump Station (and smaller supporting pumpstations), supported by stopbanks and floodgates, is essential for actively managing water levels and protecting thousands of homes and valuable farmland from flooding.

Below is a summary of the distinct drainage and flood protection features within and bordering the Bell Road catchment.

4.4. Site Geology

4.4.1. Existing Geology

ENGEO Limited has undertaken a Geotechnical Factual Report (Appendix O of the AEE), Geotechnical Interpretive Report (Appendix P of the AEE), Natural Hazards Assessment Report (Appendix Q of the

AEE), and a Hydrogeological Assessment Report **(Appendix R of the AEE)**. A summary of the relevant findings and geology of the site is provided below.

The site is located within the Maketu Basin, in a back-dune, swampy environment that forms part of the Kaituna floodplain. Its geology is the result of recent (Holocene and late Pleistocene) coastal and alluvial processes, creating a landscape of coastal dunes and swampy deposits. The northern boundary of the site marks a transition from the alluvial floodplain to coastal dune sands.

The ground profile consists of several distinct layers. A thin layer of silty topsoil (0.3 to 0.4 m thick) covers the surface. Beneath this lies a significant deposit of soft, fibrous peat containing organic clays and wood, which reaches a maximum thickness of 4 metres towards the southeast and pinches out to the north. Underlying the peat and topsoil is a 5 to 8-metre-thick layer of fine to medium dune sand. Deeper still are two fluvial (river) deposits, starting with a layer of loose sands and gravels, followed by a much denser layer of sands and gravels below 13 metres depth.

4.4.2. Existing Groundwater

ENGEO's Hydrogeological Assessment Report is focused on groundwater effects assessment rather than hydraulic flood modelling. Their report provides a comprehensive understanding of subsurface conditions, groundwater behaviour, and relative changes between pre- and post-development states. Overall, the new development will not have an adverse effect on the development or neighbouring lots in terms of groundwater.

Infiltration parameters were removed from the flood model due to the high groundwater levels across the site. With groundwater already close to the surface, any additional infiltration is minimal and therefore including it would not significantly affect model results.

Initial pond, swale and stream levels were taken from the current data.

Council has provided water level data for Pond H, which can be used as a reference for the current groundwater conditions in the surrounding area.

The dataset covers the period from May 2022 to December 2025 with 5-minute interval recordings. All levels are referenced to NZVD2016, with a recorded maximum of RL 2.35 m, a minimum of RL 1.81 m, and an average level of RL 1.87 m. For the model we adopted a water level of RL2.0m, for any future impact.

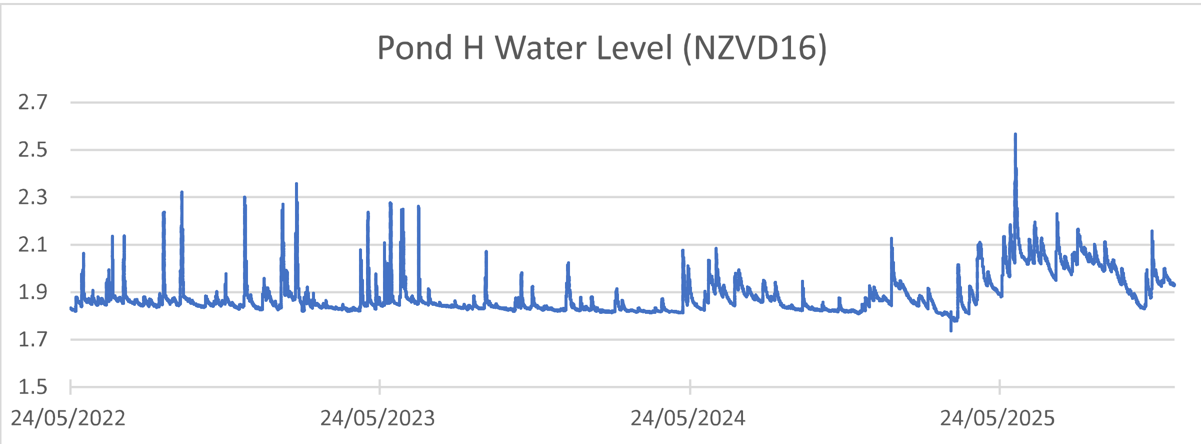

Figure 6: Pond H Water Level (Source: BOPRC)

The Maven adopted conservative and clearly documented assumptions for groundwater inflows and initial conditions, are informed by ENGEO's findings in the Hydrogeological Assessment Report. The approach conservatively aligns with the magnitude and direction of effects (for more information see section 5.3.10). These assumptions were applied consistently across pre- and post-development scenarios.

4.5. Existing Hydraulic Structures

The site is covered by a series of land drains, culverts, flood protection and controls, however there is no council urban reticulation network available to the site.

The Kaituna River floodplain features extensive stop banks. The Bell Road catchment comprises a network of artificial channels equipped with culverts and flap gates, draining into the main Bell Road Drain, which serves as the primary conveyance for the catchment. The southern boundary of the site adjoins the Kopuaroa Canal, which collects flow from the Kopuaroa catchment.

Below is a list of the key stormwater conveyance and control features:

- Bell Road Drain
- Bell Road Drain A
- Bell Road Drain Pumpstation – 3.00m^3/s capacity
- Bell Road Drain A Pumpstation – 2.37m^3/s capacity
- Bell Road B Pumpstation – 0.57m^3/s capacity
- Kopuaroa pumpstation – 1.05m^3/s capacity (south of the site, dewater Armer Farm (Part Lot 2 DP 32292)
- Bell Road Drain SH2 Box culvert crossing
- Bell Road Drain Pumpstation Floodgate culverts into the Kaituna River
- Bell Road culvert crossings
- Culverts with flapgate outlets into the Kopuaroa canal
- Pond G culvert into north-east corner of the site
- Pond H culverts at the northern boundary – west of the development site
- Kaituna River stop banks
- Kopuaroa canal stop banks
- Bell Road Drain stop banks

5. Flood Model Build and Validation

5.1. Modelling Methodology

The direct rainfall method was chosen for the Bell Road catchment because the initial model exhibited flow patterns which lack distinct and defined flow paths, known as diffuse overland flow. Direct rainfall generates a more accurate representation of how rainfall originates, disperses and infiltrates across the model area, particularly in regions with gentle slopes where water does not concentrate into specific channels. This approach combines the hydrologic and hydraulic components into an integrated model. The rainfall intensity was obtained from the existing flood model that the BOPRC provided to Maven.

Upstream catchments including flows from the Kaituna River catchment and Pāpāmoa (Wairakei north) have been modelled as boundary conditions, with inflow hydrographs representing the flows from these catchments against a time-date series. These inflow boundary conditions have been obtained from the BOPRC model.

Hydrologic modelling is the process of creating a simplified representation of real-world hydrological systems. It uses inputs like rainfall, soil properties, and land cover to estimate the runoff generated in a rainfall event and resultant surface flows at predetermined points. Input parameters are abstracted from real-world processes and captures processes such as rainfall, evapotranspiration, and infiltration.

Hydraulic modelling simulates the behaviour of fluid flow and captures more localised effects, such as the flow behaviour around hydraulic structures. Effects such as fluid acceleration, wave propagation and tailwater conditions are considered. Results produced commonly include water level, velocity, and flow direction.

Catchment delineation is affected by the spatial resolution and construction of the computational grid to provide accurate runoff. Additionally, model run times are noticeably longer than a similar hydraulic model with direct inflows. However, this approach is required to capture the flow behaviour observed in this catchment and predict flood levels with more accuracy.

For steep catchments such as the Pāpāmoa Hills, runoff was modelled in HEC-HMS using the SCS Curve Number method for rainfall losses and the Clark Unit Hydrograph method for runoff transformation.

These hydrographs were then applied as inflow boundary conditions to reduce potential uncertainty associated with rain-on-grid modelling over steep terrain. A rain-on-grid model was also developed. In areas where culvert information was not available, 450 mm culverts were assumed based on catchment size and expected discharge. Results from the rain-on-grid model were compared against the HEC-HMS hydrology outputs as a sensitivity check.

The Bell Road main drain culverts can still be utilised under the 2130 climate-adjusted design scenario (3.68°C warming and 1.59 m sea level rise). At the end of the event, upstream water levels are expected to rise slightly above the downstream level, allowing floodgates to open and release approximately 400,000 m^3 of water by gravity, reducing the volume requiring pumping. This gravity discharge would only occur once levels have equalised at the low point. Any storm event below the 100-year floodgates will not have any effect on the upstream floodwater. For the 500-year event, these floodgates will provide a major release of upstream floodwater after the storm event, releasing approximately 3.06 million m^3 of water.

5.2. Hydrological Parameters

5.2.1.Overview

The BOPRC modelling parameters required hydrologic modelling of the 5-, 10-, 50-, 100-, and 500-year ARI events under current rainfall conditions, as well as with a 3.68°C climate change adjustment and 1.59 m of sea level rise. The storm event was simulated in MIKE+ using a rainfall dataset with a 30-minute time-step over a 7-day period. In MIKE+, a spatially varying rainfall distribution was applied, whereas HEC-RAS utilised a uniform rainfall input. The design storm was represented as a centrally nested storm profile based on a HIRDS v3 rainfall hyetograph as per previous BOPRC flood model reports, reflecting an event comparable to that represented in the MIKE+ model.

Adopting a 72-hour heavy-ended (75%) storm profile, as required by the guidelines, would prevent the direct use of the MIKE+ culvert inflow hydrographs in the northern and northwestern areas. As a result, this approach would reduce the ability to undertake a direct comparison between the two models. The resulting difference in rainfall volume between the two design storms discussed further below under "Rainfall Data".

The model duration was set to 87 hours to ensure that any peak inflows from upstream catchments were fully captured within the eastern inundation area of the Hurst block, which is located immediately southeast of the site. The simulation was extended for a further 65 hours beyond the end of the storm event, during which only minor inflows continued. This extension was included to simulate the likely duration of inundation. The level of service adopted for assessment was the 5-year ARI event, as confirmed by BOPRC. Peak discharge occurred approximately 1 to 2 hours after the peak rainfall intensity. However, hydrograph recession continued beyond the initial 24-hour period. Consequently, the extended simulation duration allowed the model to capture the full hydraulic response of the catchment.

5.2.2.Model Parameters

All models assume a climate change adjustment of 3.68°C as required by BOPRC and include sea level rise of 1.59m (to NZVD16).

The rainfall depth was obtained from the BOPRC model and was checked using Table 6 in the HIRDS Version 4 Technical Report Section 4.4 Augmentation factors for HIRDS. A screenshot of the table is shown below. As there is no data available for events exceeding the 100-year storm, the climate change adjustment for the 500-year storm is based on the same change factor: an increase of 13.6%/°C of warming.

DURATION/ARI	2 YR	5 YR	10 YR	20 YR	30 YR	40 YR	50 YR	60 YR	80 YR	100 YR
1 HOUR	12.2	12.8	13.1	13.3	13.4	13.4	13.5	13.5	13.6	13.6
2 HOURS	11.7	12.3	12.6	12.8	12.9	12.9	13.0	13.0	13.1	13.1
6 HOURS	9.8	10.5	10.8	11.1	11.2	11.3	11.3	11.4	11.4	11.5
12 HOURS	8.5	9.2	9.5	9.7	9.8	9.9	9.9	10.0	10.0	10.1
24 HOURS	7.2	7.8	8.1	8.2	8.3	8.4	8.4	8.5	8.5	8.6
48 HOURS	6.1	6.7	7.0	7.2	7.3	7.3	7.4	7.4	7.5	7.5
72 HOURS	5.5	6.2	6.5	6.6	6.7	6.8	6.8	6.9	6.9	6.9
96 HOURS	5.1	5.7	6.0	6.2	6.3	6.3	6.4	6.4	6.4	6.5
120 HOURS	4.8	5.4	5.7	5.8	5.9	6.0	6.0	6.0	6.1	6.1

Figure 7: HIRDSv4 Climate (Source: NIWA, HIRDS v4)

As there were some differences in the rainfall data calculated and the data within the BOPRC Bell Road model, Maven opted to using BOPRC's model data to better align with the inputs of the BOPRC model and generally because it produced greater rainfall depths, which would produce more conservative runoff estimation.

The rainfall depths extracted from the BOPRC model used for hydrological modelling are shown below:

Table 2 - Rainfall Depth

Rainfall Event (7-day duration – BOPRC Model)	Rainfall Depth (mm)
5yr ARI current climate	141.81
10yr ARI current climate	283.28
50yr ARI current climate	415.49
100yr ARI current climate	488.53
500yr ARI current climate	710.21
5yr ARI + 3.68°C yr 2130	155.23
10yr ARI + 3.68°C yr 2130	344.82
50yr ARI + 3.68°C yr 2130	537.78
100yr ARI + 3.68°C yr 2130	632.34
500yr ARI + 3.68°C yr 2130	919.28

Only the Bell Road catchment was modelled using direct rainfall. All external contributing catchments were represented as inflow boundary conditions, sourced from the BOPRC model. The hydrologic modelling used to generate the Papamoa Hill Inflow adopted the SCS Curve Number methodology to estimate runoff. Land cover was classified as *"Pasture, grassland, or range – continuous forage for grazing" in fair condition*, consistent with Table 9-1 of USDA NEH Part 630, Chapter 9. Hydrologic Soil Group (HSG) C was confirmed across the catchment by ENGEO based on site geological assessments.

