# HDM Halswell Flood Model – Status Report

Prepared for Christchurch City Council
Prepared by Beca Limited

**9 December 2022**


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## Revision History

| Revision Nº | Prepared By | Description | Date |
|---|---|---|---|
| A | Leif Healy / Natasha Webb | First release to accompany delivery of Existing Development 2016 model. | 09/12/2022 |
| | | | |
| | | | |

## Document Acceptance

| Action | Name | Signed | Date |
|---|---|---|---|
| Prepared by | Leif Healy | | 25/11/2022 |
| Reviewed by | Elliot Tuck | | 08/12/2022 |
| Approved by | Kate Purton | | 09/12/2022 |
| on behalf of | Beca Limited | | |

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# 1 Introduction

## 1.1 Purpose of this Report

The purpose of this report is to outline the method, tools and processes used in the model build of the HDM Halswell flood model. The HDM Halswell flood model represents the Halswell River catchment of the Citywide Flood Modelling (CFM) project. This report follows the same structure as the other catchment reports with blank sections representing work that has not been completed.

## 1.2 Citywide Flood Modelling (CFM) Project

The CFM project is a three-way coupled hydraulic modelling project being undertaken by Christchurch City Council (CCC).

The purpose of the project was to develop updated river catchment models for the 'flat land' Christchurch, combined with a single hydraulic model of the city's waterways and pipe networks (greater than 300 mm), that can be used:

- By the Land Drainage Recovery Programme (LDRP) to support identification and prioritisation of repair and remedial options.
- By CCC to investigate resiliency of the city against climate change and sea level rise.

## 1.3 Specifications

The HDM Halswell flood model was developed using the following specifications and reference documents:

- CFM (LDRP044) Model Schematisation – Avon/Estuary, Heathcote, and Sumner (GHD\AECOM, 2018)
- Waterways and Wetland Design Guide (WWDG) (CCC, 2011)
- Stormwater Modelling Specification for Flood Studies (GHD, 2012)
- Christchurch City Council MIKE FLOOD Technical Specifications (DHI, 2015).

## 1.4 Model Developer

The HDM Halswell flood model has been developed by Beca Ltd (Beca) from their Christchurch Office; details listed in Table 1-1.

Table 1-1: Contact details

| | |
|---|---|
| **Name** | Beca Ltd |
| **Office Address** | ANZ Centre, Level 2<br>267 High Street<br>Christchurch 8011 |
| **Postal Address** | PO Box 13960, Armagh Street<br>Christchurch 8141 |
| **Phone** | +64 3 366 3521 |
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The primary modellers involved in the development of the HDM Halswell model are Elliot Tuck and Leif Healy. Their details are listed in Table 1-2.

Table 1-2 Primary modellers contact details

| Name | Email | Phone |
|---|---|---|
| Elliot Tuck | Elliot.Tuck@beca.com | +64 3 367 2458 |
| Leif Healy | Leif.Healy@beca.com | +64 9 300 9000 |

## 1.5 Peer review

DHI undertook a peer review of the Existing Development 2016 model at the end of 2021. The outcome of this review was advice to correct several general schematisation errors and specific features. DHI compiled their advice in a Microsoft Excel spreadsheet with the importance of each item indicated.

Beca updated the Existing Development 2016 model as per the advice in all items marked as *not satisfactory* and *still room for improvement* or responded with an explanation of why no action was taken.

The advice provided by DHI was also followed during the update from the Existing Development 2016 model to the Existing Development 2021 model. At the time of this release (draft revision A) the Existing Development 2021 model is with DHI for review.

# 2 Modelling Software

The DHI MIKE software suite is the primary software used to build the HDM Halswell model. The suite was used as MIKE can model a variety of hydrologic and hydraulic processes, including rainfall-runoff, river behaviour, pipe networks, 2D flood flows.

There are two main components of the HDM Halswell model: the hydrology and the hydraulics. Table 2-1 outlines the software used for each component and the methods that were adopted under the software. Further details for each software are provided in the following sections below.

Table 2-1 HDM Halswell model summary

| Process | Software | Method |
|---|---|---|
| Rainfall-Runoff (Hillside) | RORB | Concentrated Non-linear Storages |
| Rainfall-Runoff (Flat Land) | MIKE 21 FM | Rain on Grid |
| Flood Plains | MIKE 21 FM | 2D Shallow Water Equations |
| Pipe Network | MIKE Urban | MOUSE 1D St Venant and Continuity Equations |
| Channel Flow | MIKE 11 Classic | St Venant and Continuity Equations |

## 2.1 Hydrology Software

The hydrology was modelled using two methods in two software packages: 2D rain on grid in MIKE 21 and RORB rainfall-runoff. The use of this software was determined based on the topography of the Halswell catchments, shown in Figure 2-1, where:

- The flat land hydrology was modelled as rain on grid. Rain on grid applies a rainfall time-series directly to each active mesh element in MIKE 21. As water accumulates in each mesh cell, water will move to the adjacent element based on the amount of rain and losses applied to that element. An infiltration map within MIKE 21 uses Horton's equation to apply a spatially varying Horton's decay time series across the entire catchment area.

  The rain on grid method is consistent with the methodology outlined in CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018).

- The hillside (Port Hills) catchments were modelled using RORB. RORB is a runoff and streamflow routing program that calculates flood hydrographs from rainfall and channel inputs. It subtracts losses (initial loss and continuing loss) from the rainfall to produce excess-rainfall and routes this through catchment storage to produce runoff hydrographs at specified locations.

  RORB was used to produce outflow hydrographs for each hillside catchment which were used as point inflows to the MIKE 21 model. More information on the development and background theory of the runoff-routing method employed in RORB is contained in Appendix A[1]. Section 3.3 provides the detail of the hydrology used within the HDM Halswell model.

[1] The concepts and information provided in this Appendix A are taken from the RORB Version 6 User Manual which is copyright of Monash University and Hydrology and Risk Consulting Pty Ltd. Any material copied or otherwise reproduced from that manual must give due acknowledgement of its source.

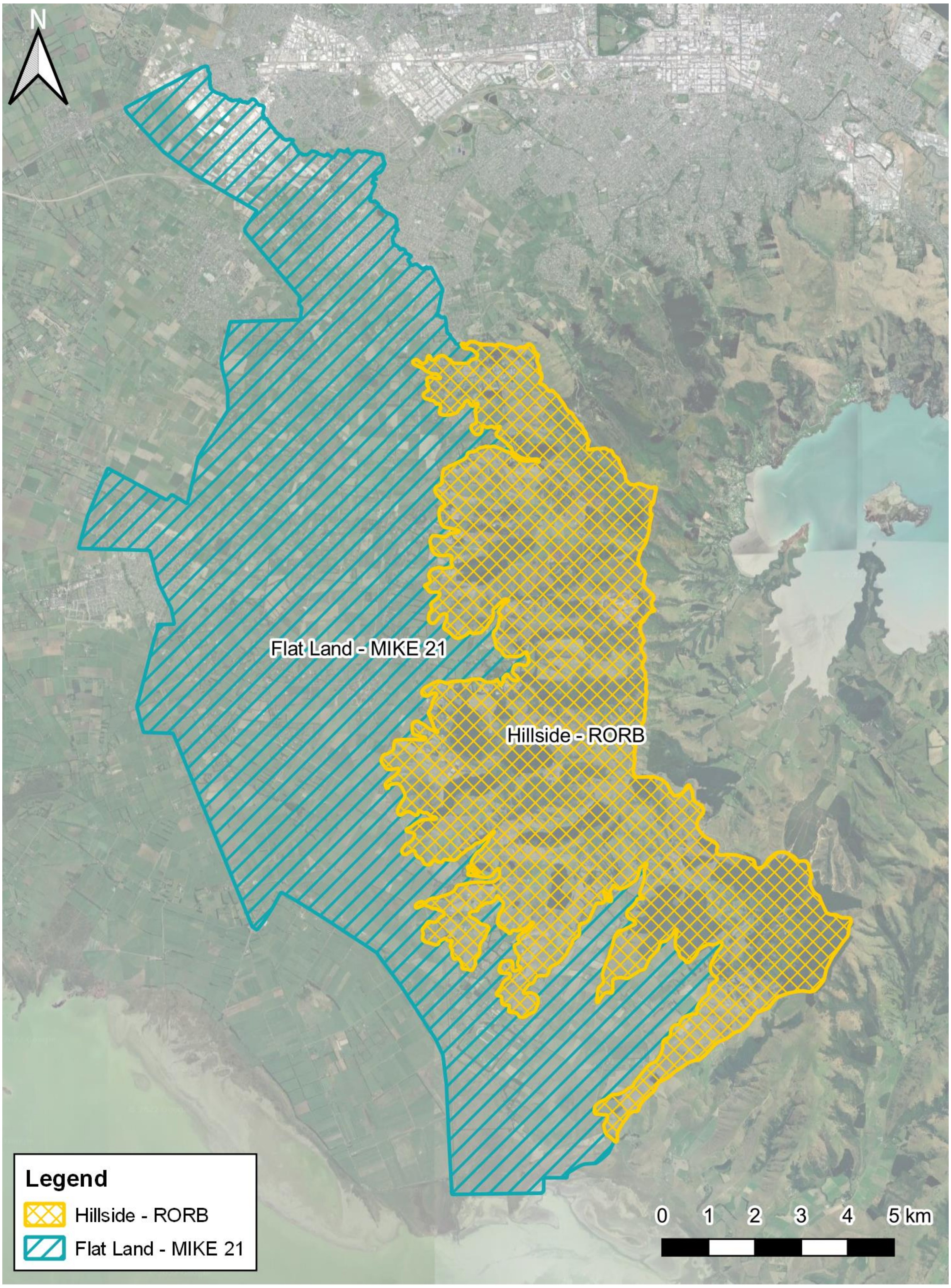


Figure 2-1 Hydrology methods applied to model extents.

## 2.2 Hydraulics Software

The HDM Halswell model consists of the three component models which are coupled together with MIKE FLOOD. Figure 2-2 shows this structure and it is listed below.

- The one-dimensional (1D) pipe flow was modelled using MIKE URBAN,
- the 1D river flow was modelled using MIKE 11, and
- the 2D overland flow was modelled using MIKE 21.

The blue arrows in Figure 2-2 represent transfer of water and momentum between the three component models accommodated by MIKE FLOOD.

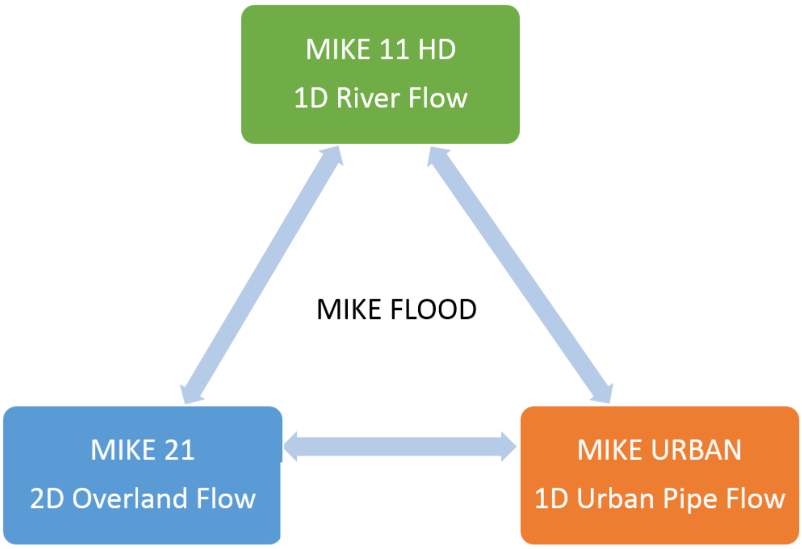


Figure 2-2 Structure of coupling in MIKE FLOOD

## 2.3 Software Versions adopted within model

The software versions that are currently used for the development of the HDM Halswell model are outlined in Table 2-1. This model has been developed over several years, and hence has moved through preceding software versions.

Table 2-1 HDM Halswell model software versions

| Model Component | Software Version |
|---|---|
| MIKE 11 | v.2020 |
| MIKE 21 | v.2020 |
| MIKE FLOOD | v.2020 |
| MIKE URBAN | v.2020 |
| RORBwin | v.6.32 (2017) |

## 2.4 Hardware Specifications

The HDM Halswell model was run on two machines to speed up the total time required for the simulations. Table 2-2 and Table 2-3 outline the hardware used to run the HDM Halswell model. The model was run using the MIKE 21 graphics processing unit function. This function was used as it increases performance through the use of multiple graphical processing units, reducing simulation run times.

Table 2-2 HDM Halswell model hardware specifications machine A

| Computation Element | Hardware Specification |
|---|---|
| Processor | Intel ® Xeon ® CPU E5-1620 v3 @ 3.50 GHz |
| Memory (RAM) | 32 GB |
| Operating System | 64-bit Operating System |
| GPU Computing Processor | NVIDIA Quadro K2200 & NVIDIA Tesla K40c |
| Core Speed | 1045 MHz (Quadro) & 745 MHz (Tesla) |
| Memory | 20 GB (Quadro) & 12 GB (Tesla) |

Table 2-3 HDM Halswell model hardware specifications machine B

| Computation Element | Hardware Specification |
|---|---|
| Processor | Intel Core i7-8700K CPU @ 3.70GHz, 3696 Mhz, 6 Core(s), 12 Logical Processor(s) |
| Memory (RAM) | 32 GB |
| Operating System | 64-bit Operating System |
| GPU Computing Processor | NVIDIA Quadro P2000 |
| Core Speed | 1076 MHz (Quadro) |
| Memory | 5 GB (Quadro) |

# 3 Model Build Introduction

## 3.1 Catchment Description

The Halswell River catchment is located to the south of Christchurch (Figure 3-1). The catchment is increasingly urbanised in the north and remains predominately rural in the south. It is characterised by:

- Spring-fed headwaters, rising in the commercial, industrial, and residential suburbs of south-west Christchurch (for example Hornby and Halswell),
- Steep, rain-fed headwaters draining from the Port Hills to the east,
- Extensive, flat alluvial plains throughout the mid-course of the river dominated by agriculture and low-density rural properties, and
- Low lying areas bordering the river where it drains into Te Waihora/Lake Ellsemere. These areas are reclaimed wetlands and prone to flooding from the lake. The river flows through a canal in its last 6km to reduce flooding issues.

The catchment area is approximately 190 $km^2$, with 65% flat and the remaining 35% is the steep hillside area of the Port Hills. Only a small percentage (15%) of the total catchment area is within the Christchurch City Council boundary (Opus, 2016). Soil type on the plains varies from free-draining Waimakariri sandy loams to poorer-draining Tai Tapu silt loams. Loess covers the slopes of the Port Hills. Overall, the soils are considered to be relatively poor draining with high groundwater levels (Opus, 2016).

The upper Halswell catchment has been identified for future residential developments, with new developments already commenced and likely to continue over several decades (Christchurch City Council, 2020).

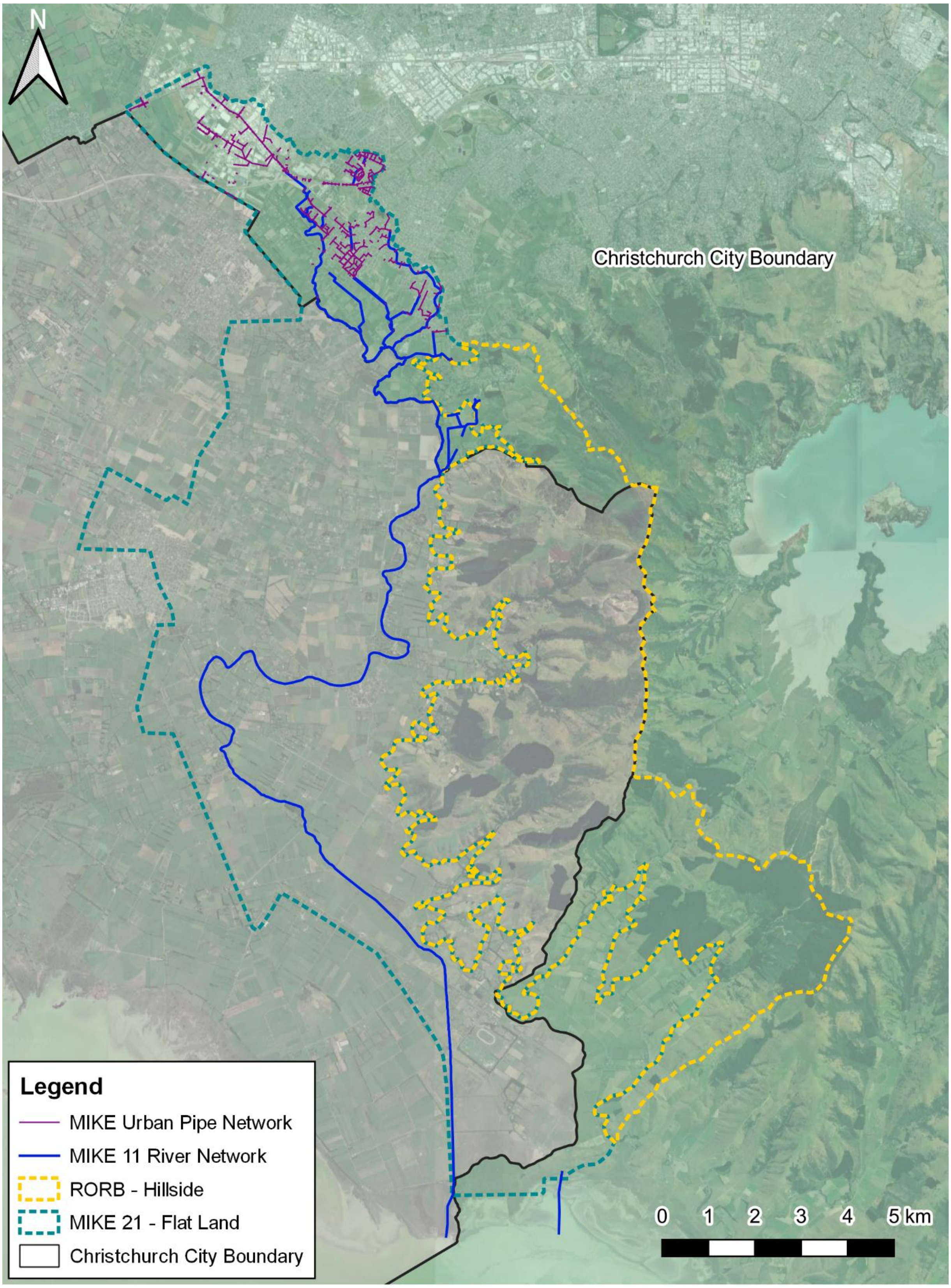


Figure 3-1 Model extent and model networks

## 3.2 Data Sources

To provide a level of detail to sufficiently quantify the effects of flooding and to support future investigation, the data required for the HDM Halswell model has been obtained from and provided by CCC and several other sources including the Stronger Christchurch Infrastructure Rebuild Team (SCIRT), Landcare Research, Environment Canterbury (ECAN) and the National Institute of Water and Atmospheric Research (NIWA). The information received has included MIKE models, drainage asset information, detailed feature survey information, CCC reports and CFM memorandums. The data has been reviewed and gaps identified. The following section provides an overview of the data provided and any infill undertaken. Table 3-1 (page 11) summaries the data inputs for the HDM Halswell model.

### 3.2.1 Documentation

The primary documents received and reviewed include, but are not limited to:

- CFM (LDRP044) Model Schematisation – Avon/Estuary. Heathcote and Sumner (GHD/AECOM, 2018),
- Christchurch City Council MIKE FLOOD Technical Specifications (DHI, 2015),
- Land Drainage Recovery Programme Survey Specification (CCC, 2015),
- Waterways, Swamps and Vegetation of Christchurch in 1856 and Baseflow Discharge in Christchurch City Streams (ECAN, 2007)
- Restoring Knights and Nottingham Stream Report, (CCC, 2014)
- Halswell River/Huritini floodplain investigation Report (ECAN, 2013),
- Halswell River Hydraulic Model Status Report (DHI, 2015),
- Knights and Nottingham Stream: Background and Modelling Log (CCC, 2013),
- Christchurch Flood Areas March 2014 (Jacobs SKM, 2014),
- Citywide Flood Modelling: “2014 Simplified” Pre-Quake Variation Description Memo (GHD, 2015),
- Citywide Flood Modelling: Proposed Technical specification for model runtimes (GHD, 2015),
- Citywide Flood Modelling: 2041 Runoff Methodology (GHD, 2015),
- Citywide Flood Modelling: Avon Calibration Approach Draft (GHD, 2017),
- LDRP044 Citywide Flood Modelling: Rain on mesh hillside hydrology/stability Memo (GHD\AECOM, 2018),
- LDRP044: Developing sub-catchment rainfall events Memo (AECOM, 2015).

### 3.2.2 Existing MIKE Models

The following MIKE models were supplied by CCC:

- Halswell River Hydraulic Model (DHI, 2015), modelled in MIKE FLOOD (1D MIKE 11/2D Mike 21).
- Halswell River/Floodplain investigation model (ECAN, 2013), modelled in MIKE FLOOD (1D MIKE 11/2D Mike 21).
- Nottingham Stream Model (CCC, 2013), model in 1D MIKE 11.
- Knights Stream Model (CCC, 2013), modelled in 1D MIKE 11.

These models were used to provide supporting information and cross reference for asset information, pipe connectivity, sub-catchments, terrain features and linkages.

### 3.2.3 Topographical Data (Rivers and Streams)

On behalf of CCC, GHD provided the processed Citywide Flood Modelling DEM developed with CCC post-quake 2011 LiDAR. For information on the development of the DEM, refer to CFM (LDRP044) Model Schematisation (GHD/AECOM, 2018). The extent of the post-quake LiDAR used in the Halswell model is shown in Figure 3-2. Beca updated the DEM in the recently developed areas in Knight Stream and Longhurst using LiDAR supplied by CCC that was recorded 2019.

River and stream cross-sectional survey were provided by CCC for:

- Halswell River,
- Nottingham Stream,
- Knight Stream,
- Halswell Outfall Drain,
- Minsons Drain,
- Cases Drain,
- Greens Stream,
- Paynes Drain,
- Quaifes Drain,
- Talbots Drain,
- Creamery Drain, and
- Creamery Ponds 1 and 2.

### 3.2.4 Topographical Data (Roads and Railways)

Terrain features for road and rail were provided by CCC and Selywn District Council (SDC) as a GIS database. The SDC dataset is clean with no identifiable spatial alignment issues. The CCC dataset needed refining due to issues with the digitising of the road centrelines and other digital inconsistencies. Methodology for refining the CCC road centre lines were as detailed in CFM (LDRP044) Model Schematisation (GHD/AECOM, 2018).

### 3.2.5 Drainage Assets

Beca downloaded stormwater pipe network information from CCC's GIS database. Supplementary data for the storm water pipe assets were provided by SCIRT from their GIS database (2015). Drawings and as-builts were provided by CCC for the following stormwater ponds:

- Halswell Junction Detention Basin,
- Creamery Ponds,
- Owaka Basin,
- Knight Stream Park Basin, and
- Murphys Drainage Basin.

Drawings of additional basin were provided in 2022. These were not incorporated into the Existing Development 2016 model.

### 3.2.6 Rainfall, River flow and Rainfall records

Spatially varying rainfall depths from NIWA's HIRDS v4 were used to create rainfall depths for each ARI, duration, and climate change emissions pathway. For the MIKE 21 rain on grid model, a 500 m spaced *.dfs2 file was generated for each rainfall event. The RORB model had the HIRDS v4 rainfall depths applied at the centroid of each sub-catchment.

Rainfall records and river flow records were collected from ECan and NIWA that were used in the model validation (see Section 6). Rainfall radar data in the form of accumulation plots was provided by CCC for the RORB model calibration.

Table 3-1 Data collected for the model

| Data Type | Source | Comment |
|---|---|---|
| CCC LiDAR 2003 | GHD on behalf of CCC | Pre-quake data. |
| CCC LiDAR 2011 | GHD on behalf of CCC | Post-quake data, compiled from a range of post-quake flyovers (Aug 2011 and Feb 2012). |
| CCC GIS network data | CCC | Asset data downloaded from CCC GIS database (2016). |
| SCIRT supplementary survey data | Stronger Christchurch Infrastructure Rebuild Team (SCIRT) | Supplementary survey data for stormwater pipe network assets (2015). |
| Cross-sectional survey data | CCC | Cross-sectional survey data collected by CCC. Where survey data was not available cross-sections were sourced from the LiDAR, as specified by CFM (LDRP044) Model Schematisation (GHD/AECOM, 2018) and WWDG (CCC, 2011). |
| Citywide Soil Drainage Map | Landcare Research/CCC | Layer outlining soil drainage across Christchurch City. |
| Stormwater retarding basins | CCC | Design drawings and as-builts of stormwater basins:<br>• Halswell Junction Detention Basin,<br>• Creamery Ponds,<br>• Owaka Basin,<br>• Knight Stream Park Basin,<br>• Mushroom Basin,<br>• Ramp Basin,<br>• Maize Maze, and<br>• Murphys Drainage Basin. |
| Halswell HIRDS v4 historic and projected rainfall depths | NIWA | Used to generate design rainfall from HIRDS v4 database. |
| Kaituna River flow record (RORB calibration) | Environment Canterbury (ECAN) | Flow record for RORB model calibration at gauge:<br>• Site 67702 - Kaituna River at Kaituna Valley Road. |
| Port Hill rainfall records (RORB calibration) | ECAN | Rainfall record for RORB model calibration at gauges:<br>• Site 326611 - Coopers Knob.<br>• Site 328711 - Kaituna Valley Road. |
| Port Hill rainfall record (RORB calibration) | NIWA | Rainfall record for RORB model calibration at gauge:<br>• Site 327804 Rainfall 4960 Chch-Akaroa Hwy. |
| Rainfall radar data (RORB calibration) | CCC | Radar dataset provided for RORB model calibration. |

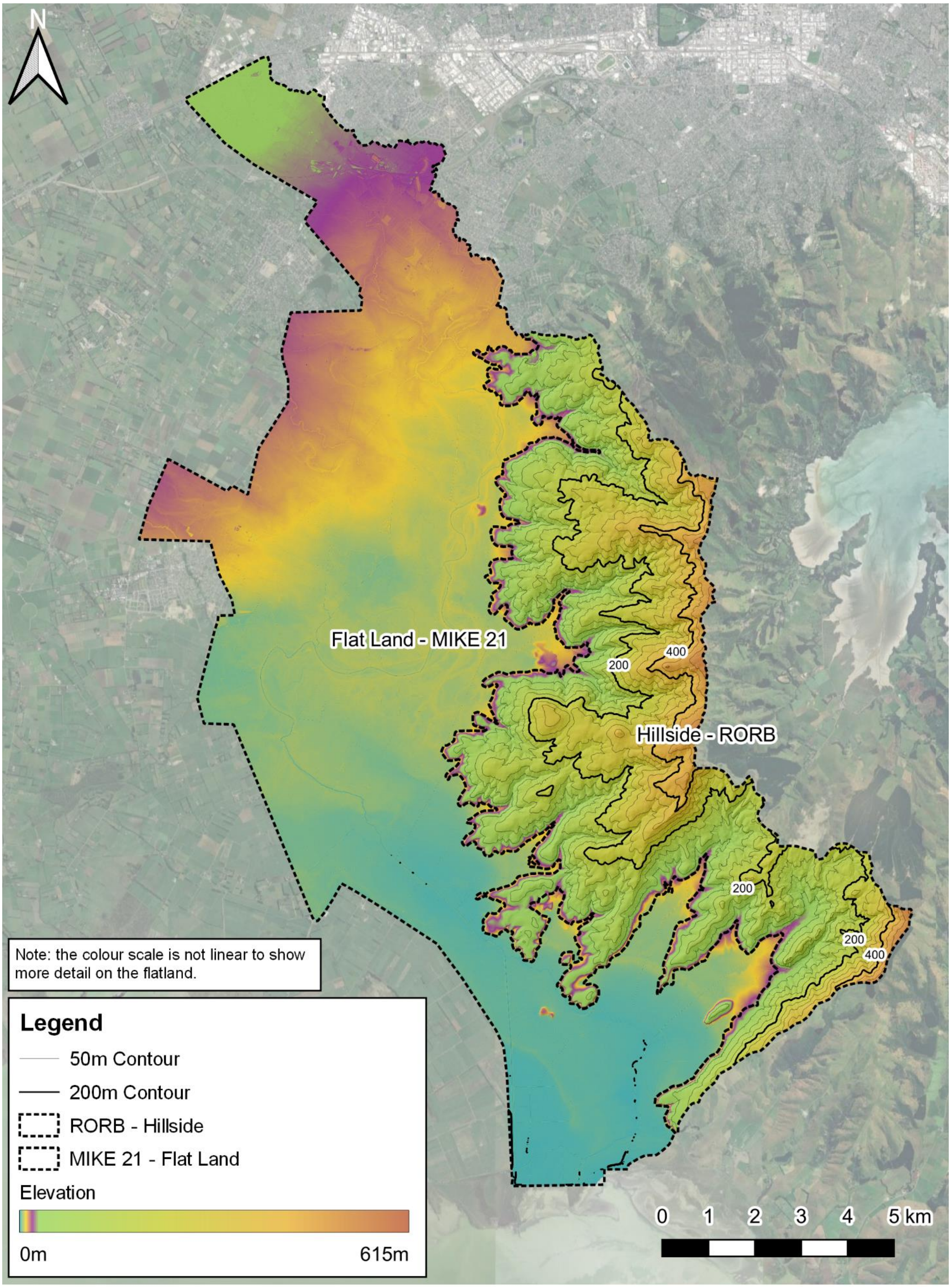


Figure 3-2 Model topography

## 3.3 Hydrology

Two hydrological methods were adopted. These were Rain on Grid (also known as direct rainfall), and RORB rainfall/runoff routing model. These approaches were applied to the Halswell catchment as shown in Figure 2-1.

Rain on Grid was adopted for the flat lands as specified by the CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). RORB was used instead of MIKE 21 to model the Port Hills due to instabilities arising in the MIKE 21 rain-on-grid method.

The following sections discuss the design rainfall, design events, base flow, evaporation, imperviousness, wetting loss, and storage loss.

### 3.3.1 Design Rainfall

As stated in Section 3.2.6, the design rainfall used is spatially varying and was sampled from NIWA's HIRDS v4. The intensity for each duration, frequency, climate change scenario and x-y coordinate was used to create a set of *.dfs2 files that contain:

- The standard triangular hyetograph profiles as prescribed in Chapter 21 of WWDG (2011). These profiles have a peak of twice their average intensity which occurs 70% through the storm. A standard normalised version is shown in Figure 3-3.
- A set of 60-hour centre-weighted nested storms. These are centre-weighted nested storms in which the depth that occurs in the middle 10 minutes matches the depth for 10-minute event in HIRDS v4 (for that point in space with that ARI). The middle 30 minutes matches the depth for 30-minute event in HIRDS v4 and so on. Overall the depth of the 60-hour nested storm matches the depth of the 60-hour event in HIRDS v4[2].

A detailed account of how these design rainfall events where created is given in HIRDSv4 data for MIKE and RORB flood modelling (Beca, 2021).

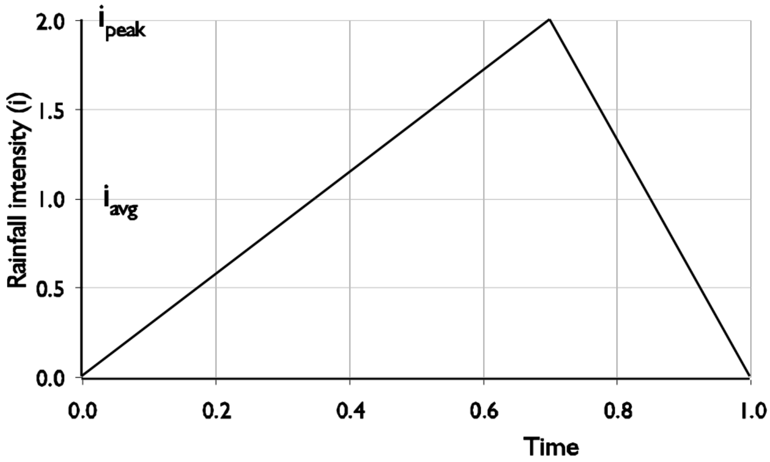


Figure 3-3 Standard dimensionless hyetograph for rainfall intensity from WWDG (2011)

[2] Actually, HIRDS v4 does not have a 60-hour duration so this has been interpolated using HIRDSv4's seven-parameter depth duration frequency model. For more detail refer to HIRDSv4 data for MIKE and RORB flood modelling (Beca, 2021).

### 3.3.2 Design Events

The HDM Halswell model was developed for the ARI events and storm durations outlined in Table 3-2, as agreed with CCC. Previous modelling by ECan identified the critical duration of the Halswell River catchment to be 60 hours.

Table 3-2 HDM Halswell model ARI events and storm durations

| ARI (years) | Storm durations (hours) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.5 | 1 | 2 | 3 | 6 | 9 | 12 | 48 | 60 | 72 | 60-hour nested |
| 10 | X | X | X | X | X | X | X | X | X | X | X |
| 50 | X | X | X | X | X | X | X | X | X | X | X |
| 200 | X | X | X | X | X | X | X | X | X | X | X |
| 500 | X | X | X | X | X | X | X | X | X | X | X |

### 3.3.3 Rain-on-Grid (Flat Land)

a. Rainfall

Rain on grid, also known as rain-on-mesh or direct rainfall, works by applying a rainfall timeseries to each active mesh element. The design rainfall was applied directly to MIKE 21 using the ‘Specified Precipitation’ function. Details on the development of the design rainfall is provided above in Section 3.3.1.

b. Imperviousness

The imperviousness of the Halswell catchment was developed by Beca from the Landcare 2012 imperious dataset provided by CCC. This data set was modified to reflect the impervious values of new roads and buildings as specified by CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). 90% imperiousness was used for all roads, excluding roads classified as ‘non-existent’, ‘proposed’ or ‘paper.’

Beca used the impervious data set to generate an impervious 2D grid file (*.dfs2). The grid file was generated with a 5 m x 5 m spacing and was applied using the ‘Bed Resistance’ function in MIKE 21 model.

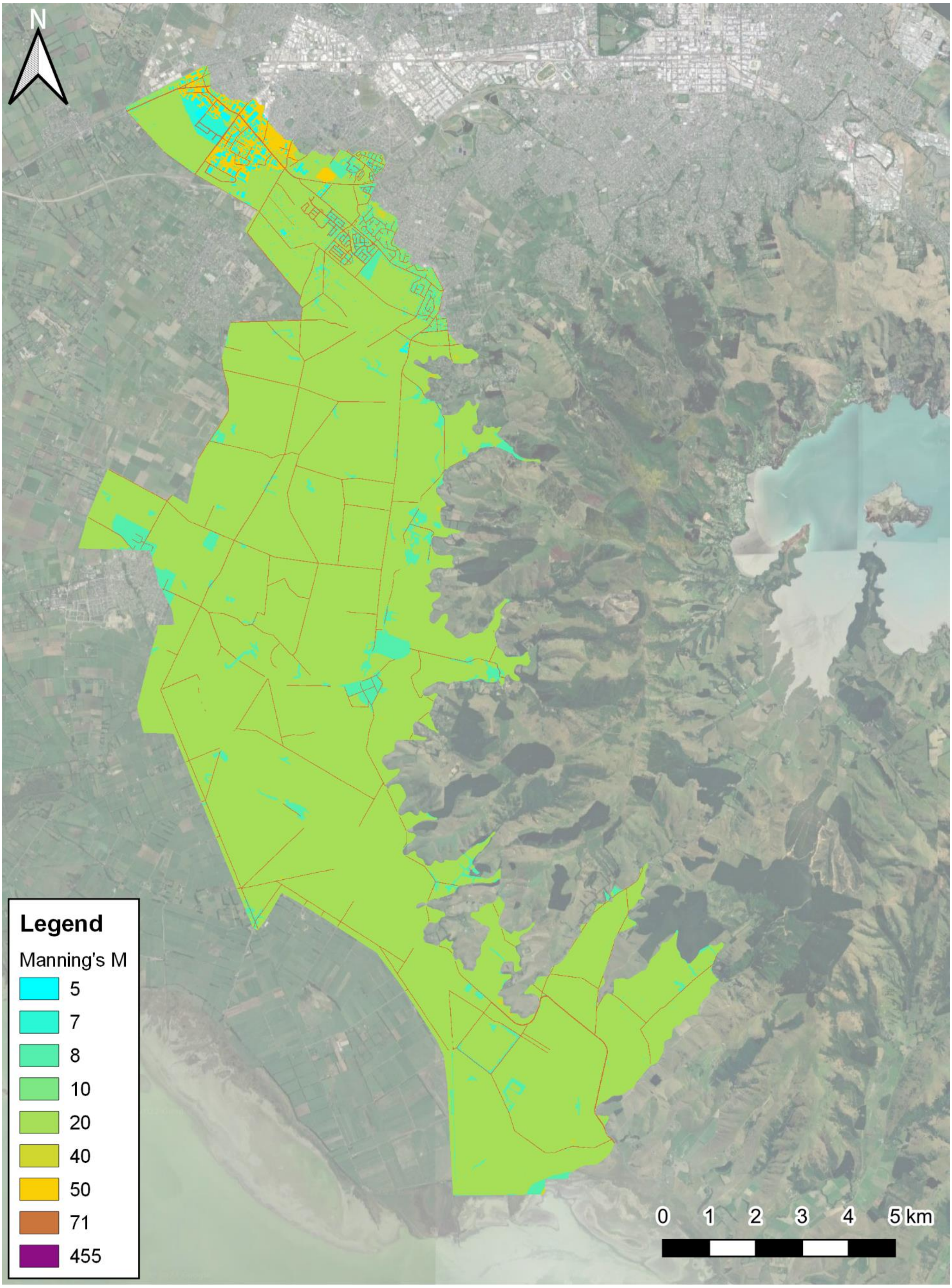


Figure 3-4 Manning's M roughness map (Existing Development 2016 model)

c. Infiltration Losses

Infiltration losses were applied as a set of 10 m x 10 m grid series (a *.dfs2 file is a grid that can change with time) in MIKE 21. Beca developed these grid series based on infiltration zones. These are areas for which infiltration is expected to be the same (any area with a single soil type and land use). These infiltration zones are shown in Figure 3-5. An initial infiltration rate, final infiltration rate and Horton's decay rate was used to create a loss time series for each infiltration zone. These were initially based on recommended values in WWDG (CCC, 2011) and CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018), however following the model validation, sensitivity tests, and peer-review, Beca and DHI decided to use the same values as those in the calibrated Heathcote model.

Beca applied the infiltration loss time series to the infiltration zones using the dfs0-to-dfs2 MIKE Zero Tool. The infiltration grids were applied in MIKE 21 using the 'Specified Evaporation' function.

The following section outlines the development of the infiltration zones (*.dfs2) and the loss time series (*.dfs0) used in the model.

d. Infiltration Zones

Beca categorised infiltration zones based on the soil-type and land-use using the soil-type map/dataset provided by Landcare (2013). The three soil drainage types used are poor, moderate, and free draining as outlined in WWDG (2011) and summarised in Table 3-3. The final infiltration zones are shown in Figure 3-5.

Table 3-3 Christchurch standard soil infiltration types (WWDG, 2011)

| Infiltration Type | Soil Description | Example of Local Soils |
|---|---|---|
| Poor | Poorly drained, low permeability | Tai Tapu silt loams and Port Hills soils |
| Moderate | Imperfectly drained, medium permeability | Kaiapoi silt loams |
| Free | Free draining, high permeability | Waimakariri silt loams |

The final infiltration parameters used for each zone are given below.