Given the consistently high groundwater levels, no initial abstraction, infiltration, or continuous losses were applied in the model. This treatment was reviewed and confirmed with ENGEO during the groundwater assessment. As such, any previously considered developed area CN assumptions fall away, as groundwater conditions remove the feasibility of assigning typical urban infiltration loss parameters.

Model parameters were initially informed by the BOPRC model and further refined through selective calibration against BOPRC outputs. No developed land and maize cropping areas were incorporated, consistent with the BOPRC model inputs. Manning's n values for the Kaituna and Kopuaroa channels were derived using a 1D calibration model based on BOPRC observed data at key locations and subsequently applied within the 2D model for comparative assessment. The Raparapahoe and Ohineangaanga waterways were assigned a global Manning's n value of 0.04, as their primary function in the model is to convey inflows to the Kaituna system.

Given the early stage of development, the modelling approach was intentionally focused on understanding overall flood behaviour across the site, rather than detailed assessment of localised, stage-based urban flooding processes.

Table 3 - Manning's n Values

Item	Curve Number	Abstraction Ratio	Manning's N
Non-Developed area	74 (N/A)*	0	0.04
Maize Field	74 (N/A)*	0	0.125
Kaituna and Kopuaroa channel	74 (N/A)*	0	0.02
Development area	90.8 (N/A)*	0	0.05***
Bell Road main drain	74 (N/A)*	0	0.04**
Swales and ponding area	74 (N/A)*	0	0.04**

- *ENGEO's Hydrogeological Assessment Report (Appendix R of the AEE) report states that no infiltration is allowed, default to N/A
- **Sensitivity check was done up to 0.06
- ***Sensitivity check was done from 0.005 to 0.1 to find the Time of concentration close to 10 minutes to simulate pipe flow

No	Type of stream Channel and Description	Manning's n Roughness Values					
		Default		Lower		Upper	
		1D	2D	1D	2D	1D	2D
1	Flood plains – Coarse gravel	0.035	0.027	0.03	0.025	0.04	0.03
2	Flood plains – Stone/Gabion	0.045	0.035	0.035	0.03	0.05	0.04
3	Flood plains – Riprap	0.06	0.045	0.045	0.04	0.065	0.05
4	Flood plains – Sand	0.025	0.020	0.02	0.017	0.033	0.025
5	Flood plains – Concrete	0.025	0.020	0.02	0.018	0.03	0.022
6	Flood plains – Bare ploughed soil	0.03	0.025	0.025	0.022	0.035	0.028
7	Flood plains – Height varying grass	0.05	0.041	0.022	0.02	0.10	0.08
8	Flood plains – Light brush and trees (incl. flax)	0.065	0.06	0.045	0.04	0.10	0.08
9	Flood plains – Medium brush and trees, pine forest	0.11	0.10	0.08	0.07	0.20	0.16
10	Flood plains – Coniferous trees, native forests, mangroves	0.16	0.15	0.11	0.10	0.25	0.20

Figure 8: - Table A6-2: Manning's N Values (Source: Auckland Council: Stormwater Modelling Specification Dec 2023)

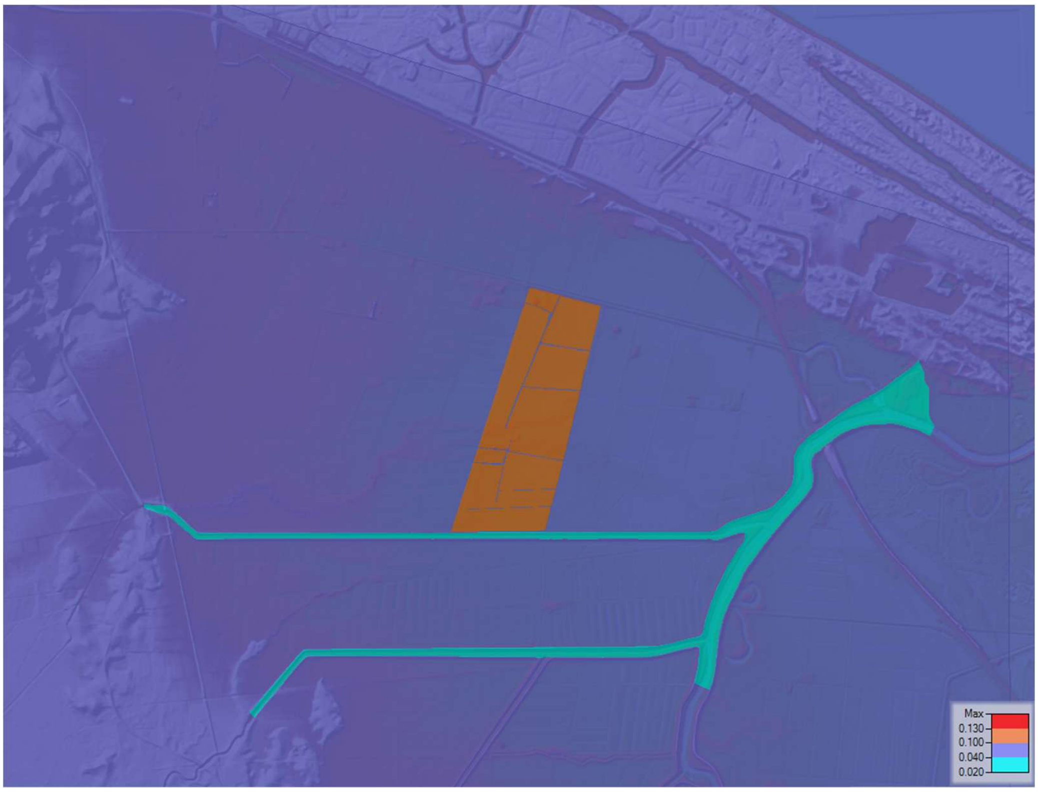

Figure 9 - Pre Development Manning’s N Values (Source: HEC-RAS)

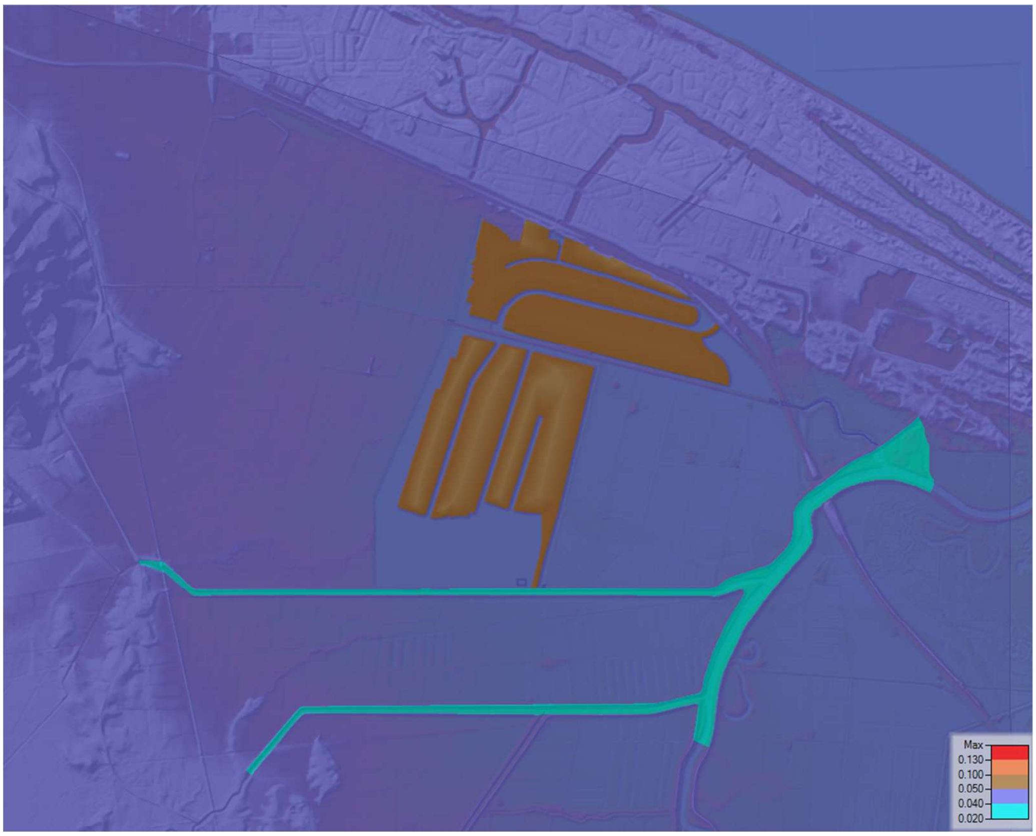

Figure 10 - Post Development Manning’s N Values (Source: HEC-RAS)

5.2.3.Rainfall Data

The design rainfall events were extracted from the BOPRC model which reporting describes the storm as a centrally nested storm profile based on a HIRDS v3 rainfall hyetograph, which was centrally nested to a 7 Day duration. This approach is consistent with the Bay of Plenty Regional Council (BOPRC) regional model, adopted BOPRC modelling parameters, and other flood modelling studies within the region.

For site-specific assessment, BOPRC has indicated the use of a 72h heavy-end nested storm profile (75%). To maintain consistency across models and inflow hydrographs, a 7-day storm duration was therefore adopted in the MIKE+ model, ensuring alignment of all contributing hydrographs.

This is included in the graph below showing the rainfall profile for the 100yr ARI 3.68°C 7 Day duration event:

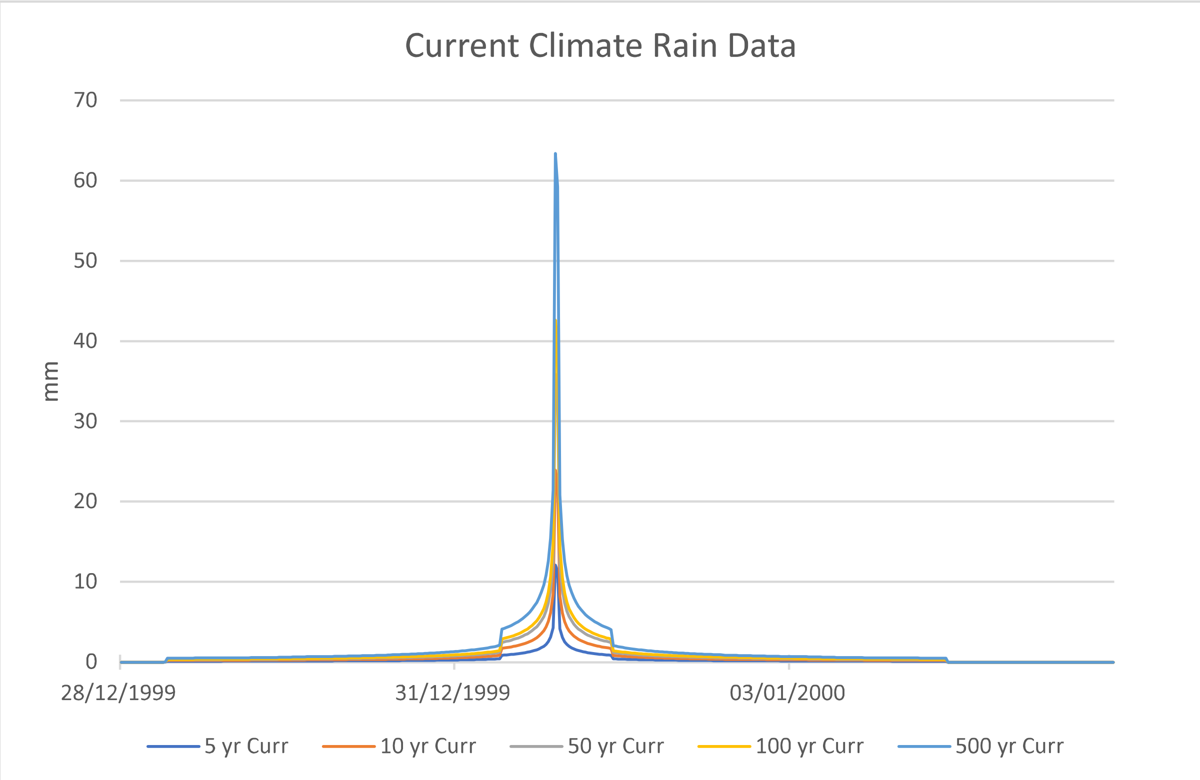

Figure 11 – Current Climate Rain Event (Source: MIKE+ model)

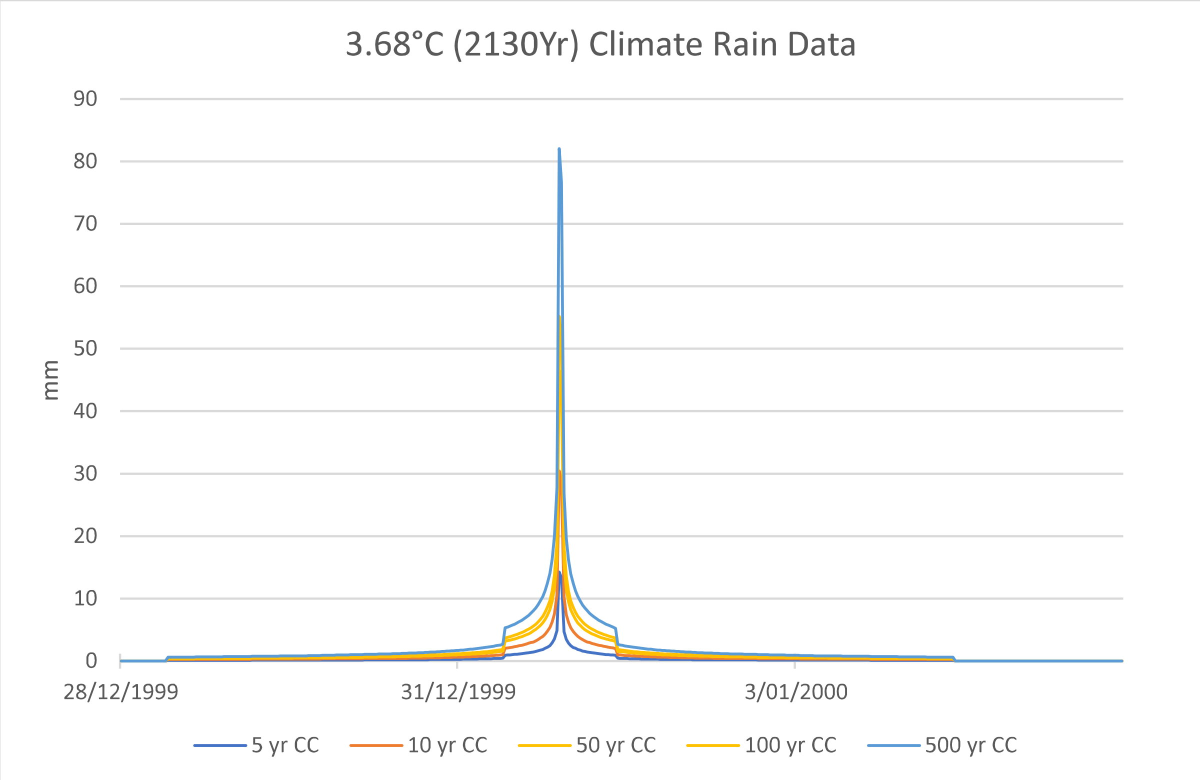

Figure 12 - Year 2130 + 3.68C Rain Event (Source: MIKE+ model)

For the 72-hour heavy-ended (75%) storm profile, the following parameters were used for the volume comparison between the 7day storm event and the 72h event.

The graph is based on a 10min time step.

Table 4 - Manning's n Values

Rainfall Event (72h duration)	Rainfall Depth (mm)
5yr ARI + 3.68°C yr 2130	219.84
10yr ARI + 3.68°C yr 2130	260.23
50yr ARI + 3.68°C yr 2130	328.81
100yr ARI + 3.68°C yr 2130	407.52

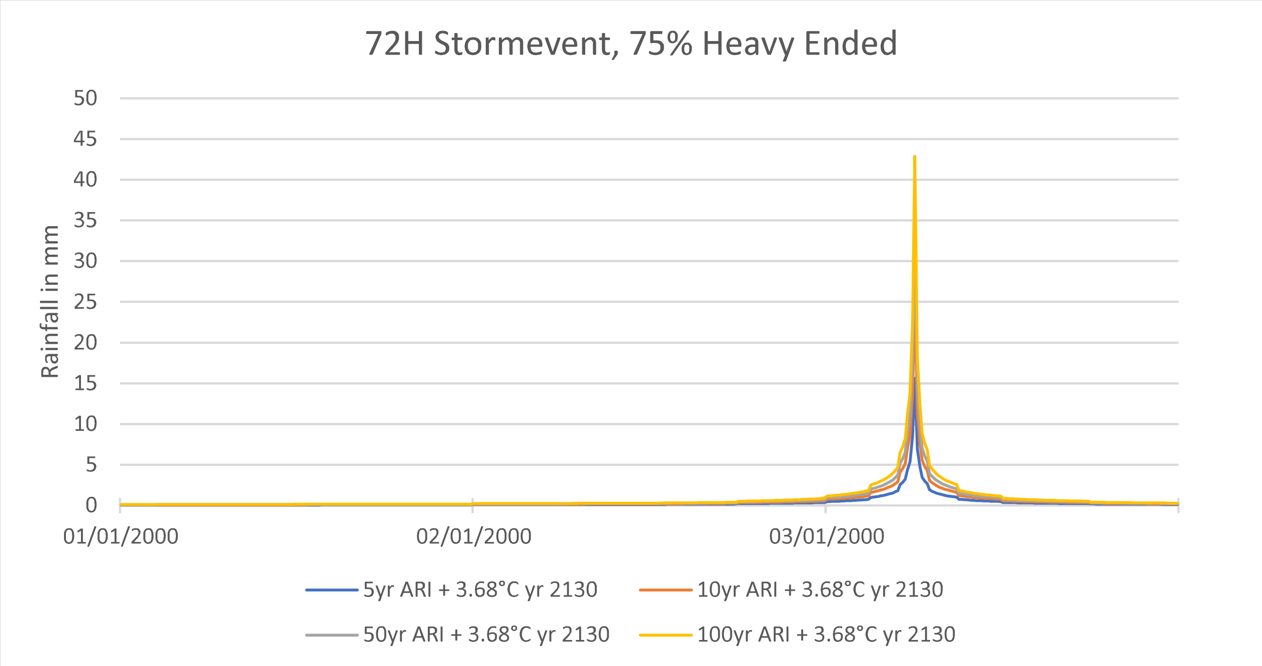

Figure 13 - Year 2130 + 3.68C Rain Event 75% Heavy Ended (Source: MIKE+ model)

Below is a zoomed-in section of the heavy-ended portion.

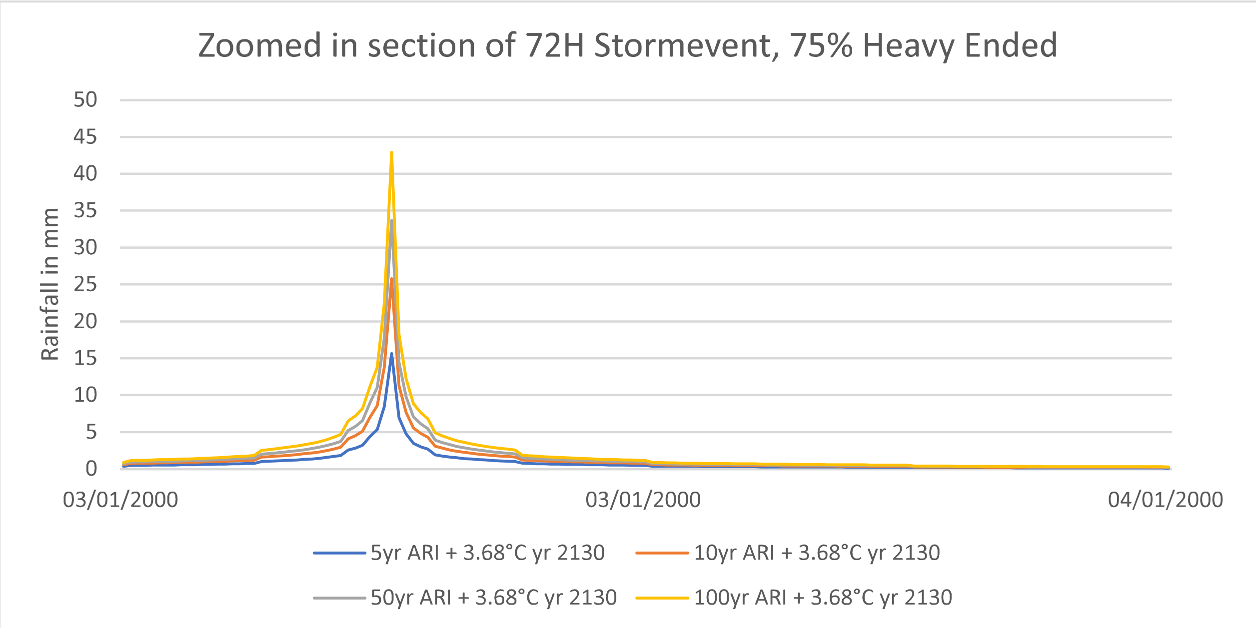

Figure 14 - Year 2130 + 3.68C Rain Event 75% Heavy Ended Detailed

(Source: MIKE+ model)

5.2.4. Key Modelling Assumptions

a. Groundwater inflows were incorporated into the model at the existing stormwater management areas known as Pond H and Pond G. The adopted water level for Pond H was confirmed with the Council data. The north and south basins allowed for 100mm of water as part of the sensitivity check that was incorporated into the main model.

b. ENGEO has estimated a groundwater inflow of approximately 3,700 m^3/day from the western and northern extents of the site. This inflow rate is not considered significant relative to the modelled stormwater volumes and is therefore not expected to materially influence flood levels within the site.

c. The tidal water levels from the coast do not propagate upstream to the site and therefore do not influence hydraulic conditions within the model domain. The tidal water levels only affect the Kopuaroa Canal and Kaituna River, in the current and future sea level rise.

d. Catchment inflows from the surrounding higher ground (Papamoa Hills) are represented in accordance with the adopted regional modelling parameters. While a rapid or short-duration runoff response from the hills is not expected to be significant, the site is sensitive to additional water volume. Consequently, all inflows into the site have been included in the model to ensure a comprehensive assessment of flood behaviour.

e. Hydraulic roughness values have been selected to represent expected land cover and surface characteristics, consistent with typical overland flood conveyance conditions. A sensitivity assessment of the swales and post-development areas was undertaken to confirm that the model results are robust to reasonable variations in the adopted roughness parameters.

f. Ponds and swales have been represented at a preliminary level within the model, noting that final detailed design, lining, and planting treatments are subject to a detailed design process. The adopted storage volumes and roughness parameters represent reasonable worst-case conditions for flood modelling purposes and will be refined during detailed design. As the development progresses, water level gauges will be installed and monitored to enable more accurate calibration of hydraulic roughness and water level predictions.

g. Publicly available terrain information sourced from LINZ and BOPRC has been used for the area outside of the site survey. Generally, public data is lower resolution, less recent, not specific to the area of study, and not as easily verified against the raw data. However, tolerances and comparisons between different sources in multiple formats (such as aerial imagery, LiDAR and REC2 catchment delineation) are consistent with each other, which increases confidence in this data.

h. Attenuation of the catchment within the modelled area was included through terrain storage. However, the accuracy of topographic data is affected by vegetated cover in reality, which may affect flow direction, terrain flood storage, stopbank overtopping levels and times. To account for the presence of roads in the model, a uniform width was assigned to all roads as part of the terrain modification process. The chosen road width had a negligible impact on the model's storage capacity and flow characteristics.

i. Infrastructure upstream of the site may impact results. Flows that may be redirected away from the catchment by underground infrastructure are not captured in the model. This is less of a risk for rural catchments, as flows are less likely to be rerouted through pipes or other conveyance systems.

j. The differences in behaviour between sheet flow, concentrated flow and stream flow have been simulated by applying a nominal depth to obtain differences in infiltration and conveyance behaviour. Assumptions could be conservative and may not correspond to actual flood behaviour.

k. Manning's n assumptions were based on BOPRC and Auckland flood modelling guidelines, with sensitivity analyses conducted to verify their appropriateness. For flood events greater than the 10-year recurrence interval, higher Manning's n values are expected to reduce due to depth-dependent effects. Following the construction of Stage 1, surface water monitoring probes should be installed to facilitate calibration, allowing adjustments as subsequent stages are constructed. A Manning's n of 0.10 was adopted for swales, with depth-dependent reduction to as low as 0.04 during higher flows. During the detailed design phase, the flow channels will be configured to convey stormwater efficiently toward the pump stations. This will also enable Pond H to be dewatered in accordance with its original design intent.

l. The 72-hour rainfall event used in the model (following BOPRC guidelines) generates a lower runoff volume compared to the 7-hour BOPRC model. With the proposed pumps, the system is expected to easily accommodate the inflows, ensuring that the stormwater management performance is not adversely affected relative to the BOPRC 7-hour scenario.