Table 3-4 Final infiltration loss parameter used in the HDM Halswell model (taken from the calibrated Heathcote model)

| Parameter | Poorly Drained | Imperfectly Drained | Well Drained |
|---|---|---|---|
| Initial (mm) | 1 | 2.5 | 3 |
| Final (mm/hr) | 1 | 1.5 | 2.5 |
| Decay rate ($s^{-1}$) | NA | 0.00004 | 0.00003 |

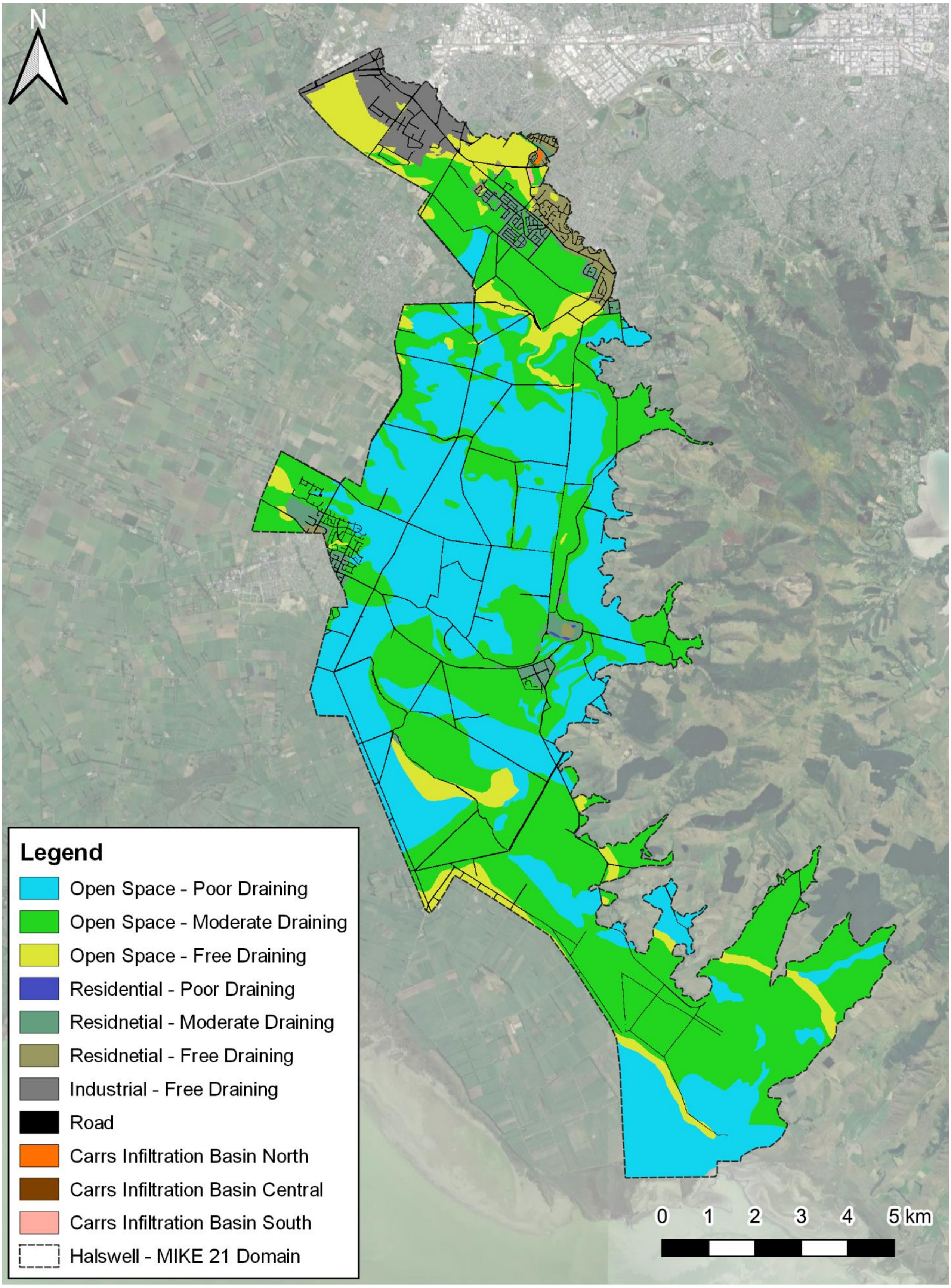


Figure 3-5 HDM Haswell model infiltration zones

### e. Infiltration Rates and Loss Time Series

For each infiltration zone, the initial and ultimate infiltration rates and Horton's decay rates were initially determined based on those recommended by WWDG (2011). Following model validation (described in section 6) and as per advice from the peer review (by DHI) lower infiltration rates were adopted which matched those used in the Heathcote catchment model. The infiltration rates were weighted based on the percentage pervious of each zone. Table 3-5 summaries the final infiltration rates and Horton's decay rates for each zone.

The loss rates in Table 3-5, were used to develop an infiltration loss time series for each of the zones. The infiltration loss time series for each zone was calculated with a 2-minute timestep and 10-minute timestep. The infiltration loss time series applicable to each design event is summarised in Table 3-6.

Figure 3-5 also shows the location of Carrs Infiltration Basin (west side of north most end of the catchment). This is an infiltration basin whose infiltration rate was set in the model based on onsite experiments undertaken by CCC engineers. The infiltration rate for the three sections of the basin were (north, central, and south) were set at a constant 1800 mm/day, 3600 mm/day, and 1200 mm/day respectively. The infiltration rates for these zones are not expected to drop throughout a storm.

Table 3-5 Summary of infiltration rates in the model

| Zone ID | Percent pervious | Initial infiltration rate $f_0$ (mm/hr) | Ultimate infiltration rate $f_c$ (mm/hr) | WEIGHTED Initial infiltration rate $f_{0WEIGHTED}$ (mm/hr) | WEIGHTED Ultimate infiltration rate $f_{cWEIGHTED}$ (mm/hr) | Horton decay rate k ($s^{-1}$) |
|---|---|---|---|---|---|---|
| Open Space – Poor Draining | 100% | 2.5 | 1.0 | 2.5 | 1.0 | 3.0E-04 |
| Open Space – Moderate Draining | 100% | 3.5 | 2.0 | 3.5 | 2.0 | 6.0E-05 |
| Open Space – Free Draining | 100% | 5 | 2.5 | 5 | 2.5 | 3.0E-05 |
| Residential – Poor Draining | 50% | 2.5 | 1.0 | 1.25 | 0.5 | 3.0E-04 |
| Residential – Moderate Draining | 50% | 3.5 | 2.0 | 1.75 | 1.0 | 6.0E-05 |
| Residential – Free Draining | 50% | 5 | 2.5 | 2.5 | 1.3 | 3.0E-05 |
| Industrial – Free Draining | 10% | 5 | 2.5 | 0.5 | 0.3 | 3.0E-05 |
| Roads – Moderate Draining | 10% | 3.5 | 2.0 | 0.35 | 0.2 | 6.0E-05 |
| Hillside | 100% | 2.5 | 1.0 | 2.5 | 1.0 | 1.5E-03 |

The infiltration loss time series were summarised in two *.dfs0 files (converted to mm/day to satisfy MIKE21): one for the 2-minute timestep series and one for the 10-minute timestep series. Each infiltration loss time series was allocated a number that matches a corresponding number for the infiltration zones in the 10 m x 10 m grid.

Table 3-6 Infiltration loss time series duration

| Event Duration | Time Step |
|---|---|
| 10 min – 2 hr | 2 min |
| 3 hr – 72 hr | 10 min |

f. Wetting Loss

A wetting loss is not compatible with rain-on-grid in MIKE 21, so none was applied. This is as specified by CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018).

g. Storage Loss

No storage losses were applied, as rain-on-grid has been adopted and it is not intended to allow for any storage loss, as specified in CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018).

### 3.3.4 RORB Model

Beca developed two RORB models for the hillside catchment of HDM Halswell model. These are shown in Figure 3-6. Two models were built based on catchment size and types:

- The Halswell CCB model was developed for the hillside catchment located within the Christchurch City Boundary (CCB), with sub-catchments ranging from 0.1 $km^2$ to 0.5 $km^2$. This provides sufficient detail for the urban/rural catchments. The Halswell CCB model sub-catchments are shown in Figure 3-7. In the figure, all RORB nodes are labelled however only nodes at the boundary of the MIKE 21 domain are inflows into the MIKE 21 model. There are also some nodes that exist in the RORB model that are within the MIKE 21 (for example, DU2 and OUTLET). These are only in the RORB for completeness and are not inflows to the MIKE 21 model.
- The Halswell 02 model was developed for the hillside area located outside the Christchurch City Boundary, with sub-catchments ranging from 0.5 $km^2$ to 5.6 $km^2$. Larger sub-catchments were used as the hillside area outside the Christchurch City Boundary is rural and does not require the same level of detail. The Halswell 02 model sub-catchments are shown in Figure 3-8.

To calibrate the two models, the Kaituna River catchment model was developed (Figure 3-9). The Kaituna catchment is a gauged rural catchment located further south around the Port Hills. It is a good representation of the characteristics of the hillside area of the HDM Halswell model.

All the RORB models were built using ArcMap and the ArcRORB tool. The RORB model-build and calibration process is outlined in the Halswell RORB Calibration Memo (Beca, 2018) located in Appendix B.

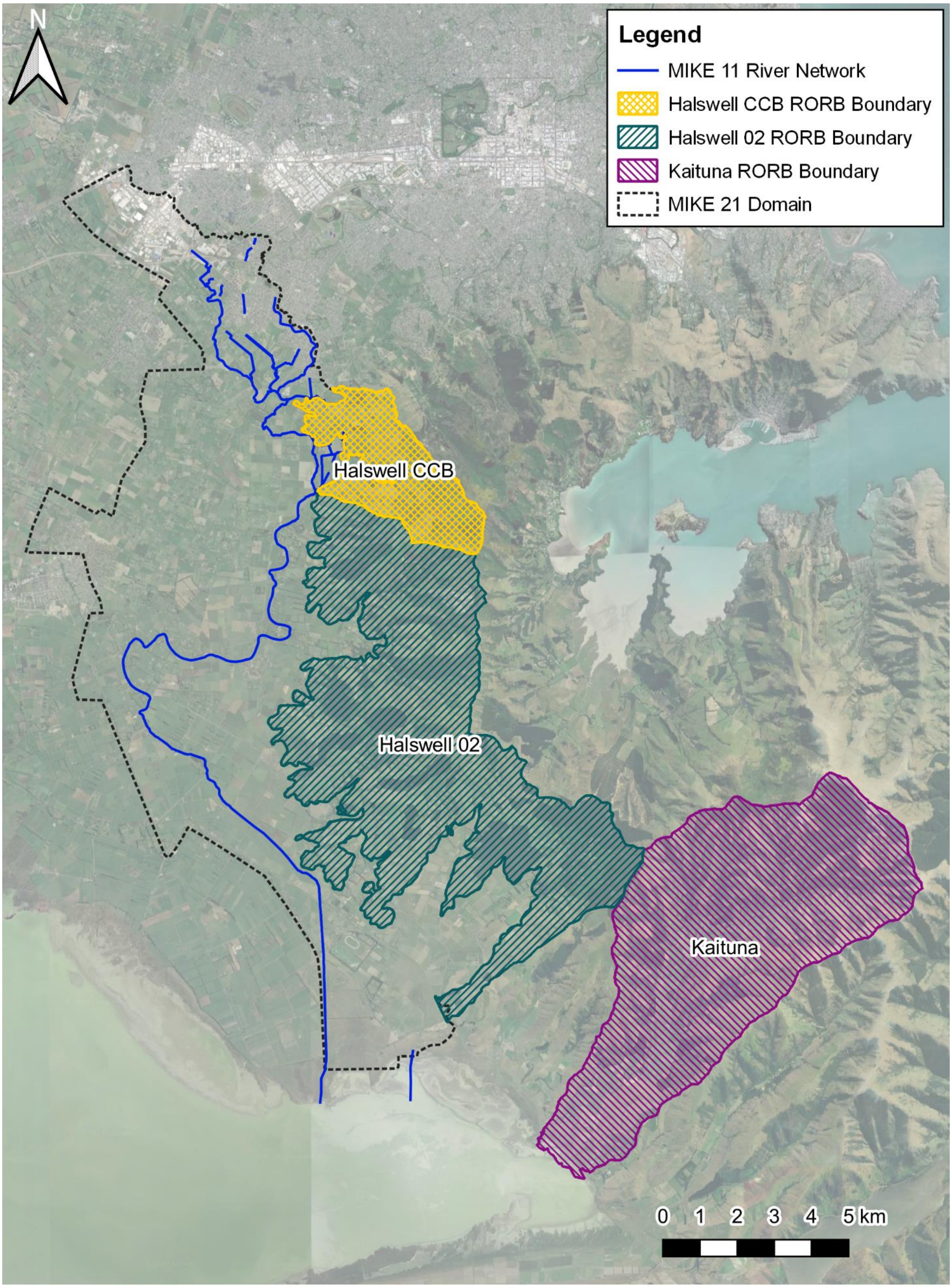


Figure 3-6 Halswell Hillside RORB catchments (Halswell CCB, Halswell 02 and Kaituna Valley)

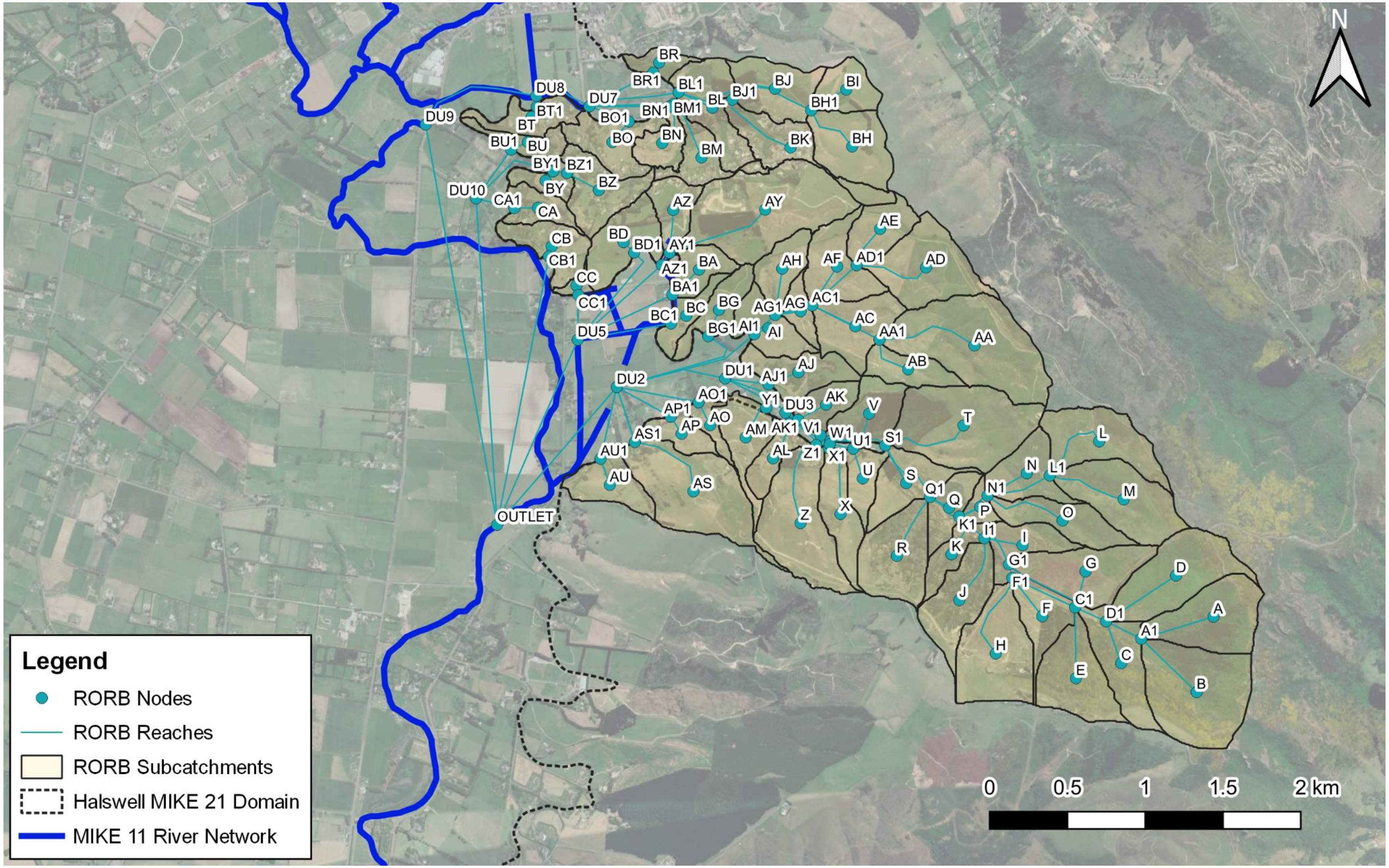


Figure 3-7 Halswell CCB RORB model sub-catchments

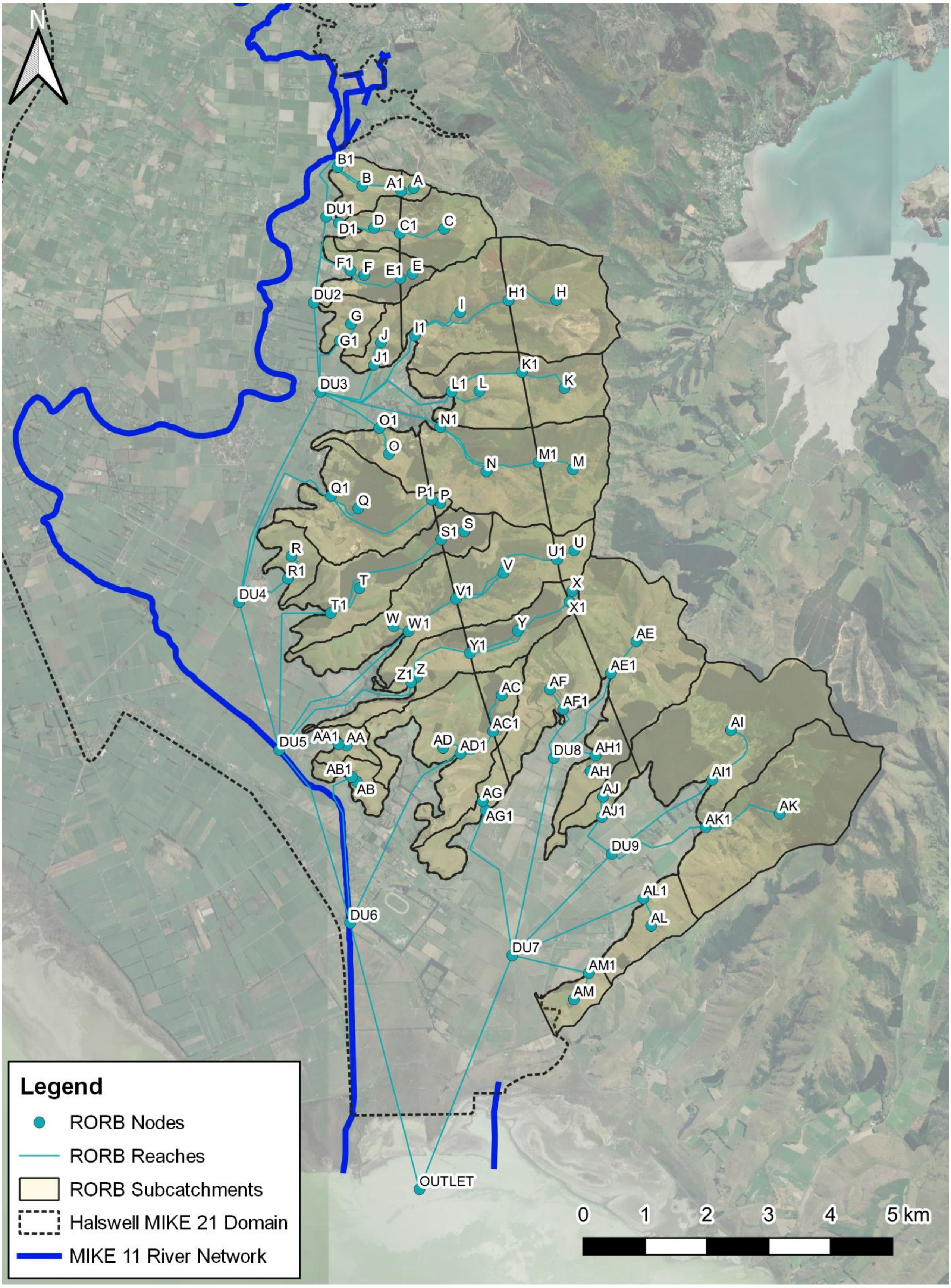


Figure 3-8 Halswell 02 RORB model sub-catchments

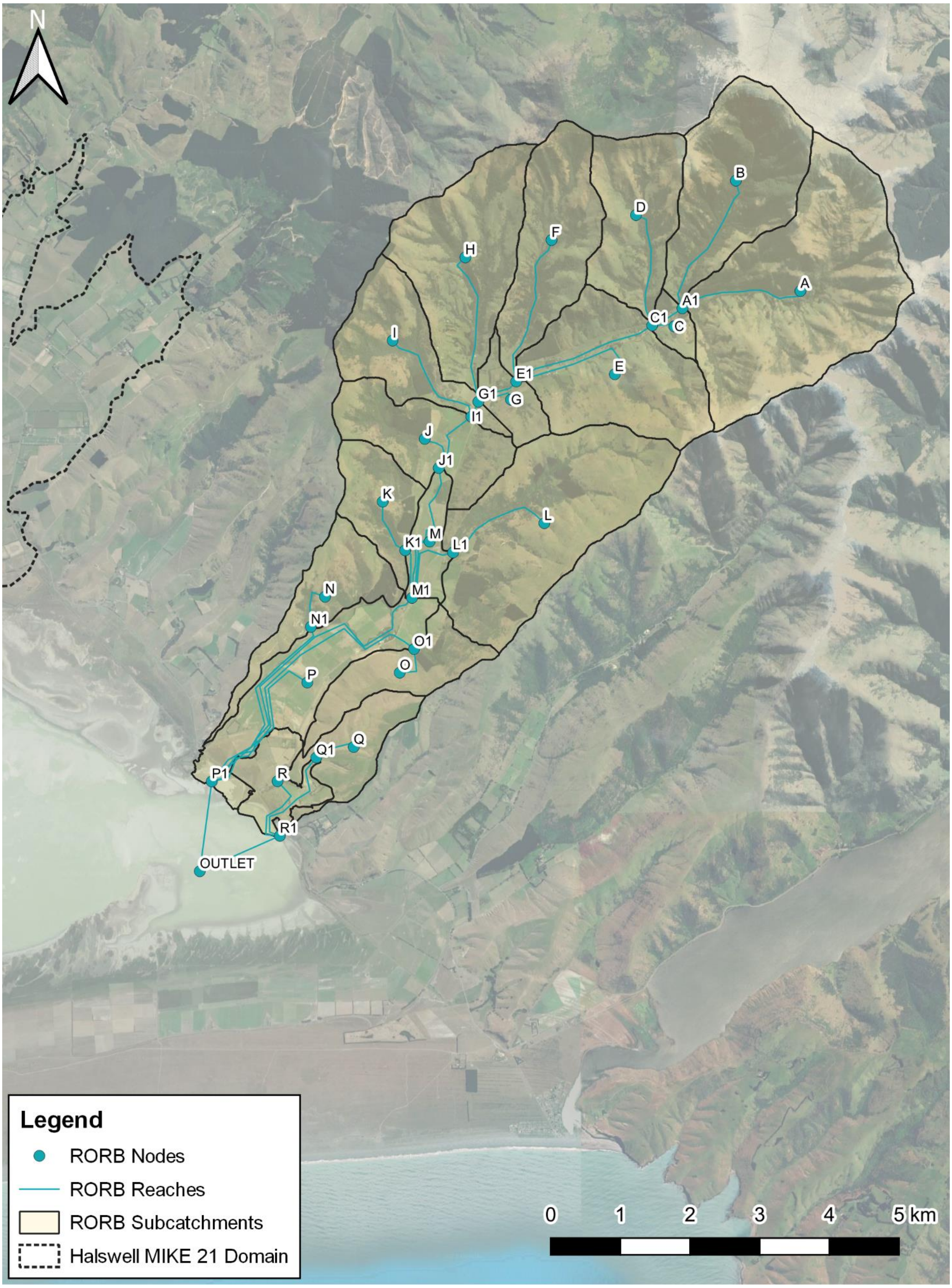


Figure 3-9 Kaituna catchment model sub-catchments

a. RORB Imperviousness

Imperviousness for the RORB models was determined based on the Christchurch City District Plan (the district plan) provided by CCC, and Chapter 21 of the WWDG (2011). The district plan was used to create district zone types for each sub-catchment. Based on the types, an average pervious and impervious percentage was applied to each sub-catchment.

The types are Living Hills and Rural – Port Hills. The Living Hills zone average effective pervious and impervious percentages are summarised in Table 3-7. The WWDG does not provide the zone average effective pervious and impervious area percentages for Rural – Port Hills zones. It was assumed that sub-catchments categorised as Rural – Port Hills would be 100% pervious.

Table 3-7 Zone average effective pervious and impervious area percentages (WWDG, 2011)

| District Zone | Pervious Area Contribution (%) | Impervious Area Contribution (%) |
|---|---|---|
| Residential: Living Hills | 50% | 90% |

The Halswell CCB model had several sub-catchments split between Living Hills and Rural – Port Hills zones. To account for Living Hills zone, the area of Living Hills was measured, and percentage imperviousness was calculated as recommended in Table 3-7. The rest of the sub-catchment was taken to be Rural – Port Hills and set to zero.

The Halswell 02 model and the Kaituna catchment model sub-catchments were categorised as a Rural – Port Hill zone and the fraction imperviousness was set to zero (100% pervious).

b. RORB Reaches

To join sub-catchments to one another reaches are required. Beca set the reach type for Halswell CCB, Halswell 02 and Kaituna catchment models as Reach Type 1. Reach Type 1 is for natural channels and was assumed for all reach types as the Halswell hillside is predominantly rural. Beca determined the slope of the reaches using the 5 m contours generated from the post-quake LiDAR which was provided by CCC.

c. RORB Calibration

Beca calibrated the Kaituna model using four rainfall events (Table 3-8). These events were selected as recommended by CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). Rainfall records for each event were collected from the Kaituna, Coopers Knob and Akaroa Highway rain gauges. A flow record was used for the Halswell River at Kaituna River. These records were provided by ECan and NIWA as listed in Table 3-1 in Section 3.2.

Table 3-8 Calibration events for the Kaituna RORB model

| Calibration Event | Event Dates |
|---|---|
| October 2000 | 12th October 2000 |
| May 2006 | 12th – 13th May 2006 |
| June 2013 | 15th – 22 June 2013 |
| March 2014 | 4th – 5th March 2014 |

Calibration was done through RORB using the design run process and parameters configured by varying routing parameters by intersection area. The process of calibration of the RORB model is provided in Halswell RORB Calibration Memo (Beca, 2018) located in Appendix B.

The RORB model parameters determined through calibration are summarised below.

d. Coefficient $k_c$

$k_c$ is an empirical coefficient that is the principal parameter of the model. A $k_c$ of 13 was applied to the Halswell hillside models as per the Kaituna model calibration. A $k_c$ of 13 is close to the recommend $k_c$ (from the RORB manual) of 13.9 and aligns the timing of peak river flows with the peak runoff for the October 2000, May 2006, and June 2013 events. The closest alignment occurs for the June 2013 event.

e. Exponent *m*

Exponent *m* is a dimensionless exponent and a measure of a catchment's non-linearity, where a value of one implies a linear catchment. A value of 0.8 was taken for *m*, as recommended by the RORB manual for ungauged catchments.

f. Losses

Loss processes were modelled with an initial loss and a continuing loss rate. These were determined through calibration of the Kaituna model. An initial loss of 17.5 mm and a continuing loss of 1.25 mm/hour were found to be suitable in the Kaituna catchment, and by extension the study catchments within the Halswell model. These values provide the closest alignment of river flows and runoff.

g. Runoff Hydrographs

The RORB models were used to generate runoff hydrographs at junction nodes located on the flat land (2D surface) boundary. Runoff hydrographs were generated for each design event (refer to Section 3.3.2). The RORB hydrographs were converted from *.csv files into *.dfs0 files and the *.dfs0 files were applied as sources in the HDM Halswell MIKE 21 Model.

## 3.4 MIKE 11

### 3.4.1 Branches

There are 24 branches modelled in the MIKE 11 component of the HDM Halswell model. These branches represent streams, field drains, open channels, two basins and the Halswell River. To model the connections to main river channel the Halswell River branch was split into three sections.

The MIKE 11 network is shown in Figure 3-10 and Figure 3-11, and includes the following branches:

- Cases Drain,
- Creamery Drain,
- Creamery Pond 1,
- Creamery Pond 2,
- Glovers Drain,
- Greens Drain,
- Halswell Junction Outfall,
- Halswell River 1,
- Halswell River 2,
- Halswell River 3,
- Knights Stream,
- Minsons Branch 1,
- Minsons Drain,
- Minsons Early Valley Drain,
- Minsons Farm Drain 2,
- Minsons Farm Drain 3,
- Minsons Farm Drain 1,
- Nottingham Stream,
- Old Channel,
- Paynes Drain,
- Quaifes Drain 1,
- Quaifes Drain 2,
- Talbots Drain 1, and
- Talbots Drain 2.

The branch “Hals.OldChannel” models the downstream end of the historic channel of the Halswell River. The branch does not capture the whole historic channel which extends further north towards Gebbies Valley and Motukarara. The rest of the historic channel is therefore modelled in the MIKE 21FM component.

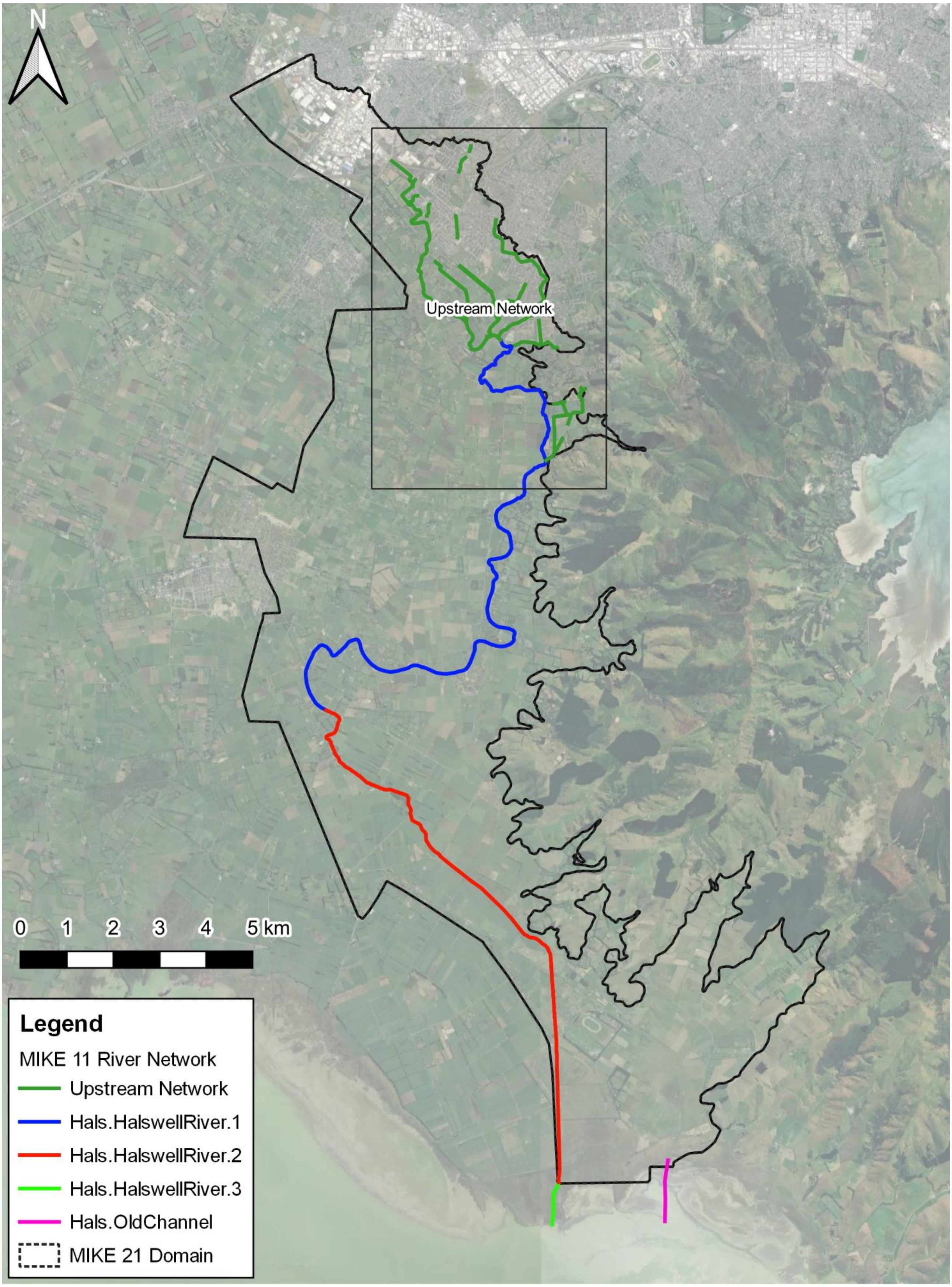


Figure 3-10 HDM Halswell MIKE 11 network – full extent

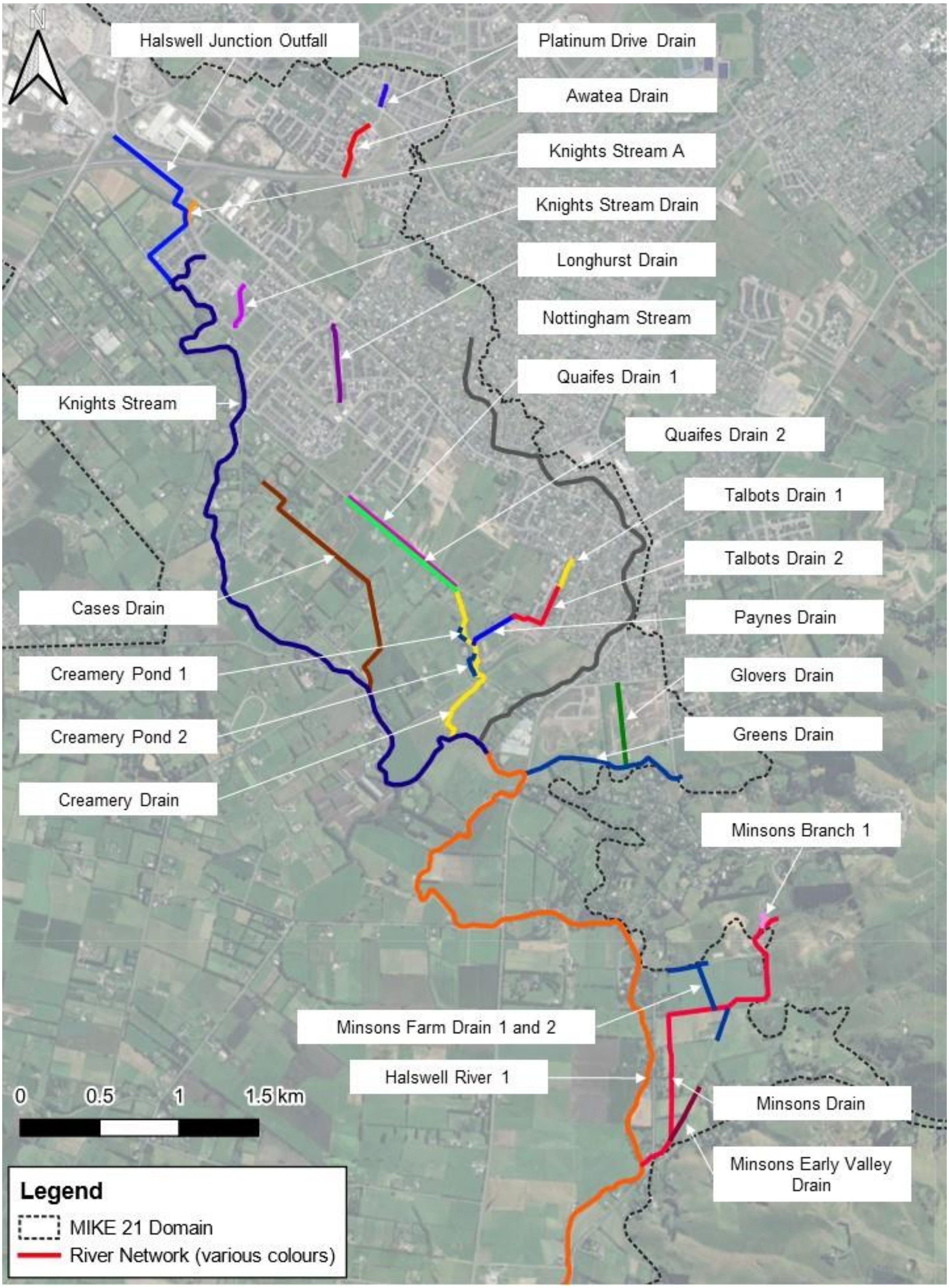


Figure 3-11 HDM Halswell MIKE 11 network - upstream network

a. Cross-sections

The MIKE 11 cross-sections used in this model were sourced from detailed ground survey provided by CCC. Where this information was not available, cross-sections were developed from CCC LiDAR. The approach to generate a MIKE 11 cross-section database from the detailed ground survey and LiDAR was as specified in CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). The cross-sections were applied to the 24 branches defined in the MIKE 11 network. The cross-sections were applied to open channels between the banks to represent 1D flow with the corresponding elements removed from the mesh to avoid double counting flow.

b. Cross-sections from detailed ground survey data

Ground survey cross-sections were imported into MIKE 11 format without interpretation or manipulation of the point-coordinates and levels. The data was reformatted from “x, y, z” format into the MIKE 11 “x, y” format.

The bank markers of the ground survey cross-sections were manually reviewed so that the top of bank represents the top of bank levels (markers 1 and 3) and that these marker locations were suitable for lateral linking to the MIKE 21 mesh.

This process was completed for the branches and detailed ground survey listed in Table 3-9.

Table 3-9 Ground Survey cross-sections used to generate MIKE 11 cross-sections

| Branch ID | Survey |
|---|---|
| Cases Drain | CCC Minson and Cases survey 2016 |
| Creamery Drain | CCC Creamery drain survey 04-08-2015 |
| Glovers Drain | CCC Glovers drain survey 04-08-2015 |
| Greens Drain | CCC Greens drain survey 04-08-2015 |
| Halswell Junction Outfall | CCC HJO drain survey 04-08-2015 |
| Knights Stream | CCC Knight Stream survey February 2013 |
| Minsons Branch 1 | CCC Minson and Cases survey 2016 |
| Minsons Drain | CCC Minson and Cases survey 2016 |
| Minsons Early Valley Drain | CCC Minson and Cases survey 2016 |
| Minsons Farm Drain 2 | CCC Minson and Cases survey 2016 |
| Minsons Farm Drain 3 | CCC Minson and Cases survey 2016 |
| Minsons Farm Drain 1 | CCC Minson and Cases survey 2016 |
| Nottingham Stream | CCC Nottingham Stream 1996[3] |
| Paynes Drain | CCC Paynes drain survey 04-08-2015 |
| Quaifes Drain 1 | CCC Quaifes drain No 1 survey 04-08-2015 |

[3] Note: there is more up to date (2013) survey for Nottingham Stream, however it has errors that need to be resolved. In the ED2016 model the 1996 survey was used. This has been recorded as an anomly in the ED2016 model record. The 1996 survey was qualitatively checked against the 2013 survey and it is broadly similar.

| Branch ID | Survey |
| --- | --- |
| Quaifes Drain 2 | CCC Quafies drain No 2 survey 04-08-2015 |
| Talbots Drain 1 | CCC Talbots drain 1 survey 04-08-2015 |
| Talbots Drain 2 | CCC Talbots drain 2 survey 04-08-2015 |

i. Cross-section from LiDAR

Cross-sections were generated for the Halswell River and Old Channel branches, from the pre-quake and post-quake LiDAR provided by CCC. Cross-sectional points were sampled every 1 m along the cross-section line and spaced every 10 m along the Halswell River and Old Channel. The initial cross-sections were generated with a width of 30 m, which were then adjusted to reflect and capture wider or narrower sections of these water courses.

The cross-section inverts were identified as the lowest elevation point (or mid-point of the lowest elevation points). The initial top-of-bank markers were identified based on the maximum elevation differences to the true left and true right of the invert. These marker positions were then further refined to match the position of the MIKE 21/MIKE 11 block-out extents, thus producing bank marker positions symmetrical on the centreline.

LiDAR cross-sections were checked against survey cross-section supplied by ECan. These checks were mainly in the lower Halswell river and showed that LiDAR represented the channel reasonably well given the intent of this model to show flooding within the CCC limits.