5.3. Hydraulic Modelling

5.3.1. Overview

All models used the same inputs – terrain, geometry, parameters to maintain consistency, except for hyetographs and hydrographs, which were changed based on the total rainfall depth for the different storm events. The model boundary extent is shown below.

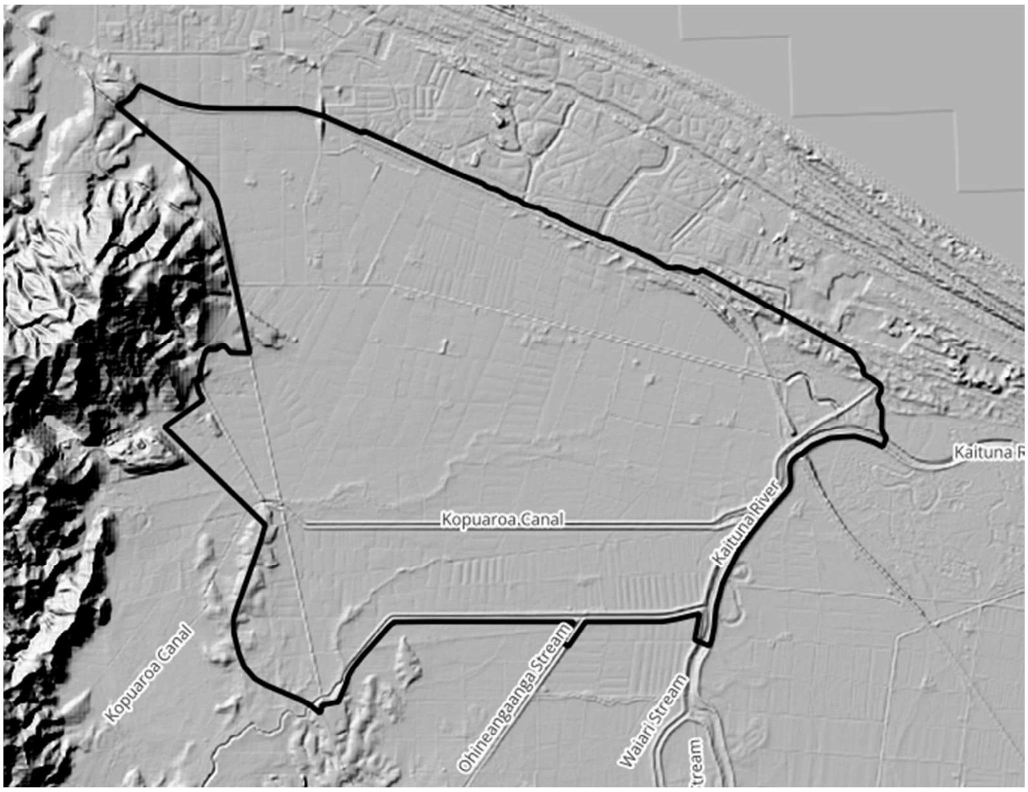

Figure 15 - Model Boundary overlay shown on the HEC-RAS view (Source: QGIS)

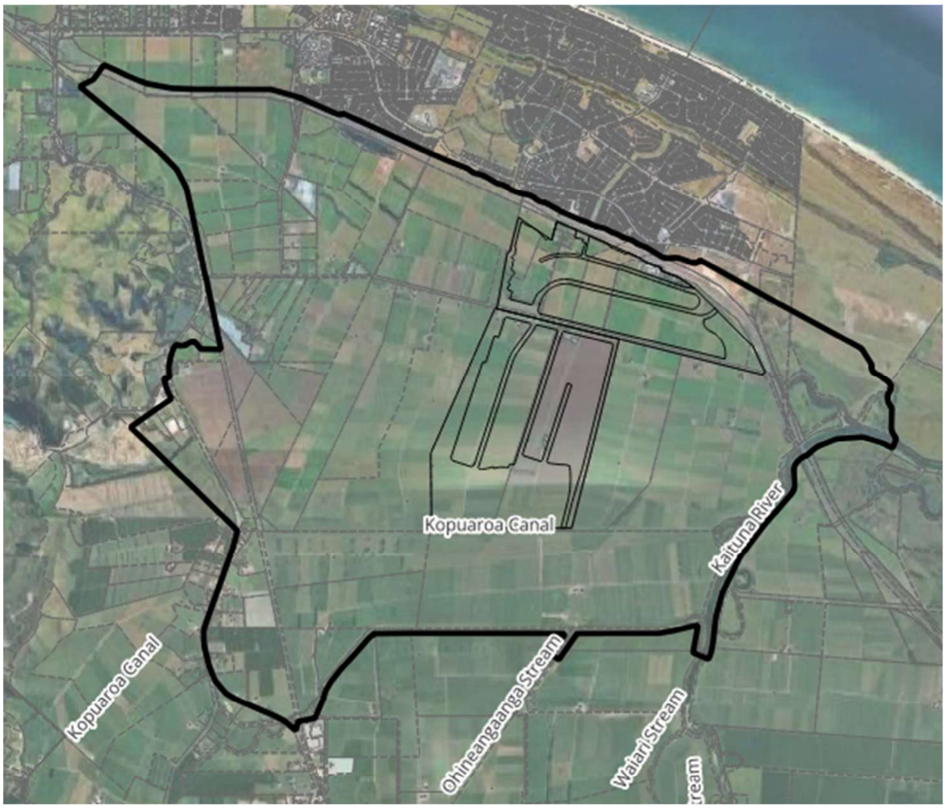

Figure 16 - Model Boundary overlay shown with aerial image and Wairakei South development site shown (Source: HEC-RAS with Google Earth)

5.3.2. Modelling Software

The hydraulic models for the site have been developed using HEC-RAS (River Analysis System) version 6.6 developed by Hydraulic Engineering Center (HEC) of the United States Army of Corps of Engineers (USACE).

For hydrologic analysis, HEC-HMS version 4.12 was used to model the Papamoa Hills catchment and to perform rainfall comparison analyses, assessing differences in runoff volume under various rainfall scenarios.

5.3.3. Catchment Attenuation & Storage

Across the catchment there is natural depression storage within the existing terrain, as well as man-made structures provide attenuation storage for stormwater runoff upstream of the catchment. The Bell Road catchment itself being an outlet controlled by floodgates and pumpstations provides attenuation storage.

The proposed development is expected to generate a runoff volume of 1.42×10^6 m^3 during the 100-year ARI, 72-hour rainfall event. For comparison, the BOPRC model indicates a runoff volume of 2.2×10^6 m^3 for the 100-year ARI, 7-day rainfall event.

The proposed South Block pump capacity of 12.8 m^3/s will dewater approximately 3.0×10^6 m^3 during the 100-year ARI 7-day event (BORPC Model). In the model, the pump operation begins at 15:00 on 31 December 1999 and continues until 08:50 on 3 January 2000, within a total simulation duration of 87 hours. Pumping would have continued beyond this period; however, the simulation ceased when the input boundary time series ended.

5.3.4. Time of Concentration

The use of BOPRC's model, aligned with inflow boundaries for the Kaituna River and Kopuaroa Canal catchments, together with direct rainfall modelling of the Bell Road catchment, indicates that upstream time of concentration (Tc) is not a controlling factor for the model setup or results. Tc calculations were performed where required to inform the model, but they do not materially influence the model configuration or outcomes. Given the site has only three outflows, two of which are pumped, the site response is dominated by storage and pump capacity once the site begins to fill; the existing pumps and flap gate are unable to discharge rapidly enough for Tc to be a significant driver of results.

Inflow from Papamoa hills has been added as a boundary inflow condition and will allow for sheet flow towards the development.

5.3.5.Modelling Scenarios

BOPRC provided guidance on the modelling scenarios to be assessed at project initiation (letter dated 29 January 2025). These scenarios included 5-, 10-, 20-, 50-, 100- and 500-year ARI events for both current climate conditions and a future climate scenario incorporating a +3.68°C temperature increase and 1.59 m sea level rise (SLR).For the purposes of this assessment, the modelling has focused on the future climate scenario incorporating the +3.68°C temperature increase and 1.59 m SLR. This approach was adopted as the Flood Risk Assessment is primarily concerned with evaluating future flood risk and the long-term performance of the proposed development under climate change conditions, which are expected to represent the more conservative scenario for assessing potential effects.

Maven has initially worked on setting up the 100-year ARI model to be calibrated and validated against BOPRC’s model in the pre-development event. This has been used to form the basis for all modelling scenarios for pre and post development, with the only changes between scenarios being the landform between pre and post development, and rainfall depths for the different return periods and climate change adjustments.

The 20-year ARI event has not been included in the current modelling assessment as the relevant model inputs were not provided within the BOPRC MIKE+ model package. In addition, the 5-, 10- and 50-year events provide sufficient resolution to characterise the intermediate level-of-service performance of the system. Modelling of a stopbank breach scenario has not yet been completed, as confirmation of breach location and modelling parameters is required in consultation with BOPRC. This is considered a future work stream.

Scenarios modelled include:

- 5Yr + 3.68°C + 1.59 SLR
- 10Yr + 3.68°C + 1.59 SLR
- 50Yr + 3.68°C + 1.59 SLR
- 100Yr + 3.68°C + 1.59 SLR100Yr + 3.68°C + 1.59 SLR + Full Pump Failure.
- 500Yr + 3.68°C + 1.59 SLR

5.3.6. Pre Development Model

In addition to the hydrological parameters discussed in the section above, the main parameters used to build the hydraulic models are described below, in sequential order of inputs into the model:

a. Terrain data contains a representation of the landform shape and is processed using the method outlined in the Terrain Data section under Catchment Context above. This data was imported into the model as a DEM. Buildings were not added to the model because the building density is sufficiently sparse to have a negligible effect on flood behaviour overall.
b. Geometry files store the spatial and GIS information relating to the model. These files include definitions for storage areas, flow areas, 1D hydraulic elements (such as culverts), and boundary condition lines. Each geometry file has an associated projection. A grid size of 5m was used across the 2D flow area. Breaklines were added at ridgelines, stopbanks and drains/canals and areas where refinement was required to modify the positions of local computation points. All breaklines were set at a resolution of <2m. The computational mesh immediately around the site is shown in Figure 18 below.
c. Before running the 5 m grid model, preliminary simulations were completed using a 50m grid (20m average). Breaklines were set at 5–10m spacing in areas needing greater detail. Within stormwater conveyance channels, cell size was refined to achieve at least 3–4 cells across the channel width.

A minor difference was noticed with the 20m average grid size compared to the 5m grid size.

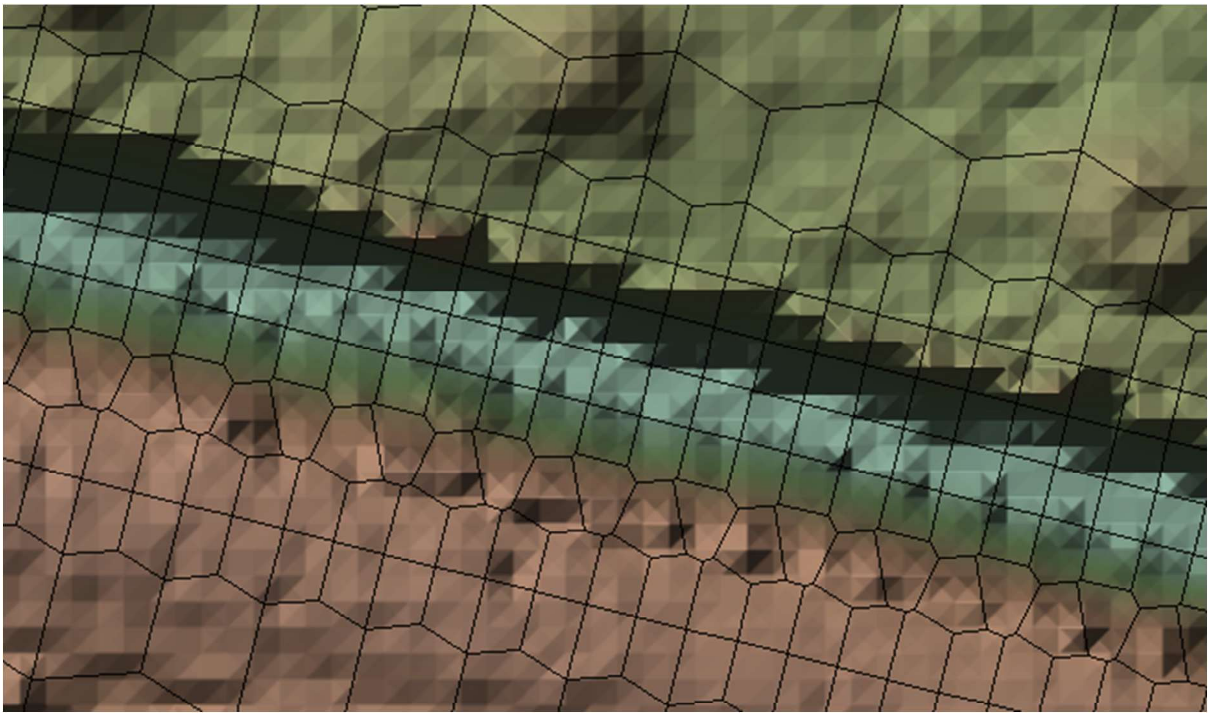

Figure 17 – Channel Computational Mesh (Source: HEC-RAS)

Several culverts were included in the model. Small-diameter farm culverts (300mm or less) were not input into the model as it was deemed they were of little consequence to the model results, particularly for larger storms where such culverts would be flooded out. The main culverts included in the pre-development model are outlined in the following section.

5.3.7. Hydraulic Structures

The hydraulic model incorporates key site infrastructure, including major culverts, pump stations, and stop banks, along with initial water levels for major ponds and drains. These features have been represented to ensure accurate simulation of flood conveyance and storage across the site. Details of the included structures, pump station capacities, stop banks, and initial water levels are summarised below:

- Bell Road Drain SH2 crossing
- Bell Road Drain Pumpstation Floodgate culverts
- Bell Road crossings
- Culverts with flapgate outlets into the Kopuaroa Canal
- Pond G culvert into north-east corner of the site
- Pond H culverts at the northern boundary – west of the development site
- There are several pumpstations also included in the model including:
- Bell Road Drain Pumpstation – 3m^3/s capacity
- Bell Road Drain A Pumpstation – 2.37m^3/s capacity
- Bell Road B Pumpstation – 0.57m^3/s capacity
- Kopuaroa Pumpstation – 1.05m^3/s capacity (south of the site)
- Additionally, the following stop banks border the site and have a break line delineation along these stop banks to improve model computation:
- Kaituna River stop banks
- Kopuaroa Canal stop banks
- Bell Road Drain stop banks
- Raparapahoe stop banks (Using 2021 LINZ data)
- Ohineangaanga stop banks (Using 2021 LINZ data)
- Any other Bell Road catchment stop bank of significant value
- Initial elevation water level
- Pond H water level RL2.0m (NZVD 16), council data confirmed an average level over 3 years of RL1.87m
- Pond D water level RL1.3m (NZVD 16)
- Bell Road main drain water level RL0.52 to 1.8m (NZVD 16)
- Pond H Drain, RL1.5 – RL2.1m (NZVD 16)
- Raparapahoe Channel RL6.0m (NZVD 16) (artificially creates a peak at the start of the development)

Flow data contained data linked to the boundary conditions in the geometry files. Inflow boundary conditions used hydrographs obtained through the process outlined in the Model Parameters and Rainfall Data sections under Catchment Context above. This assessment used unsteady models with varying flow rates through time.

The HEC-RAS plan and geometry settings govern how the simulation is computed. The 2D model was solved using the Shallow Water (full momentum) equations, with computational timesteps controlled using the Courant-Friedrichs-Lewy (CFL) stability criterion. Compared with the diffusion wave equation set, the full momentum solver better represents inertial effects and rapidly varying flows, improving accuracy where these processes are significant. 1D/2D iterative coupling was enabled to improve model stability and consistency between the 1D and 2D domains.

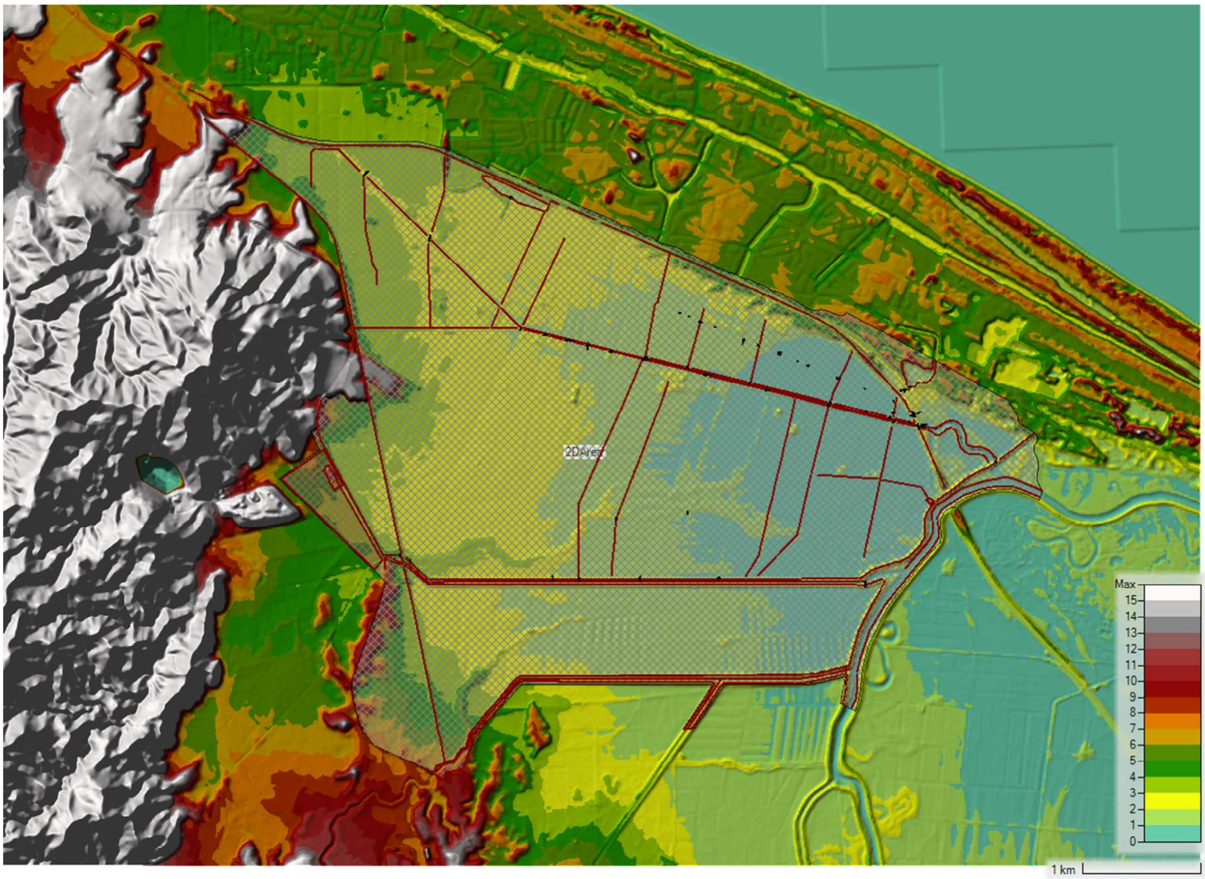

Figure 18 - Pre-Development Terrain and Computational Mesh (Source: HEC-RAS)

5.3.8.MIKE+ Spatial vs HEC-RAS Nested Rainfall Profile

MIKE+ allows the use of spatially distributed rainfall, enabling local variations in storm intensity to influence the timing and magnitude of inflows across the catchment. HEC-RAS applies a uniform rainfall, producing more uniform hydrographs across the catchment.

A nested rainfall profile consistent across the HEC-RAS 2D domain was applied. This ensures that all parts of the 2D floodplain receive the same temporal rainfall pattern, simplifying interpretation of results and maintaining consistency in flood volume and extent calculations. The 2D HEC-RAS model simulates overland flow across the defined domain but does not inherently account for spatial variability in rainfall.

The primary implication of using a uniform rainfall profile is that local peak flows, longer peak duration, and flood volumes may be slightly higher in areas where rainfall would naturally vary. This may result in an earlier and slightly longer flood peak, but it does not cause a significant change in overall water levels across the floodplain. For large-scale floodplain modelling and comparison of flood extents, this assumption is generally acceptable. Applying a consistent rainfall profile also allows it to be used as part of an overall sensitivity check, helping to isolate the effects of hydraulic parameters and floodplain interactions without introducing additional variability from spatially varying rainfall.

5.3.9. Boundary Conditions

Boundary conditions were placed upstream and downstream of the Bell Road catchment model, simulating inflows from the Kaituna River catchment, Pāpāmoa. A stage boundary condition downstream of the Bell Road Drain Pumpstation on the Kaituna River was applied to simulate tidal influence.

All Pāpāmoa culverts have been excluded from the model as separate boundary inflows. The comparison between pre- and post-development scenarios indicates that there would be no additional backflow into Pāpāmoa.

This information was obtained from the BOPRC Bell Road Model (MIKE 2025), as part of the shared model agreement.

Pāpāmoa Hills, in the BOPRC Bell Road Model (MIKE 2025), has a significantly lower inflow than expected. As Bell Road is flood prone, we did not use BOPRC Bell Road Model (MIKE 2025) inflow data but rather generated our own model to more accurately represent the inflow from this part of the catchment. As we have noticed no weir, water depth or flow data in the area, we used a HEC-RAS rain on grid model and HEC-HMS model to provide an indication of the volume and peak flow from Pāpāmoa Hills.

The HEC-RAS rain-on-grid model simulates overland flow directly across the 2D floodplain, capturing spatial variations in flow pathways and local storage. For this model, a 450 mm culvert was assumed, based on the catchment size and the Level of Service (LOS) that the council requires for a 5-year event. This configuration allows for more reasonable outflow through the culvert compared with potential road overtopping, while enabling smaller storm events to build up and flow through the culvert without causing premature flooding.

In contrast, the HEC-HMS model uses a hydrologic approach with sub catchments and the Clark Unit Hydrograph method, producing flow hydrographs at the catchment outlets rather than explicitly simulating flow across the floodplain surface.

The HEC-RAS model was built using a Curve Number (CN) of 74 and an infiltration rate of 0.05. A sensitivity analysis was also conducted assuming no infiltration for the 50-year ARI and 100-year ARI events, with results compared to the BOPRC Bell Road Model.

The HEC-RAS model employed a 10 m grid with breaklines aligned to the contour lines, with a smaller cell size applied in the vicinity of Te Puke State Highway to capture finer topographic detail.

A HEC-HMS model was also applied for the same area to provide a comparative check, with results from the rain-on-grid approach compared against traditional sub catchment calculations using the Clark Unit Hydrograph method.

There are some limitations of the HMS and RAS models due to calibration data, these are;

- Peak flows and flood volumes
- Flood timing
- Local inundation patterns
- Comparisons between pre- and post-development scenarios.
  The images below, taken from HEC, show the Pāpāmoa Hills boundary area and the inflow points taken into account in our model.

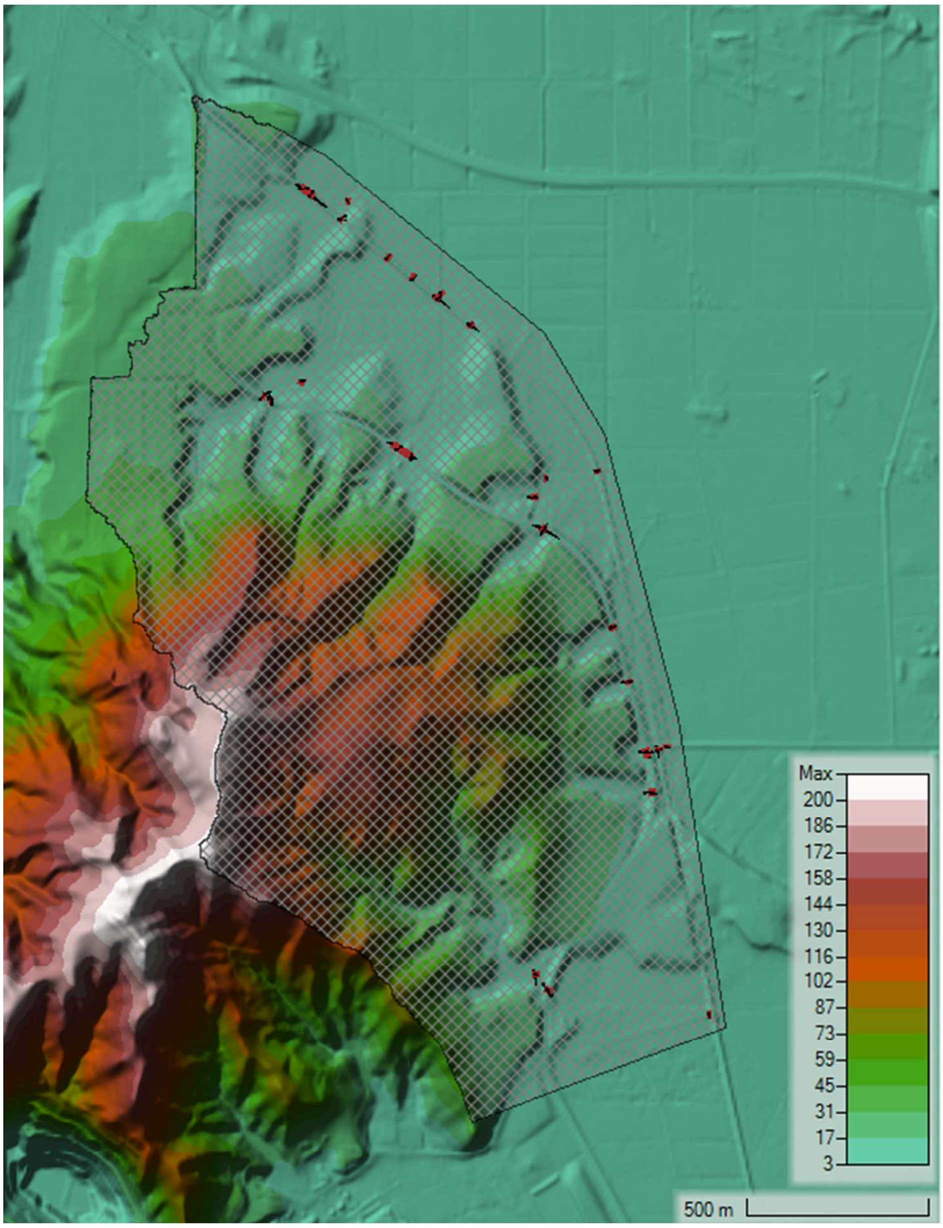

Figure 19 - Boundary Area for Pāpāmoa Hills (Source: HEC-RAS)