Further information on the generation of cross-sections from LiDAR is provided in Section 5.2.2 of CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018).

ii. Cross-sections from Drawings

Creamery pond retarding basin cross-sections were taken from as-builts for ponds 1 and pond 2. These cross-sections were imported into the model without interpretation and/or manipulation of the levels. The data was converted into the required MIKE 11 “x, y” format.

iii. Transitional cross-sections

Transitional cross-sections were used in the HDM Halswell model to smooth abrupt changes in flow conveyance between the network branches and to remove the risk of the MIKE 11 computation of structure Q-H relationship failing. Transitional cross-sections were copied from the wider downstream branch and placed as the last cross-section in the incoming stream.

### 3.4.2 Naming Convention and Data Flagging

The naming convention and data source flagging for rivers, cross-sections and structures in the MIKE 11 model was as specified in Section 5.8, Section 5.9 and 5.10 of CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018).

For example:

- river branches “Hals.QuaifesDrain.1”, “Hals.QuaifesDrain.2” and “Hals.HalswellRiver.2”,
- cross-sections “Survey.sect_160.07”, “Bridge_US.SF 1280”, “LIDAR2019.INTERPOLATED.3”,
- structures “26 John Patterson Drv_Weir”, and “23 Halswell Ryans Bridge”.

The naming convention for cross-sections has many different forms depending on the source of the data so only a subset of examples is provided here.

## 3.5 MIKE URBAN

### 3.5.1 Pipe Network

The stormwater reticulation network was modelled in MIKE URBAN. The HDM Halswell MIKE URBAN model network is shown in Figure 3-12, and was developed from CCC and SCIRT GIS. The network was simplified by excluding pipes smaller than 300mm. 'Short' pipes (less than 10m) and manholes that are close together were merged except sump leads. Figure 3-12 also shows the locations of basins in the MIKE 21 model, these are discussed in section 3.6.1.

### 3.5.2 Culverts

Culverts were modelled in the MIKE URBAN model for the cross drainages for roads, such as Christchurch Southern Motorway and Halswell Junction Road, and the culverts connecting upstream and downstream open channels at Longhurst and Knight Stream Park developments.

### 3.5.3 Naming Convention and Asset Flagging

The naming convention and data source flagging for the MIKE Urban network was as specified in Section 6.10 of CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018).

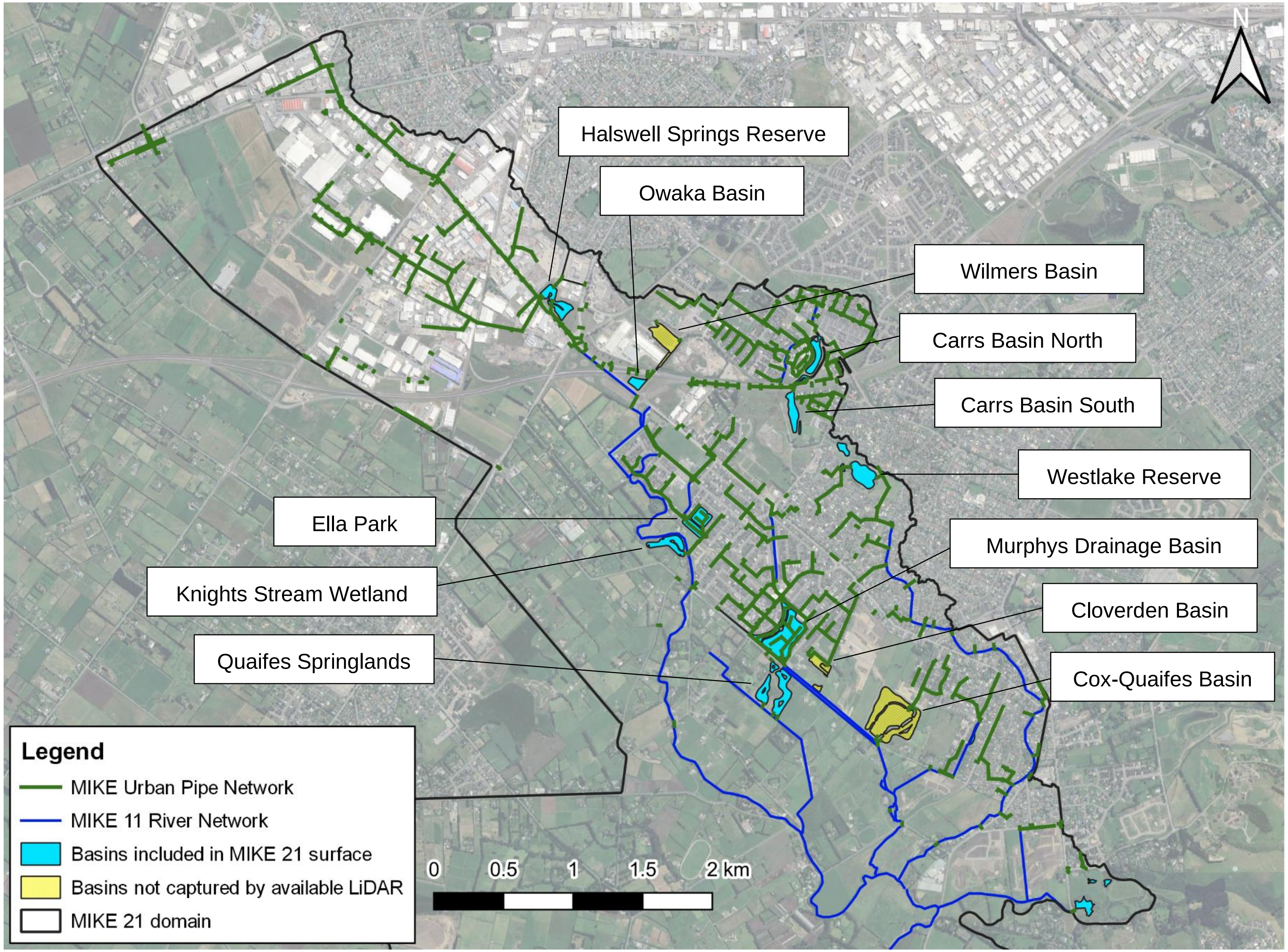


Figure 3-12 HDM Halswell MIKE Urban Network, basins in and out of the Existing Development 2016 model

## 3.6 MIKE 21FM

### 3.6.1 Mesh

The mesh developed for the HDM Halswell model was based on Section 4 of CFM (LDRP044) Model Schematisation (GHD/AECOM, 2018) and was generated for the flat land of the Halswell catchment. The specific methodology used to generate the mesh is outlined in the Beca Christchurch City Council's Citywide Modelling Methodology Memo (2017) located in Appendix C. The HDM Halswell model mesh is a triangular flexible mesh and was generated with the criteria summarised in Table 3-10.

Table 3-10 HDM Halswell Mesh Generation Criteria

| Mesh Generation Criteria | Value |
|---|---|
| Maximum element area | 200 $m^2$ |
| Smallest allowable angle | 26 |
| Maximum number of nodes | 10,0000,000 |

a. Road and Rail

Roads and rail networks in the Halswell catchment were defined by the 'three row method' outlined in Section 4.3.1 of CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). This method defines the road as three rows of triangles, with the central row used to represent the centreline crown. For more information on mesh development, see the Beca Road Point Methodology Memo (2021) located in Appendix D. The elevation of the road uses an adaption of the DHI's 'depth correction' function to control the elevation of each element. For more information on depth correction, see Section 3.6.2 below.

b. Mitigation Basins

A total of 8 mitigation basins were represented in the MIKE 21 mesh in the Existing Development 2016 model, including:

- Halswell Junction Detention Basin,
- Owaka Basin,
- Carrs Basin,
- Knight Stream Park Basin,
- Knight Stream Wetland,
- Murphys Drainage Basin,
- Westlake Ponds,
- Halswell Quarry Pond, and
- Quaifes Road Springlands.

The locations of these basins are shown in Figure 3-12. Some basins were not captured by the available LiDAR so could not be included in the model.

The extent of each basin was defined by triangles for both the top-of-bank and the toe-of-bank. The top-of-bank uses elevations taken from LiDAR. The toe-of-bank locations were drawn at 8m from the inside of the top-of-bank location to avoid small triangle formation and avoid negative bias in the basis volumes. Elevations for the toe-of-bank were set at drawing/as-built/LiDAR levels. The top-of-basin element could not always be relied to accurately capture the crest due to being close to the road elements, so dykes were also added. The insertion of dykes better defines the crest level and is as per the CWM scheme. The dykes' crests use 2018-2019 LiDAR. Wilmers, Cox-Quaifes, Mushroom, and Owaka Bains dyke levels were set using 2020-2021 LiDAR as they were constructed after the 2018-2019 LiDAR was flown (and they were added to the existing development 2021 model later in the project). A resulting example is shown in Figure 3-13.

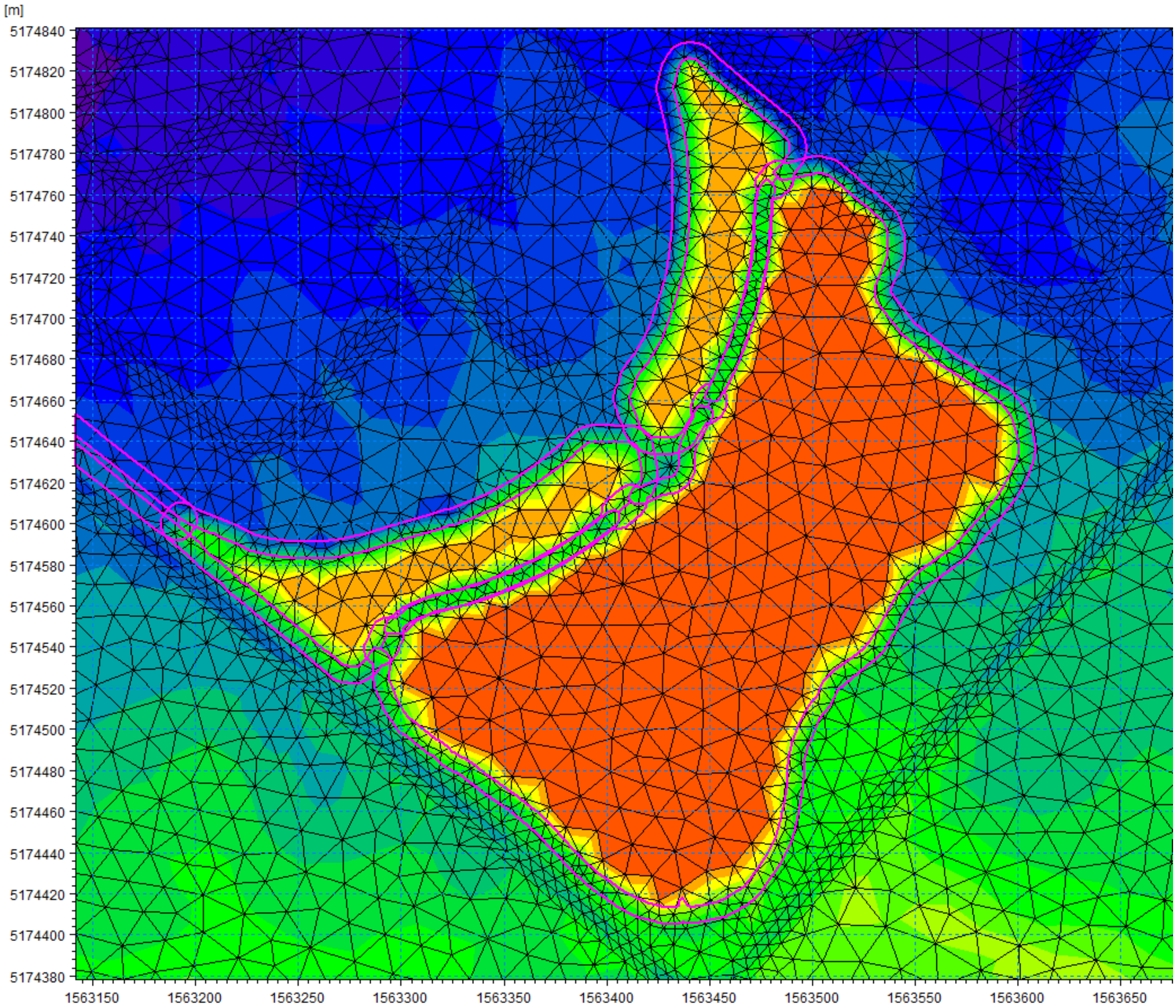


Figure 3-13 Example of basin definition in HDM Halswell model – Murphys Drainage Basin

### 3.6.2 Depth correction

Usually with a flexible mesh, the bathymetry is applied to the mesh elements by assigning an elevation value based on an average of their corner vertices. In this case, the mesh elevation was set through the 'depth correction' function. The 'depth correction' function assigns a given elevation to each flexible mesh element centroid and eliminates the use of the element vertex elevations.

The depth correction file, a *.dfsu, was developed from a digital elevation model (DEM) by overlaying the mesh triangle areas in GIS and using zonal statistics to calculate the average elevations from the DEM for each triangle mesh element. For the detailed process, see the Beca Depth Correction Memo (2021) located in Appendix E.

The DEM surface was generated based on the CCC LiDAR data that was modified to include a variety of real physical features, as outlined in the Section 4.4 of CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). The processed surface is referred to as the Z4. The Z4 for the Existing Development 2016 model was provided by CCC and GHD on 7 February 2017. It was updated by GHD using 2018-2019 LiDAR within the Christchurch City Boundary provided by Beca on 5 July 2021. The resulting layer was provided back to Beca by GHD on 17 August 2021.

# 4 Model Data Summary and Computation Parameters

The key parameters used within each MIKE model component of the HDM Halswell model (MIKE 11, MIKE 21, and MIKE Urban) are discussed in the following sections. These parameters are critical to the model's behaviour and results.

## 4.1 MIKE 11

The MIKE 11 model of the HDM Halswell model consists of 5 primary files:

- Network File (*.nkw11),
- Cross-section File (*.xns11),
- Boundary Data File (*.bnd11),
- HD Parameters File (*.hd11), and
- MIKE 11 (MIKE11.ini file).

A summary of the parameters and inputs for each of these MIKE 11 files is provided in the sections below.

### 4.1.1 Network File

The extent of the MIKE 11 network is shown in Figure 3-10 in Section 3.4, with the components of the network summarised below in Table 4-1.

Table 4-1 Network file breakdown for MIKE 11 model component of HDM Halswell model

| Type | Input Parameter | Existing Development 2021 Model |
|---|---|---|
| Network | Branches | 29 |
| | Points | 1481 |
| Structures | Weirs | 54 |
| | Culverts | 60 |

a. Points

Points defined were specified with x and y coordinates, with chainage types a mixture of both "user" and "system defined". "User" was selected at the start and end of branches and where needed to specify specific coupling locations.

b. Branches

Branches of the model were developed based on the CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018), where:

- All branches were set as "regular," which is the normal branch type used within MIKE 11 and comprises several calculation points that are defined by the cross-sections specified in the cross-section file.
- The dx values for all branches in MIKE 11 were set to be within a range of 1 to 8 times the element length in MIKE 21. For some branches, this requirement could not be met due to long MIKE 11 structures. Here additional cross sections were added (interpolated) so there are sufficient H points (head points) for proper coupling of lateral links.

Table 4-2 summarises the upstream (US) and downstream (DS) chainages, topographical identifications and dx values of the HDM Halswell model branches.

Table 4-2 HDM Halswell Model MIKE 11 Branches

| Branch ID | Topo ID | US Chainage (m) | DS Chainage (m) | dx (m) |
|---|---|---|---|---|
| Hals.CreameryDrain | CREAMERY DRAIN CCC survey 04-08-2015 | 0 | 1173 | 100 |
| Hals.GloversDrain | GLOVERS DRAIN CCC survey 04-08-2015 | 0 | 517 | 100 |
| Hals.GreenDrain | GREEN DRAIN CCC survey 04-08-2015 | -22 | 1057.5 | 100 |
| Hals.HalswellRiver.1 | Topo | 0 | 17048.7 | 100 |
| Hals.KnightsStreamA | FEB2019A | 0 | 148 | 50 |
| Hals.NottinghamStream | SURVEY 1996 | 0 | 3873 | 50 |
| Hals.PaynesDrain | Paynes Drain CCC survey 04-08-2015 | 0 | 312 | 100 |
| Hals.QuaifesDrain.1 | Quaifes Drain No 1 CCC Survey 04-08-2015 | 0 | 859 | 100 |
| Hals.QuaifesDrain.2 | Quaifes Drain No 2 CCC Survey 04-08-2015 | 0 | 883 | 100 |
| Hals.TalbotsDrain.1 | TALBOTS DRAIN 1 CCC survey 04-08-2015 | 0 | 196.7 | 100 |
| Hals.TalbotsDrains.2 | TALBOTS DRAIN 2 CCC survey 04-08-2015 | 0 | 432 | 100 |
| Hals.HalswellJunctionOutfall | HJO CCC survey 04-08-2015 | 0 | 1323.6 | 120 |
| Hals.HalswellRiver.3 | Couple | 0 | 1000 | 100 |
| Hals.OldChannel | Couple | 0 | 500 | 100 |
| Hals.HalswellRiver.2 | XS from ECan LiDAR | 17050 | 29979 | 100 |
| Hals.MinsonsDrain | CCC Minson and Cases Survey 2016 | 988 | 3256 | 100 |
| Hals.MinsonsFarmDrain.1 | CCC Minson and Cases Survey 2016 | 0 | 186 | 100 |
| Hals.MinsonEarlyValleyDrain | CCC Minson and Cases Survey 2016 | 0 | 360 | 100 |
| Hals.MinsonsBranch.1 | CCC Minson and Cases Survey 2016 | 30 | 104 | 100 |
| Hals.MinsonsFarmDrain.2 | CCC Minson and Cases Survey 2016 | 0 | 343 | 100 |
| Hals.MinsonsFarmDrain.3 | CCC Minson and Cases Survey 2016 | 0 | 179 | 100 |
| Hals.CasesDrain | CCC Minson and Cases Survey 2016 | 0 | 1693 | 100 |
| Hals.CreameryPonds.1 | DRAWINGS | 0 | 77 | 100 |
| Hals.CreameryPonds.2 | DRAWINGS | 0 | 149 | 100 |

c. Culverts

A total of 232 culverts were modelled in MIKE 11. They were modelled as closed sections and structures as specified in CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018), with the culvert geometries ranging from Circular, Rectangular, Irregular – Depth-Width Table and Cross Section DB.

The Manning's '*n*' for the majority of culverts was set of 0.013. Default values for the head loss factors were used to model the culverts in MIKE 11 and are summarised in Table 4-3.

Table 4-3 Culverts and bridges head loss factors for HDM Halswell MIKE 11 Model

| Head Loss Factor | HDM Halswell MIKE 11 Culverts/Bridges |
|---|---|
| Inflow Positive | 0.5 |
| Outflow Positive | 1 |
| Free Flow Positive | 1 |
| Bends Positive | 0 |
| Inflow Negative | 0.5 |
| Outflow Negative | 1 |
| Free Flow Negative | 1 |
| Bends Negative | 0 |

d. Weirs

A total of 54 weirs were modelled in MIKE 11 and were modelled as Regular and Broad Crested Weir with Level-Width geometries. Default values for the head loss factors were used to model the weir in MIKE 11 and are summarised in Table 4-4.

Table 4-4 Weir Head Loss Factors for HDM Halswell MIKE 11 Model

| Head Loss Factor | HDM Halswell MIKE 11 Weir |
|---|---|
| Inflow Positive | 0.5 |
| Outflow Positive | 1 |
| Free Flow Positive | 1 |
| Inflow Negative | 0.5 |
| Outflow Negative | 1 |
| Free Flow Negative | 1 |

e. Bridges

A total of 28 single-span bridges were modelled as culverts in MIKE 11. The culvert structure was used for the bridges as specified by CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). The culvert function forms a uniform cross-section of the span of the bridge, where the bridge's geometries were set as Circular, Irregular – Depth-Width Table or Cross-section DB.

The Manning's '*n*' for the bridges ranged from 0.013 to 0.024. Manning 'n' value was determined based on the bridge's material, the stream beds roughness, and the bridge opening cross-section shape.

Default values for the head loss were used for all the bridges except for Culvert 8 (Hals.KnightStream Chainage 3392 diameter 1200 mm) and are summarised in Table 4-3. For Culvert 8, the inflow positive and inflow negative factors were set to 0.7.

### 4.1.2 Cross sections

The MIKE 11 cross sections developed were applied to the 24 branches defined in the MIKE 11 network. The cross-sections were applied to the branches between the banks to represent 1D flow and were spaced to represent the conveyance along the watercourse. The key cross-sectional properties are specified below.

a. Topo IDs

The Topo IDs linking the cross-sections with the MIKE 11 Branches are outlined in Table 4-2.

b. Cross-section ID

Cross-section ID were named as outlined in Section 5.91 of CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018). This specific naming convention has been used to help identify and track cross-section sources and any modifications made to the cross-sections.

c. Cross-section properties

The following properties were applied for all cross-sections in the MIKE 11 of the Halswell model:

- The section types were set as open (for open channels).
- The radius types were set to “Total Area, Hydraulic Radius.”
- No coordinates were applied.
- No correction of the x- coordinates was applied.
- The morphological model has not been used.
- Resistance numbers were set using uniform distribution, with a Manning’s ‘*n*’ of 0.035.

### 4.1.3 Boundary Data

The 41 boundary conditions used in the MIKE 11 for the HDM Halswell model are summarised in Table 4-5. This table outlines the location of the boundary (branch name and chainage), the boundary nature (open, point source, distributed and so on) and the boundary type (namely, inflows, water levels and so on) and the corresponding boundary values.

Boundary 12 and Boundary 34 represent the downstream lake level boundary at Lake Ellesmere. This open water level boundary is a timeseries that varies from 9.543 m RL to 11.293 m RL and was taken from the Halswell River/Huritini Floodplain Investigation (ECan, 2013). The average reoccurrence interval for a lake level of this elevation is 200 years (annual exceedance probability of 0.5%).

Boundary 11 represents the flow of water into the start of Greens drain (from the MIKE 21 model). The water level was set to equal the invert level of the cross-section at chainage -22 m however the actual water level is set based on the current level in MIKE 21 and flow is transferred between the models each timestep. At this location (approximately 80 m north of the Halswell Quarry entrance along Kennedys Bush Rd) there is a wetland which is modelled in MIKE 21. The same type of boundary is also used at boundaries 19, 21, 25, and 34.

Table 4-5 MIKE 11 boundary conditions for HDM Halswell Model

| No. | Branch Name | Chainage | Nature | Type | Data Type | Value |
|---|---|---|---|---|---|---|
| 1 | Hals.AwateaDrain | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 2 | Hals.AwateaDrain | 240 | Point Source | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 3 | Hals.AwateaDrain | 337 | Point Source | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 4 | Hals.AwateaDrain | 364 | Open | Water Level | Constant Water Level | 27.7m CDD |
| 5 | Hals.CasesDrain | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 6 | Hals.CreameryPonds.1 | 56.75154326868 | Point Source | Inflow | Constant Discharge | 0.05 $m^3/s$ |
| 7 | Hals.CreameryPonds.2 | 0 | Point Source | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 8 | Hals.GloversDrain | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 9 | Hals.GreenDrain | -22 | Open | Water Level | Constant Water Level | 20.45 m CDD |
| 10 | Hals.HalswellJunctionOutfall | 0 | Open | Inflow | Constant Discharge | 0.02 $m^3/s$ |
| 11 | Hals.HalswellRiver.1 | 5612 | Distributed Source | Inflow | Constant Discharge | 0.05 $m^3/s$ |
| 12 | Hals.HalswellRiver.3 | 1000 | Open | Water Level | Water Level Time Series | 9.543 m CDD to 11.293 m CDD |
| 13 | Hals.KnightsStream | 530 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 14 | Hals.KnightsStream | 2150 | Point Source | Inflow | Constant Discharge | 0.07 $m^3/s$ |
| 15 | Hals.KnightsStream | 3260 | Point Source | Inflow | Constant Discharge | 0.15 $m^3/s$ |
| 16 | Hals.KnightsStream | 4730 | Point Source | Inflow | Constant Discharge | 0.12 $m^3/s$ |
| 17 | Hals.KnightsStream | 5500 | Point Source | Inflow | Constant Discharge | 0.081 $m^3/s$ |
| 18 | Hals.KnightsStreamA | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 19 | Hals.KnightsStreamA | 148 | Open | Water Level | Constant Water Level | 28.03 m CDD |
| 20 | Hals.KnightStreamDrain | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 21 | Hals.KnightStreamDrain | 268 | Open | Water Level | Constant Water Level | 23.15 m CDD |
| 22 | Hals.LonghurstDrain | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |

| No. | Branch Name | Chainage | Nature | Type | Data Type | Value |
|---|---|---|---|---|---|---|
| 23 | Hals.LonghurstDrain | 88 | Point Source | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 24 | Hals.LonghurstDrain | 160 | Point Source | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 25 | Hals.LonghurstDrain | 474 | Open | Water Level | Constant Water Level | 23 m CDD |
| 26 | Hals.MinsonEarlyValleyDrain | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 27 | Hals.MinsonsBranch.1 | 30 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 28 | Hals.MinsonsDrain | 988 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 29 | Hals.MinsonsFarmDrain.1 | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 30 | Hals.MinsonsFarmDrain.2 | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 31 | Hals.MinsonsFarmDrain.3 | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 32 | Hals.NottinghamStream | 0 | Open | Inflow | Constant Discharge | 0.016 $m^3/s$ |
| 33 | Hals.OldChannel | 0 | Open | Inflow | Constant Discharge | 0.1 $m^3/s$ |
| 34 | Hals.OldChannel | 500 | Open | Water Level | Water Level Time Series | 9.543 m CDD to 11.293 m CDD |
| 35 | Hals.Platinum.Drive.Drain | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 36 | Hals.Platinum.Drive.Drain | 78 | Point Source | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 37 | Hals.Platinum.Drive.Drain | 120 | Open | Water Level | Constant Water Level | 28.442 m CDD |
| 38 | Hals.QuaifesDrain.1 | 0 | Open | Inflow | Constant Discharge | 0.021 $m^3/s$ |
| 39 | Hals.QuaifesDrain.2 | 0 | Open | Inflow | Constant Discharge | 0.02 $m^3/s$ |
| 40 | Hals.TalbotsDrain.1 | 0 | Open | Inflow | Constant Discharge | 0.01 $m^3/s$ |
| 41 | Hals.TalbotsDrains.2 | 0 | Point Source | Inflow | Constant Discharge | 0.02 $m^3/s$ |

#### 4.1.4 HD Parameters

The hydrodynamic (HD) parameters provided are used for setting supplementary data for model runs. Most of the parameters were set to default values, with many parameters set as global for the entire model. The HD parameters used within the MIKE 11 for the HDM Halswell model are summarised below.

a. Default Computational Parameters

Table 4-6 outlines the default computation parameters used in the MIKE 11 for the HDM Halswell model. These parameters are essential for the computational scheme of MIKE 11 and defaults values were used for all computation parameters except for Delta.

In the HDM Halswell model, Delta was set to 0.9 (default value is 0.5). The Delta value can significantly influence model dynamics as Delta affects the time-centring of the gravity term in the momentum equation. A high value of Delta has a dampening effect. Delta was set to 0.9 to stabilise the linkages from MIKE 11 to MIKE FLOOD. The MIKE 11 manual recommends a Delta of 0.85 for MIKE FLOOD models with small timesteps. The adopted value of 0.9 is slightly higher.

Table 4-6 MIKE 11 HD default computational parameters for the HDM Halswell model

| Parameter | Value |
|---|---|
| Delta | 0.9 |
| Delhs | 0.01 |
| Delh | 0.1 |
| Alpha | 1 |
| Theta | 1 |
| Eps | 0.0001 |
| Dh Node | 0.01 |
| Zeta Min | 0.1 |
| Stru Fac | 0 |
| Inter1Maz | 10 |
| Nolter | 1 |
| MaxIterSteady | 100 |
| Froudemax | -1 |
| FroudeExp | -1 |

b. Initial Conditions

The initial water depth has been set globally at 0 m in the MIKE 11 component.

c. Wind

Wind factors were not considered in the HDM Halswell model.

d. Bed Resistance

A global Manning's '*M*' of 30 has been applied for the MIKE 11 network of the HDM Halswell model. However, since the cross sections use local Manning's n values the global parameter is not used in the simulation.

e. Wave Approximation

The wave approximation uses a High Order Fully Dynamic approximation.

f. Quasi Steady Parameters

The default values for the quasi-steady simulation computational parameters and no steady state options were used in the MIKE 11 model. Table 4-7 outlines the quasi-steady simulation computational parameters used in the MIKE 11 HD file.

Table 4-7 MIKE 11 HD quasi-steady simulation computational parameters

| Parameter | Value |
|---|---|
| Relax | 0.5 |
| Beta_Limit | 1E-8 |
| Fac_0 | 2.5 |
| Qconv_factor | 0.001 |
| Hconv_factor | 0.01 |
| Min_Hconv_In_Branch | 1E-5 |
| Q-struc_Factor | 0.005 |
| H_stop | 0.0001 |
| Steady State Option | Not Used |

g. Heat Balance

Heat exchange was not used.

h. Stratification

Stratification parameters used in the MIKE 11 of the HDM Halswell model are summarised in Table 4-8. These are the default parameters in MIKE 11. There are no stratification branches in the Halswell model so the parameters are not used in the model. Stratification branches are defined in the 'branches' section of the *.nwk11 file.

Table 4-8 MIKE 11 HD Stratification Parameter for HDM Halswell model

| Parameter | Value |
|---|---|
| No of Layers | 10 |
| Densities Calculated | Yes |
| Turbulence model in fluid | k-eps model |
| Turbulence model Viscosity | 0.003 |
| Turbulence model at bed | Drag coefficient |
| Richardson numbers correction | Yes |
| Baroclinic Pressure – Factor | 1 |
| Baroclinic Pressure – Local Bed Slope | 0 |
| Convection / Advection – Factor horizontal momentum | 1 |
| Convection / Advection – Factor vertical momentum | 1 |
| Convection / Advection – Factor advection | 1 |
| Dispersion – Factor horizontal viscosity | 1 |
| Dispersion – Factor vertical viscosity | 1 |

i. Groundwater Leakage

Groundwater leakage was not used in the MIKE 11 part of the HDM Halswell model.

j. Flood Plain Resistance

The default value of -99 for flood plain resistance was applied in the MIKE 11 model. A value of -99 indicates that the flood plain resistance should be calculated from the raw data of the cross-sections database (*.xns11 file).

k. Encroachment

Encroachment simulations were not used in the MIKE 11 of the HDM Halswell model.

### 4.1.5 MIKE 11.Ini file

The MIKE11.ini file offers functionality to adjust settings within the numerical engine of MIKE 11. There are 48 variables that can be changes for hydrodynamic simulations. One default was changed for the MIKE 11 component of the HDM Halswell model.

The water level exceedance factor was set to 8 (usually it is 4). This parameter defines the factor at which MIKE 11 will halt and give an error due to a river reach being too deep compared to its bank height. For example, if a bank is 1m tall MIKE 11 will halt with an error when the depth reaches 8 x 1m = 8m. When the water depth exceeds the hight of its banks four times this usually indicates an error in the model set up or that the MIKE 11 model has become unstable. In HDM Halswell model however, Greens Drain has a realistic water level that exceeds the height of its bank by more than 4 times (but less than 8 times) for the large events. These are the 0.5% and 0.2% annual exceedance probability events.

## 4.2 MIKE URBAN

MIKE Urban computation was setup for network run with long term statistics (LTS) simulation. The LTS modelling of the network, with dynamic wave simulation, allows for a time-efficient long-term simulation, covering a long simulation period while providing the computation of relevant statistics for the variables of interest.

### 4.2.1 Pipes

The pipes included in the model are 300 mm diameter and greater. Pipes with lengths less than 10 m were merged, where possible, to simplify the network.

All the pipes in the MIKE Urban model are circular.

a. Roughness

The material classification for the pipes was provided by the CCC and SCIRT GIS data sets. The pipes in Halswell catchment are either concrete or plastic. Concrete was applied as the default classification for pipes where no material classification was provided.

The roughness values for concrete and plastic pipes were taken from Section 6.9 of by CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018) and are summarised in Table 4-9. The plastic pipe roughness provided was a homogenised roughness value for all plastic pipe types used in the CCC GIS data set.

The formation for hydraulic friction losses were set as Mannings explicit.

Table 4-9 Pipe Roughness Parameters for HDM Halswell MIKE Urban Model

| Material Type | Manning's '*M*' | Manning's '*n*' |
|---|---|---|
| Concrete | 76.9 | 0.013 |
| Plastic (PE, PVC/uPVC, PP) | 90.9 | 0.011 |

### 4.2.2 Manholes

All manholes were specified as 'Normal' type. Manhole head losses were set to 'MOUSE Classic (Engelund)' or 'MOUSE_Sharp_Edged', with a $K_m$ coefficient set to 0.5. Zero soakage (infiltration) was applied.

a. Manhole Diameter

The manhole sizes were provided by CCC and SCIRT GIS data sets. Where manhole diameter was unavailable, the standard manhole sizes were used as a default. These standard sizes, based on connecting pipe sizes, are summarised in Table 4-10.

Table 4-10 Standard Manhole Diameter used in HDM Halswell MIKE Urban Model

| Connecting Pipe Size (mm) | Standard Manhole Diameter (m) |
|---|---|
| < D900 | 1.05 |
| D900 to D1200 | 1.35 |
| D1200 to D1350 | 1.5 |
| > D1350 | 2.3 |

b. Ground Level

The ground level for all manholes was set from the elevation values from the MIKE 21 mesh.

c. Invert Level

Invert levels were taken from CCC GIS system and checked against SCIRT data. As the HDM Halswell model is in Christchurch Drainage Datum, inverts were adjusted to the datum by raising levels by 9.04 m.

As specified in CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018), negative and zero grades in the MIKE URBAN pipelines were removed by raising the upstream invert level of the pipe. Wherever there was a greater than 200 mm difference between the upstream and downstream invert levels, it was assumed that the negative gradient is real and was hence left in the model.

d. Dummy Manholes

Dummy manholes were used to for the culverts connecting to the open channels. The dummy manhole was a standard diameter selected based on the culvert size (as outlined above). The inlet losses for the culverts were set to ‘MOUSE_Sharp_Edged’ with a $K_m$ set to 0.5.

### 4.2.3 Outlets

Outlets that discharge to a river (represented in MIKE 11) or into flood plain (2D surface in MIKE 21) were represented as outlets in the MIKE Urban network. Outlet energy losses were set to ‘MOUSE Classic (Engelund)’ with a $K_m$ set to 0.5.

## 4.3 MIKE 21FM

The MIKE 21 model of the HDM Halswell model was set up as a hydrodynamic triangular flexible mesh model. The MIKE 21 model of the HDM Halswell model uses the following key files:

- Mesh file (Hals_ED2016.mesh)
- Depth correction file (Hals_ED2016.dfsu)
- Roughness file (Roughness_citywide_Hals_postEQ_ED2016_5m.dfs2)
- Infiltration files (for example, Infiltration_Hals_ED2016_01to24hr_2min_mmday.dfs2)
- Rainfall files (for example, RAIN_Hv4_0200ARI_001HR_HISTORICAL.dfs2)
- Sources files (for example, Halswell_002_Hv4_200ARI_002HR_HISTO.dfs0)
- Initial condition file (Hals_ED2016_IC_DepthUV.dfsu)

### 4.3.1 Timestep and Solution Technique

The maximum timestep used in the MIKE 21 model was 0.5 second intervals. MIKE 21 uses an explicit variable timestep that requires a Courant-Friedrich-Lévy condition below 1 to be stable.

The solution technique used in the HDM Halswell MIKE 21 model for shallow water technique was low order fast algorithm scheme. The parameters used in the solution technique are outlined in Table 4-11. The Courant-Friedrich-Lévy (CFL) was set to the default value of 0.8.

Table 4-11 Solution technique parameters for HDM Halswell MIKE 21 model

| Parameter | | Value |
|---|---|---|
| Shallow Water Technique | Time Integration | Low order, fast algorithm |
| | Space discretization | Low order, fast algorithm |
| | Minimum Time Step | 0.02 sec |
| | Maximum Time | 0.5 sec |
| | Critical CFL number | 0.8 sec |
| Transport Equation | Minimum Time Step | 0.01 sec |
| | Maximum Time | 0.25 sec |
| | Critical CFL number | 0.8 sec |

### 4.3.2 Depth

Depth correction file, as described in Section 3.6.2, was applied in the process outlined in the Beca Depth Correction Memo (2021) located in Appendix E.

### 4.3.3 Drying, flooding, and wetting depths

Drying and wetting depths were set to 1 mm, 3 mm, respectively. These values were advised by DHI and CCC.

### 4.3.4 Eddy Viscosity

Eddy viscosity in the HDM Halswell model was set to constant, with a value of 0.1 $m^2/s$.

### 4.3.5 Bed resistance

A 5 m by 5 m roughness grid file (dfs2) was generated to apply the roughness (Manning's '*M*') values associated with the land use classifications in the HDM Halswell model 2D domain. The roughness values were derived from the values provided in Table 3 of the CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018) and the land use classification provided from various sources, including Landcare data, CCC City Plan Zoning, CCC Street Centreline and CCC Building Footprints. Composite roughness values were developed and applied the residential and commercial/industrial classification based on an assessment of the extent of various elements (driveways, lawns, gardens and so on) in sample areas.

In the HDM Halswell MIKE 21 model, the roughness grid was applied as a Manning's number that was constant in time and varying in space.

### 4.3.6 Precipitation Evaporation

The 2D design rainfall grids, described in Section 3.3.3a, were applied using the Precipitation function in MIKE 21. The rainfalls were applied as specified precipitation, varying in time and domain and with a soft start interval of 0 sec (no soft start).

### 4.3.7 Infiltration

The 2D infiltration grids, described in Section 3.3.3c, were applied using the Evaporation function in MIKE 21. This function sits under Precipitation - Evaporation in MIKE 21. The infiltration grids were applied as varying in time and domain with a soft start interval of 0 sec (no soft start).

### 4.3.8 Culverts

A total of 20 culverts have been modelled in MIKE 21. These culverts have a range of culvert geometries including circular, rectangular, irregular, and level-width.

The Manning's '*n*' for all the culverts was set to 0.013. Default values for the head loss factors were used for all the MIKE 21 culverts. These are summarised in Table 4-12.

Table 4-12 Culverts and bridges Head Loss Factors for HDM Halswell MIKE 21 Model

| Head Loss Factor | HDM Halswell MIKE 11 Culverts/Bridges |
|---|---|
| Inflow Positive | 0.5 |
| Outflow Positive | 1 |
| Free Flow Positive | 1 |
| Bends Positive | 0 |
| Inflow Negative | 0.5 |
| Outflow Negative | 1 |
| Free Flow Negative | 1 |
| Bends Negative | 0 |

### 4.3.9 Sources Nodes

Source nodes have been used to apply the runoff hydrographs from the RORB rainfall runoff model, as described in Section 3.3.4. There is a total of 57 source points applied to the MIKE 21 grid, as shown in Figure 4-1. These 57 source points represent the location of the RORB nodes located on the hillside/flat land boundary and that will provide the runoff onto the 2D surface.