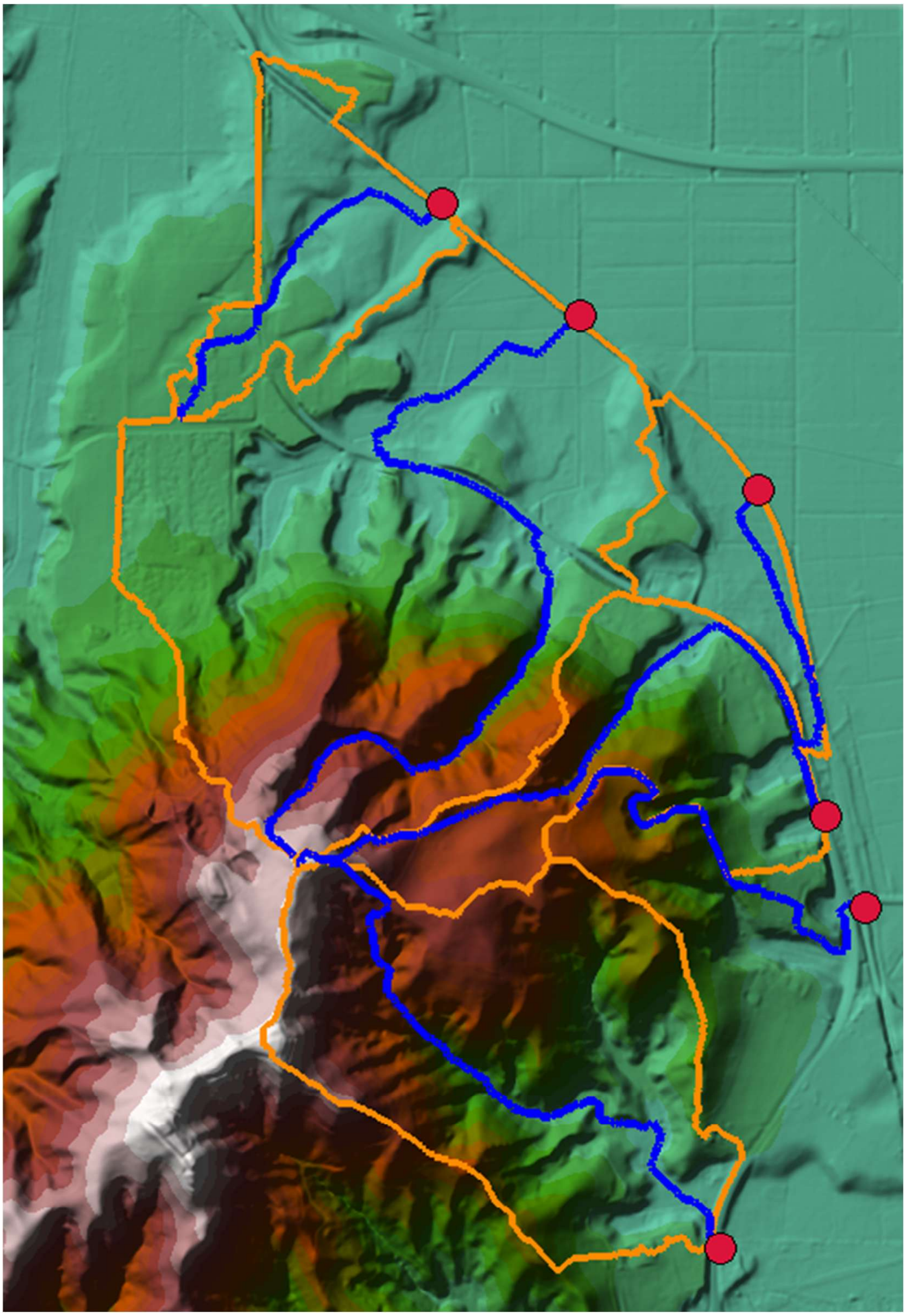

Figure 20 - Boundary Area for Pāpāmoa Hills and Flow Paths (Source: HEC-RAS)

The image below, taken from HEC, shows the Pre and Post boundary inflow locations.

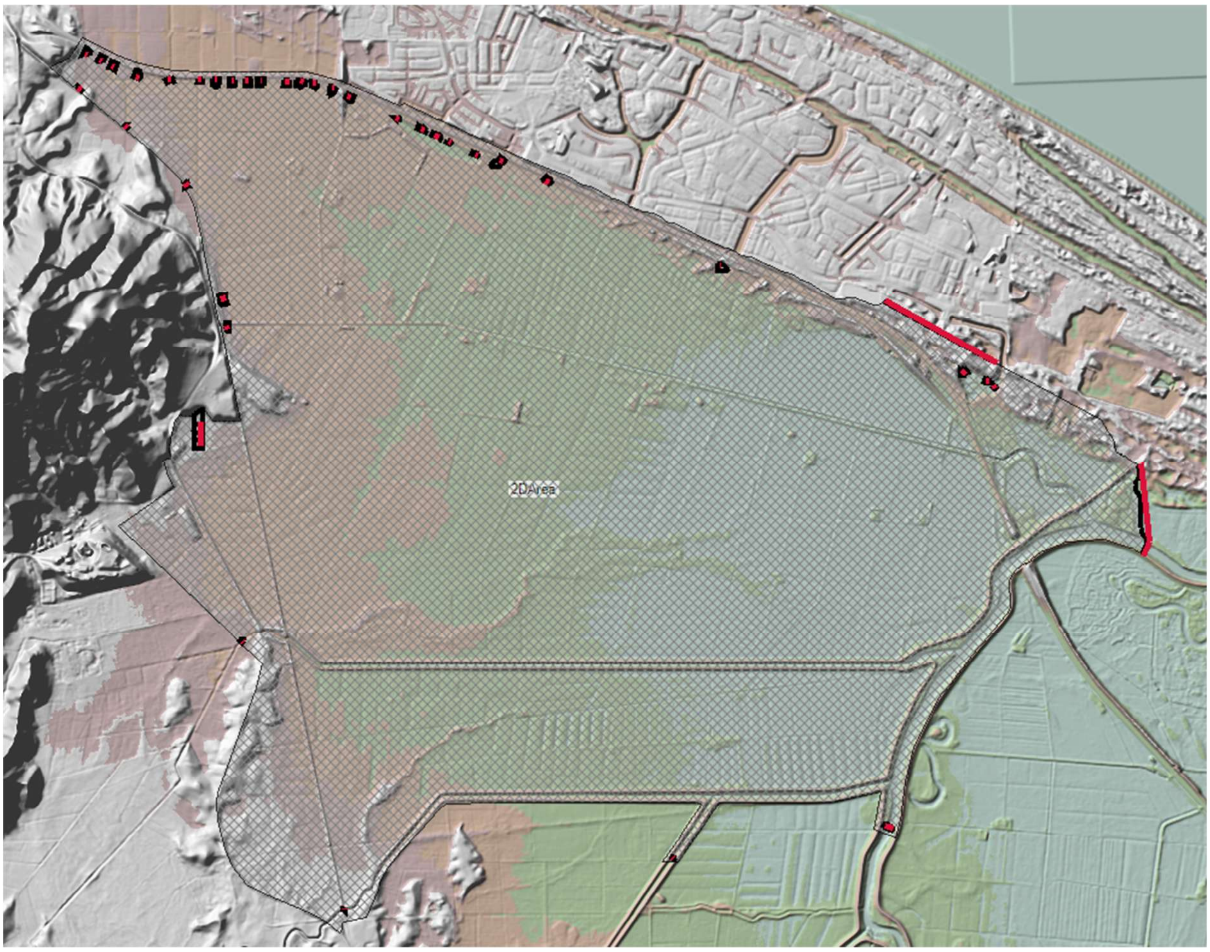

Figure 21 - Pre and Post Boundary Inflow Locations (Source: HEC-RAS)

5.3.10. Post Development Model

In the post-development model, the same base geometry, mesh, terrain, direct rainfall and boundary condition data was utilized as for the pre-development model, with the following changes to model the post development scenarios:

- The design landform terrain was modelled in AutoCAD Civil 3d and then exported as a DEM .tif file and merged with the pre-development terrain. This included the raising of the development site and lowering of conveyance channels and stormwater reserves as storage areas. The minimum elevation used in the southern block of development was RL 0.7m to imitate the average groundwater level across this region of the site and hence ensure the storage volume available is not overrepresented.
- For the northern block, a minimum elevation of RL 0.4m was adopted using the same approach.
- Groundwater levels were developed in close coordination with ENGEO to account for any groundwater inflow during a storm event. ENGEO's Hydrogeological Assessment Report (Appendix R of the AEE) estimates a groundwater inflow of 3,700m^3/day (0.04m^3/s). The

model represents this using seven inflow nodes at 0.1 m³/s each to provide contingency for uncertainty. More information on groundwater and the hydrogeology of the site can be found in the ENGEO Hydrogeological Assessment Report.

- The southern stormwater reserves include an additional 433,000m³ of storage compared to pre-development, and there is an additional 149,000m³ of storage volume within the northern stormwater reserve.
- A new 12.8m³/s pumpstation is included within the southern block stormwater reserve.
- The Bell Road Drain A pumpstation will be upgraded to 8.2m³/s maximum capacity (an additional 6m³/s capacity).
- A new 100m wide causeway across Bell Road connecting the overland flows and channels on the northern side of Bell Road to the new major conveyance channel on the southern side of Bell Road.
- New culverts and causeways within the site provide conveyance for stormwater flow.
- Internal swale culverts and box culverts have not been added to the model; this will be part of the detailed design modelling for the lesser storm events at engineering approval stage.

Figure 22 - Model Geometry for Proposed Development (Source: HEC-RAS)

Data on the key causeways and culverts is as follows:

- Causeway and Box culvert sizing
  - Causeway A
    - Span 100m, Rise 1.5m
    - Mannings N 0.011

        - Peak flow 100Yr 2130 – 65.64m³/s
        - Upstream RL 0.99m
        - Downstream RL 0.92m
    - New Box Culvert A (low flow for Bell Road)
        - Span 8m, Rise 1m
        - Mannings N 0.013
        - Peak flow 100Yr 2130 – 6.60m³/s
        - Upstream RL 0.2m
        - Downstream RL 0.1m
        - Low flow channel toward the South pump station will be made in detailed design.
    - Causeway B
        - Span 100m, Rise 1.5m
        - Mannings N 0.015
        - Peak flow 100Yr 2130 - 118.1m³/s
        - Upstream RL 0.42m
        - Downstream RL 0.41m
    - New Box Culvert B
        - Span 35m, Rise 2m
        - Mannings N 0.013
        - Peak flow 100Yr 2130 – 20.14m³/s
        - Upstream RL 0.71m
        - Downstream RL 0.71m

5.3.11. Calibration

Maven's model which was produced in HEC-RAS v6.6 was calibrated against the BOPRC Bell Road Catchment Model which was produced in MIKE+ 2025.

Input data and parameters have been extracted from the MIKE+ 2025 council model and replicated in the HEC-RAS model, with multiple instances of verification of the data being used.

Model analysis runs produced in HEC-RAS were calibrated across multiple iterations to align Maven's model to the BOPRC model with a reasonable fit in terms of flood depths, water surface levels and peak flows.