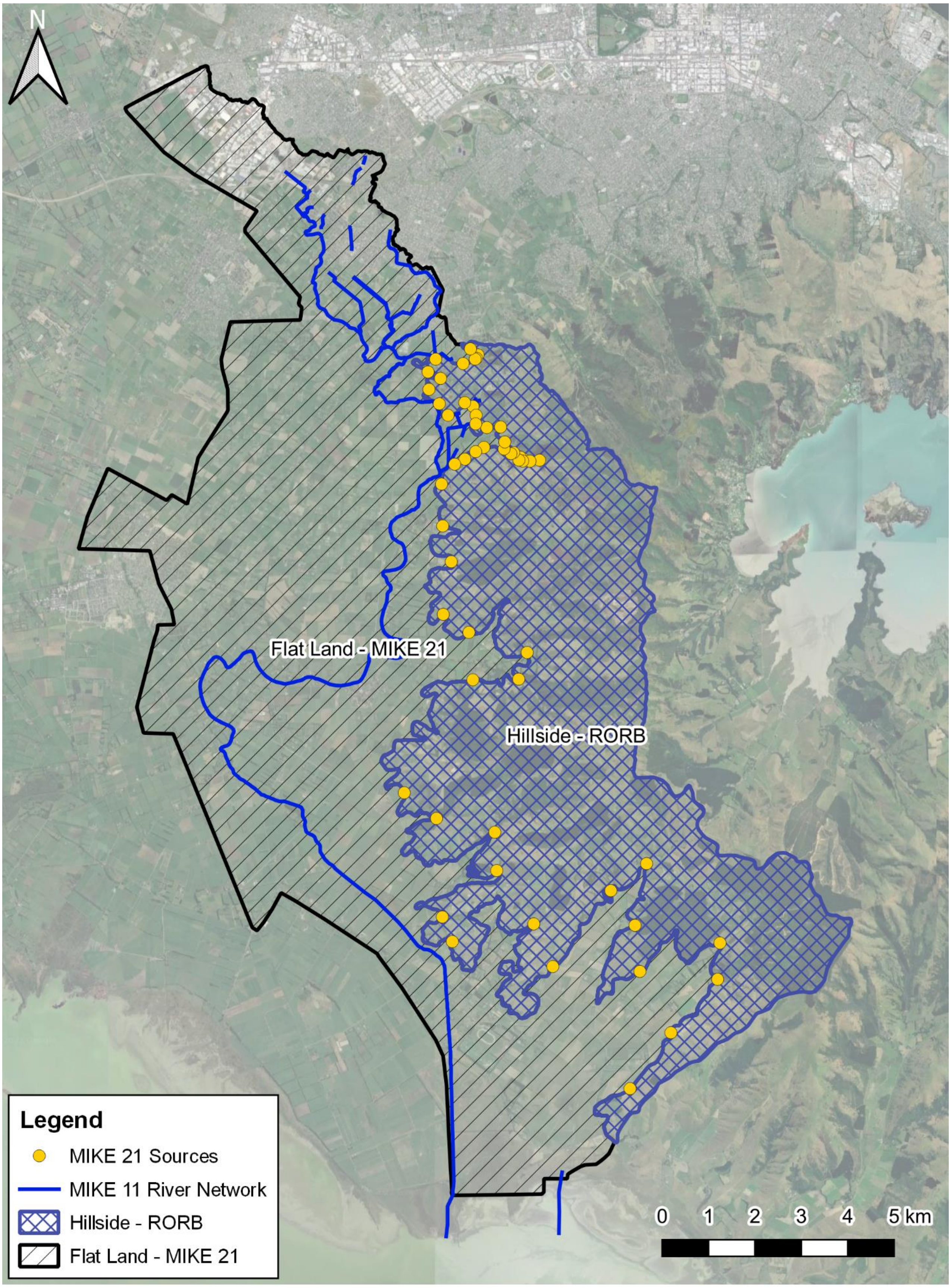


Figure 4-1 HDM Halswell MIKE 21 RORB Hydrograph Source Nodes

#### 4.3.10 Other Parameters

Wind, Coriolis forcing, ice coverage, tidal potential and wave radiation were not included as part of the Halswell MIKE 21 model.

#### 4.3.11 MIKE 21 FM Boundary

Most of the edges of the MIKE 21 model have land boundaries. These boundaries do not allow flow into or out of the model and are hence good for walls or areas of the model that remain dry. Initial results showed water was unrealistically pooling against the outside boundary in some locations. In these locations a water level boundary was created to allow water to flow out of the model. To encourage stability at these water level boundaries the depth correction layer (bathymetry) was lowered to be approximately 200mm below the level of the boundary. Figure 4-2 shows the location of these boundaries.

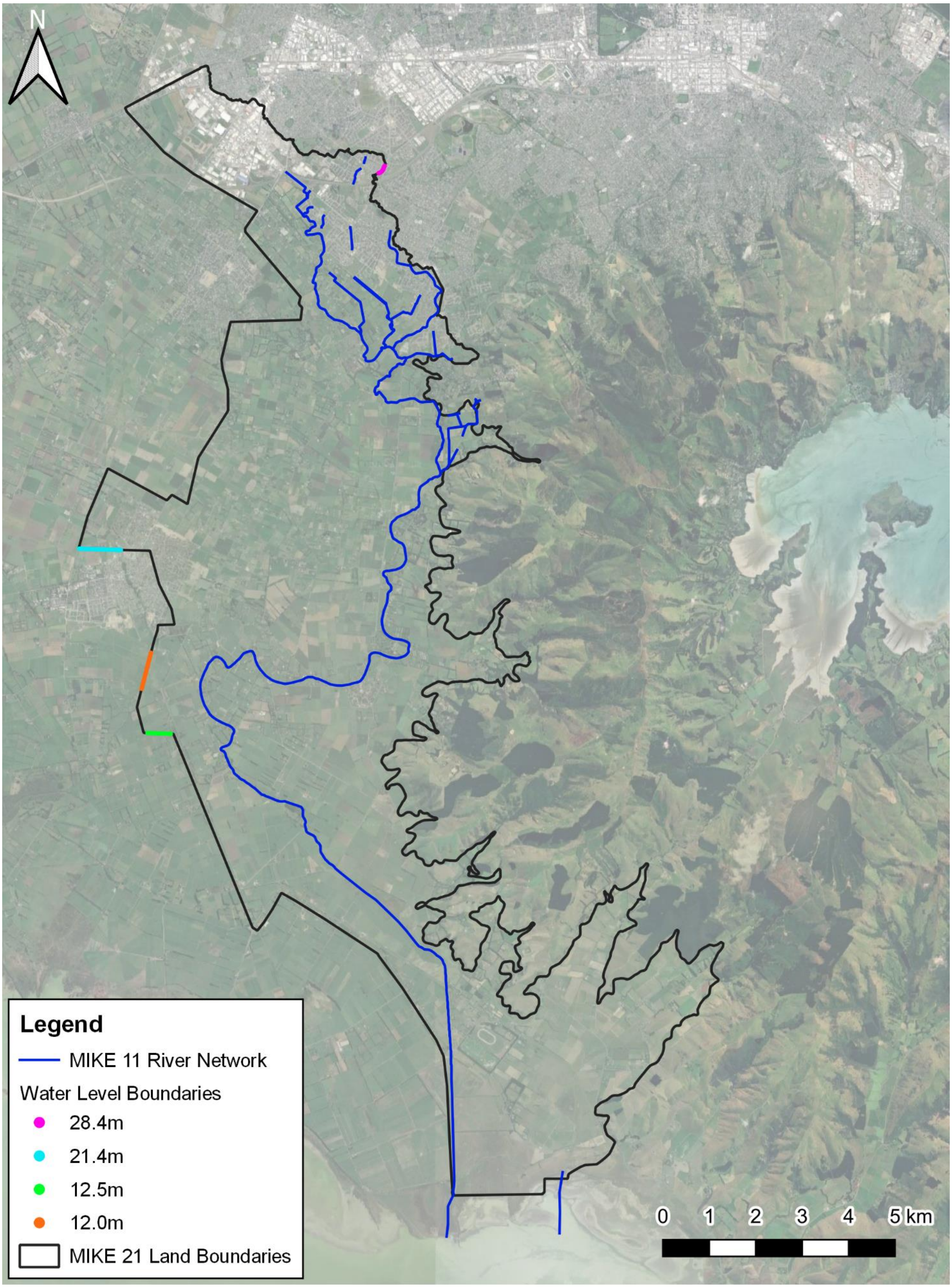


Figure 4-2: Water level boundaries allowing flow out of the model

## 4.4 MIKE FLOOD

MIKE FLOOD was used to couple the MIKE 11, MIKE 21, and MIKE Urban models, allowing for the transfer of water from the 1D engine to the 2D engine vice versa to predict flooding during the design storm events. The sections below summarise the linkages between the three MIKE models.

### 4.4.1 Lateral Links

Lateral links were used to link all the open channels (rivers, streams, and drains) in MIKE 11 to the 2D surfaces. The lateral links were places along the banks of each branch using the MIKE 11 1D cross-sections. Table 4-13 summarises the Lateral Link parameters set in the Halswell MIKE FLOOD model.

Table 4-13 Lateral link parameters for HDM Halswell MIKE FLOOD model

| Parameter | Setting | Comment |
|---|---|---|
| Type | Weir 1 | Default value. MIKE 11 Weir Formula 1 with friction term included in the computation. |
| Source | M21 and HGH | As specified in CFM (LDRP044) Model Schematisation (GHD\AECOM, 2018) these are generally set to MIKE 21 except in locations with known bank levels above the MIKE 21. |
| Depth tolerance | 0.100 | Default value. |
| Weir Coefficient | 1.838 | Default value. |
| Friction (Manning's '*n*') | 0.050 | Default value. |
| Exponential Smoothing Factor | 0.2 to 1 | Exponential smoothing factor dampens short timescale oscillations; however it can create a time lag in the transfer of water levels across the link if it is set inappropriately. For links with initial unstable behaviour (oscillates), the MIKE FLOOD user manual suggests a factor between 0.2 to 0.4. |

### 4.4.2 Standard Links

Standard links were used to link the MIKE 11 branches, where the river branch discharges directly to the 2D surface and vice versa. This only applies to Greens Drain which was linked at the start of the drain (at chainage -22). Table 4-14 summarises the standard link parameters used.

Table 4-14 Standard Link parameters for HDM Halswell MIKE FLOOD model

| Parameter | Setting/Value | Comment |
|---|---|---|
| Momentum Factor | 0 | Set to zero meaning no momentum is transferred through to the MIKE 11 computation. |
| Depth Adjust | No | Depth adjustment not active for the link. |
| Extrapolating Factor | 0 | Default value. |
| Exponential Smoothing Factor | 0.2 | Default value. Factor of 1 represents no smoothing. |

### 4.4.3 Urban Links

Manholes in the HDM Halswell MIKE Urban model were linked to the 2D surface through Urban links, with the 'M21 to intlet' function. Manholes were linked at the location of each connected sump inlet. A sump was considered as 'connected' when it is located within the hydraulic extent, within the mesh extent and within 10 m of the MIKE Urban manhole. The urban coupling linkages for the HDM Halswell MIKE FLOOD model were provided by GHD on 9 November 2017. During peer review DHI identified errors in the coupling linkages provided by GHD. Beca resolved these by reviewing and manually updating them.

The MIKE Urban outlets discharging to the 2D surface via the Urban links using the ‘M21 to outlet’ type.

Table 4-15 summaries the Urban link parameters set in model.

Table 4-15 Urban Link Parameters for HDM Halswell MIKE FLOOD model

| Parameter | Setting/Value | Comment |
|---|---|---|
| Max Flow ($m^3/s$) | Quad Sump: 0.8<br>Triple Sump: 0.6<br>Double Sump: 0.4<br>Single Sump (& other types): 0.2<br>Outlets: 10 | Based on the inlet capacity defined in CCC GIS. |
| Inlet Area ($m^2$) | 0.16 | Default value. |
| Inlet Method | Orifice Equation | Default value. |
| Discharge Coefficient | 0.98 | Default value. |
| QdH Factor | 0 | Default value. |
| Exponential Smoothing Factor | 0.2 | Exponential smoothing factor dampens short timescale oscillations; however it can create a time lag in the transfer of water levels across the link if it is set inappropriately. For links with initial unstable behaviour (oscillates), the MIKE FLOOD user manual suggests a factor between 0.2 to 0.4. |

### 4.4.4 River/Urban Links

River/Urban linkages were used to connect the MIKE Urban network to the MIKE 11 river branches. The majority of links were set to ‘MIKE Urban Outlet to Mike 11’ type. Five links were set to ‘MIKE 11 water level boundary’ where a river branch flows into the MIKE Urban. The exponential smoothing factor for River/Urban links were set to 0.4 as MIKE FLOOD user manual suggest a factor between 0.2 to 0.4 for links with initial unstable behaviour.

# 5 Model Stability Status

This section outlines the stability status of the HDM Halswell model. It provides the stability standard methodology, information on the tools, and the model stability status to date.

Here instability is identified/defined by ‘oscillations’ in the water level results. An oscillation is calculated to be the smaller of the level changes either side of a local extreme point. For example, if the water level in a manhole jumps up by 20mm in a timestep then down by 25mm in the next, the oscillation recorded for that manhole is 20mm. The max oscillation is the largest such event during the simulation.

## 5.1 Stability standard

### 5.1.1 Stability standard methodology summary

Stability assessments were done in batches that include several events of different ARIs and durations. The 1D and 2D results were analysed and stability was improved to meet agreed standards.

The stabilization process is outlined below.

a. 1D results

For 1D results, the following methodology is to be used:

1. 1D results (MIKE URBAN or MIKE 11) are to be loaded into MIKE view excluding the first 25%.
2. Water level oscillations are to be evaluated within the 1D results file using the MIKE View oscillation tool. The tool is discussed further in Section 5.1.1b (MIKE View oscillation tool). The following process is to be applied:
    i. All sites with oscillation amplitude greater than 150 mm tabulated and their locations are to be mapped (shapefile).
    ii. All sites’ max oscillation are to be calculated in the XLS and tabulated.
    iii. All MU sites in this set are to have their associated M21 time series results extracted from the cell above the node. If the node was not coupled, the lowest coupled cell; or if there was any MIKE Urban-MIKE 21 coupling (using “Data Extraction FM” tool with a list of named x-y coordinates) are also to be plotted.
3. Work to reduce instabilities is then to proceeded as follows:
    i. For oscillations between 150mm to 300mm, typically a “do nothing” approach is to be taken and are to be reported except for any MIKE Urban sites where the associated MIKE 21 levels show level oscillations greater than 100mm. For the MIKE Urban sites where the associated MIKE 21 level shows oscillations greater than 100mm, one attempt to fix them is to be taken and the sites are to be reported thereafter whatever the outcome.
    ii. For oscillations larger than 300mm, an initial attempt to fix them is to be taken. If not solved initially, typically a “do nothing” approach is to be taken and they are to be reported, except for any MIKE Urban sites where the associated MIKE 21 levels show level oscillations larger than 100mm. For these sites, a second attempt is to be taken and they are then to be reported whatever the outcome.

b. MIKE View oscillation tool

The MIKE View oscillation tool evaluates and provides a summary report on instabilities in the MIKE Urban and MIKE 11 model results. It is available under the MIKE View tools and the stability standard settings are shown in the Table 5-1.

Table 5-1 MIKE View oscillation tools settings

| Parameter | Value |
|---|---|
| Data Type | Node Level for MIKE Urban<br>Water Level for MIKE 11 |
| Calculation Period | All data (the first 25% was not loaded) |
| Option Selection | Oscillations |
| Limit for Number of Extremes | 0 |
| Duration of Time Interval | Default Value |
| Min Difference for Extremes | 0.15 or 0.30 m |

c. 2D results

For 2D results, the MIKE 21 results are to be run through a DHI tool, which produces overall oscillation frequency summary data and a DFSU of max oscillation 'map' for each stability run.

The DHI MIKE 21 stabilisation tool was provided by DHI on 07/06/2022. An earlier version was also provided by CCC and GHD (developed by DHI) in 2017 but this version did not work with MIKE 2020. The earlier version was used for the existing development 2016 model stabilization rounds. The tool generates two outputs:

- The first is a *.csv file which contains a set of 'bins' and a corresponding count of how many cells there are whose largest oscillation falls into that bin. For example, one simulation could have 1,000,000 elements whose largest oscillation is smaller than 50 mm, 500 elements whose largest oscillation is between 50 and 100mm, and so on. The bins that are to be used as part of the stability methodology are 0 mm to 50 mm, 50 mm to 150 mm, 150 mm to 300 mm, 300 mm to 500 mm, 500 mm to 1 m, 1 m to 2 m, and 2 m to 5 m.
- The second output is a *.dfsu map of the largest oscillation in the simulation by element. For more information on the tool, see Appendix F.

## 5.2 Existing Development 2016 Stability Runs and Status

### 5.2.1 ED2016 Stability Batch Events

The batches used for the Existing Development 2016 model stabilization included the:

- 10-year ARI 3-hour event,
- 10-year ARI 6-hour event,
- 10-year ARI 60-hour event,
- 50-year ARI 3-hour event,
- 50-year ARI 6-hour event,
- 50-year ARI 60-hour event,
- 200-year ARI 3-hour event,
- 200-year ARI 6-hour event, and the
- 200-year ARI 60-hour event.

### 5.2.2 Stabilization process

Prior to stabilisation several basic decisions were made with the goal of building a stable model. For example, the minimum timestep was set appropriately low such that there were few CFL violations during the simulation and an exponential smoothing factor was applied to lateral links that caused instability in initial runs.

During the first batch there were no oscillations in the MIKE 21 that were greater than 100 mm and that corresponded with oscillations in either of the 1D models. Therefore, as per the standard, the only trigger for updating the model was a 1D model's largest oscillation being greater than 300 mm.

Across the whole batch this occurred 32 times. It occurred three times at chainage 0 m of Longhurst Drain and the remainder were at manholes. Actions taken to resolve these instabilities included:

- Updating MIKE 11 cross section and MIKE Urban pipe/manhole inverts to match across River-urban couplings (seven instances).
- Coupling additional MIKE Urban manholes along inlet links to Longhurst channel.
- Coupling outlet links going into Murphys Drainage basin from Longhurst channel.

#### 5.2.3 Existing Development 2016 model - Stability Status

In the second batch there were a total of 61 points (manholes in MIKE Urban or H points in MIKE 11) that show oscillations greater than 150 mm. Of the 32 locations where the oscillations were larger than 300 mm, 10 were resolved and 22 remained. As stated above, none of these corresponded to oscillations in MIKE 21.

#### 5.2.4 Peer Review and impact on Stability

After stabilization, the existing development 2016 model was provided to DHI to peer review (September 2021). This review raised several issues that both needed resolution and were likely to impact the stability of the model. For example, at the time of stabilization the existing development 2016 model used an unrealistically low exponential smoothing factor for some lateral links. Resolving these issues (which was done in late 2021) likely increased oscillations in some areas. Re-doing the full stabilization was outside the scope so only spot checks were made prior to delivering the model results (17 March 2022).

## 5.3 Existing development 2021 Stability Status

The existing development 2021 model stability status is summarised below.

- It has been stabilized according to the stabilization methodology (summarized in section 5.1.1).
- Unfortunately, it has persistent oscillations in the MIKE 21 model that are larger than 100 mm.
- It has several MIKE Urban manholes that have oscillations larger than 150 mm (approximately 13% of all manholes) and some that are larger than 300 mm (approximately 2% of all manholes). None of these correspond to oscillations larger than 100 mm in the MIKE 21 model.
- It has three locations where there are persistent oscillations larger than 300mm in the MIKE 11 model (upstream end of Green Drain, and downstream end of Minsons Drain and Knights Stream Drain). These do not correspond to oscillations in the MIKE 21 model larger than 100 mm.
- As per section 5.2.2, shapefiles have been generated to report the locations of the oscillations referred to above.

A supporting file-note is appended (Appendix H) that provides detail on the stabilization process for the existing development 2021 model.

# 6 Model Validation

## 6.1 Introduction

Unfortunately, due to insufficient or inaccurate data it was not realistically possible to calibrate the Halswell model. To calibrate a model properly we need:

- river stage/flow data that is near the area of interest, or
- quality aerial photography of flooding (or mapped aerials) in the area of interest with timestamps indicating when the flooding occurred relative to the rainfall, and
- a rainfall timeseries near the area of interest or in the catchment upstream, and
- good information about the physical hydrological system at the time of the calibration events (for example, percentage imperviousness and accurate flow paths).

The data that was available is discussed in detail in section 6.2, but does not meet these general requirements (for example, the only river flow recorder available during the selected events is far away from the Christchurch City Boundary). Therefore, the Existing Development 2016 (ED2016) Halswell model was only validated not calibrated. The validation events selected are from: June 2013, July 1977, and June 1975.

### 6.1.1 Validation approach

The model validation was undertaken in 2021 using the Existing Development 2016 model as a base. The model was validated against aerial flood extent maps (1975 and 1977 events) and level/flow recordings at Ryans Road Bridge in Tai Tapu (2013 event). The flood extent maps were supplied by ECan and are discussed in section 6.2.3.

Once the Halswell river rises high enough the flow can bypass the recorder at Ryans Road. Therefore, it is likely to underestimate the flow during extreme events. The distance from the area of interest (upper catchment and in particular within the Christchurch City Boundary) also reduces the amount of weighting we can give a comparison to this recorder. A model that predicts water levels well in one area does not necessarily predict water levels well far away from that location.

### 6.1.2 Event Selection

Events were selected based on available aerial flood extent maps, rainfall records, and flow and level gauges with the aim to align the selected events with those used in the Heathcote River model calibration.

### 6.1.3 Antecedent Conditions

No adjustment for antecedent conditions was made for the validation runs. However, the 2013 event had two peaks with approximately 36 hours between them. Since both peaks were modelled it is likely that the first peak helped to create more realistic antecedent conditions for the second peak.

## 6.2 Available data summary

### 6.2.1 Rainfall gauges

For the June 2013 event, the Christchurch Airport gauge was used to apply spatially uniform rainfall to the MIKE 21 model. The Port Hills were modelled in RORB using spatially uniform rainfall recorded from the Coopers Knob gauge. These gauges are shown in Figure 6-1 (overleaf).

For the June 1975 and July 1977 events, the Christchurch Airport gauge was used to create both the MIKE 21 and RORB model inputs as the Coopers Knob rainfall gauge was not available.

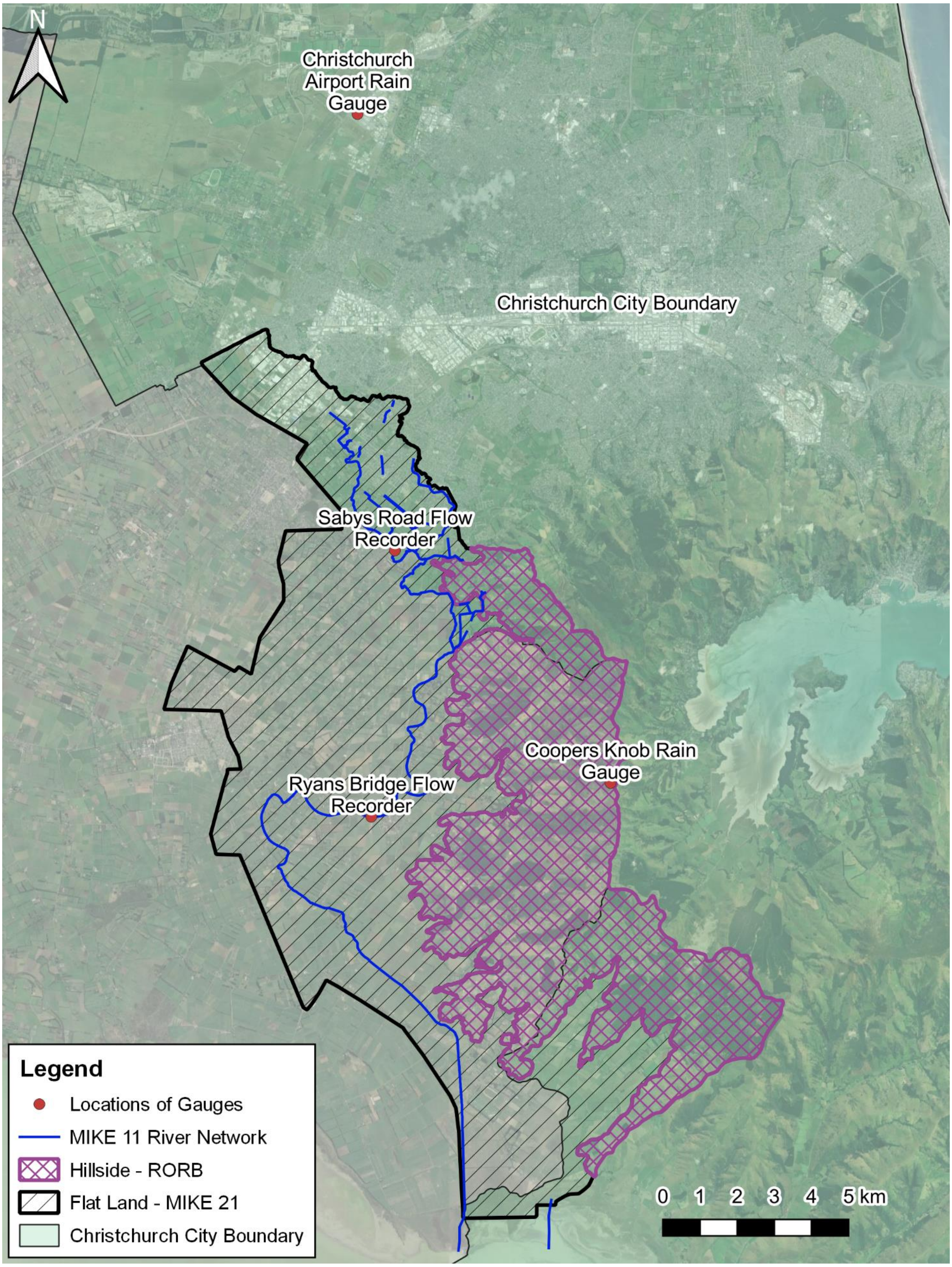


Figure 6-1 Halswell rain gauge locations

Table 6-1 shows the duration, total depth, and peak intensity of the three validation events. The 1975 and 1977 events were similar in all three variables. Despite these similarities the flood extent maps provided by

ECan for these events are different. This could be due to recording error or timing of the photos (for example, one photo set may have been taken during the peak water level while the other may have been taken after the water level had significantly receded).

Table 6-1 also shows the approximate average reoccurrence interval (ARI) for the rainfall of the three validation events for the 6-hour and 48-hour durations. These ARIs were estimated by calculating the highest depth that fell in any 6-hour and 48-hour period during the event and comparing this to the depth-duration-frequency tables provided by NIWA's High Intensity Rainfall Design System (HIRDS v4). The 6-hour duration ARI is provided because it is close to the critical duration of the upper catchment (within the Christchurch City Boundary) and the 48-hour duration ARI is provided because it is close to the critical duration for the whole Halswell catchment (the generally agreed critical duration for the whole catchment is 60-hours, however HIRDSv4 does not provide a 60-hour depth-frequency table). The ARIs indicate the size of the events against which the model was validated. The location sampled in HIRDS is in the upper catchment (within the Christchurch City Boundary but not on the Port Hills where rainfall is expected to be higher).

The 2013 event had a lower peak intensity but the total rainfall depth was higher and the total duration was approximately twice that of 1975 and 1977 events.

Table 6-1 Validation events summary and comparison with HIRDSv4 ARIs

| Event | Duration (hours) | Total depth (mm) | Peak intensity (mm/hour) | 6hr ARI (years) | 48hr ARI (years) |
|---|---|---|---|---|---|
| 2013 | 170 | 155 | 5.2 | 1.58 years | ~8 years |
| 1977 | 58 | 128 | 7.6 | 5 years | 30 years |
| 1975 | 66 | 125 | 7.6 | 5 years | 40 years |

Figure 6-2 shows rainfall intensity of the three events over their duration. The 2013 event peak was preceded by approximately 3 days of lower intensity rainfall (less than 3 mm/hr).

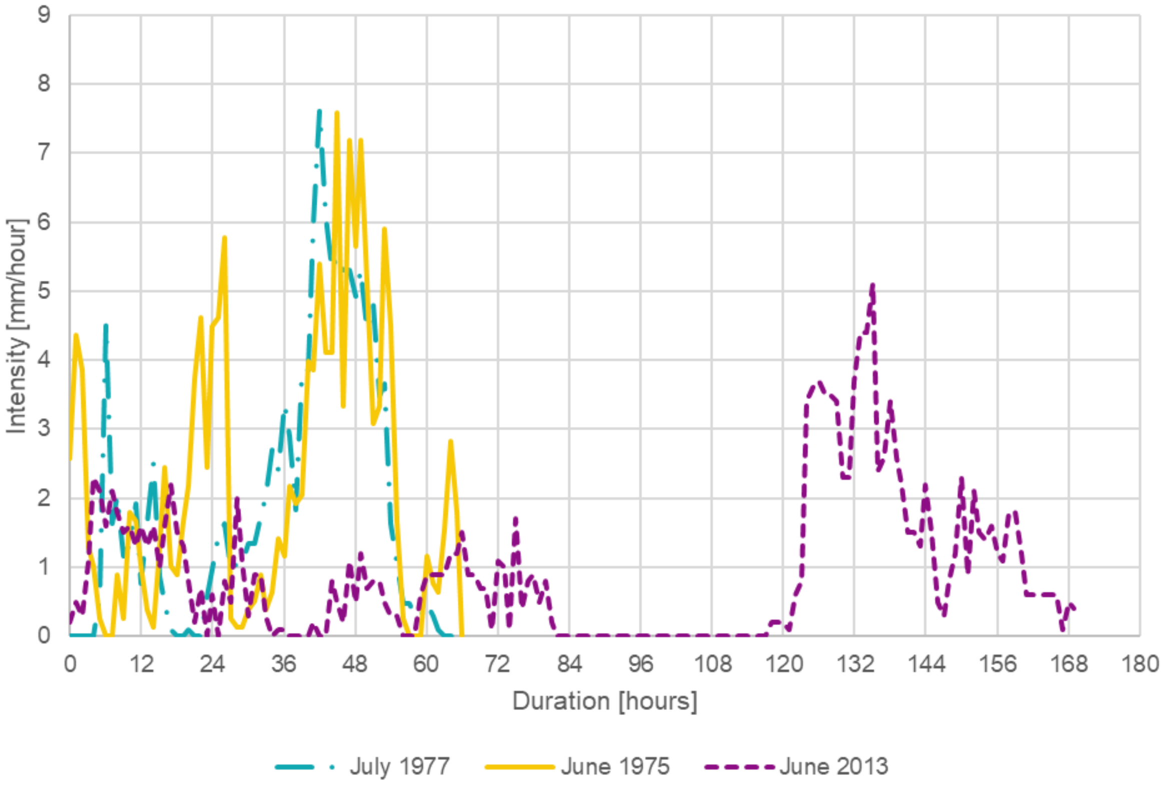


Figure 6-2 Rainfall timeseries of validation events

### 6.2.2 Flow and level gauges

The Ryans Road Bridge recorder is approximately 11 km from Halswell and records flow from a catchment area of approximately 76 $km^2$ (the catchment area within the Christchurch City Boundary is 20 $km^2$). The site has been recording since 1996 (25 years) and therefore does not have stage/flow data for the 1977 and 1975 events. It does have data available for the 2013 validation event.

A recorder was installed on the Halswell River at the Sabys Road bridge after the 2013 event. This site is near the Christchurch City Boundary and is ideally located to assess changes in flow in the upper catchment. Beca recommends that this site is used for any future flood event calibration. The record was checked for large events since its installation (which was in 2013 but after the validation event), but it was agreed that none were sufficient to warrant a model calibration.

A level recorder exists in Te Waihora/Lake Ellesmere. The 2% annual exceedance probability (AEP) level has been used for the 1975 and 1977 runs. The 2% AEP level is 10.343 m RL. The 2013 event validation uses the recorded lake level which peaks at 10.57 m RL.

### 6.2.3 Aerial mapping

ECan has supplied aerial mapped flood extents for each of the validation events. The maps were created based on aerial photos and observations after each event. While these flood extents are useful for validation, care needs to be taken when comparing model results against these mapped extents. Some issues that may arise from the use of these maps are:

- **The area mapped:** ECan's main focus is on the area south of Halswell and therefore the mapped extent is limited in the upper catchment (nothing within the Christchurch City Boundary). The Halswell model, although reaching to Te Waihora/Lake Ellesmere, has limited detail in the lower reaches. These lower reaches have numerous farm drains and minor culverts which are not included in the model (including Dawsons Creek) and have not been accurately captured in the mesh. We should also assume that not every flooded area has been mapped as aerial photos can be limited and may not have captured the full extent of the ponded water in the catchment.
- **The timing of the mapping:** With no indication of when the mapping occurred it is then difficult to assess whether these are peak water levels or water levels when the flood is receding.
- **Accuracy of the mapping**: It is possible that ECan mapped against general extent and did not cross reference the extents to the topography. In particular, areas that didn't flood in 1977 that should have, based on flood extents seen in 2013.

Although they have their limitations, the maps are a useful tool that show expected areas of flooding. The ECan flood extent maps are compared with peak water levels from the model in section 6.4.

## 6.3 June 2013 Validation

### 6.3.1 Model geometry changes

No adjustments were made to the Existing Development 2016 (ED2016) model for the June 2013 event because the differences in development between the two scenarios are negligible.

### 6.3.2 Infiltration losses

The infiltration rates were set based on values from the Waterways, Wetlands and Drainage Guide (CCC 2003) and a soil type dataset as described in section 3.3.3c. The resulting Horton's loss parameters are given in Table 6-2 below. Following the validation and as peer advice from the peer review (DHI) the final infiltration rates used in the model (ED2016 and ED2020) were updated to those given in section 3.3.3c.

Table 6-2: Horton's infiltration parameters used for the validation events

| Zone ID | Percent pervious | Initial infiltration rate $f_0$ (mm/hr) | Ultimate infiltration rate $f_c$ (mm/hr) | WEIGHTED Initial infiltration rate $f_{0WEIGHTED}$ (mm/hr) | WEIGHTED Ultimate infiltration rate $f_{cWEIGHTED}$ (mm/hr) | Horton decay rate k ($s^{-1}$) |
|---|---|---|---|---|---|---|
| Open Space – Poor Draining | 100% | 2.5 | 1.0 | 2.5 | 1.0 | 1.50E-03 |
| Open Space – Moderate Draining | 100% | 7.5 | 2.5 | 7.5 | 2.5 | 1.00E-04 |
| Open Space – Free Draining | 100% | 12.5 | 5.0 | 12.5 | 5.0 | 3.00E-05 |
| Residential – Poor Draining | 50% | 2.5 | 1.0 | 1.25 | 0.5 | 1.50E-03 |
| Residential – Moderate Draining | 50% | 7.5 | 2.5 | 3.75 | 1.3 | 1.00E-04 |
| Residential – Free Draining | 50% | 12.5 | 5.0 | 6.25 | 2.5 | 3.00E-05 |
| Industrial – Free Draining | 10% | 12.5 | 5.0 | 1.25 | 0.5 | 3.00E-05 |
| Roads – Moderate Draining | 10% | 7.5 | 2.5 | 0.75 | 0.3 | 1.00E-04 |
| Hillside | 100% | 2.5 | 1.0 | 2.5 | 1.0 | 1.50E-03 |

These Horton's losses were applied in the 2013 validation event using the same infiltration zones (2D distribution) used in the ED2016 model, which is presented in Figure 3-5 and described in section 3.3.3d.

### 6.3.3 Roughness adjustments

No adjustments were made in 2D surface roughness in the MIKE 21 model.

### 6.3.4 Results

The two main sources of information for comparison are:

- the extent maps provided by ECan (based on aerial photos generally in the lower catchment), and
- the Ryans Rd Bridge recorder.

A recorder was installed by CCC on the Halswell River at Sabys Road bridge in the upper catchment but recording did not start until after this event. ECan supplied information on the event which is presented below:

*A sustained high rainfall event occurred on 16-22 June 2013. In many areas, water levels were as high as have been recorded since July 1977. The levels exceeded the 1977 event in Ahuriri Lagoon and* *downstream. Te Waihora/Lake Ellesmere rose about 0.8m during the event, to the highest level recorded since 1941. 204mm of rain fell in 7 days in Tai Tapu. The rainfall fell in two distinct periods;*

- *a 20% Annual Exceedance Probability (5-year) 2-day event on 16-17 June, followed by 2 days with little rainfall, then*
- *a 20% AEP (5-year) 3-day event on 20-22 June*

*The overall chance of this combination of events has not been calculated but may be of the order of 2-5%AEP (20-50 years).*

*The first part of the rain event was characterised by very even, steady rainfall. The second part of the event coincided with a major snowfall event throughout Canterbury, including some snowfall on the Port Hills part of the catchment. The snow did not appear to have any significant effect on the Halswell River.*

*At Tai Tapu, the peak flow was recorded as 14.4 cumecs on the 23rd, (compared with 0.9 cumecs prior to the event) and the peak level was 6.56m, about 2m higher than the normal winter level and 0.23m higher than a 5-year event in August 2012.*

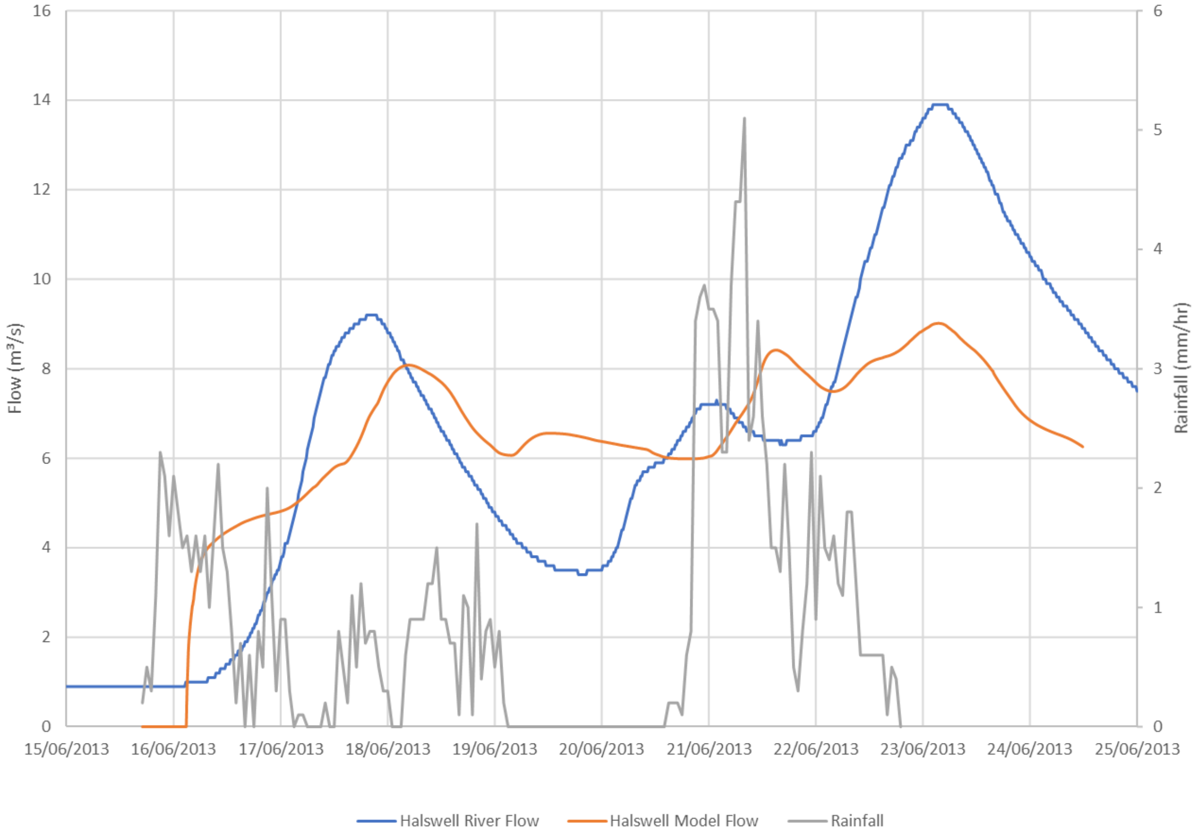


Figure 6-3 Halswell River recorded flow and modelled flow

Comparing the recorded flow at Ryans Road Bridge and the modelled flow in the 2013 event shows that the model does not predict the same response to runoff. The response is subdued resulting in a lower peak flow. This could be due to the connectivity of the farm drains in the model. The lower reaches of the model have limited detail of the connecting drains that help to fill the Halswell River. This results in a significantly lower peak flow in the model when compared to the recorder (9 $m^3/s$ vs 14 $m^3/s$).

A check of the rating curve in the modelled cross-section near the recorder showed that it is similar to that of the recorded site based on the flow and level record.

The volumes of water passing the site for the 7 days from 17 - 23 June are also similar. Observed runoff was 4,600,170 $m^3$, compared to modelled 4,282,991 $m^3$.

Figure 6-4 shows the full catchment modelled maximum flood extent compared to the aerial flood map from the June 2013 event. At a catchment scale the modelled results generally have less inundated area than aerial photo records. Depths differ by 300–800 mm in some areas but in the upper catchment some of the areas are similar. Negative values indicate areas where water depth is underestimated in the model. The figure also highlights some areas where missing infrastructure is affecting the flood extent.

Not all the areas shown in Figure 6-4 are directly connected to the river or are flooding as a result of the river level. Areas like this could show the rainfall in the catchment was more intense than that recorded at the Christchurch Airport or that infiltration is less than that used in the model. As stated, the calibrated infiltration parameters used in the Heathcote model (as advised by DHI) are lower than those used in this validation model. The final Existing Development 2016 model uses the infiltration parameters from the calibrated Heathcote model.

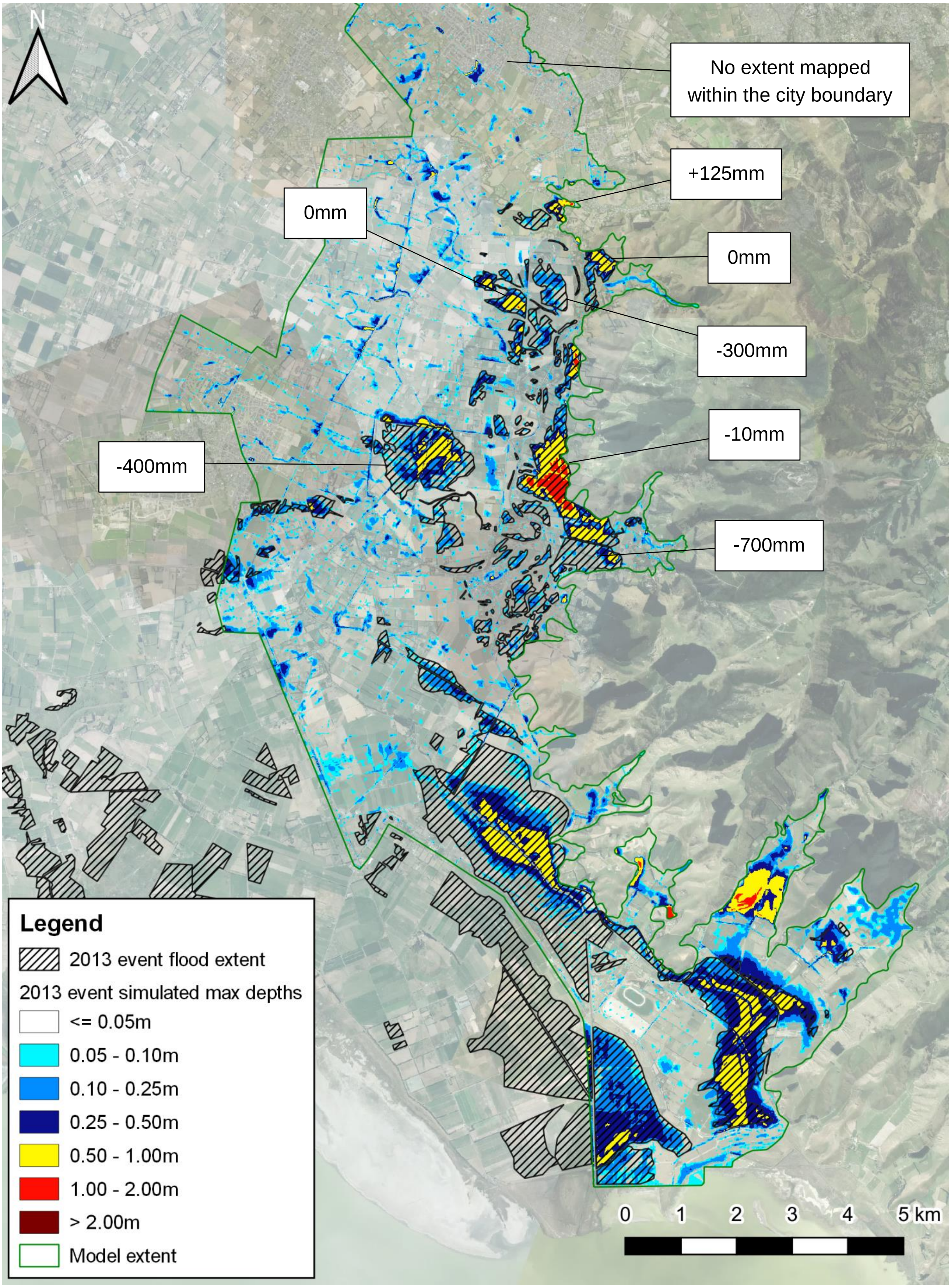


Figure 6-4 Modelled difference in water level for 2013 event

## 6.4 June 1975 and July 1977 Validation

### 6.4.1 Model geometry changes

The model infiltration map was changed to reflect an approximate level of development based on historic aerial photos of Halswell between 1975-1979. This resulted in a reduction of imperviousness in the upper catchment. The same level of development was used for both the 1975 and 1977 events. No changes were made to the flow paths (MIKE 21 depth correction, MIKE Urban pipe network, and MIKE 11 river network, so these match the Existing Development 2016 model).

### 6.4.2 Infiltration losses

Infiltration rates were set based on the recommended values in the Waterways, Wetlands and Drainage Guide (as with the 2013 validation event discussed in section 6.3.2).

### 6.4.3 Results

As with the June 2013 event, the modelled results generally have less inundated area than aerial photo records. Depths differ by 300–800mm between modelled results and observed flooding. Refer to Figure 6-5 and Figure 6-6 for a comparison of modelled and observed flood extents for the July 1977 and June 1975 events respectively.

The mapping of the extents appears to be more detailed for 1977. Where a flooding area is mapped in both events the extent is similar.

An overall underestimation of flood depths may be due to the Christchurch Airport rain gauge not providing a representative rainfall for the Halswell River catchment. This will certainly be the case for the Port Hills portion of the catchment which are likely to contribute a lot of the ponding in the eastern and southern areas of the model. Another possible reason is the infiltration used was higher than reality (as with the 2013 validation event).

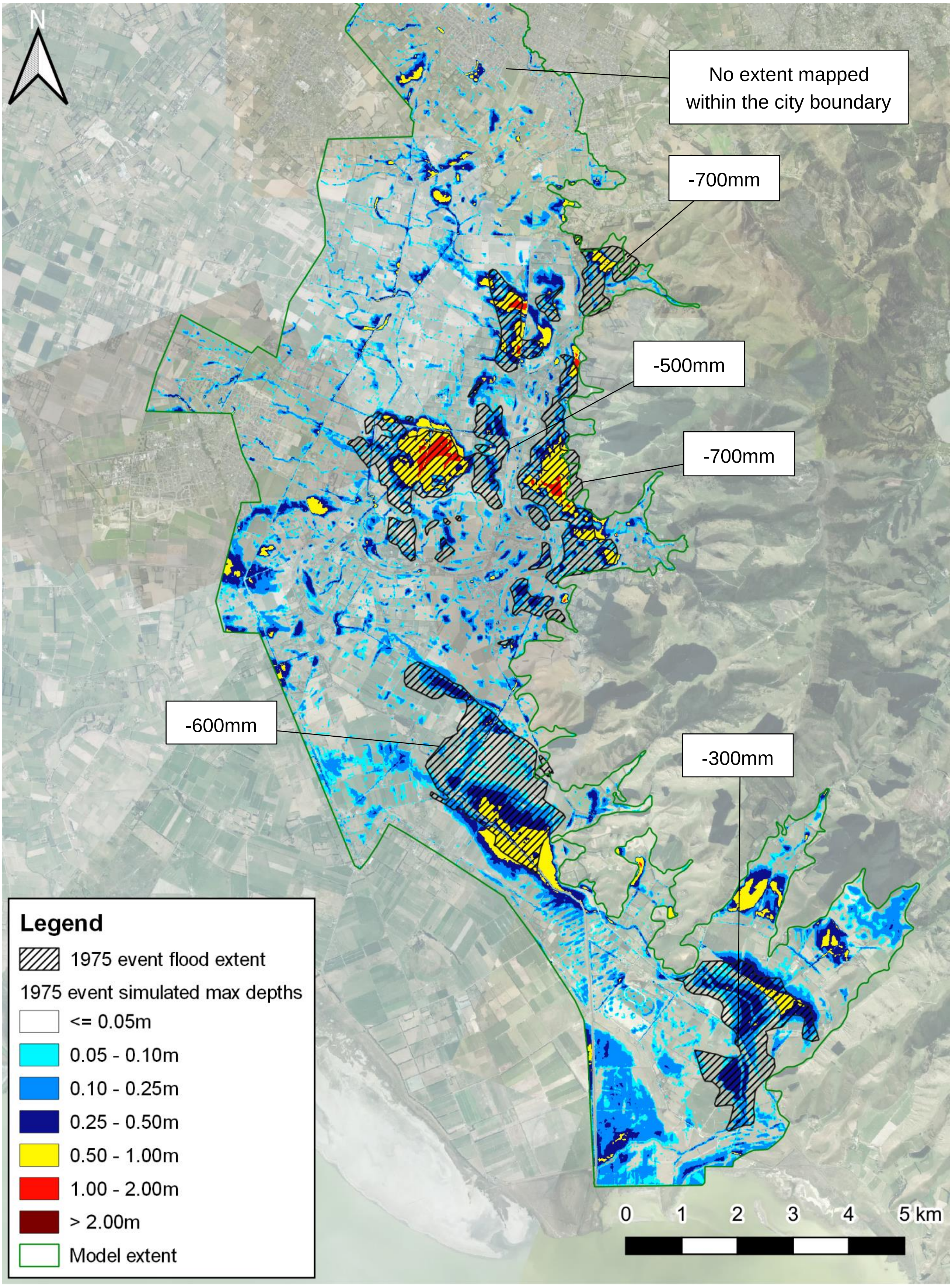


Figure 6-5: 1975 event simulated maximum depths and observed flood extent

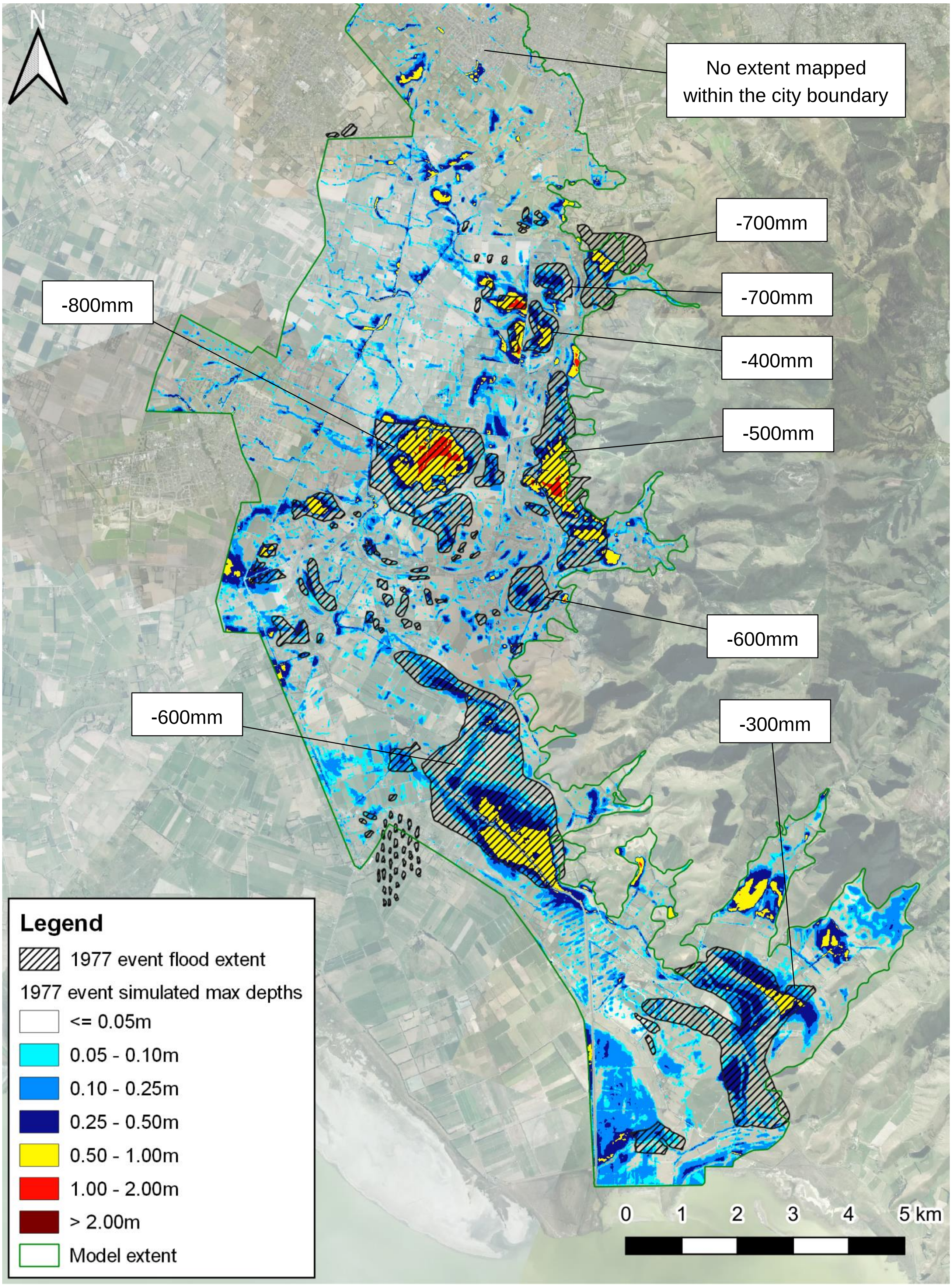


Figure 6-6: 1977 event simulated maximum water depths and observed flood extents

## 6.5 Validation Conclusion

There is insufficient data available to perform a full calibration of the HDM Halswell model. In particularly, the flow metering is far from the area of interest, flow can bypass the meter in large events, and we cannot be confident that the available flood extent maps accurately represent peak water levels. For these reasons the model was validated against historical events but not calibrated.

Comparing the simulated flow at Ryans Road bridge with the recorded flow shows there is a difference between the simulation and the event. The simulated response is subdued with a lower peak flow. This may be due to limited detail in the lower reaches of the model (farm drains may not be accurately captured by the MIKE 21 mesh).

The three modelled validation events generally showed the model underestimates flood levels when compared to the mapped extents. Due to the limited accuracy of the mapped extents drawing conclusions from them is difficult. As all three events show a trend of under estimation the infiltration rates were reduced to match those used in the calibration of the Heathcote model. This agrees with advice from the peer reviewer that the initial infiltration rates appeared high considering the values in the Heathcote model. The lack of alignment with the validation events could be due to:

- **Missing rural drainage.** Given the definition of the model in the rural area small local drain will not be represented in the mesh as well as culverts. This could be restricting the ability of runoff to enter the Halswell River.
- **Rainfall.** The model uses rainfall from the Christchurch Airport. Although not far from the headwaters of the catchment it is a considerable distance from the recorder at Tai Tapu (Ryans Rd Bridge).
- **Limitation of mapped extents.** The maps provide an indication of where flooding occurred in the lower catchment but are not necessarily accurate representations of the peak water level. Using them as a direct comparison of peak water level is beyond their accuracy.

It is unlikely that the reduction in infiltration rates will fully align the 2013 event with the recorded flow, but they will give a consistent approach across the two catchments whose headwaters boarder each other. The infiltration rates have been reduced in the final Existing Development 2016 model runs.

# 7 Model Sensitivity

## 7.1 Approach

Following the conclusions from model validation, the sensitivity checks were determined to be:

- A 20% increase in 2D surface and 1D channel roughness for the 6-hour duration 50-year ARI event, and
- A reduction in infiltration for the 6-hour duration 50-year ARI event

The 6-hour duration 50-year ARI event was selected because it is near the critical duration for the upper part of the catchment. This means it is more indicative of the potential flooding within the Christchurch City Boundary, which is the most important area to CCC.

### 7.1.1 Roughness adjustments

Given the Existing Development 2016 model underestimated historic flood levels in the validation, the model roughness was increased to force more flow out of the 1D channels and onto the 2D floodplain. The model roughness was increased by 20% for both the 1D channels in MIKE 11 and 2D surface in MIKE 21. As stated in section 4.3.5, the initial roughness parameters used in the model come from table 3 of the CFM – Model Schema Report (GHD 2018). For example, building footprints which previously had a Mannings ‘n’ of 0.2, have a Mannings ‘n’ of 0.24 in the sensitivity runs and open spaces which previously had a Mannings ‘n’ of 0.05, have a Mannings ‘n’ of 0.06 in the roughness sensitivity run.

### 7.1.2 Infiltration losses

Infiltration rates were reduced to match the values used in the calibrated Heathcote Model for the June 2013 event:

| Parameter | Poorly Drained | Imperfectly Drained | Well Drained |
|---|---|---|---|
| Initial (mm) | 1 | 2.5 | 3 |
| Final (mm/hr) | 1 | 1.5 | 2.5 |
| Decay rate ($s^{-1}$) | NA | 0.00004 | 0.00003 |

Figure 7-1 Infiltration rates used in sensitivity testing

## 7.2 Results

### 7.2.1 Roughness adjustment

Increasing the roughness of the MIKE 11 and MIKE 21 by 20% results in higher water levels in the upper reaches of the Halswell river catchment. This is shown in Figure 7-2 where warm colours show an increase and cool colours show a decrease in maximum water level due to increasing the roughness. The size of the change is between -100mm (lower reaches of Nottingham Stream) and +500mm (in Knights Stream and Cases Drain). In general, the increase in roughness results in higher water levels throughout the channels in the upper catchment by approximately 50mm. The lower reaches (for example, the main Halswell river channel) are not shown in Figure 7-2 as the main area of interest is within the Christchurch City Boundary and the duration of the event used is shorter than critical duration for the lower catchment.

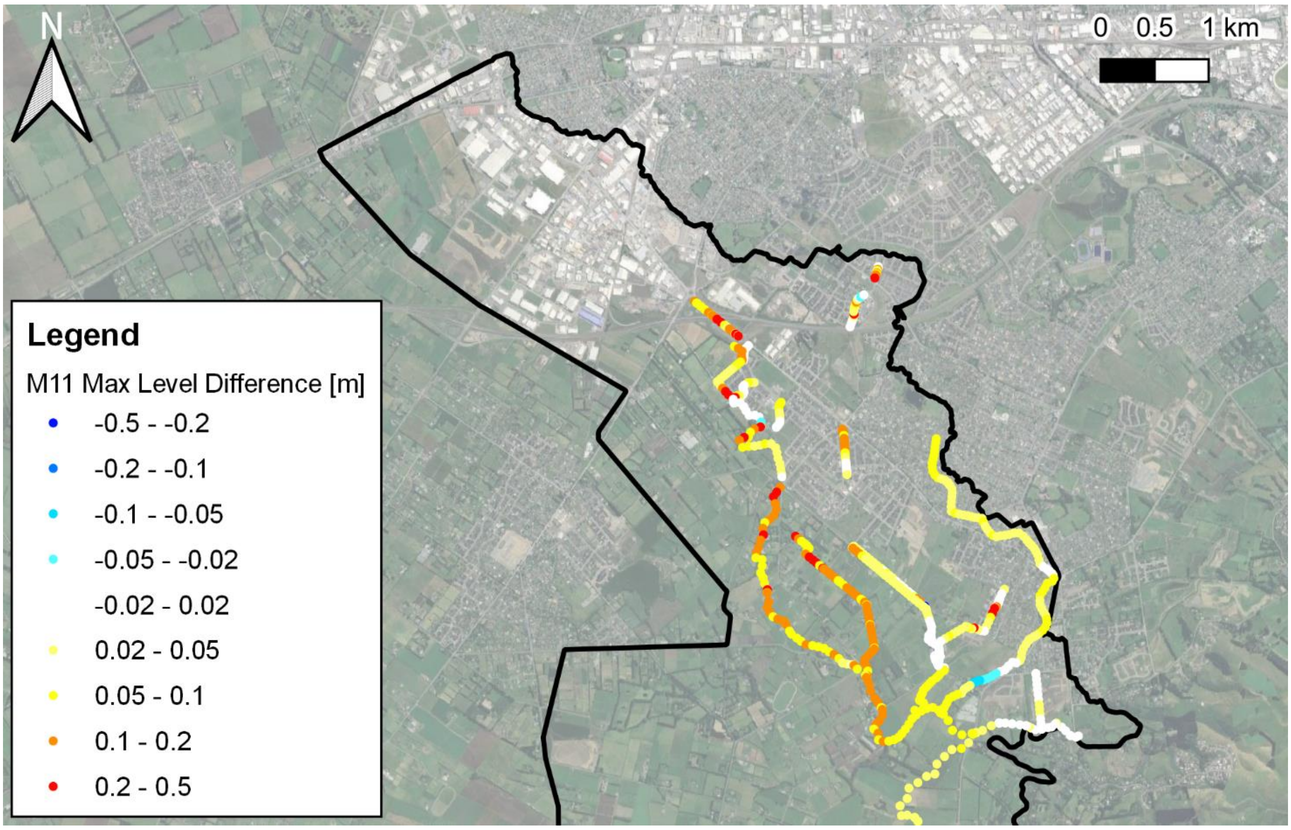


Figure 7-2 Difference in maximum water level created by increasing roughness values by 20%

The change in level at Creamery Ponds is very small (less than 20mm in magnitude).

The impact that increasing the roughness by 20% has on flooding on the MIKE 21 surface is minor. Most of the catchment is unchanged with some small areas to the west of the flatland changing by up to 200mm and the majority of these changing by less than 50mm (Figure 7-3). The upper catchment is mostly unaffected by increasing the roughness; however, Ella Park basin has its peak water level reduced by 25mm (blue patch within the Christchurch City Boundary). This is possibly due to water entering the basin slower from Knights Stream Drain (north of the basin).

The location of each basin is indicated on page 33 (Figure 3-12). The duration of the event used for this sensitivity is shorter than the critical duration for the lower catchment so the difference in maximum water level may be higher for longer duration events which are simulated long enough for the water to concentrate there.

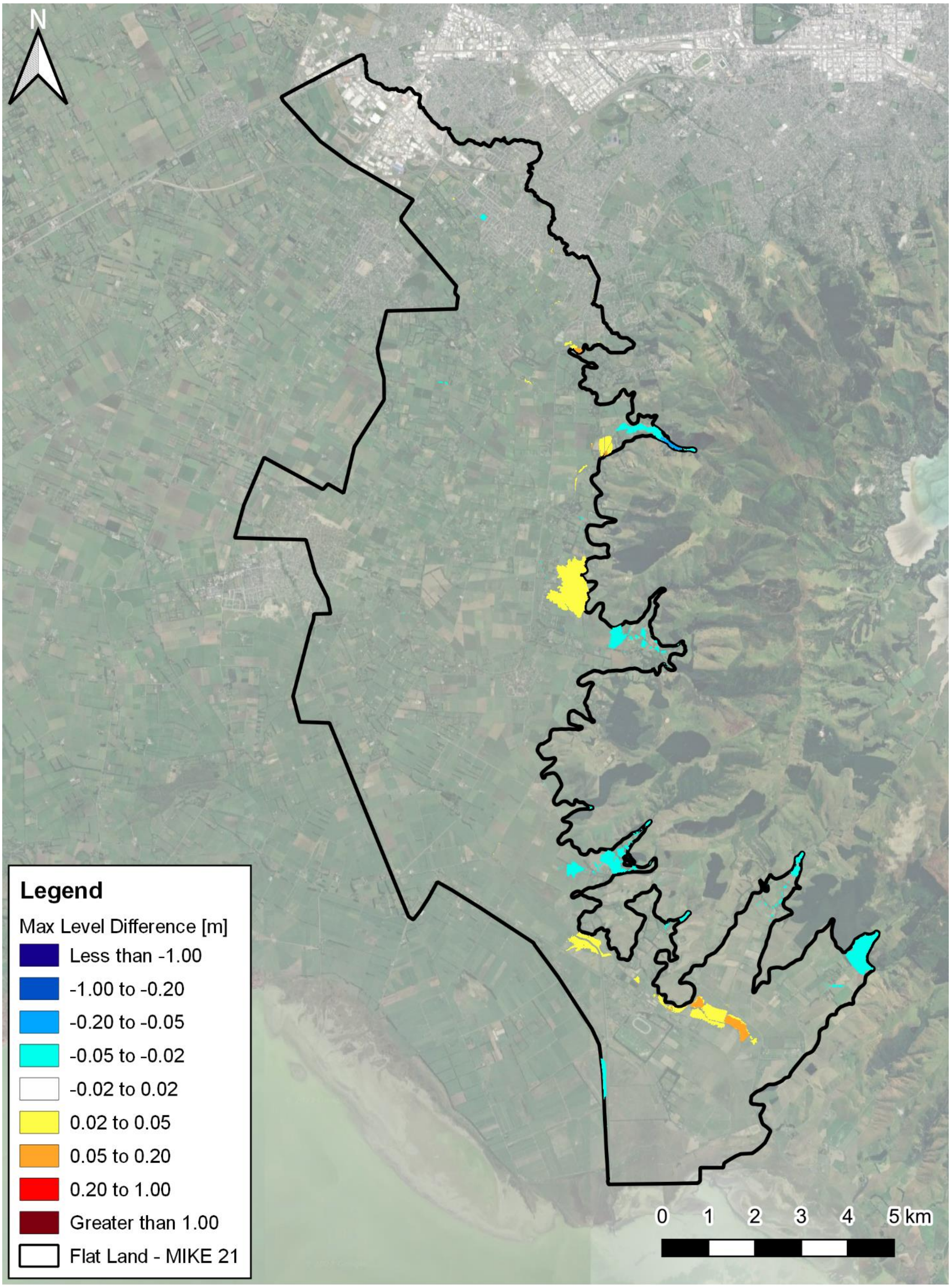


Figure 7-3 Maximum water level difference [m] caused by increasing the roughness by 20%

### 7.2.2 Infiltration losses

Reducing the infiltration losses to match the calibrated Heathcote values results in a significant increase in the water level of the rivers and streams within the Christchurch City Boundary (Figure 7-4). Several streams including Nottingham Stream, Quaifes Drain, Talbots Drain and Cases Drain show an increase in maximum water level of up to 100mm while Knights Stream, Halswell Junction Outfall, Awatea Drain and Platinum Drive Drain show an increase of up to 500mm. The names of the watercourses are given on page 29 (Figure 3-11).

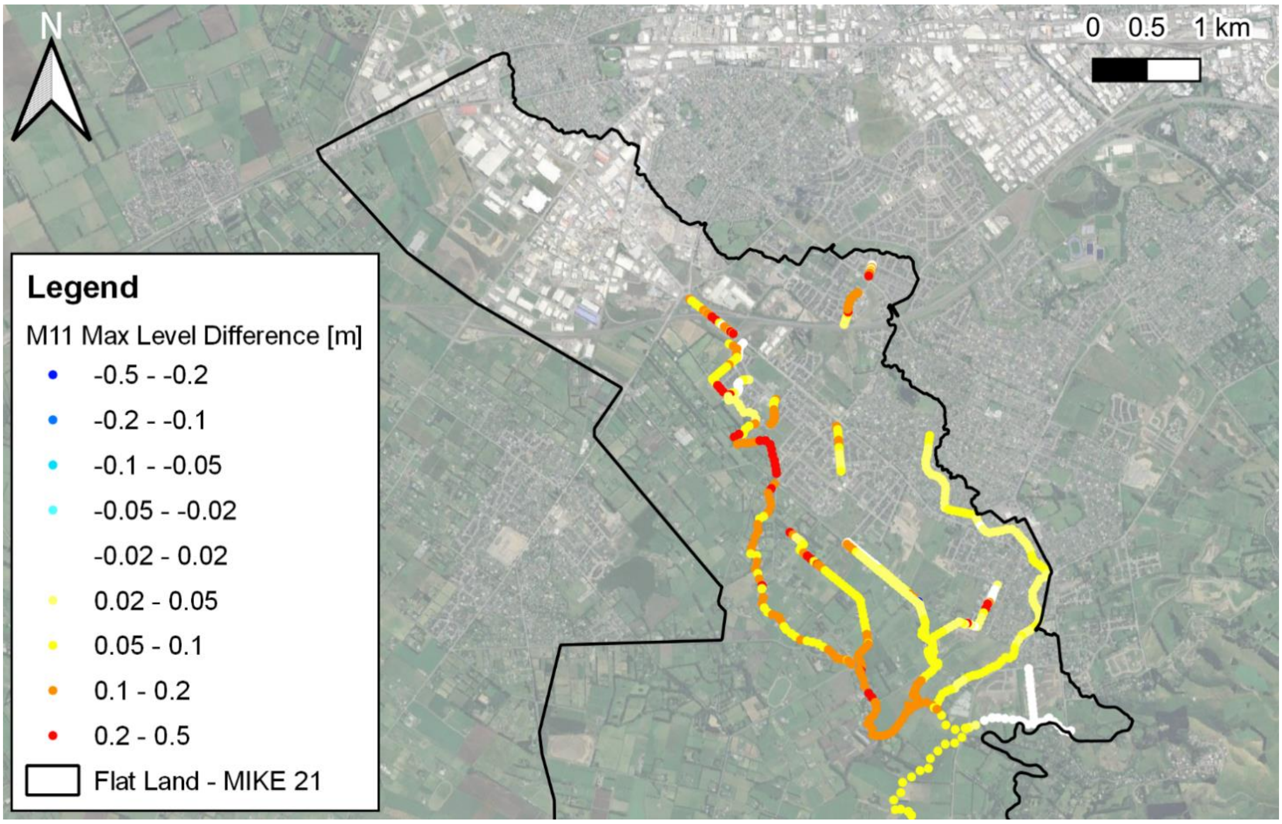


Figure 7-4 Difference in maximum water level created by reducing infiltration to match calibrated values for the Heathcote model

Reducing the infiltration losses results in a significant increase in maximum water level on the MIKE 21 surface (Figure 7-5, next page). The increase is seen both within the Christchurch City Boundary and in the lower catchment. Within the Christchurch City Boundary there is an increase in maximum water level up to 50mm south of the industrial area and an increase in maximum water level of up to 200mm in several basins including Halswell Junction Detention Basin, Owaka Basin, Murphys Basin. The maximum water levels at Westlake Reserve and Ella Park Basin increase by 35mm and 270mm respectively. The maximum water level also increases in the lower catchment with most areas increasing by up to 50mm and some increasing by up to 200mm.

For the event tested, decreasing the infiltration does not broadly change how the system operates. There are some additional overland flow paths south of the industrial area that were not active prior to decreasing the infiltration to match the Heathcote model.

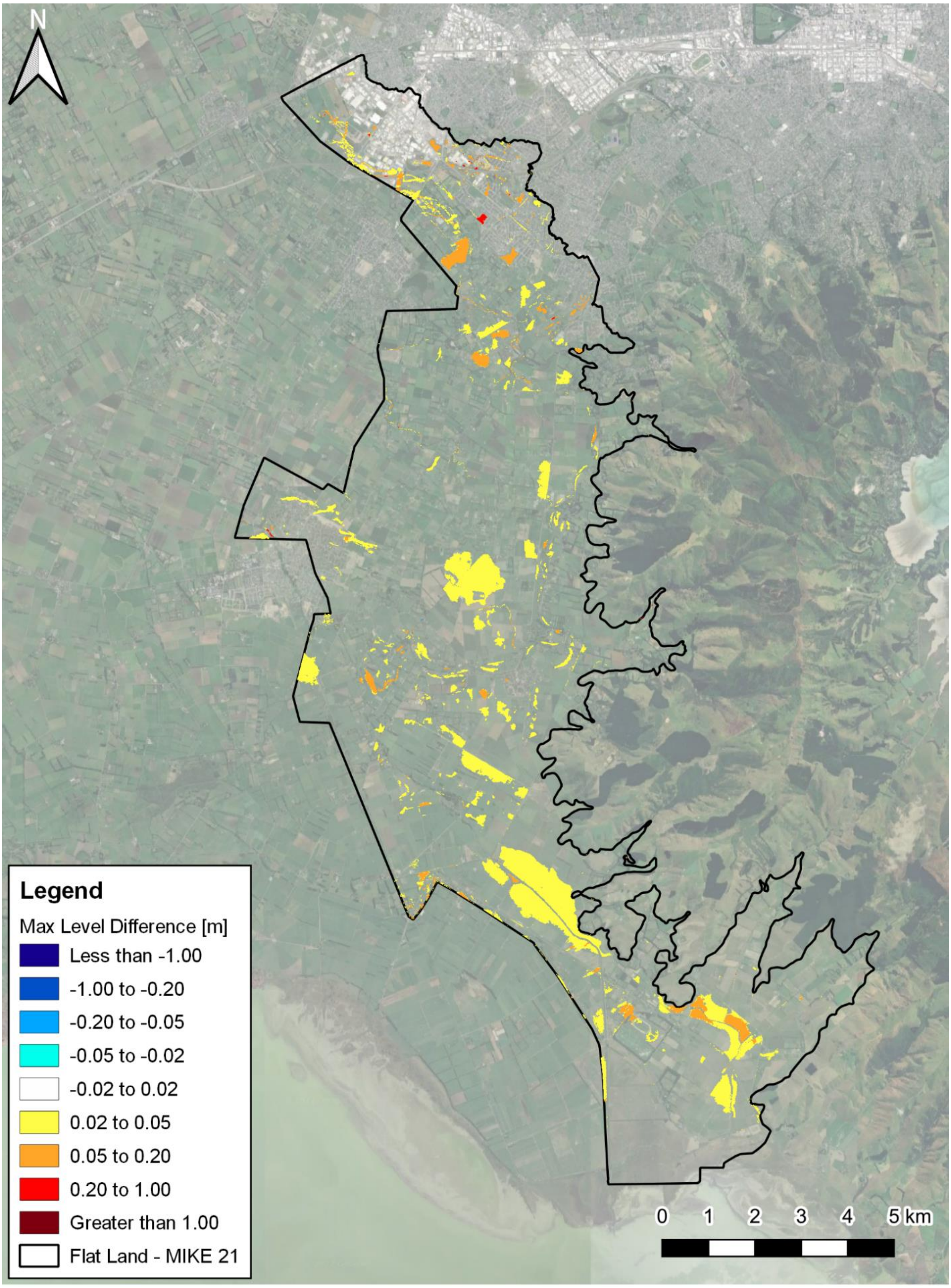


Figure 7-5 Difference in maximum level [m] on the MIKE 21 surface when the infiltration losses are reduced to match the calibrated Heathcote values

## 7.3 Conclusions

Increasing the roughness increases the maximum water level in the river channels within the Christchurch City Boundary and increases the maximum water level in some areas of the MIKE 21 surface. It also decreased the maximum water level in other areas. As discussed in detail in section 6 (Model Validation) there is not enough real-world data to properly calibrate the model. Therefore, the roughness values in the final model were left as is (namely, there was no 20% increase applied to the values from the CWM schema report).

Decreasing the infiltration parameters to match the Heathcote calibrated values increased the maximum water level in the rivers, streams, and basins within the Christchurch City Boundary and on the plains in the lower catchment. Generally, this increase was less than 50mm but in Ella Park Basin was as high as 270mm. This did not change how the system broadly operates in this event, however there are some additional flow paths south of the industrial zones.

The infiltration parameter used in the Heathcote model were calibrated so there are reasonable grounds to adopt them in the Halswell model. This is consistent with the advice from the peer reviewer that the initial infiltration parameters were too high. For these reasons, the final Halswell Existing Development 2016 model and simulations use these values (presented in section 3.3.3d).

A combination of increased roughness and reduced infiltration could be checked to further refine the model. It may also be useful to run a combination of these parameters for the June 2013 event to check if it provides a closer match to observed flooding.

# 8 Model Runs Setup

## 8.1 Specification for design run settings

MIKE 21 results were recorded every 20 minutes for all the design storm durations. The results saved are water level, depth, east velocity, north velocity, current speed, and Courant Friedrichs Lewy number (CFL number). Table 8-1 provides the storm durations, their overall run duration (total duration simulated), and the number of time steps saved.

Table 8-1 storm duration, run duration and number of time steps saved for each design storm simulation

| Storm duration [hours] | Run duration [hour] | Number of time steps saved |
|---|---|---|
| 0.5 | 60.5 | 242 |
| 1 | 61 | 244 |
| 2 | 62 | 248 |
| 3 | 63 | 252 |
| 6 | 66 | 264 |
| 9 | 69 | 276 |
| 12 | 72 | 288 |
| 48 | 108 | 432 |
| 60 | 120 | 480 |
| 72 | 132 | 528 |

The events simulated include the durations given in Table 8-1 and a 60-hour duration nested storm combined with the annual exceedance probabilities 10%, 2%, 0.5%, 0.2% (all combinations so 40 triangular storms and 4 nested storms).

No other parameters were changed associated with the different runs (except the duration, rainfall, and inflows associated with the event). For example, the infiltration parameters remain the same throughout all the design simulations.

# 9 Model Runs Results

## 9.1 Runtime performance

A memo written by GHD titled Citywide Flood Modelling - Proposed technical specification for model runtimes dated (15 July 2015) outlines the runtime objectives for the city-wide modelling projects. The target runtime specified is 7 hours for draft runs and 'not required' for final model runs.

The time taken to simulate the HDM Halswell models depends on:

- the hardware of the machine used to run the simulations,
- the software environment that the runs are simulated in (for example, the operating system and MIKE Flood specific settings),
- the other processes that are running on the machine in parallel. In particular, the proportions of the graphics processing unit (GPU) and central processing unit (CPU) that are available to be used by MIKE Flood, and
- several other simulation/event specific factors (for example, flow velocity and its impact on Courant number).

The Halswell model was run on two machines to speed up the total simulation time. See section 2.4 for the hardware specification of these machines. Figure 9-1 shows the time taken to simulate each design event with the Existing Development 2016 model. Most events take less than 18 hours. Generally, events that are longer in duration or have higher rainfall (lower annual exceedance probability) take longer. The events with higher rainfall likely take longer because there is more water on the MIKE 21 surface which requires more computation and, in some cases, smaller timesteps.

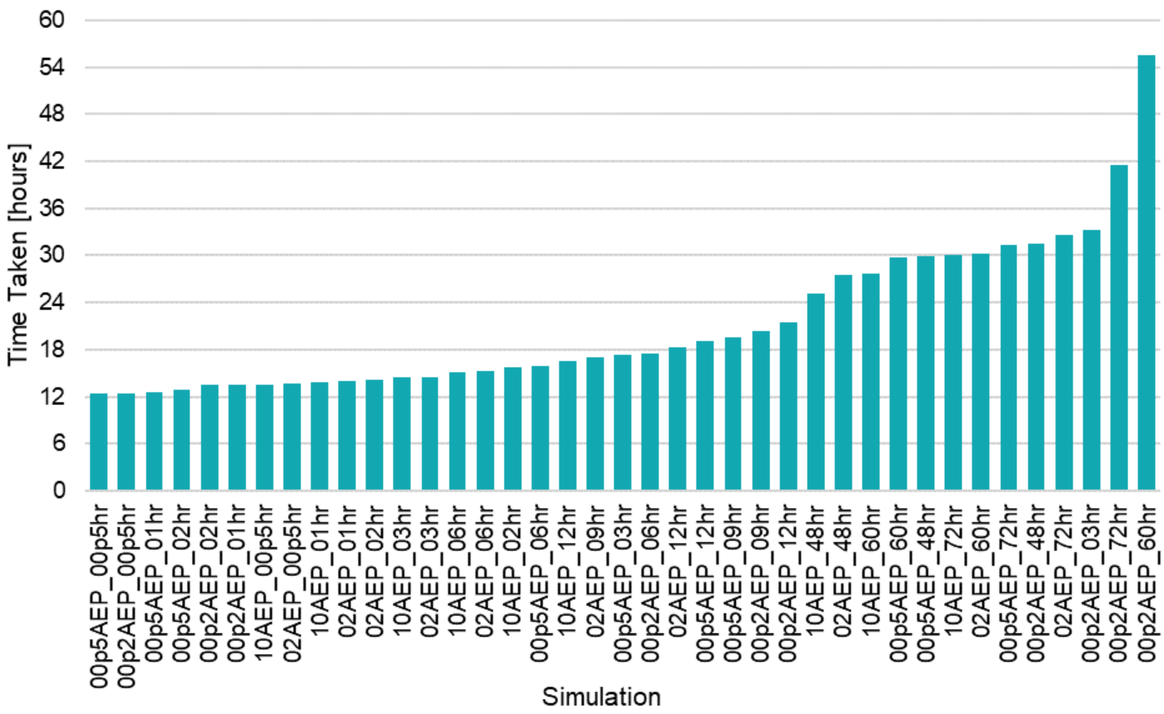


Figure 9-1: Runtime performance using two machines with the Existing Development 2016 model

# 10 Compliance with Specifications

## 10.1 Identified issues of low importance

While building the Halswell model and upon reviewing the results some minor issues were identified. Generally, as issues were found they were resolved but those that required significant effort with little improvement were instead recorded in an anomalies-register. This was done to keep the project moving and allow important results to be provided in a timely manner without spending too much time on inconsequential details. Here are two examples:

- The Existing Development 2016 model contains dykes representing top-of-banks around basins that were not built at the time of the most recent LiDAR being flown. They have been disabled in the model so do not impact the results at all. They can be removed in future to avoid confusion.
- A pipe on Westlake drive flows to Westlake Reserve from a small suburban catchment. It then flows on to Nottingham Stream. This flow is accounted for using a network load in MIKE Urban that is set to the pipe full capacity for the duration of the rainfall event. This is accurate enough but may overestimate the volume during long events. The model extent could be increased to include the small suburban catchment so that runoff from it is modelled explicitly.
- In several locations in the south of the catchment (well south of the Christchurch City Boundary) there are farm drain culverts whose location was erroneous. This means water would pond and overtop the road leading to deeper flooding upstream and less flooding downstream.