The Kopuaroa Canal and Kaituna River have been calibrated using the BOPRC model to derive appropriate Manning's *n* values for the 2D domain. The hydraulic system was initially developed in a 1D model, with subsequent verification carried out in the 2D model to ensure consistency and performance across both modelling platforms.

5.3.11.1. Kopuaroa Canal Calibration

The peak water level was obtained from MIKE+ model, which was set as the observation height for the calibration model in 1D. The 100Yr 3.68°C 1.59SLR was used for the calibration. Diagrams showing a representation of the calibration points are provided below.

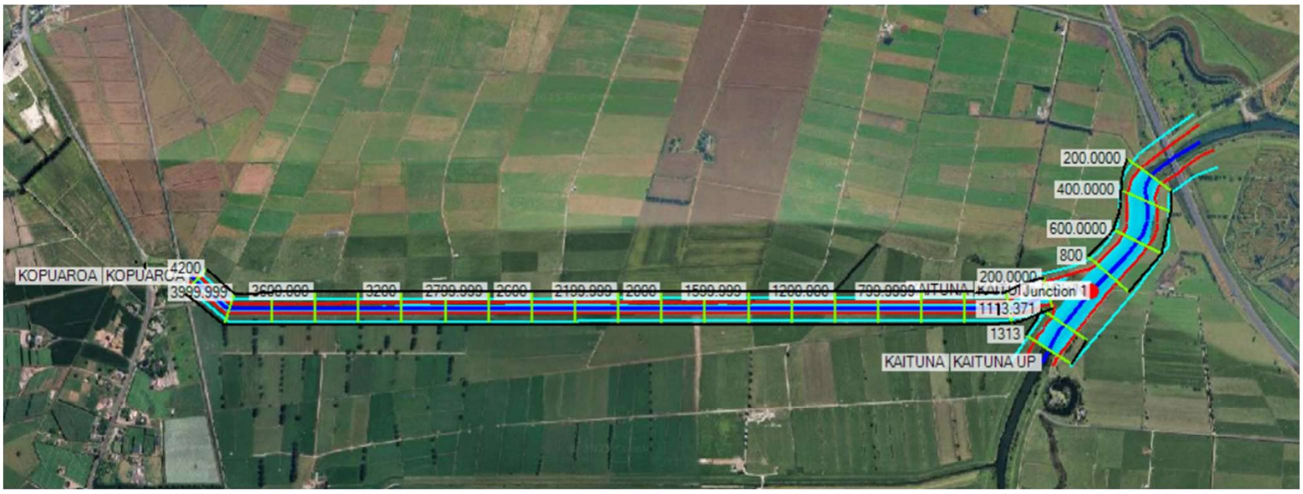

Figure 23 – Kopuaroa and Kaituna Calibration 1D (Source: HEC-RAS)

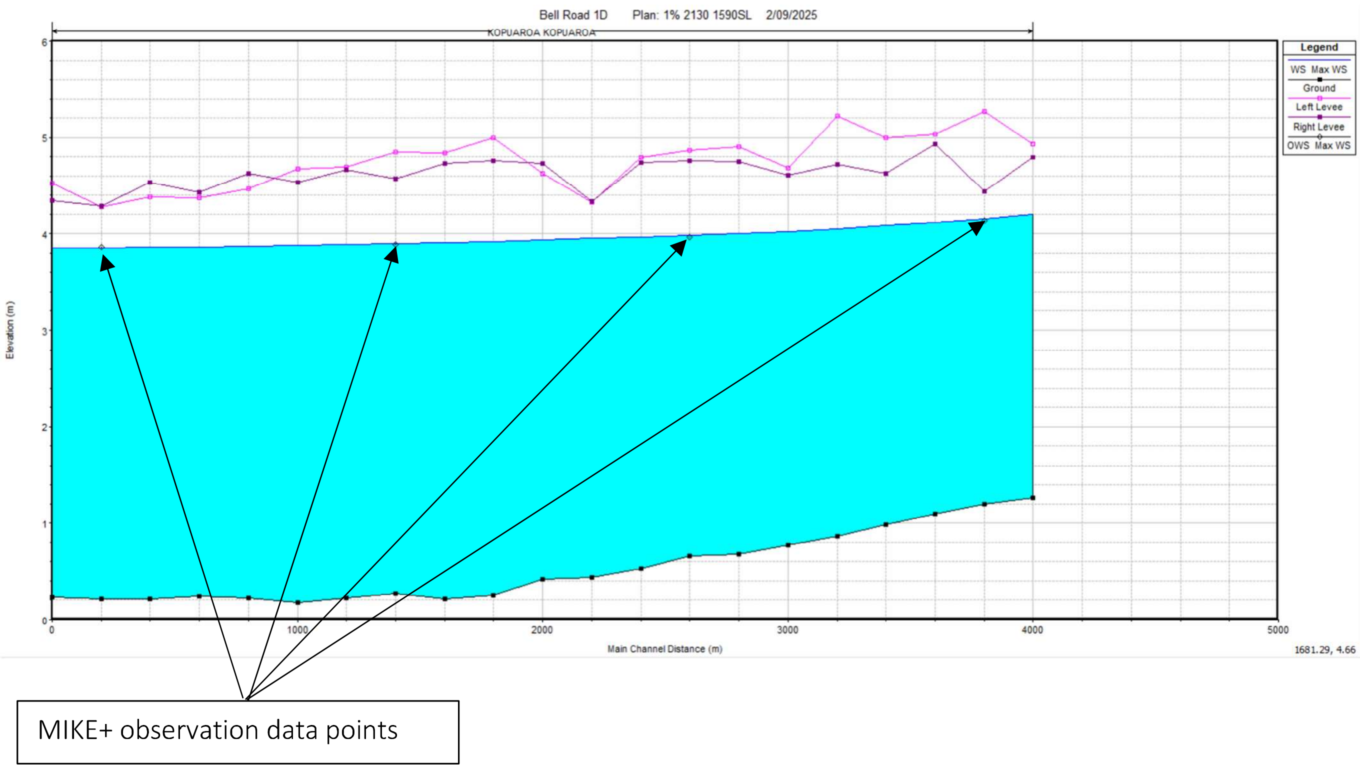

Figure 24 – Kopuaroa Canal Long Section 1D (Source: HEC-RAS)

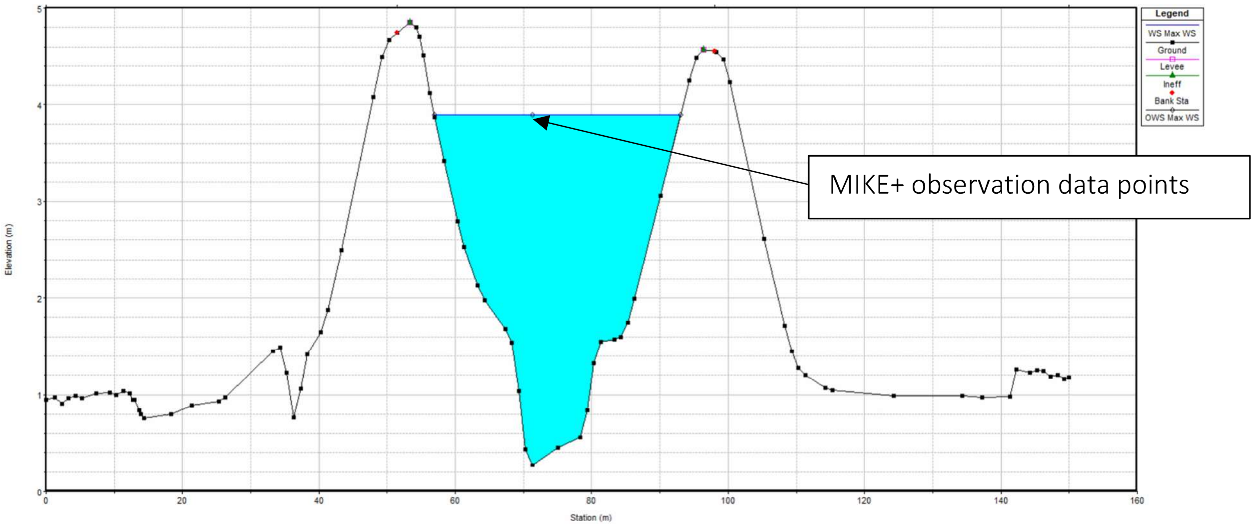

Figure 25 – Kopuaroa Canal Cross Section 1D (Source: HEC-RAS)

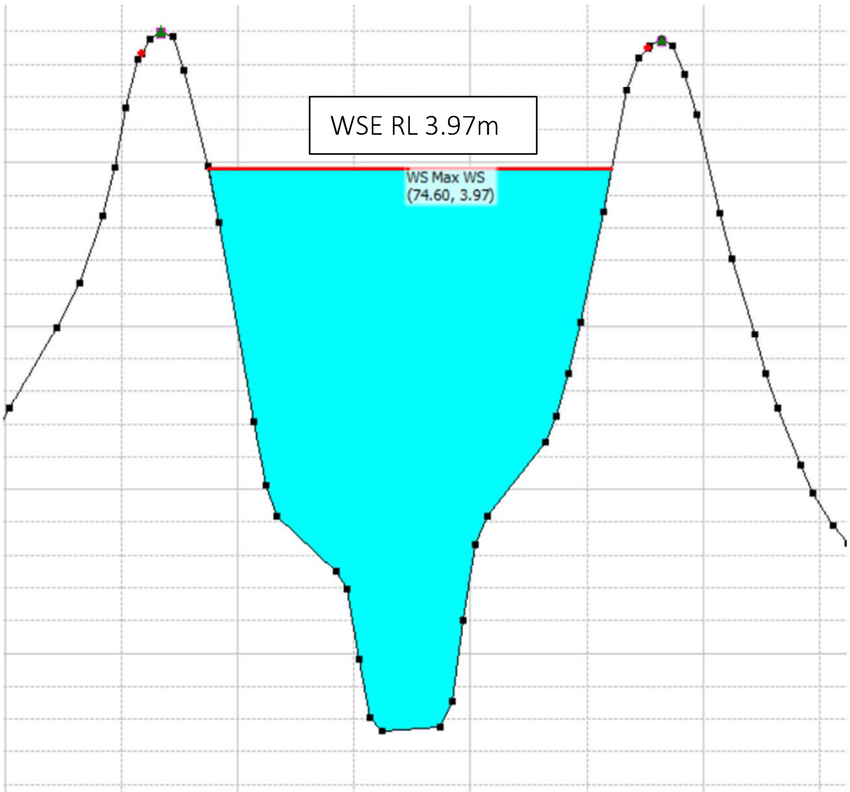

Figure 26 – Kopuaroa Cross Section 1D: Ch 2600 (Source: HEC-RAS)

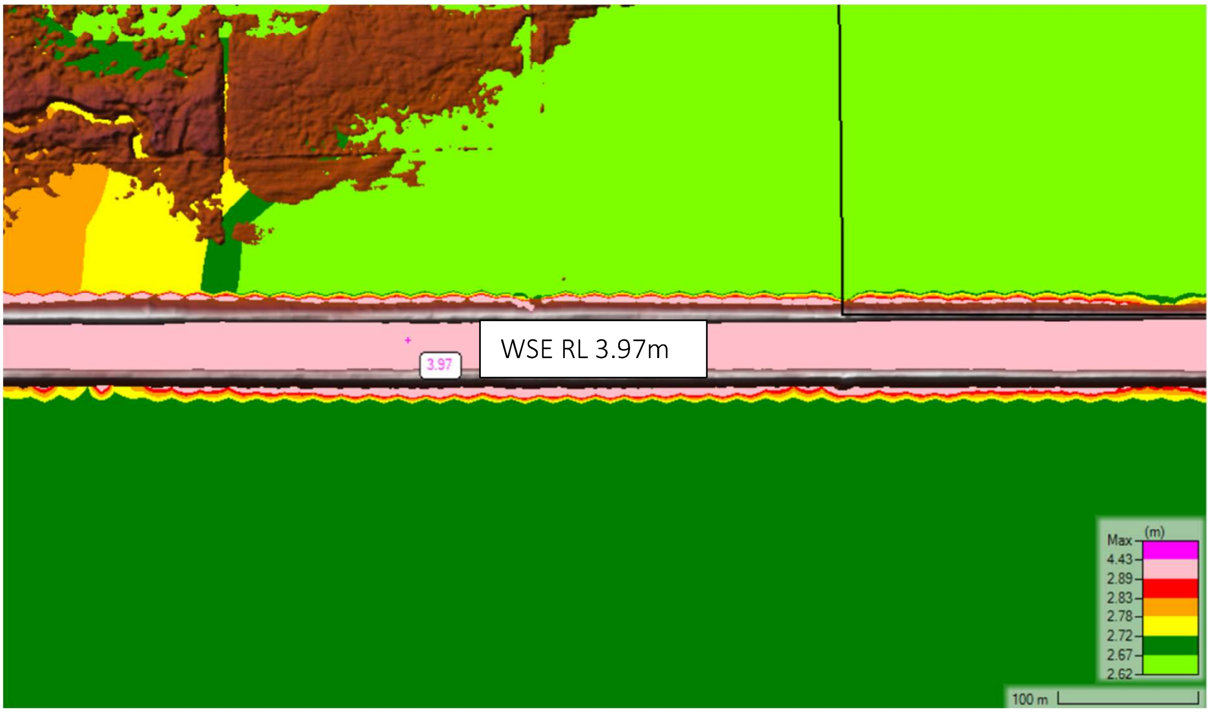

Figure 27 – Kopuaroa Canal Cross Section 2D: Ch 2600 (Source: HEC-RAS)

A similar process was followed for the Kaituna River before starting the 2D model.