## 10.2 Recommendations for Model Improvement

This section is to be completed after the completion of the Existing Development 2021 runs.

# 11 References


Christchurch City Council. (2020, 10 22). *Christchurch District Plan Maps*. Retrieved from https://districtplan.ccc.govt.nz/pages/plan/book.aspx?exhibit=DistrictPlan

Opus. (2016). *Huritini / Halswell River Catchment: Vision and Values.* Christchurch City Council.

# A

Appendix A – RORB Manual Runoff Concepts

# APPENDIX A: RUNOFF ROUTING CONCEPTS

The purpose of this appendix is to introduce readers to some of the concepts of runoff-routing as they apply to RORB, i.e. to help explain why the model was formulated the way it was. It begins (Section A.1) by showing how a series of concentrated storages can represent the attenuation and translation effects of a catchment on the rainfall-excess hyetograph. Section A.2 provides theoretical and empirical justification of the power function equation $S = kQ^m$ used to simulate reach storage-discharge behaviour and indicates the factors affecting $k$ values. Some justification for splitting the parameter $k$ into catchment ($k_c$) and reach ($k_r$) components is given in Section A.3, along with the ideas behind the use of $d_{av}$ as an intermediate parameter in the model. To conclude, Section A.4 summarizes the main advantages of the approach adopted in the RORB program.

## A.1 Simulation of Catchment Behaviour

We begin by considering a storm large enough to produce runoff at the outlet of a catchment. If the hyetograph of rainfall-excess (i.e. the rainfall hyetograph less the losses due to infiltration, etc.) is plotted *on the same scale as* the observed hydrograph at the catchment outlet, the resulting figure shows the relationship of one to the other. Figure A-1 shows typical `input' and `output' curves in hydrograph units.

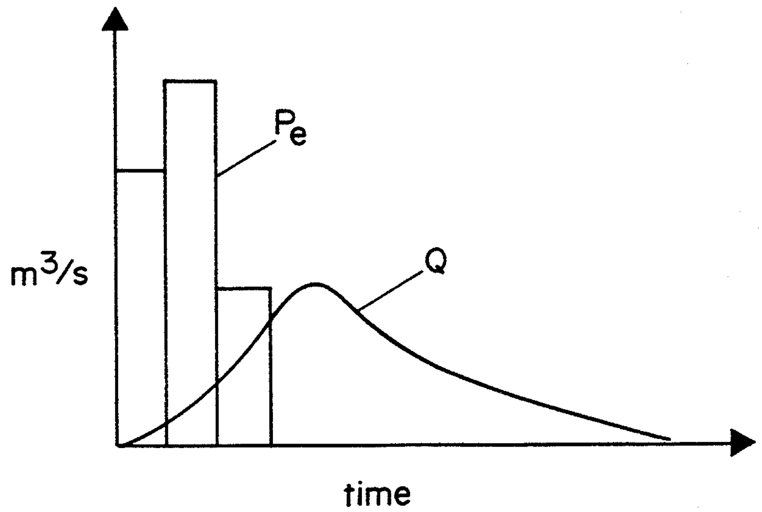


**Figure A-1 The Effect of Catchment Storage**

In Figure A-1, the volumes beneath the rainfall-excess hyetograph and the runoff `hydrograph' are equal. However, the latter is much flatter than the former, and its peak occurs later in time. That is, the effect of a catchment on the input hydrograph is like that of a storage in that it causes:

- *attenuation* of the input pattern; and
- *translation* of peak flows in time.

As shown in Figure A-2, the attenuation and translation effects of a catchment on the rainfall-excess hyetograph can be *simulated* by routing the input through a series of concentrated storages. The first routing through such a storage produces a hydrograph which peaks on the falling limb of the input hyetograph. The routing through the second storage produces further attenuation, and a peak more remote from the original input. Additional routings continue to flatten the hydrograph and to shift its peak further to the right. The routing of the rainfall-excess hyetograph through a string of concentrated storages thus can be seen to simulate the effect of a catchment on its rainfall-excess input.

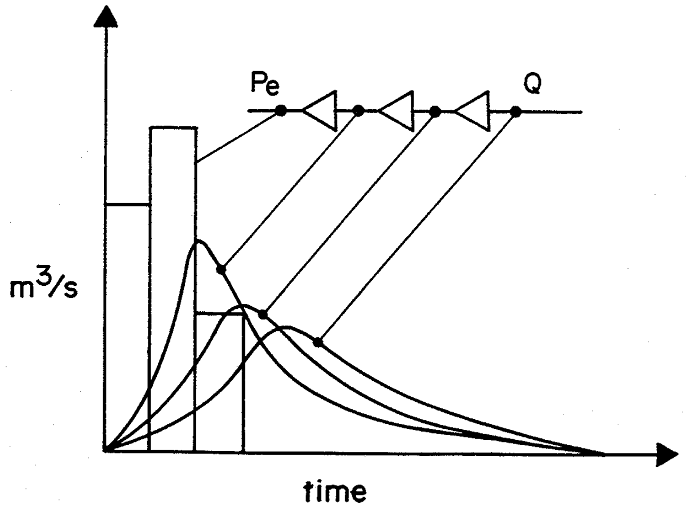


**Figure A-2 Simulation of Catchment Effect with a Series of Concentrated Storages**

Consider now Figure A-3, in which the concentrated storages have been placed on the major stream reaches of the catchment, which itself has been divided into a number of sub-areas draining to the stream system. The figure shows that the rainfall-excess for sub-area A, entered at the node on A, is routed through a series of storages on its way to the catchment outlet; runoff from sub-area B is acted on by the storages which lie on the streams between it and the outlet. Similarly for C, D, and E. With this formulation, the rainfall-excess hyetographs from different parts of the catchment are routed through an amount of concentrated storage which depends on the remoteness of each area from the outlet.

As will be explained in Section A.4, such subdivision of the catchment provides the opportunity to model spatial variability of rainfall and losses, different reach characteristics, and separate allowance for major storages which may exist (or are proposed) on the catchment.

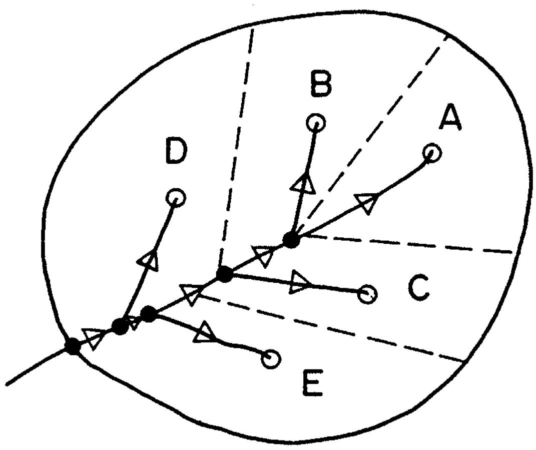


**Figure A-3 Distributed Storage Network to Represent a Catchment**

## A.2 Representation of Reach Storage ($S = kQ^m$)

The form of the storage equation $S = kQ^m$ is justifiable by both theoretical and empirical means. Theoretically, application of a uniform flow equation in a prismatic channel will yield an equation of this form as shown below.

Consider Figure A-4 which shows a reach (length, $L$) of a channel with a triangular cross-section. The depth of flow in the channel is $y$, the Manning roughness is $n$, the bed slope is $S_b$ and the channel side slopes are $z$ horizontal to 1 vertical. The water 'stored' in the channel is given by the product of the cross-section (area $A$) and the length, i.e.

$$S = AL = zy^2L \qquad \textit{Equation A-1}$$

Using Manning's equation, the discharge $Q$ can be expressed as

$$Q = vA = \frac{1}{n}\left(\frac{zy^2}{2y\sqrt{1+z^2}}\right)^{2/3} S_b^{1/2} zy^2 \qquad \textit{Equation A-2}$$

*Equation A-1* and Equation A-2 can be combined to eliminate $y$. The result is an equation in the form $S = kQ^m$ where $m = 0.75$ and

$$k = \frac{n^{0.75}(2\sqrt{1+z^2})^{0.5} L}{z^{0.25} S_b^{0.375}} \qquad \textit{Equation A-3}$$

Similar analyses can be made for other cross-section shapes (e.g. Ref. 1)

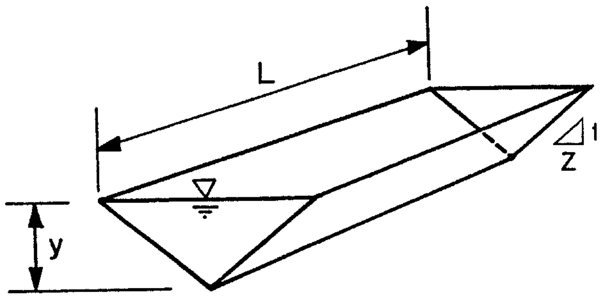


**Figure A-4 Uniform Flow in a Prismatic Channel of Triangular Cross-section**

### A.2.1 Value of exponent *m*

Analyses like the one above, can be performed for other cross-section shapes to produce the following exponent values:

| | |
|---|---|
| Triangular cross-section | m = 0.75 |
| Trapezoidal cross-section | m = 0.74 |
| Parabolic cross-section | m = 0.69 |
| Wide rectangular channel | m = 0.60 |

The results are for *uniform flow* in open channels. However, similar *m* values, ranging from 0.68 to 0.8, have been reported in *field studies* using natural catchments (Ref. 1); these studies included two events with overbank flow. More recent experience with runoff-routing models has led the authors to adopt an 'average' *m* value of 0.8, although other values in the range 0.6-1.0 are frequently encountered. [It might be noted here that an *m* value of 1.0 implies a *linear* model of catchment response.]

### A.2.2 Value of *k*

The relation for *k* given by Equation A-3 shows the nature of its dependence on channel roughness, cross-section shape, bed-slope, and length. Such a relationship could be of use for routing in *prismatic* channels where these parameters can be estimated. In *natural* river channels the effects of slope and roughness often tend to be compensatory; in such cases, the *k* value can be taken as proportional to *L*. In *urban* catchments with lined channels, this compensating effect is missing so the effect of slope on reach storage must be specifically included.

## A.3 Splitting $k$ into $k_c$ and $k_r$

The time between the occurrence of a particular element of rainfall-excess at a particular point on the catchment and its effect at the outlet is called the storage delay time for that point. On a linear catchment, it is constant for any point but on a nonlinear one it varies with discharge.

It was proposed by Laurenson (Ref. 2) that the *lag* of a catchment (i.e. the time between the centroid of rainfall-excess and the centroid of the resulting surface runoff) was equal to the *average storage delay time* for all elements of rainfall-excess throughout the storm and over the entire catchment. Assuming areally uniform rainfall-excess, this means that lag is equal to the storage delay time of points on the catchment corresponding to the centroid of the time-area diagram. On nonlinear catchments, both the lag and the storage delay times of all points were assumed to vary with discharge. It was also assumed that the storage-delay time value of the centroid of the time-area diagram varied with outlet discharge in the same way that lag varied with mean outlet discharge of the flood from which it was derived. This latter relation was found empirically to have the form

$$t = k_c Q^p \qquad \textit{Equation A-4}$$

where $t$ = lag($h$) and $Q$ = mean outlet discharge $m^3/s$ and $k_c$ and $p$ are constants, $p$ having a value of the order of -0.25. Adopting a catchment storage model of the form shown in Figure A-3, the storage delay time (in hours) of a storage i is thus assumed to have the form

$$k_i = k_c k_r Q^p \qquad \textit{Equation A-5}$$

implying a storage function

$$S_i = 3600\, k_c k_r Q^{p+1} \qquad \textit{Equation A-6}$$

where $k_r$ is the *relative* delay time of storage i, i.e. the ratio of its delay time to that of the centroid of the time-area diagram and $S_i$ is in $m^3$.

The RORB program computes the catchment mean travel distance (the distance from the outlet to the centroid of the time-area diagram) from the reach lengths input in the catchment data file. If there are $n$ sub-catchments and the **i**th one has area $a_i$ (km$^2$) and a travel distance to the *catchment* outlet $d_i$(km), then the mean travel distance $d$ is given by:

$$d_{av} = \sum_{i=1}^{n} (a_i d_i) / A \qquad \textit{Equation A-7}$$

where A is the total catchment area (km$^2$). Considering the catchment storage between two adjacent nodes of the model, **i** upstream and **j** downstream, its $k_r$ value is given in dimensionless form by

$$k_r = (d_i - d_j)/d_{av} \qquad \textit{Equation A-8}$$

Such relative delay times will have values of the order of 0.1.

With $k_r$ expressed in its dimensionless form, $k_c$ becomes equal to the coefficient of the formula for delay time of the centroid of the time-area diagram and of the lag-mean discharge formula (*Equation A-4*). The advantage of this is that $k_c$ can be estimated directly from lag formulae.

## A.4 Advantages of the Runoff Routing Approach

The main advantage of the runoff-routing procedure over methods which are based on unitgraphs is its **flexibility**; the catchment formulation adopted in RORB can easily account for:

- variability of rainfall depth over the catchment;
- variability of rainfall pattern over the catchment; variability of losses over a catchment;
- nonlinearity of catchment response;

- the modelling of existing or proposed storages in the catchment, using equations appropriate to each specific storage;
- the modelling of the effects of reaches `drowned out' by reservoir inundation;
- the modelling of flows in branched networks;
- the effects of urbanization of all, or part of, the catchment; and,
- the ability to represent other forms of runoff-routing model.

This flexibility comes from the subdivision of the catchment into sub-areas (assumed homogenous), the routing of the runoff from each part of the catchment through the appropriate amount of reach storage, and the separate modelling of significant artificial or natural storages . The formulation allows for the output of calculated hydrographs at any point in the catchment, and thus for the user to compute design flows for any region of interest.

*Authors' Note*: The power function form of the storage-discharge relation (Section A.2), while valid for most applications, may not always be the most suitable for a stream reach. The user should decide – using FIT runs and judgement - whether the formulation is suited for the task at hand.

# B

Appendix B – Halswell RORB Calibration

# Memorandum

| | | | |
|---|---|---|---|
| **To:** | Elliot Tuck | **Date:** | 9 May 2018 |
| **From:** | Natasha Webb | **Our Ref:** | 3361713 |
| **Copy:** | Mike Law | | |
| **Subject:** | Halswell Calibration | | |

This memorandum outlines the Halswell Hillside RORB modelling. The modelling was split into three RORB models, where two models for the Halswell Hillside catchment area and one calibration model of the Kaituna catchment. The memo sets out the catchment delineation, model build, calibration and calibration results.



## 1 Catchment Delineation

The three RORB models developed are shown in Figure 1 and the following section outlines the catchment delineation for each model.

### 1.1 Halswell Hillside Catchments

The Halswell Hillside area modelled in RORB, as shown in Figure 1, has a total catchment area of 67.41km2. To simplify the model, the hillside area was split into two separate RORB models based on catchment size and type.

For the hillside area located within Christchurch City Council boundary, the sub-catchments were sized from 0.1km2 to 0.5km2 to provide sufficient detail for urban/rural catchments. This level of detail is the same used in the City Wide Model currently under construction. This RORB model was called Halswell CCB and is shown in Figure 2. Halswell CCB has a total catchment area of 10.49km2.

For the hillside area located outside the Christchurch City boundary, the sub-catchments were sized from between 0.5km2 to 5.6km2. The larger sub-catchment sizes were used as the hillside area of the city is rural and does not require the same level of detail. The second RORB model is called Halswell 02 and is shown in Figure 3. Halswell 02 has a total catchment area of 56.58km2.

### 1.2 Kaituna Catchment

The Kaituna catchment was selected to calibrate the Halswell RORB model because it is a rural catchment located further south around the Port Hills from the catchment used to calibrate the Heathcote River model. The Kaituna catchment provides a better representation of the hillside area within the Halswell River Model. The Kaituna catchment, shown in Figure 4, has a total catchment area of 39.93km2 and is a gauged catchment (operated by Environment Canterbury, ECan), with flow recorder and rainfall gauge located in the valley floor at Kaituna Road (located at node M1). As the Kaituna catchment is located outside the city boundary, the sub-catchments were sized from 0.5km2 to 5.6km2.

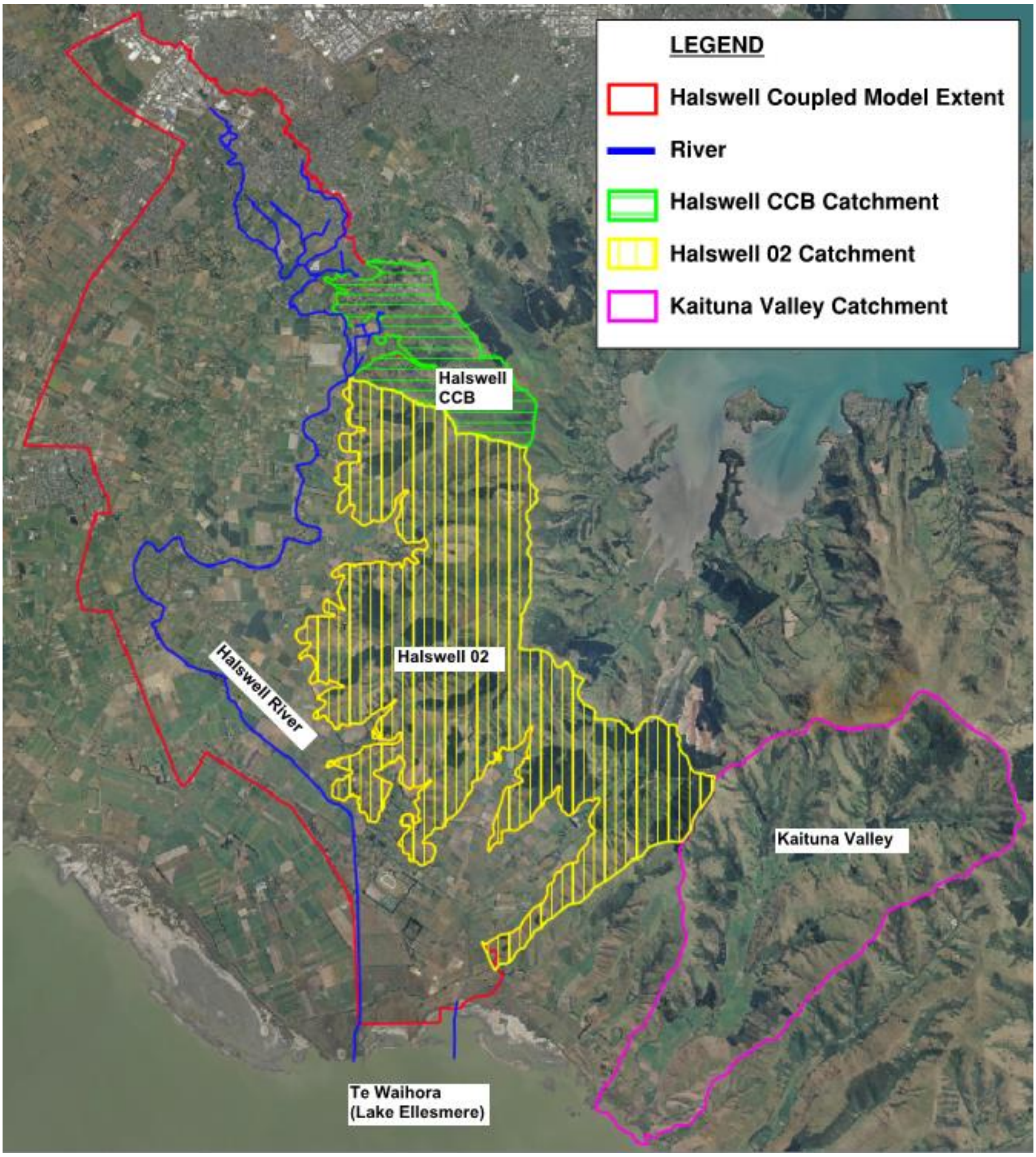


**Figure 1 – Halswell Hillside RORB model catchments (Halswell CCB, Halswell 02 and Kaituna Valley)**

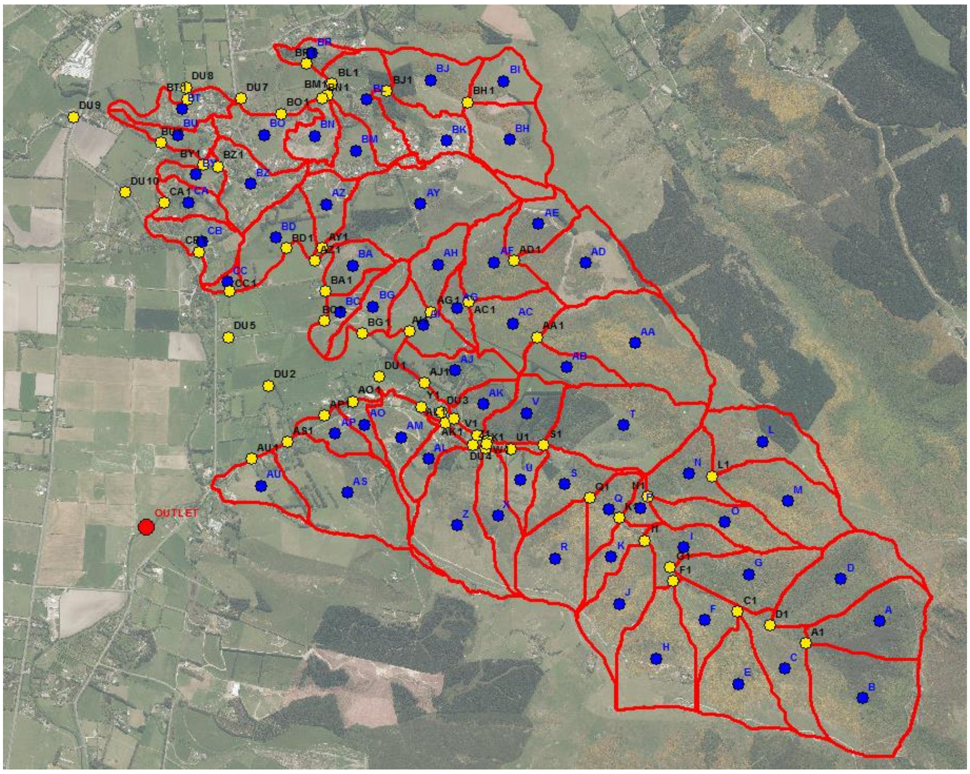

**Figure 2 – Halswell CCB RORB Model Sub-catchments**

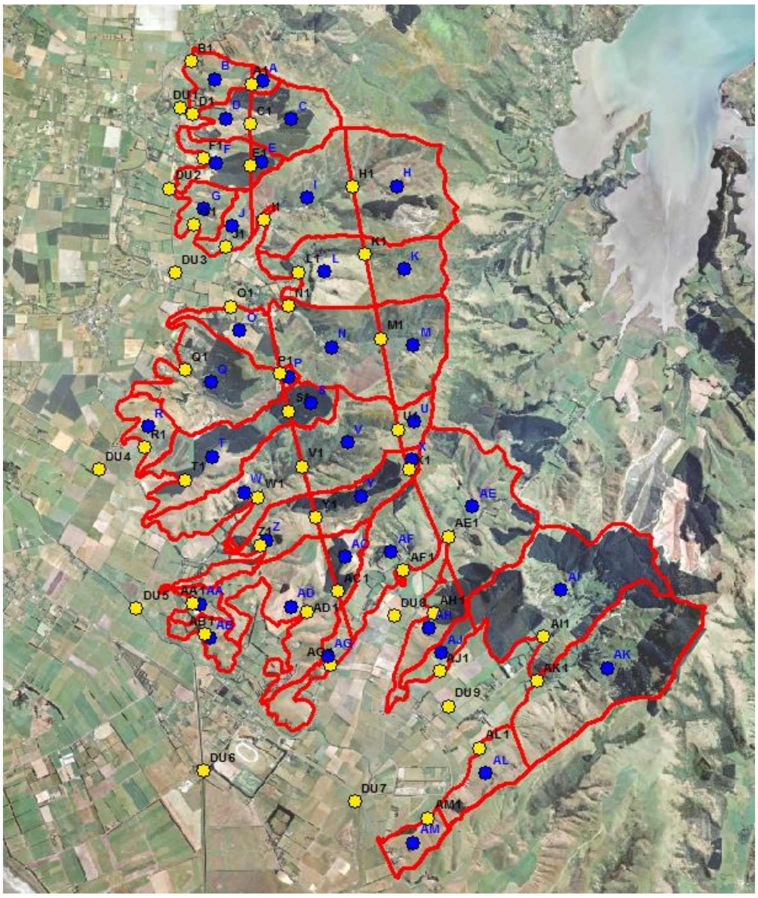

**Figure 3 – Halswell 02 RORB Model Sub-catchments**

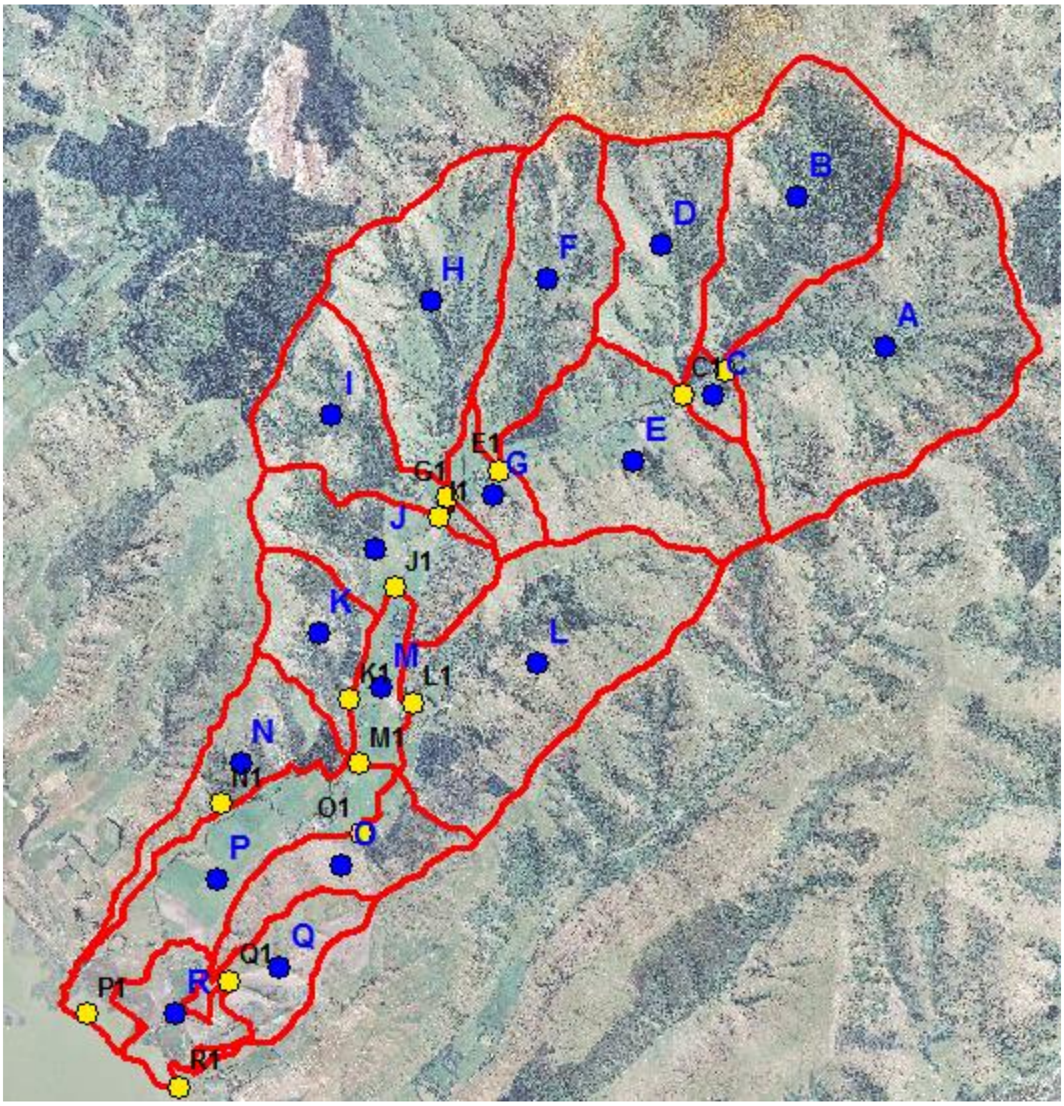


**Figure 4 – Kaituna Catchments**

## 2 RORB Model Build

All the RORB models were built using ArcMap and the arcRORB tool. Using the arcRORB tool, the sub-catchments were converted into RORB and the reaches, catchment centroids (shown as Blue dots) and junctions (shown as Yellow dots) were defined. In both RORB models, the reaches were defined as natural channels, where reach slope was generated by arcRORB tool using the equal area method and 5m contours.

For the urban sub-catchments in Halswell CCB model, the fraction impervious was calculated based on the Christchurch City Council's Waterways, Wetland and Drainage Guidelines (WWDG, 2003) for district zone LH. As the Halswell 02 model and Kaituna model are rural catchments, the sub-catchments were set as 100% pervious.

# 3 Calibration

The Kaituna RORB model was calibrated for the four calibration events listed in Table 1. For each of these events, the Kaituna River flow and Kaituna rainfall records were collected from Ecan. The rainfall records were processed into a RORB storm file and run through RORB to generate the runoff hydrographs at the flow recorder location (see M1 in Figure 4).

**Table 1 – Calibration Events for Kaituna RORB Model**

| Calibration Event | Event Dates |
|---|---|
| October 2000 | 12th October 2000 |
| May 2006 | 12th – 13th May 2006 |
| June 2013 | 15th – 22 June 2013 |
| March 2014 | 4th – 5th March 2014 |

However, initial calibration runs showed that for all of the calibration events, the rainfall depth recorded at the Kaituna gauge was too low to generate the runoff volumes required to meet Kaituna River flow volumes[1]. We confirmed that there was insufficient rainfall in the Kaituna record by calculating the expected total rainfall depth required to produce the required river flow volume (see Table 2). Some of this volume maybe accounted for as base flow, which cannot be added to the RORB model.

The Kaituna rainfall gauge is located on the valley floor and may not be representative of the rainfall the catchment receives, especially on the upper slopes. For this reason the decision was taken to, use additional rainfall gauges located at higher altitudes nearby thereby providing a comparison to the Kaituna rain gauge and to calibrate the RORB model. The two additional gauges selected are Coopers Knob (ECan site) and Akaroa Highway (NIWA site).

At both sites, the rainfall record was processed for each event, run through the Kaituna RORB model and the total rain depth calculated (shown in Table 2). Coopers Knob gauge did not have a complete record for October 2000 event and this was event was not include in the calibration runs. Akaroa Highway gauge open in November 2012 and accordingly, the October 2000 and May 2006 events were not included in the calibration run.

## 3.1 Calibration Results and Discussion

Table 2 summaries the expected excess rainfall (Total rainfall minus losses) depth calculation and the total rainfall depths for each gauge. As mentioned above, the excess rainfall depth at the Kaituna flow recorder is lower than expected. Although Cooper’s Knob shows similar rainfall depths to the expected rainfall depth, Cooper’s Knob gauge rainfall may not have enough rainfall to meet river flow volumes since this depth calculation does not take into account infiltration losses across the catchments. Akaroa Highway has recorded a lot more rainfall than either the Kaituna or Coopers Knob gauges.

[1] Note that this approach assumes that the flow rating for the Kaituna flow recorder is accurate, especially at high flows.

**Table 2 – Total Rainfall Depth Comparison**

| Calibration Events | Expected Excess Rainfall Depth – calculated from Kaituna River flows (mm) | Total Rainfall Depth (mm) | | |
|---|---|---|---|---|
| | | Kaituna Rainfall Gauge | Coopers Knob Gauge | Akaroa Highway Gauge |
| October 2000 | 37.8 | 59.5 | | |
| May 2006 | 42.3 | 38.5 | 54 | |
| June 2013 | 230.2 | 168 | 225 | 294 |
| March 2014 | 60.6 | 70 | 88 | 341.2 |

Figure A1 to Figure A8 (see attachments) show the best fit of the runoff hydrographs for the each calibration event at each of the gauge sites. Table 3 outlines the design parameters for each event at each gauge site. Figure A1 to Figure A8 show that both the Kaituna rainfall and Cooper's Knob rainfall do not have enough rainfall to meet river flow volumes. Conversely, the Akaroa rainfall provides too much rainfall, which requires large unrealistic initial losses (up to 30 mm) and continuing losses (up to 11.25 mm/h) for the March 2014 event.

These figures also show the runoff hydrographs of AECOMs proposed RORB design parameters for a $k_c$ of 2, m of 0.8, an initial loss of 17mm and a continuing loss of 1.25 mm/h. From these graphs, it can be seen that AECOMs values of a kc of 2 has generated hydrograph peaks higher and quicker than what was recorded and therefore does not provide a good fit for the Kaituna catchment. This is true for all of the calibration events.

Table 3 suggests that the Kaituna catchment requires a kc between 11.9 to 13 (for the June 2013 and March 2014 events) or a $k_c$ between 5 to 7 for (October 2000 and May 2006 events) to match up the peak discharge and timing from the catchments. In the RORB manual, the calculation for $k_c$ for a catchment area of 39.93km2 is 13.9. <u>Therefore, a $k_c$ of 13 would be reasonable for the Kaituna catchment.</u>

Similarly, if the Akaroa rainfall runs are not used, the initial losses are between 13mm to 17.5mm and the continuing losses are between 0.3mm/h to 1.27mm/h, which are similar to AECOMS proposed loss parameters. Therefore, an initial loss of 17.5mm and a continuing loss of 1.25mm/h would be reasonable for the Kaituna catchments.

**Table 3 – Design Parameters for each Calibration event**

| Calibration Event | Gauge | $k_c$ | m | Initial Loss (mm) | Continuing (mm/h) |
|---|---|---|---|---|---|
| **AECOM** | AECOM | 2.0 | 0.8 | 17.5 | 1.25 |
| **Oct-00** | Kaituna | 5.0 | 0.8 | 17.5 | 1.27 |
| May-06 | Kaituna | 7.0 | 0.8 | 15.0 | 0.30 |
| | Coopers Knob | 6.3 | 0.8 | 17.0 | 1.72 |
| Jun-13 | Kaituna | 12.5 | 0.8 | 25.0 | 0.00 |
| | Coopers Knob | 11.9 | 0.8 | 25.0 | 0.45 |
| | Akaroa Highway | 12.5 | 0.8 | 13.0 | 1.00 |
| Mar-14 | Kaituna | 13.0 | 0.8 | 30.0 | 0.00 |
| | Coopers Knob | 11.9 | 0.8 | 30.0 | 0.00 |
| | Akaroa Highway | 16.25 | 0.8 | 30.0 | 11.25 |

Using these design parameter selections, the calibration events at each rainfall gauge were rerun. Figure 5 to Figure 8 show the runoff hydrographs for the each calibration event at the gauge site that provided the best fit. The June 2013 event runoff hydrograph shows the best match to the Kaituna river flow (see Figure 7). Although there is not enough rainfall in the Kaituna rainfall record, using the selected design parameter has matched up the timings of the peaks for the June 2013 event. For the other events, the selected design parameters will not match the Kaituna river flow hydrograph. However, the resulting runoff hydrograph match the timings of the peak river flows for the October 2000 and May 2006 events.

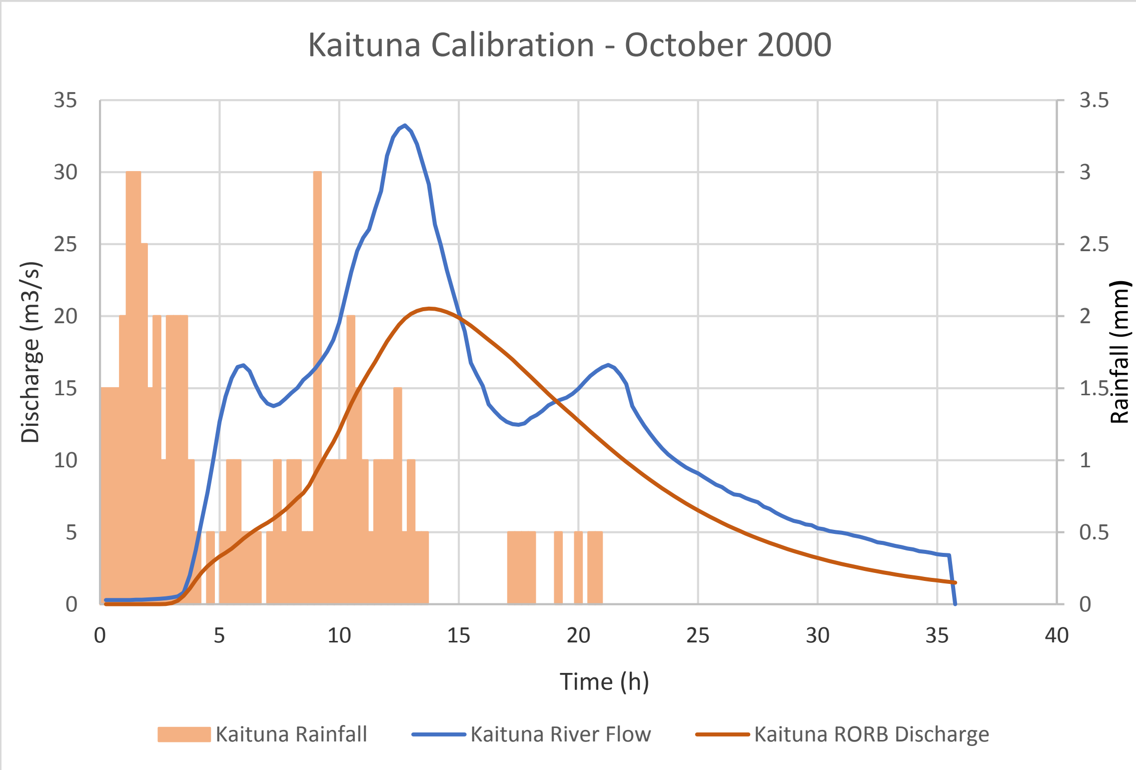


**Figure 5 – October 2000 – Kaituna Gauge**

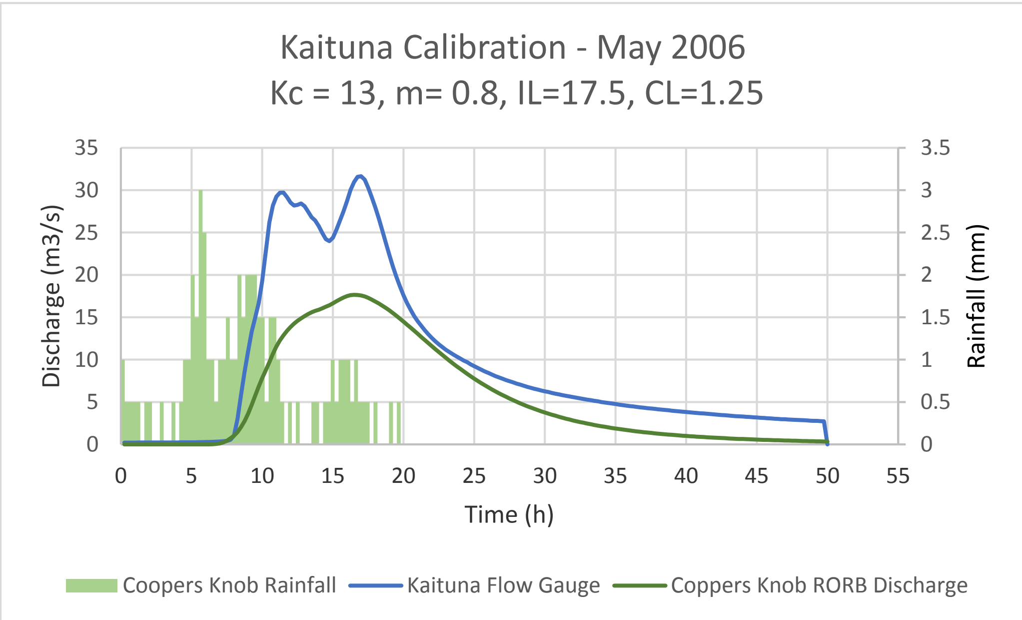


**Figure 6 – May 2006 Calibration Event – Coopers Knob Gauge**

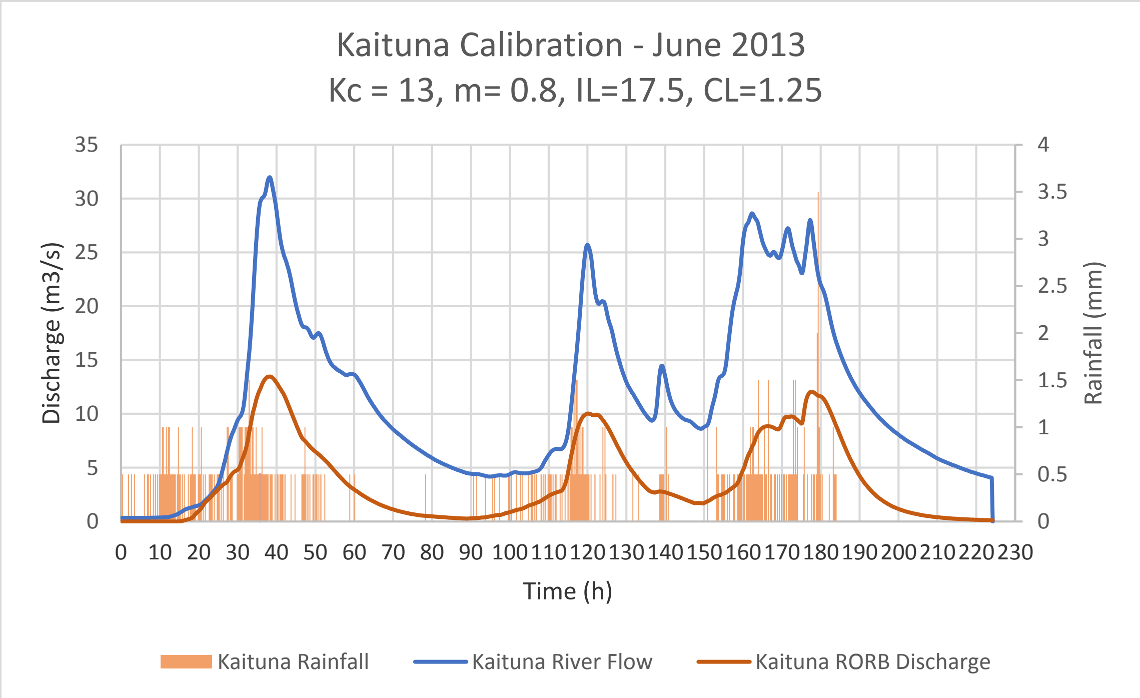


**Figure 7 – June 2013 Calibration Event – Kaituna Gauge**

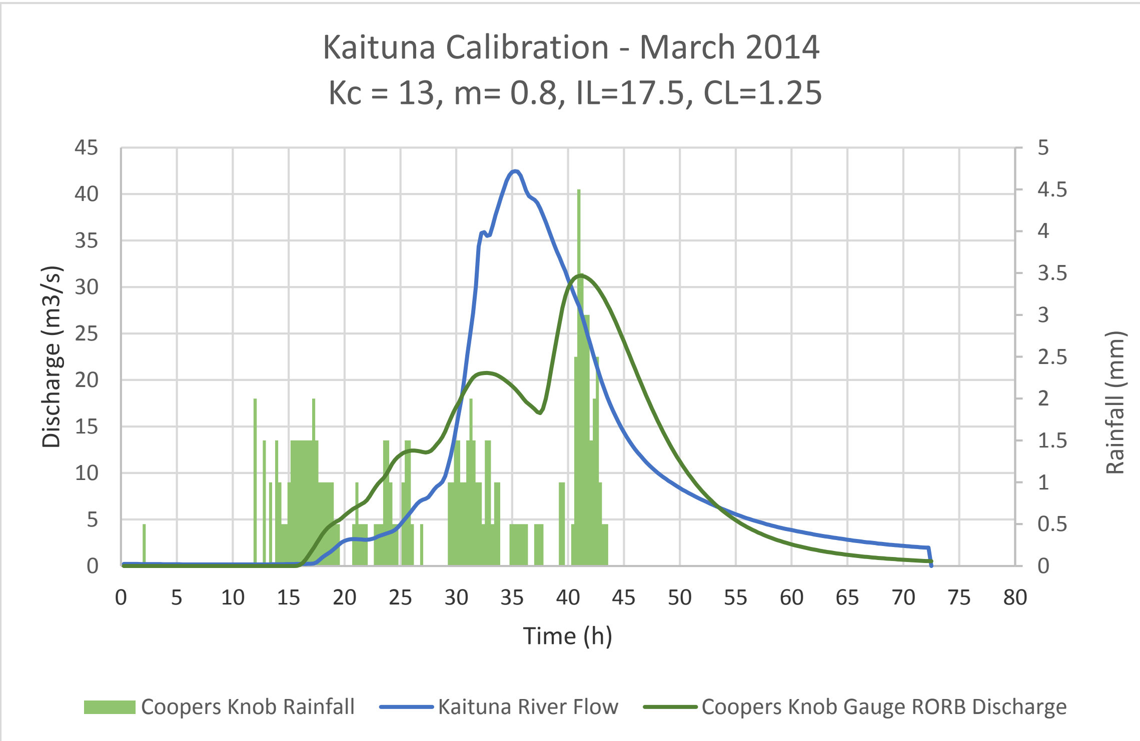


**Figure 8 – March 2014 Calibration Event – Coopers Knob Gauge**

## 4 Conclusion

The Kaituna catchment was selected to calibrate the Halswell Hillside RORB models. The Kaituna model was run for four calibration events and with the rainfall records from the Kaituna, Coopers Knob and Akaroa Highway gauges. The runoff results were compared with the Kaituna River flow record. From these calibration runs, the following conclusions were made:

- The Kaituna rainfall gauge under estimates rainfall depths for all calibration events. Lower rainfall has meant that in comparison to the Kaituna river flow gauge, the runoff does not match the peak river flows. Similarly, Coopers Knob rainfall gauge also underestimates the runoff required to meet the Kaituna river flows for all calibration events.

- The Akaroa highway rain gauge over estimates the rain in the Kaituna Valley and consequently, produced unrealistic loss parameters during calibration.

- A $k_c$ of 2, as suggested by AECOM, is too low for rural catchments that include a flat area at the base of the hills. In the Kaituna model, it creates faster and peakier runoff hydrographs and cannot be used for the Halswell Hillside catchments.

- A $k_c$ of 13 should be applied to the Kaituna and the Halswell hillside model. A $k_c$ of 13 is close to the recommend $k_c$ (from the RORB manual) of 13.9 and aligns the timing of peak river flows with the peak runoff for the October 2000, May 2006 and June 2013 events. The closes alignment occurring for the June 2013 event.

- The initial loss of 17.5mm and a continuing loss of 1.25mm are reasonable to be used for Halswell hillside models. These loss values provide the closes alignment of river flows and runoff. They are the same values as AECOM's proposed loss parameters.

The under estimating and over estimation of rainfall depths has made calibration of the model difficult. Consequently, the selected parameters do not provide a good fit with recorded flows for all calibration events. However, the selected parameters are robust, aligning peaks and are close to the expected RORB values.

**Natasha Webb**
Civil Engineer
Direct Dial: +61 2 8216 4607
Email: natasha.webb@beca.com

## Attachments

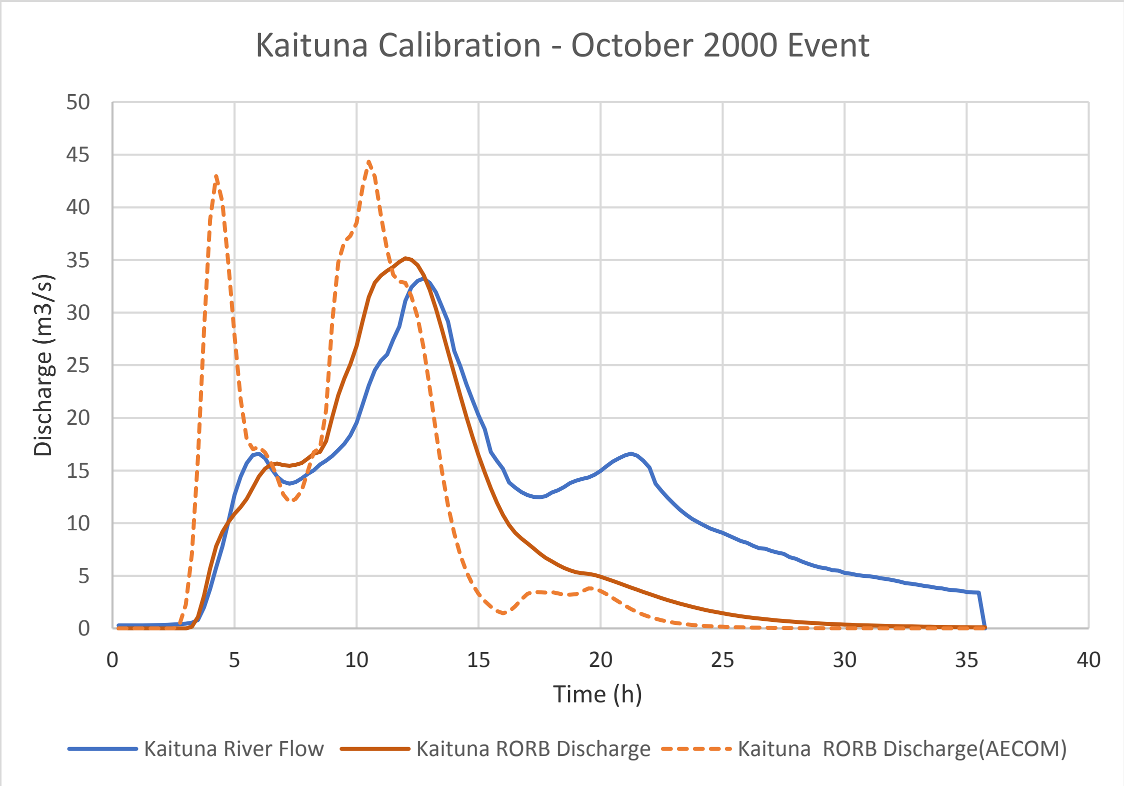


**Figure A1 – October 2000 Calibration Event**

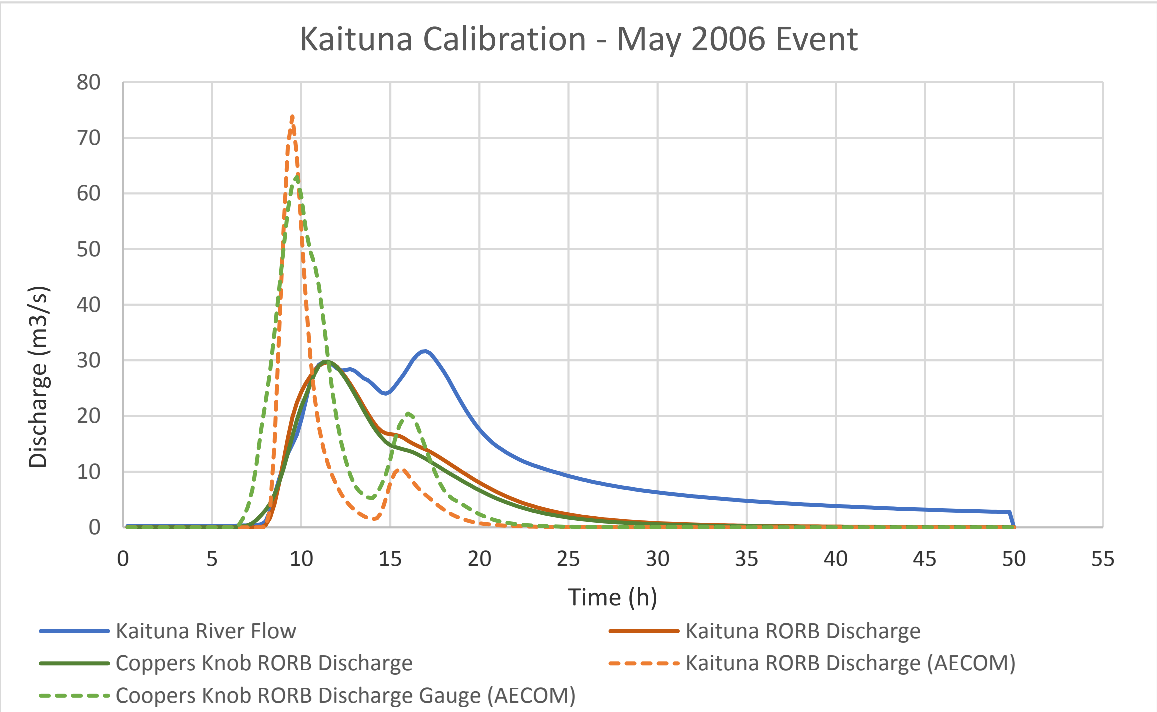


**Figure A2 - May 2006 Calibration Event**

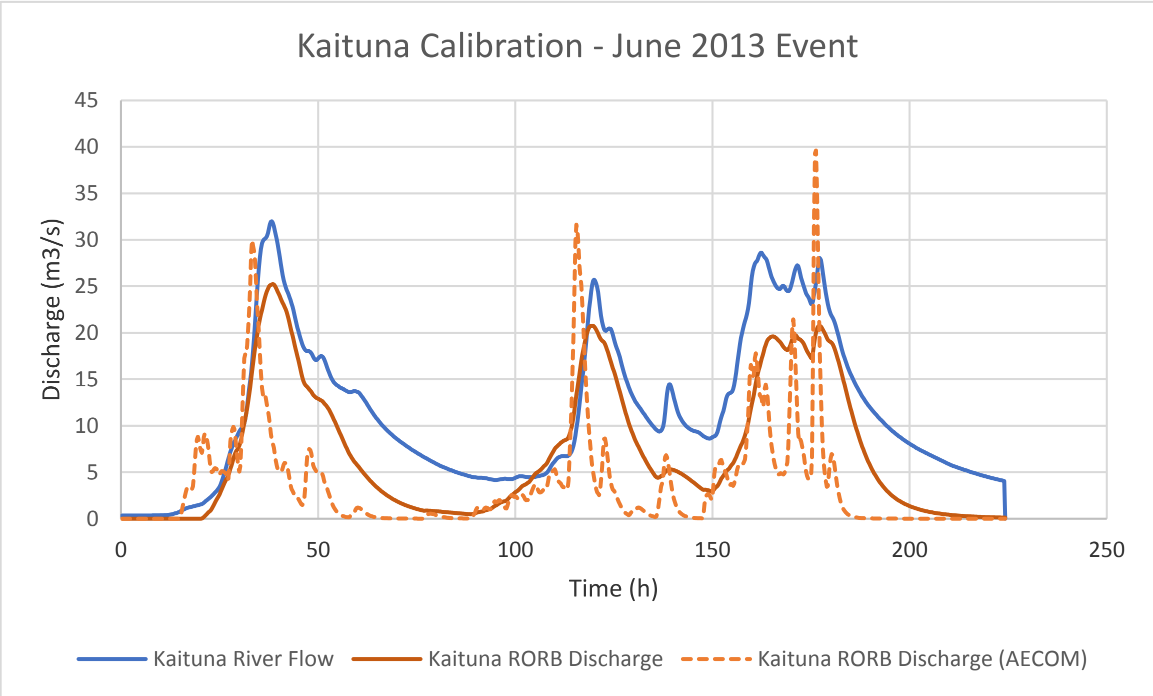


**Figure A3 – June 2013 Calibration Event – Kaituna Gauge**

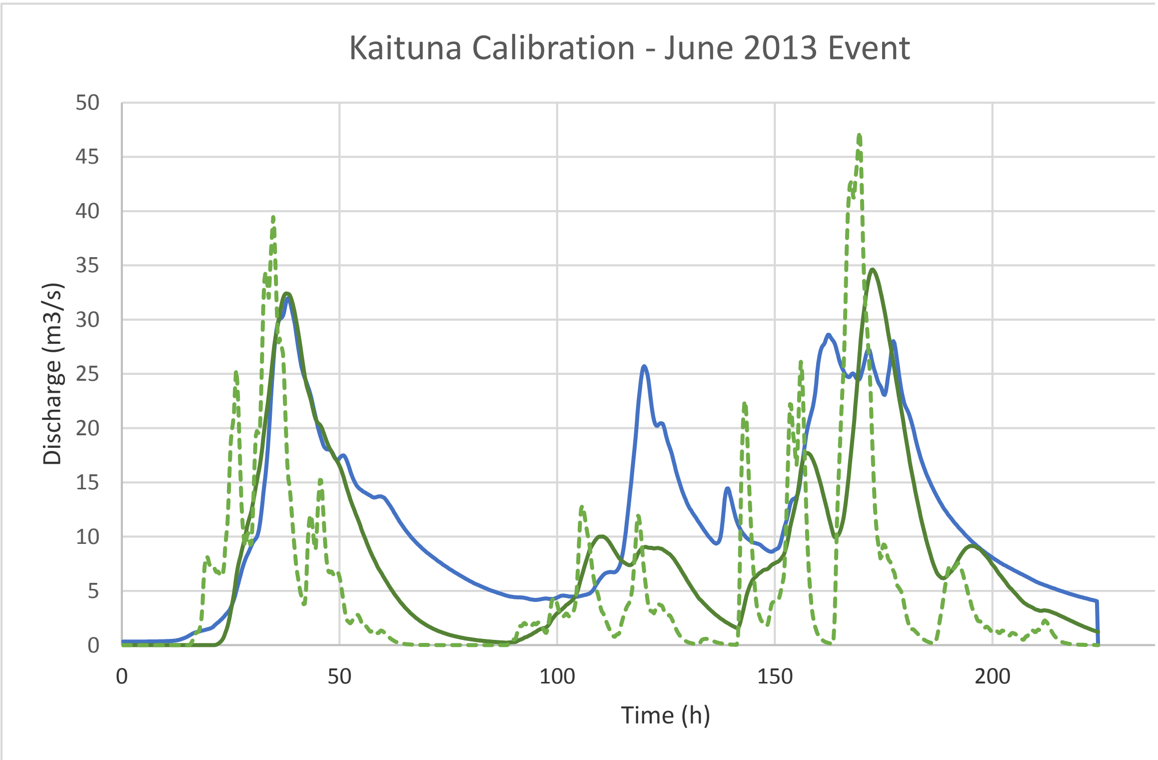


**Figure A4 –June 2013 Calibration Event – Coopers Knob Gauge**

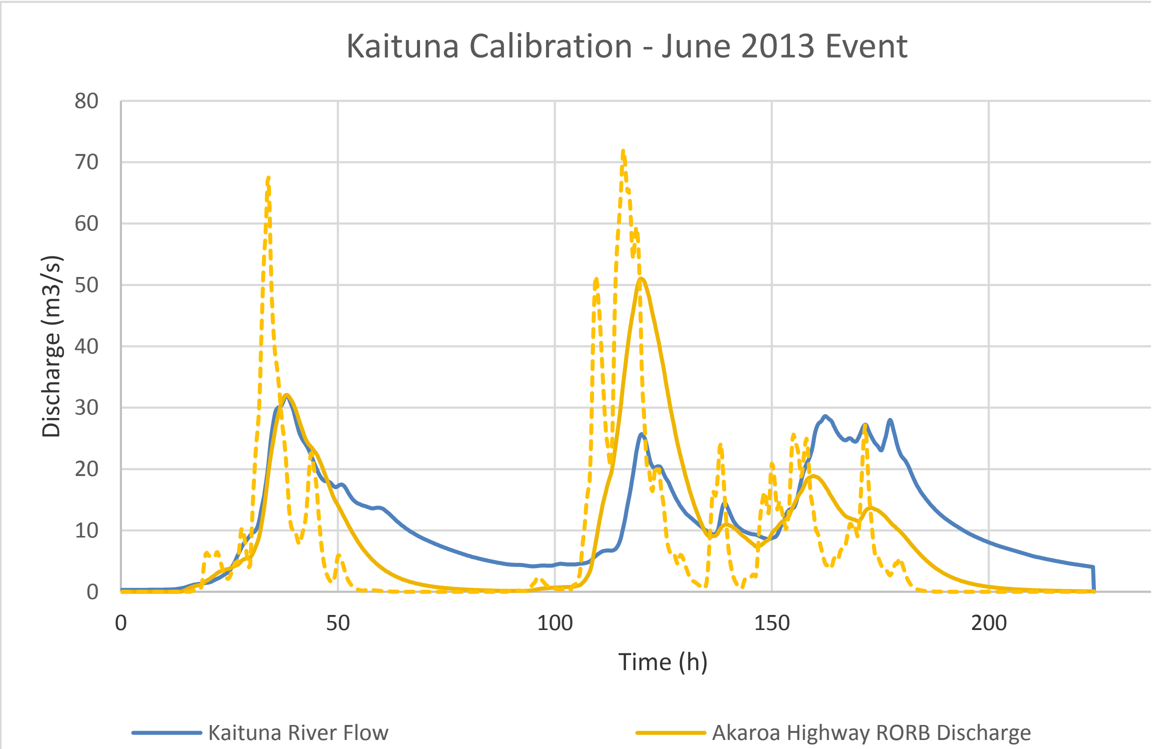


**Figure A5 – June 2013 Calibration Event – Akaroa Highway Gauge**

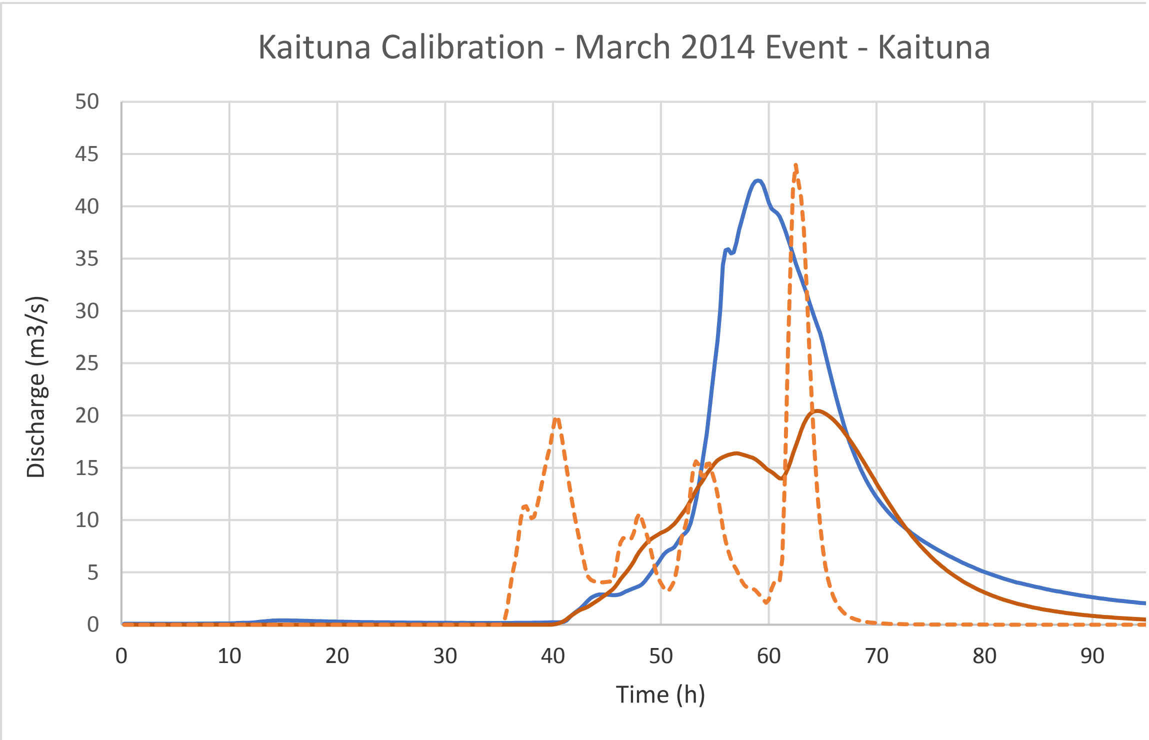


**Figure A6 – March 2014 Calibration Event – Kaituna Gauge**

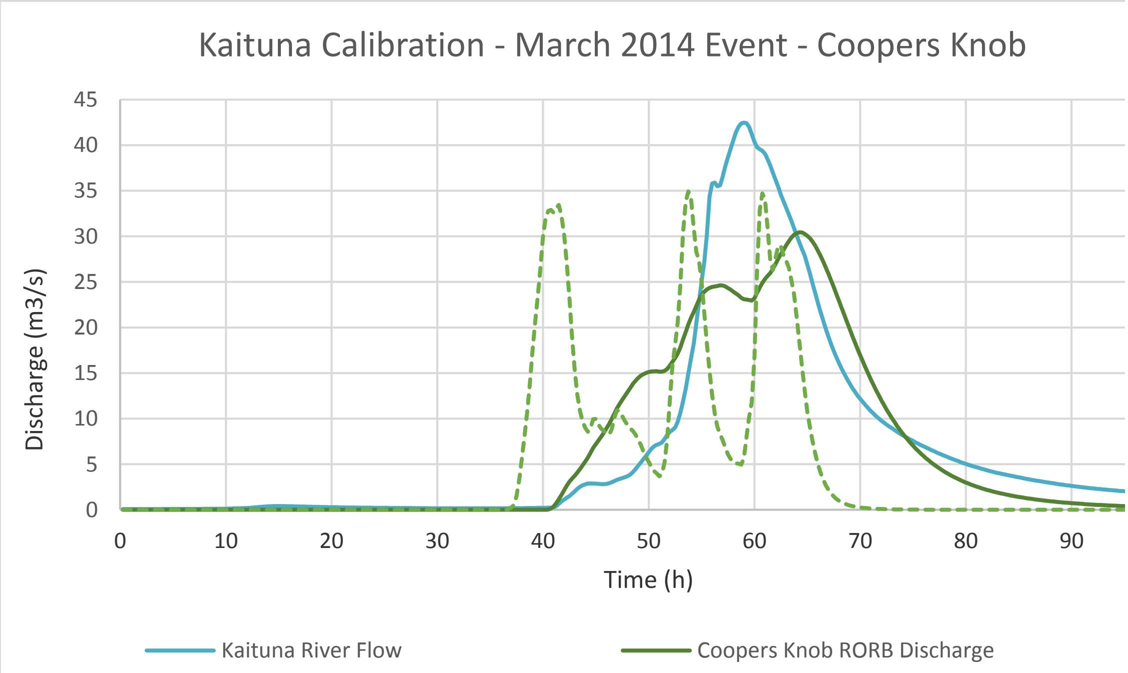


**Figure A7 – March 2014 Calibration Event – Coopers Knob Gauge**

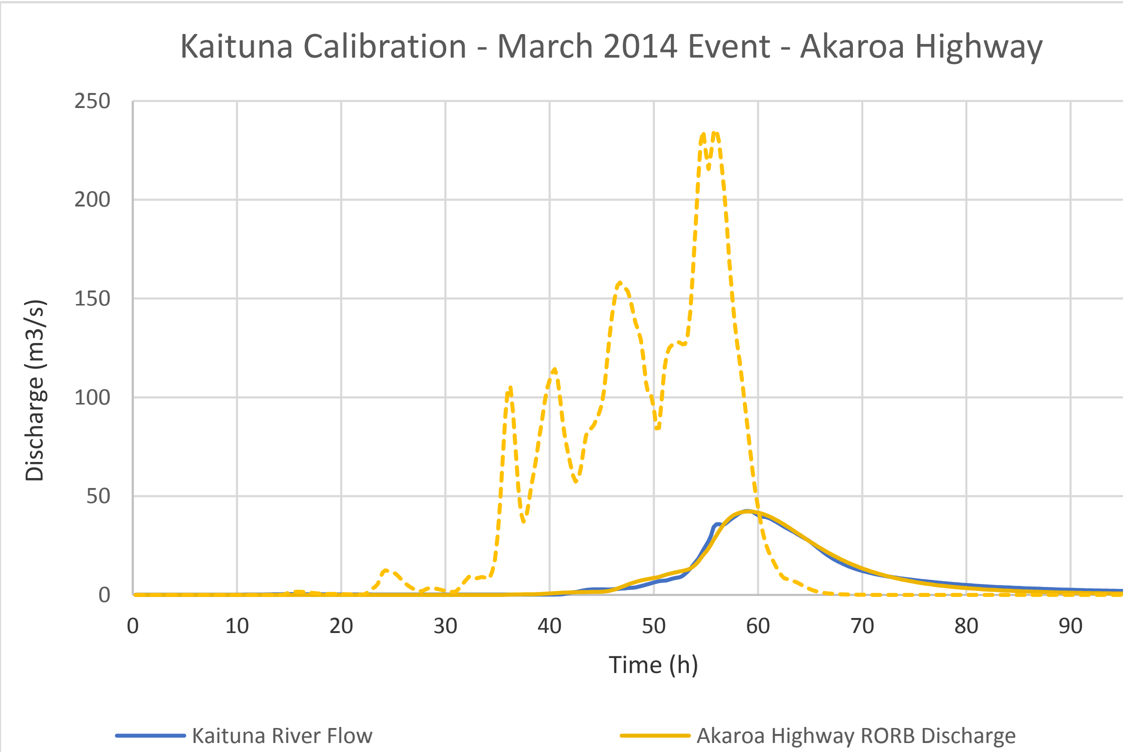


**Figure A8 – March 2014 Calibration Event – Kaituna Gauge**

# C

Appendix C – CCC Citywide Methodology, A Simplified Approach

# Memorandum

**To:** Elliot Tuck **Date:** 22 September 2017

**From:** Sher Khan **Our Ref:** 3361713

**Copy:** Graham Levy; Kate Purton

**Subject:** Christchurch City Council's Citywide Modelling Methodology. A Simplified Approach

## 1 Background

Christchurch City Council introduced a new citywide modelling approach (CCC-Citywide Flood Modelling (LDRP 044), 2017). The purpose was to keep the modelling work consistent throughout CCC area and increase the level of details in Mike by DHI models. At the time of this memo the modelling methodology was still in draft form and changes were expected in the final report.

The relevant specifications to this Citywide Flood Modelling Methodology are:

- Waterway and Wetland Design Guide (2011), CCC
- Stormwater modelling Specification for Flood Studies (2012), GHD
- Christchurch City Council MIKE FLOOD Technical Specification (2015), DHI

## 2 Purpose of this memo

The purpose of this memo is to simplify the Mike 21FM mesh generation process. The memo list the steps to generate the flexible mesh in an easy to understand way. This method does not require Spatial Analyst Tool of ArcGIS or GHD code to write DEM values onto mesh instead it uses QGIS which is available for free.

## 3 Generating Mesh as per Citywide Modelling Requirements

The following are the steps to generate the mesh as per citywide requirement

### 3.1 Generating mesh polygons

- Convert shapefiles of catchment boundary, roads, railways and channels to produce *.xyz files
- Open *.MDF file and set it projection.
- Import the catchment file in *.MDF file to define the mesh boundaries as follows:
  - Data →import boundary. Import the catchment boundary with the option of X,Y and Connectivity / NZTM/ [m]/[m]/use connectivity information/discard arc <2 vertices.
  - Define the polygons to exclude or include in the mesh and set the mesh triangles area. For Halswell area, set the maximum mesh element size for flat area and hillside to 200 m² and 1000 m² respectively.
- Import the river boundaries (1D) as above
- Import the road and rail boundaries as follows:
  - Data→Manage scatter data→Add and brows to the road/rail point (xyz) data.
  - Now convert the above road/rail ponts to mesh nodes. Convert→Convert scatter data file to Mesh nodes.

- Import the hill streams' data and convert it to mesh nodes same as road/rail data.
- Import correction points (if any) and convert it to mesh nodes same as road/rail data.
- Generate the mesh with the criteria given in Table 1

| | |
|---|---|
| Maximum element area | 1000 m² |
| Smallest allowable angle | 26 |
| Maximum number of nodes | 10,0000,000 |

- Smooth mesh by10 iterations and tick the box for "constrained by mesh criteria".
- Export mesh and DFSU files (Don't assign Z values to mesh at this stage). When exporting DFU *set the item type to "Bed level change".* You can change ITEM type at any stage by opening the DFSU file in Data Manager and browsing under View→Items
- Convert DFSU file to GIS using Mike Zero tool *"Mike2Shp"* available under Mike Zero Tool Box. Always convert DFSU file to shape file not the MESH file.

### 3.2 Mapping Z values on shape file

- For mapping Z values on shape file use QGIS.
- Load shape file generated from DFSU file
- Load DEM data (tiff or asci)
- In QGIS under raster Tab, use zonal statistics to calculate the mean values within each triangle as shown in Figure 1. Note that a column "mean values" will be add in the attribute table. You need to give it same name as was used for the ITEM type in the DFSU file. At the end multiply the values with -1. Syntax is (Bathymetry * -1).

### 3.3 Mapping Z values on DFSU file

For mapping Z values on the DFSU file use the DHI tool *"update_dfsu_z_value_from_shp".*

- Open DHI tool and run the *"DHI.NZSOL.UpdateDfsuZValueFromShp.exe"* file and brows to SHP file (section 3.1) and DFSU file.
- Hit process and wait until word "Success!" appears
- A copy of the DFSU will be generated in the same folder where the original DFSU files was placed.
- Note that you could run this tool from any drive and all files do not need to be in the same folder.

## 4 Assigning Depth Correction in Mike21FM

- In MIKE 21 FM, go to Hydrodynamic Module →Depth and set the depth correction type to specified bed level change. Browse to the newly created DFSU file containing additive inverse topography values. Save the MIKE 21 setup.
- Open the Mike21FM (*.m21fm) file in a text editor. Search for DEPTH and Change the type from 1 to 2 as shown in Figure 2. This will ensure that the values in the mesh are ignored and overwritten by the depth correction values.
- Note that the depth correction type will be greyed out when you will open the Mike 21 model. If it is not then make sure that the parameter "Type" is set to 2. Refer to Figure 3.

**Sher Khan**
Modeller
Phone Number: +64 3 366 3521
Email: sher.khan@beca.com

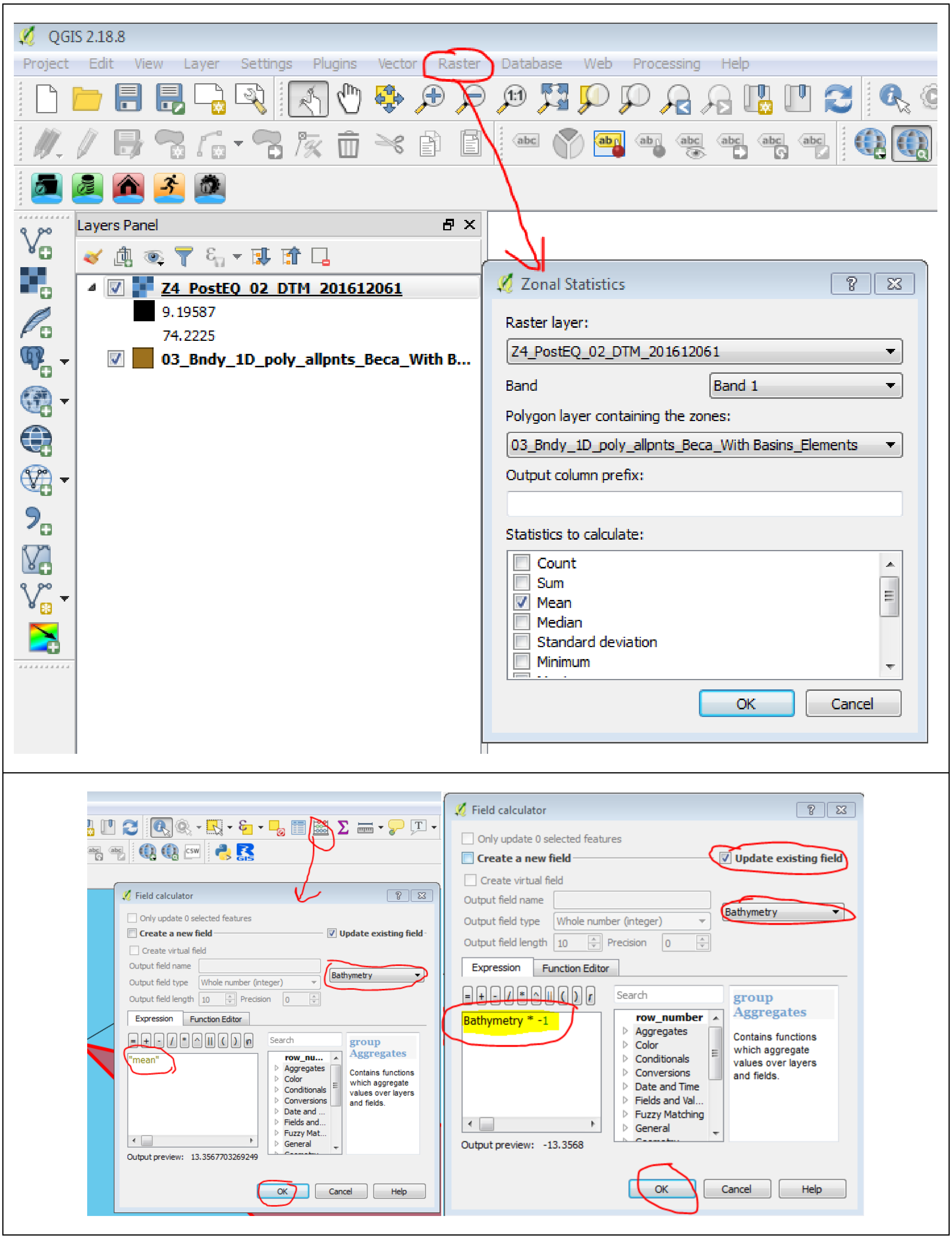


**Figure 1: Zonal Statistics in QGIS**

```
[DEPTH]
   Touched = 1
   type = 2
   format = 2
   file_name = |.\BedLevelChange.dfsu|
   item_number = 1
EndSect  // DEPTH

[FLOOD_AND_DRY]
   Touched = 1
   type = 2
   drying_depth = 0.02
```

**Figure 2: Mike 21 FM edits in text editor**

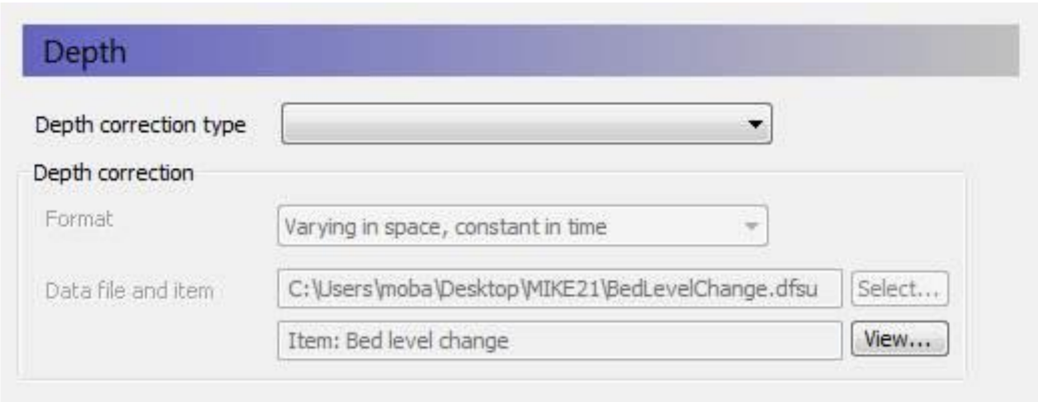


**Figure 3: Mike 21 FM depth correction**

# D

## Appendix D – Road Mesh Points Memorandum

ANZ Centre, 267 High Street,
PO Box 13960, Christchurch,
8141, New Zealand
**T:** +64 3 366 3521 // **F:** +64 3 366 3188
**E:** info@beca.com // www.beca.com

14 May 2021

**Attention: Antionette Tan (DHI)**

Dear Antoinette,

Thank you for your review of Beca's proposed revisions to the mesh generation and depth correction layer for the Halswell model. The reviewed documents were "Depth Correction Memo – Variation Request 6b" and "Road Mesh Points Memo – Variation Request 6b" both dated 9th April 2021. Below is a response to the points raised in your review. We have attached revised versions of the methodology memorandums, as well as your review letter.