5.4. Modelling Results

Modelling results presented have been generated on the 3rd of March 2026 using HEC RAS software (version 6.6)

5.4.1. Numerical Accuracy and Model Stability

The model was developed using a uniform 2D cell size selected to suit the flat terrain and drainage characteristics of the catchment. Numerical stability was managed by applying variable time steps constrained by Courant Number targets to ensure stable solution behaviour throughout the simulation.

As is common in direct-rainfall 2D models, some instability occurred during the early and late stages of each event, when shallow depths spread across a wide area. These instabilities diminished as the simulation progressed and did not persist into the peak of the hydrograph, where the critical flood level results occur. The model remained stable during peak flood conditions across all events, ensuring that key outputs—water surface elevations, flow paths, and hazard mapping—are reliable at the times that matter most.

Numerical convergence indicators within the HEC-RAS computation log, including maximum water-surface-elevation (WSE) error and mass-balance behaviour, were reviewed for each model run. Although WSE convergence was weaker during low-depth periods, this aligns with expectations for direct-rainfall simulations and does not affect prediction reliability at peak flows. Volume accounting errors remained low across all scenarios, supporting confidence that the model achieves acceptable mass conservation and overall numerical robustness.

The HEC RAS simulation computation log file overall volume accounting errors are shown below as a percentage:

Table 5 – Volume Accounting Error (%) for Current Climate Scenario

	10-year - Current	50-year - Current	100-year - Current	500-year - Current
Pre-Development	0.23%	0.02%	0.01%	0.05%
Post-Development	0.02%	0.04%	0.06%	0.10%

Table 6 - Volume Accounting Error (%) for 2130 + 3.68C Climate Scenario

	10-year – 2130 & 3.68°C	50-year – 2130 & 3.68°C	100-year – 2130 & 3.68°C	500-year – 2130 & 3.68°C
Pre-Development	0.13 %	0.05%	0.05%	0.03%
Post-Development	0.15%	0.08%	0.11%	0.09%

In both the current climate and climate change scenarios represented, the overall volume accounting errors can be considered as negligible.

5.4.2.Outflow Volume Check

Volume accounting results were reviewed to confirm mass conservation throughout each simulation. The following tables present the HEC-RAS computation log volume errors as reported in the original model runs. Percentage errors remain well within typical industry tolerances (<1%) as shown in section 5.4.1 above.

HEC RAS simulation computation log file overall volume accounting error in 1000 m3 can be found in the tables below.

Table 7 – Volume Accounting Error (in 1000m^3) for Current Climate Scenario

	10-year - Current	50-year - Current	100-year - Current	500-year - Current
Pre-Development	12.67	12.20	9.93	49.27
Post-Development	9.27	29.19	42.87	97.77

Table 8 – Volume Accounting Error (in 1000m^3) for 2130 + 3.68C Climate Scenario

	10-year – 2130 & 3.68°C	50-year – 2130 & 3.68°C	100-year – 2130 & 3.68°C	500-year – 2130 & 3.68°C
Pre-Development	78.39	45.14	42.00	30.83
Post-Development	95.14	70.09	99.07	101.4

5.4.3.Model Validation

During validation its was generally found that there was good agreement between the Maven model and BOPRC model, particularly in terms of flood extents, depths and water surface levels, with some differences to be expected due to Maven's terrain data including drone and topographical survey which generally will have a better accuracy than the publicly available LiDAR used by BOPRC.

The Hurst Block to the southeast (Lot 1 DP 30374 and Section 19 SO 490071) was used as the benchmark given its proximity and higher flood susceptibility.

The increase shown below reflects additional inflow from Papamoa Hills, which is not included in the BOPRC model.

The differences noted during validation of the pre-development scenario are shown below (for the 100 yr event only):

Table 9 – BOPRC vs Maven Model Differences

	BOPRC Model	Maven Model (NZVD16)	Difference +/-
Flood Extents – South Block - Hurst Block	General agreement between models		
Pre Dev - Maximum Water Surface Elevation m (100yr 3.68°C SLR1.59)	2.56 (MOT53) 2.329(NZVD16)	2.60	+0.27m
Pre Dev - Maximum Flood Depth m (100yr 3.68°C SLR1.59)	1.84	2.14	+0.30m

Notably, the larger storm event (100yr) is more aligned between the two models. This is likely due to catchment features such as culverts and drains not being accurately represented in either model which impacts the lesser events. In the 100yr scenario, these features are largely flooded out.

A comparison was undertaken between the 50 m grid model (with an average grid size of 20 m) and a 5 m grid model using the 100-year, 3.68 °C, 1.59 SL event. The analysis indicated that the surface elevation in the Hurst Block was approximately 20 mm lower in the 5 m grid model. In addition, inflow immediately upstream of the development peaked at the same time in both models, with only minor instabilities observed. These results provide confidence that the 50 m grid model is fit for purpose.

Time series water levels between the two models

The 100-year event incorporating a 3.68 °C climate change allowance and a sea level rise of 1.59 m was compared at Bell Road near the State Highway and at the Bell Road "A" Pump Drain located in the Hurst Block. This configuration captures inflows from both the northern and southern catchment blocks and enables a direct comparison of water level time series between the two models.

Differences in modelled water levels are observed at the commencement of the simulation, reflecting variations in initial water levels and starting conditions between the models. As the storm event develops and approaches the peak, the predicted water levels converge for Bell Road "A" (Figure 32) and "B" (Figure 34) pump stations and main drain (Figure 35) and there are differences for Bell Road main drain pump station (Figure 33). This strong alignment at peak water level conditions in certain locations provides confidence that the models are behaving consistently and that the resulting outputs are robust and reliable in those locations.

Figure 28 – HEC RAS Model, Pre *Dev:* 100Yr 3.68C 1.59SLR (Source: HEC-RAS)

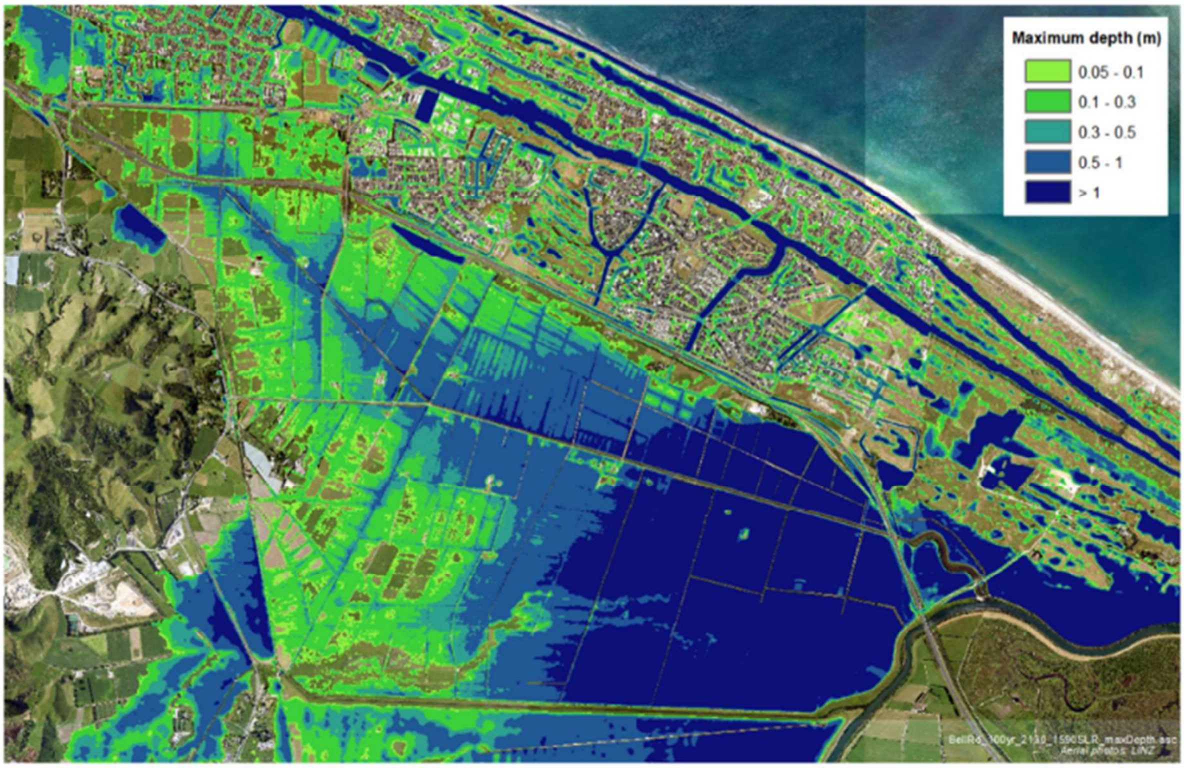

Figure 29 – Mike + Model, by River Edge Consulting Limited June 2025 (Source: 2025 June REC Bell Road 2025 model updated LiDAR *Freeboard* FINAL)

5.4.4. Specific Limitations – 5-Year ARI Event

The 5-year ARI model results are preliminary and intended to inform iterative design development rather than to provide a validated assessment. Awa's peer review confirms that the 5-year event is not yet verified against MIKE+ and exhibits routing differences, including excess flow directed south instead of toward the Bell Road Drain / TEL culvert. These routing discrepancies are expected to be resolved through improvements to channel geometry, culvert representation, and grid resolution as part of detailed design.

Given that the 5-year event represents a low-hazard scenario, the key consideration is drain-down performance and avoidance of prolonged ponding, rather than risks to people or property. The proposed pump station capacity and conveyance system are expected to improve service levels for frequent events once refined design inputs are incorporated.

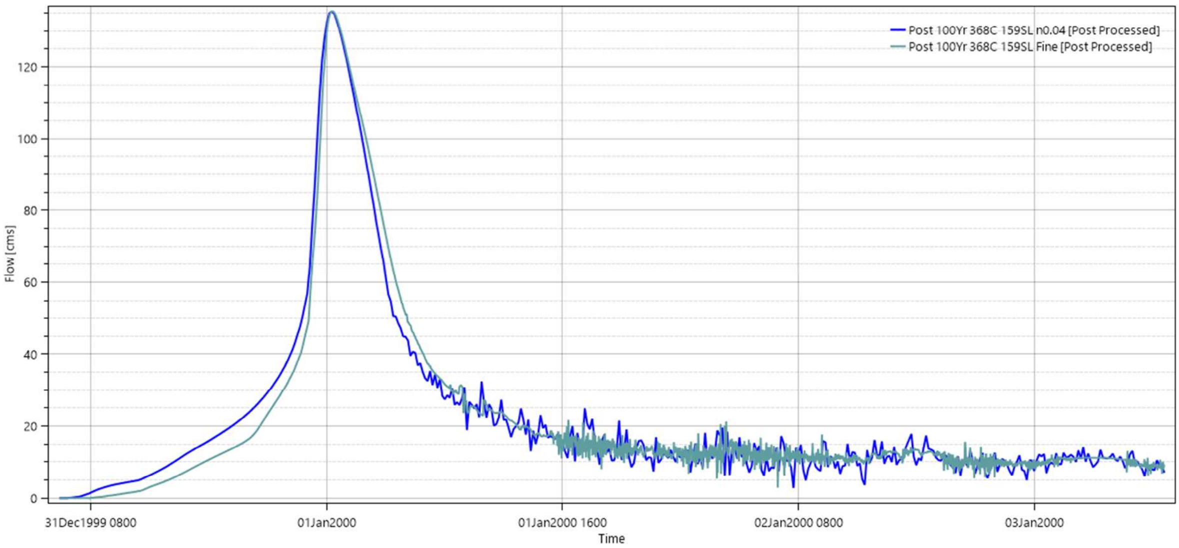

Figure 30 – Development Inflow Comparison (Source: HEC-RAS)

To further strengthen confidence in the modelling outcomes, additional validation checks were undertaken, including detailed comparisons of water level time series, discharge rates, and pump performance between the two models, confirming overall consistency and alignment of results.