## Road mesh points method

Your review did not provide any comments on the methodology or the accompanying memorandum and therefore we have finalised these in their original form.

## Depth correction method

Your review highlighted several potential methodology issues, asked for clarification in the text, and posed additional questions. Our response to your review is provided below.

### 1.1 Methodology issues

**Issue 1 -** Under the section titled "Implications of the methodology and changes from the original CWM schematisation" you raised a concern that if a large difference existed between the road crest and the gutter point elevation, the MIKE21 v2016 flooding and drying algorithm could produce instabilities in the model. The original CFM methodology limited the road crossfall slope to 6 % to improve model stability. You suggested that the v2020 algorithm is more stable in this regard, but without running the model these stability issues could not be assessed.

> **Outcome -** We agree there is a potential stability issue, which should be limited to rural roads due to the location of the buffer lines. We do not propose to change the model to v2020 at this stage, rather we will continue with v2016. We propose to resolve any instabilities associated with this issue during the model stabilisation step (ie we will treat any instabilities in the same way we would treat instabilities from other model components). We therefore do not propose any further modifications to the methodology. This could be revisited if there is widespread instability that stops the model from running and effects large areas within the city boundaries.

**Issue 2 -** You identified a limitation of our proposed methodology relating to larger roundabouts. For such structures you rightly point out that they are poorly represented as mesh sizes become larger near the wider edges of the roundabouts. As you have indicated this was also a limitation in the original methodology also.

**Outcome** – We have discussed this matter with Council and have agreed that matching the level of detail in the original methodology is sufficient for their purposes. We therefore propose to continue with the methodology as described.

### 1.2 Text clarifications

All comments in the section titled “Clarifications in the text” have been addressed. Updated versions of these memorandums have been attached.

### 1.3 Additional questions

Clarification was requested about the extent of change for the depth correction layer update (i.e. only updating the new model areas and new roads and leaving the remaining areas with previous values vs. updating all areas of the model). Our proposed methodology is to update all depth correction values for the model extent because a new mesh had to be generated for the entire model domain. There will be slight discrepancies in the mesh element alignment between the previous model mesh and the updated model mesh (even for older areas in the model that haven’t been changed), so all of the model mesh depth correction values have to be updated.

A question was raised about generating a DEM for the model extent, similar to the CWM DEM showing the overlayed road surface DEM. Given that our methodology doesn’t create a seperate road surface DEM, but only returns various zonal statistic values for each mesh element (mean, minimum and maximum), the only DEM that can be delivered was the .tiff shared during the initial review (i.e. a TIN with elevation values assigned to each element centroid).

## Closure

Based on the above we intend to proceed with the model development and subsequent stabilisation runs. If there are any points you feel have not been address appropriately please contact us to discuss.

Yours sincerely

**Josh Cumberland**
Hydraulic Modeller

on behalf of

**Beca Limited**

Phone Number:
Email: Josh.Cumberland@beca.com

**Copy**

Stephen Holder, Christchurch City Council (CCC)

# E

## Appendix E – Beca Response to DHI Review of Road Methodology

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8141, New Zealand
**T:** +64 3 366 3521 // **F:** +64 3 366 3188
**E:** info@beca.com // www.beca.com

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## Closure

Based on the above we intend to proceed with the model development and subsequent stabilisation runs. If there are any points you feel have not been address appropriately please contact us to discuss.

Yours sincerely

**Josh Cumberland**
Hydraulic Modeller

on behalf of

**Beca Limited**

Phone Number:
Email: Josh.Cumberland@beca.com

**Copy**

Stephen Holder, Christchurch City Council (CCC)

# F

Appendix F – DHI Oscillations Tool Scope Definition

# Memorandum

15 July 2015

| | | | |
|---|---|---|---|
| To | Tom Parsons | | |
| Copy to | Peter Kinley, Greg Whyte, Chris Maguire, Iris Brookland, Graham Harrington, Peter Christensen | | |
| From | Tim Preston | Tel | 027-6414301 |
| Subject | Citywide Flood Modelling - Proposed technical specification for model runtimes | Job no. | 51/33275/ |

Hi Tom,

At the technical meeting 19/6, you asked GHD to prepare our recommendations in terms of improving the CCC specification for model runtimes. This memo is produced to meet that objective and we would recommend that the following specifications be adopted for the Citywide Flood Modelling project:

The recommendation specifies runtime requirements for draft (common) model runs. There is a general expectation that on occasions either during or on conclusion of some modelling work that higher specification (slower) model run(s) will be required either as a progress check or as a final design confirmation. Runtime requirements for such runs are not specified because they are much less common and will actually be quite varied according to the modelling need.

For closely related sets of (similar) model runs, evidence of satisfactory stability and accuracy with low order calculations may only need to be thoroughly checked for one selected model run with confidence in the similar model runs inferred. The check run should verify acceptable water balance result, stability through checking velocities and CFL numbers as well as quantifying 2D grid max water level differences between low order and high order calculations. The method of checking results shall be included in reporting associated with the runs and the extent to which suggest inferred confidence is used as a QA tool is at the discretion of the senior modelling person involved.

**Table 1 Run specification and runtime parameters**

| Runtime Parameter Description | Draft model runs | Final design / or check model runs |
|---|---|---|
| **1. Precision** | Single | Single |
| **2. Order of computation engine (both in time and space)** | Low order | High order |
| **3. Auto water balance** | Off | On (discretionary) |
| **4. Courant number** | 0.9 | 0.8 |



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| Runtime Parameter Description | Draft model runs | Final design / or check model runs |
|---|---|---|
| **5. Model stability (2D)** | Qualitative inspection of 2D results and sample timeseries results | Qualitative inspection |
| **6. Maximum values calculated at runtime** | No | Discretionary |
| **7. Data saving fields** | Depth, level, speed, CFL | Discretionary |
| **8. Data saving timesteps** | 10 minute | Discretionary |
| **9. Rainfall type** | Single event | Multiple events batched |
| **10. Rain event duration** | Per river catchment critical duration | Up to 72 hours |
| **11. Flood event** | 50yr ARI, climate change rainfall and sea level rise | Optional |
| **12. Stopbanks** | Included in modelling | Optional |
| **13. Computer specification** | Refer below except single GPU | Dual or quad GPU |
| **14. Geography** | Single river catchment | Single river or Citywide catchment |
| **15. Runtime requirement** | 7 hours | Not required |

**Computer specification:** A suitable computer is a 16 core Dell Precision T7610 with Intel Xeon processors, 32 GB memory and running 64-bit Windows 7 operating system, with 2 Nvidia GeForce GTX Titan GPUs, with a solid state local hard drive (mostly adopted from DHI specification Mar 2015, issued as App C to the RFP)

**GHD Discussion / rationale:**

1. For models in Christchurch with small vertical extents (eg: 100m) that single precision 8 sig figs, produces with a minimum vertical resolution for numerical calculations of 0.01mm
2. This relates to the number of terms used in the numerical approximation (computation). Low order should be considered as a ‘draft’ run, with significant decisions such as design or floor level settings using high order
3. Auto water balance slows down the runtimes significantly and should be used sufficiently to derive confidence in the model stability, but is not required for most model runs
4. The higher 0.9 courant number will accelerate runtimes but increase risks of model instability, reducing the number to 0.8 for final runs should improve outcomes in item 5 below.
5. 2D instabilities are generally identified by high CFL numbers and unreasonably high velocities. To our knowledge there is no way to specify what is considered acceptable, but in general small amounts (both in time and space) of instability that do not appear to affect overall results of interest are considered acceptable. This needs to be judged instinctively by a senior modelling person, in conjunction with ‘reasonableness’ of the overall results and Council may want to review this.
6. Processing of maximum values post runtime is usually preferred.

7. With FM software it is tedious to post process depth into level (or vica versa), hence specification to produce both and velocity is required in order to assess stability. Sometime other parameters may be required for specific purposes, but generally three should be sufficient for most purposes.
8. Recommendation is consistent with DHI specification and strikes a balance between speed and results files sizes vs the need for resolution driven by the critical use to inspect areas of potential instability
9. Once the critical duration is established for a typical design purpose, a single event should be sufficient for most design runs. Running a ‘final proof’ using the full suite of storm durations would be recommended for final design runs.
10. Recommendation is essentially following current practice of 60, 18, 48 hrs for the Heathcote, Avon and Styx models respectively. Recommendation to adopt river specific standards will advantage the Avon/estuary model which is likely to be the ‘heaviest’ in terms of detail and runtime requirements.
11. The 50 year flood event with climate change rainfall and sea level rise will be the most common design model run undertaken. Larger events may be expected to take longer to run.
12. Stopbanks included will be the most common design model run undertaken. Including the stopbanks will reduce wetted cells and be of some benefit to runtimes.
13. We have found that procurement of dual GPU computers is both expensive and administratively onerous as it requires going outside of our IT standard procurement policy in terms of CPU box and potential need to violate suppliers warrantee. We expect this will be a problem for most consultants and hence recommendation for single GPU. We have appended a requirement for solid state local hard drives as these improve results saving performance (prerogative being that results are saved locally to maximise speed).
14. This specification would apply to any of the major CCC river catchments (Styx, Avon, Heathcote or Halswell) individually but not the aggregate Citywide model.
15. Seven hours, includes for model initialisation time and runtime, but excludes any hotstart runtime. This time allows for two runs to be completed per working day (one daytime, one overnight).

Regards

**Tim Preston**
National Technical Leader Water Planning

# G

Appendix G –Technical memo on runtime specification

# Memorandum

15 July 2015

| | | | |
|---|---|---|---|
| To | Tom Parsons | | |
| Copy to | Peter Kinley, Greg Whyte, Chris Maguire, Iris Brookland, Graham Harrington, Peter Christensen | | |
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8. Recommendation is consistent with DHI specification and strikes a balance between speed and results files sizes vs the need for resolution driven by the critical use to inspect areas of potential instability
9. Once the critical duration is established for a typical design purpose, a single event should be sufficient for most design runs. Running a ‘final proof’ using the full suite of storm durations would be recommended for final design runs.
10. Recommendation is essentially following current practice of 60, 18, 48 hrs for the Heathcote, Avon and Styx models respectively. Recommendation to adopt river specific standards will advantage the Avon/estuary model which is likely to be the ‘heaviest’ in terms of detail and runtime requirements.
11. The 50 year flood event with climate change rainfall and sea level rise will be the most common design model run undertaken. Larger events may be expected to take longer to run.
12. Stopbanks included will be the most common design model run undertaken. Including the stopbanks will reduce wetted cells and be of some benefit to runtimes.
13. We have found that procurement of dual GPU computers is both expensive and administratively onerous as it requires going outside of our IT standard procurement policy in terms of CPU box and potential need to violate suppliers warrantee. We expect this will be a problem for most consultants and hence recommendation for single GPU. We have appended a requirement for solid state local hard drives as these improve results saving performance (prerogative being that results are saved locally to maximise speed).
14. This specification would apply to any of the  major CCC river catchments (Styx, Avon, Heathcote or Halswell) individually but not the aggregate Citywide model.
15. Seven hours, includes for model initialisation time and runtime, but excludes any hotstart runtime. This time allows for two runs to be completed per working day (one daytime, one overnight).

Regards

**Tim Preston**
National Technical Leader Water Planning

# H

Appendix H – Existing Development 2021 stabilization file note

# File Note

**By:** Leif Healy **Date:** 10 November 2022

**Subject:** Existing Development 2021 stabilization **Our Ref:** 3364860

# 1 Background

This document records changes made to the HDM Halswell hydraulic model as part of stabilization of the Existing Development 2021 model.

## 1.1 Batch of runs used

Eight model scenarios were simulated. They are given below along with their abbreviated name:

- 0.5% AEP 0.5-hour design event – V002_Hals_02_ED2020_00p5AEP_00p5hr
- 0.5% AEP 3-hour design event – V002_Hals_02_ED2020_00p5AEP_03hr
- 0.5% AEP 9-hour design event – V002_Hals_02_ED2020_00p5AEP_09hr
- 0.5% AEP 60-hour design event – V002_Hals_02_ED2020_00p5AEP_60hr
- 0.2% AEP 0.5-hour design event – V002_Hals_02_ED2020_00p2AEP_00p5hr
- 0.2% AEP 3-hour design event – V002_Hals_02_ED2020_00p2AEP_03hr
- 0.2% AEP 9-hour design event – V002_Hals_02_ED2020_00p2AEP_09hr
- 0.2% AEP 60-hour design event – V002_Hals_02_ED2020_00p2AEP_60hr

Results for these simulations were analysed according to the city-wide modelling methodology (the methodology, including in Appendix A).

## 1.2 Pre-stabilisation work

To get the model to run more quickly rational steps were taken to stabilize the model prior to following the city-wide stabilization methodology. This included:

- Setting the exponential smoothing factor to 0.2, 0.4 or 0.8 for lateral links that were known to cause instability in the existing development 2016 model.
- Setting the exponential smoothing factor to 0.2 for River/Urban links.
- Setting the exponential smoothing factor to 0.2 for Urban-Overland links.
- Adding closed cross sections at the downstream end of river branches that are linked to the pipe network and were known to cause instability (as per the city-wide methodology).
- Setting the overall timestep (under ‘Time’ in the MIKE 21 FM model) to 0.25 seconds. This facilitates frequent hydraulic linking of the 1D and 2D models which should improve the stability of the links. In large events the pipe network becomes completely surcharged so small changes in flow at the river/urban links create large changes in head in the pipe network (compared to when the pipes are partially full). When the pipes are partially full the remaining volume can absorb the fluctuations.
- Locations with CFL violation warnings were mapped in GIS to focus improvements to the mesh. Some small mesh elements were merged where this would not impact the results. This was done in the *.MDF file’s nodes and arcs so the new mesh could be easily updated without needing to reapply manual edits.

These steps were successful in preventing the model from becoming unstable and allowed it to run in a practical timeframe.

# 2 Stabilization Round One

## 2.1 MIKE 21 oscillations

As per the city-wide methodology, oscillations in the 'Surface elevation' result were mapped as a set of shapefiles. A Python script was prepared to automatically convert these to raster files (*.tif) for faster visualisation.

There were only three places in the MIKE 21 domain where there were oscillations larger than 100 mm as a result of linkages with the MIKE 11 and MIKE Urban models. The largest was near Ella Park at the downstream (southern) end of the river branch 'Hals.Knightstream.Drain' (Figure 2-1).

The second largest was near the confluence of 'Hals.MinsonsDrain' and 'Hals.HalswellRiver.1' which is near the intersection of Old Tai Tapu Road and Early Valley Road (Figure 2-2). The smallest was on the true left bank of 'Hals.PaynesDrain' near its link to 'Hals.CreameryDrain' (Figure 2-3). The actions taken to resolve these are discussed in section 2.4 and the result is reported in section 3.1.

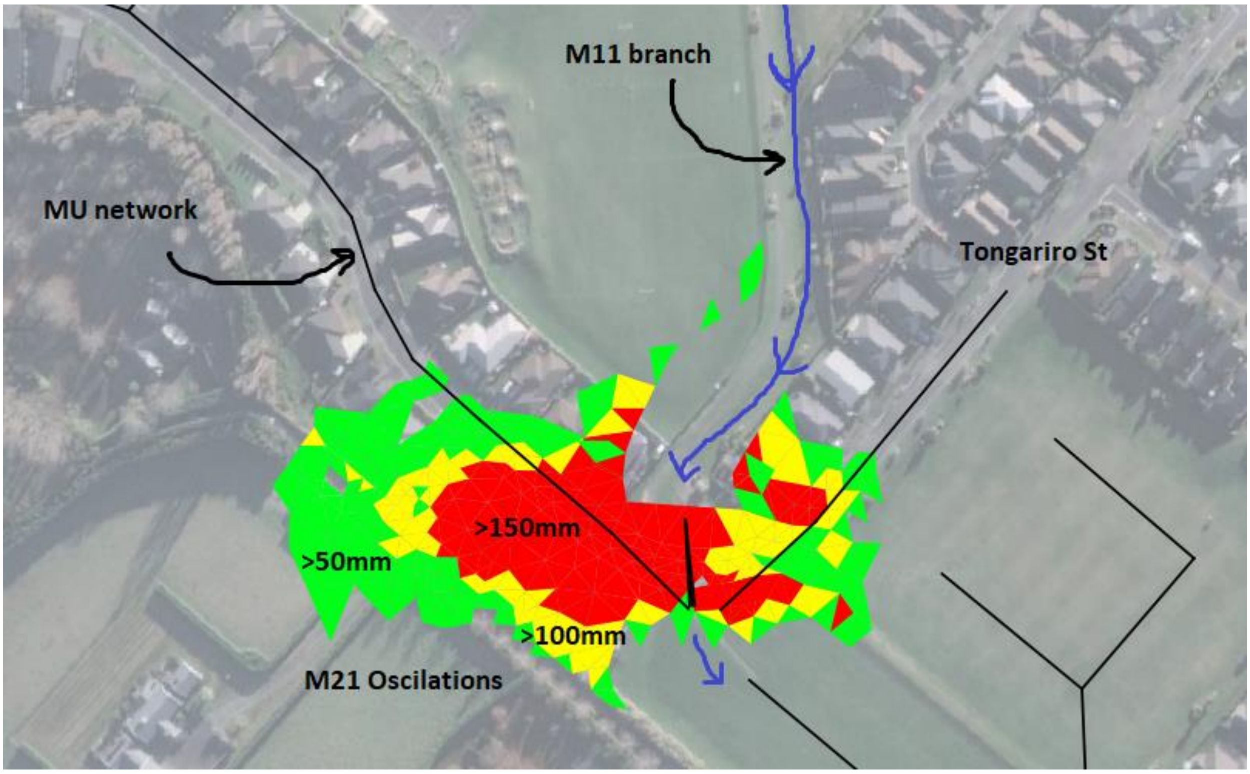



Figure 2-1: MIKE 21 oscillations near Ella Park at the bottom of 'Hals.Knightstream.Drain' river branch. NZ Basemaps by Toitū Te Whenua.

# File Note

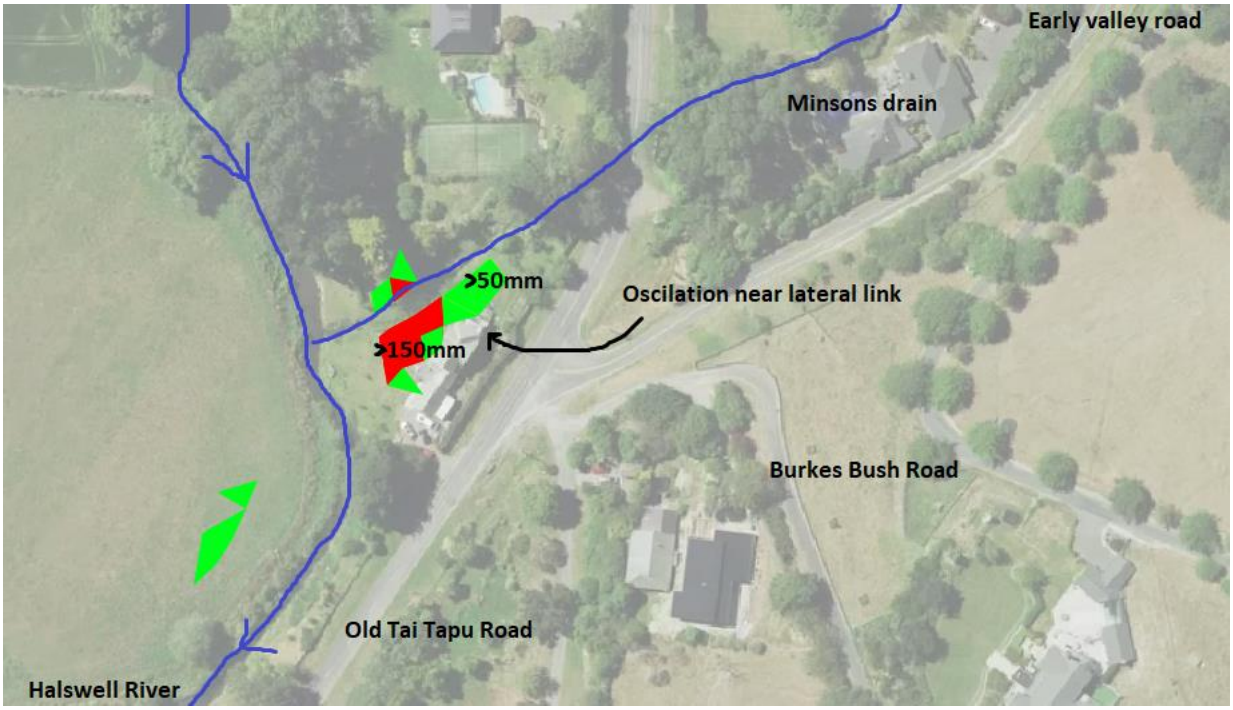



Figure 2-2: Oscillations near confluence of Minsons Drain and Halswell River. NZ Basemaps by Toitū Te Whenua.

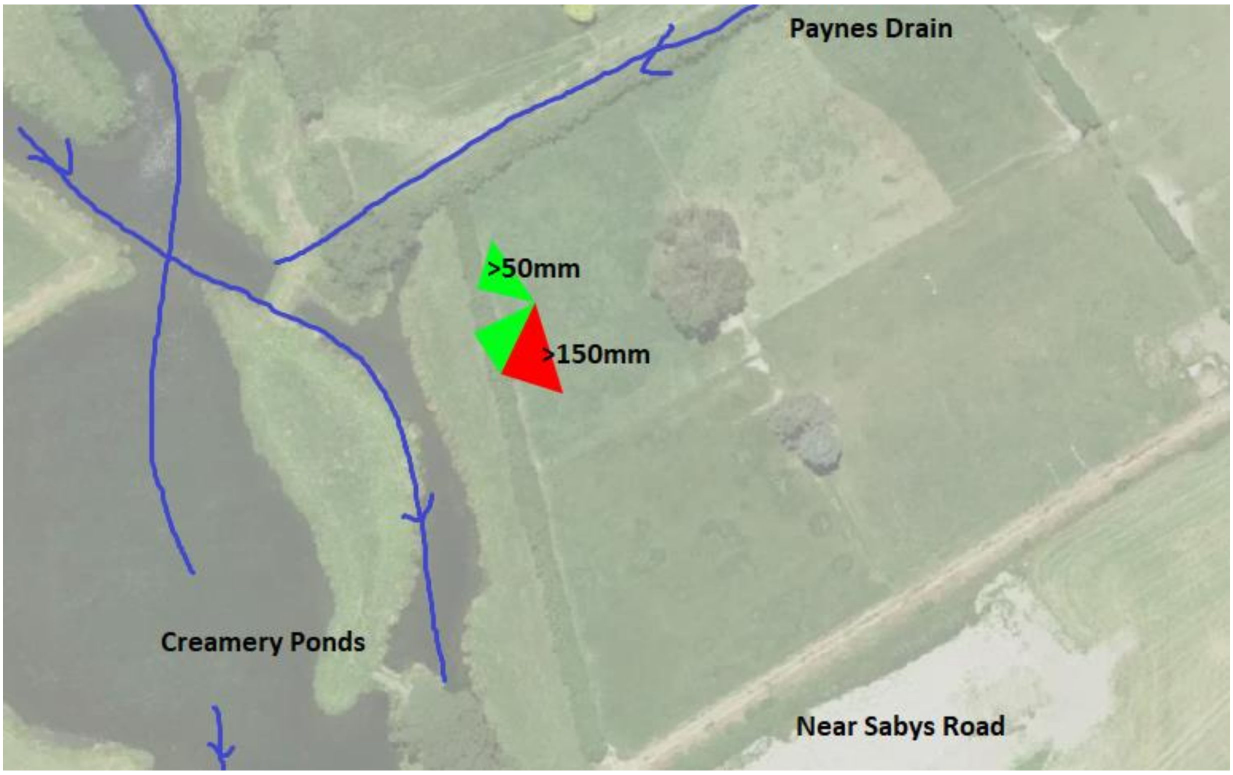



Figure 2-3: Oscillation near Paynes Drain and Creamery Ponds confluence. NZ Basemaps by Toitū Te Whenua.

# File Note

There are some other locations where oscillations were observed in the MIKE 21 domain that are not associated the 1D models or coupling. Where Dawson Creek crosses Ellesmere Road there is a culvert in the MIKE 21 model and some associated oscillations (Figure 2-4). Dawson Creek is small and is not modelled in MIKE 11. It is modelled as a simple overland flow in MIKE 21. There were also oscillations observed at the internal dike in Halswell Springs Reserve (Figure 2-5). Oscillations associated with both of these locations were only observed in one simulation: 0.2% AEP 60-hour design event. The MIKE Flood log was checked and there were no CFL violation warnings, indicating they are not associated with small elements and high velocity/wave speed.

The causes were found:

- At Dawson Creek near Ellesmere Road there was a MIKE culvert whose x-y coordinates were not correct. This meant it was connecting two small cells that were next to each other on the road. This was resolved as part of the first round of stabilization.
- At the internal dike in Halswell Springs Reserve, the water level rises and is very similar on both sides of the dike. The dike uses the MIKE 'empirical formulation' which has a rapid change in discharge at low water level difference across the dike. The MIKE documentation recommends increasing the critical level difference parameter on the dike if this causes instability.

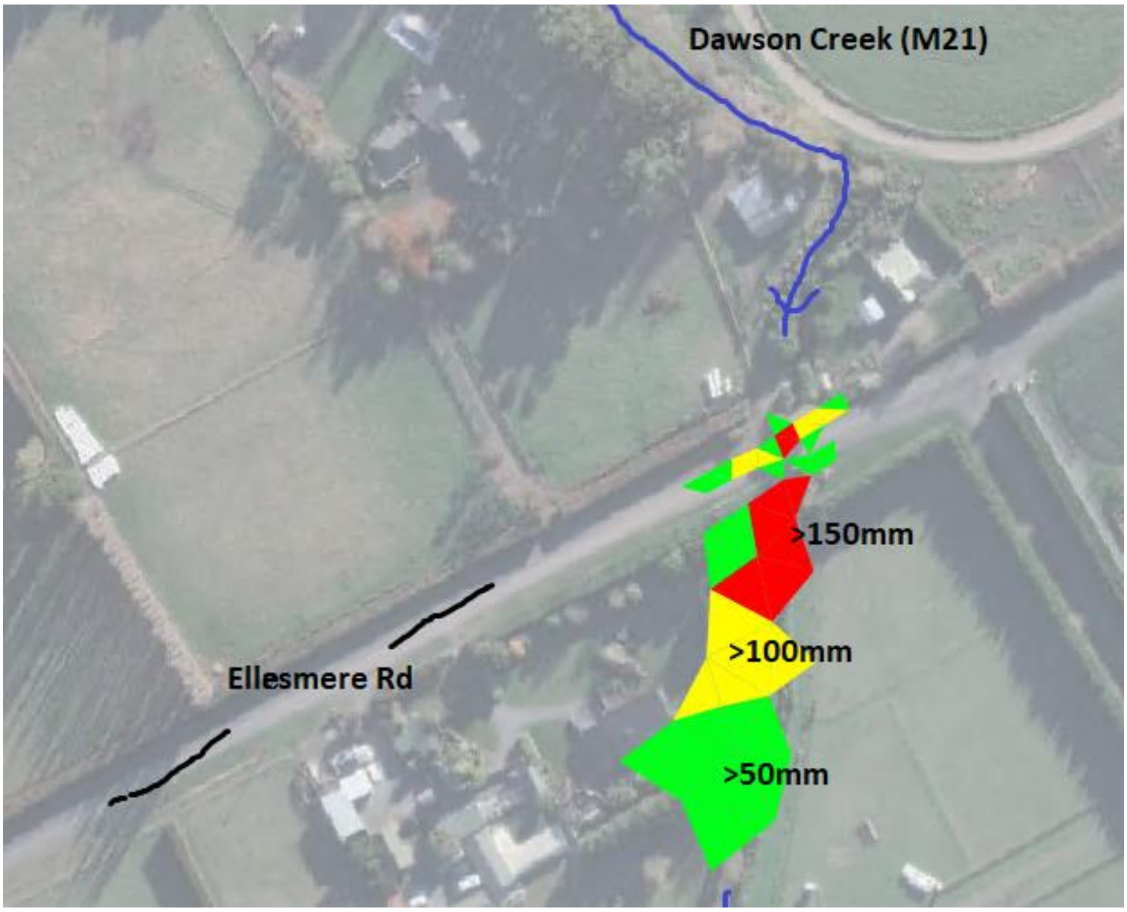



Figure 2-4: Oscillations at Dawson Creek at Ellesmere Road. NZ Basemaps by Toitū Te Whenua.

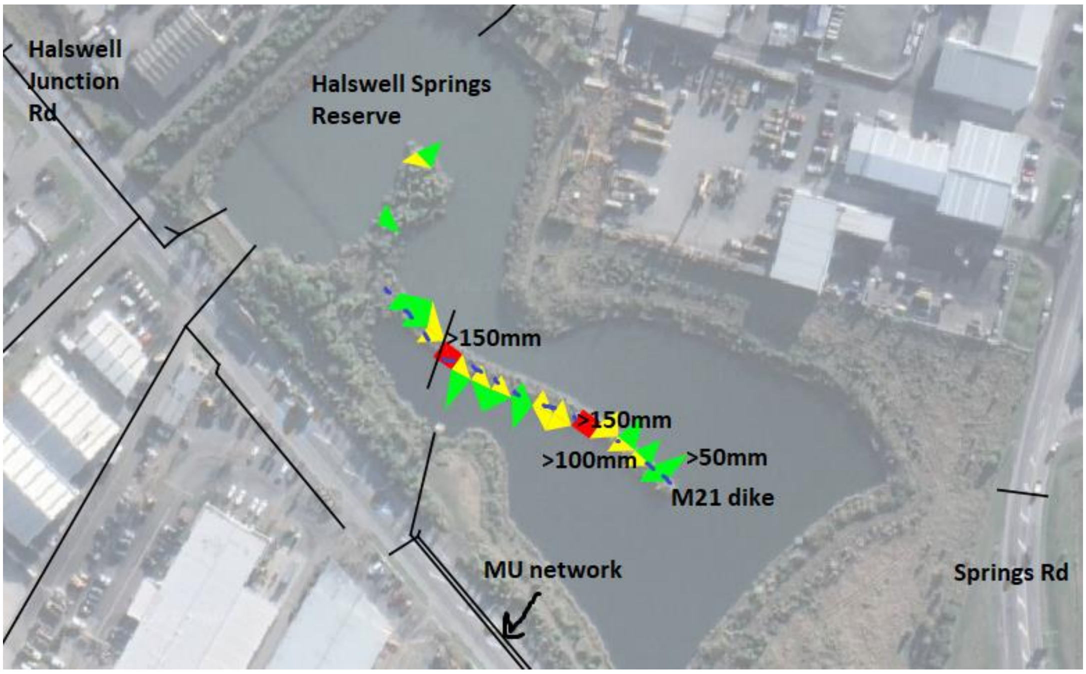



Figure 2-5: Oscillations at internal Halswell Springs Reserve dike. NZ Basemaps by Toitū Te Whenua.

## 2.2 MIKE Urban oscillations

In round one several manholes showed oscillations greater than 150mm (228 out of 1685 or 13.5%). These were recorded in a shapefile as per the methodology. The methodology says to ‘do nothing and report’ except where they are associated with a MIKE 21 oscillation greater than 100mm. This only occurred at one location (near Ella Park in Figure 2-1).

Some manholes showed oscillations greater than 300mm (40 out of 1685 or 2.4%). The methodology says one attempt should be made to resolve these and those that are not resolved should be reported. Prior to running the formal stabilization runs it was noted that some manholes had oscillations larger than 300mm. Therefore, the overall timestep was reduced from 0.5s to 0.25s. The overall timestep is when information is passed between the 1D and 2D models and therefore reducing it is likely to reduce oscillations associated with overflowing manholes/sumps. The exponential smoothing factor was also reduced to 0.2 for all manholes coupled to the MIKE 21 model.

## 2.3 MIKE 11 oscillations

In round one three MIKE 11 branches show oscillations greater than 150mm. These were Hals.MinsonsDrain, Hals.KnightstreamDrain, and Hals.GreenDrain. In at least one simulation in the batch, each of these also showed oscillations greater than 300mm.

## 2.4 Actions taken to resolve oscillations

- For Hals.MinsonsDrain at confluence with Hals.HalswellRiver.1 we:
  - Changed Hals.MinsonsDrain left and right bank to use M21 as their structure source parameter. The true left bank of Hals.HalswellRiver1 here uses MIKE 21 as its structure source parameter. This means there is no difference in the coupling between the two rivers at the confluence.
  - Changed the exponential smoothing factor for the lateral links on Hals.HalswellRiver.1 to 0.2 from 0.8.
- At the true left bank of Hals.PaynesDrain near Creamery Ponds we:
  - Reduced the exponential smoothing factor for the lateral links at Hals.PaynesDrain to 0.2 from 1.
  - Reduced the exponential smoothing factor for the lateral links at Hals.CreameryPonds.1 to 0.2 from 1.
- For the Halswell Springs Reserve internal dike we:
  - Increased the critical level difference parameter on the dike from 10mm to 50mm.
- For the Dawson Creek at Ellesmere Road MIKE 21 culvert we:
  - Increased the ‘alpha zero’ parameter from 10mm to 100mm.
  - Corrected the culvert location.
- For the downstream end of Hals.KnightstreamDrain we:
  - Added dummy pipe with a large diameter at the bottom of Hals.LonghurstDrain to absorb some of the oscillations.
  - Added a dummy/transitional cross section to the bottom of Hals.LonghurstDrain to smooth the transition into the pipe that goes to Ella Park.
- At the start of Hals.GreenDrain we:
  - Reduced the exponential smoothing factor from 1 to 0.2.

# 3 Stabilization Round Two

## 3.1 MIKE 21 oscillations

At the bottom of Hals.LonghurstDrain oscillations in the MIKE 21 were significantly reduced. They are now smaller than 100mm (Figure 3-1) across all runs in the batch (listed in section 1.1).

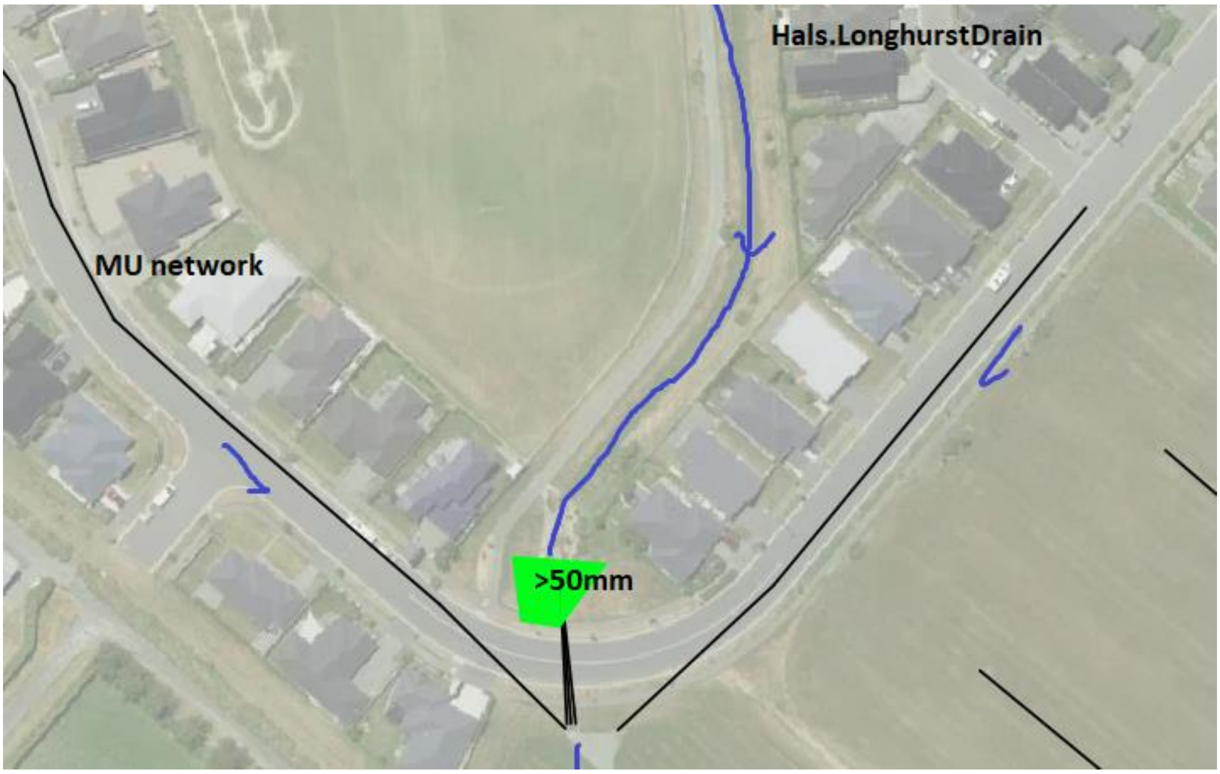



Figure 3-1: MIKE 21 oscillations at downstream end of Hals.LonghurstDrain

Oscillations larger than 150mm were observed in the second round near the confluence of Hals.MinsonsDrain and Hals.HalswellRiver.1 (Figure 3-2).

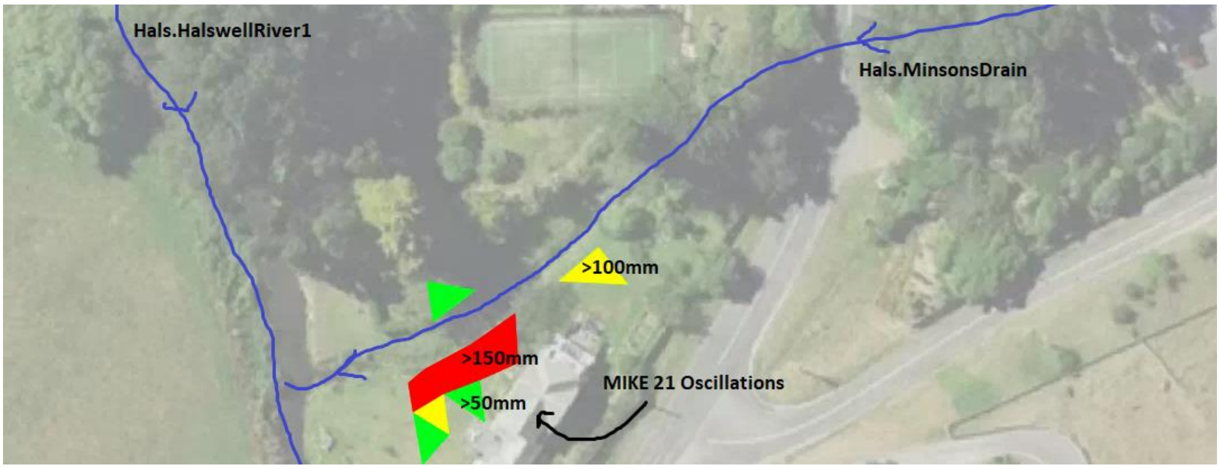



Figure 3-2: MIKE 21 oscillations at the confluence of Hals.MinsonsDrain and Hals.HalswellRiver.1

# File Note

There were no oscillations at Paynes Drain at Creamery Ponds in the second round (but there were in round one - Figure 2-3). Oscillations larger than 150mm were observed at Dawsons Creek at Ellesmere Road in the second-round batch (Figure 3-3).

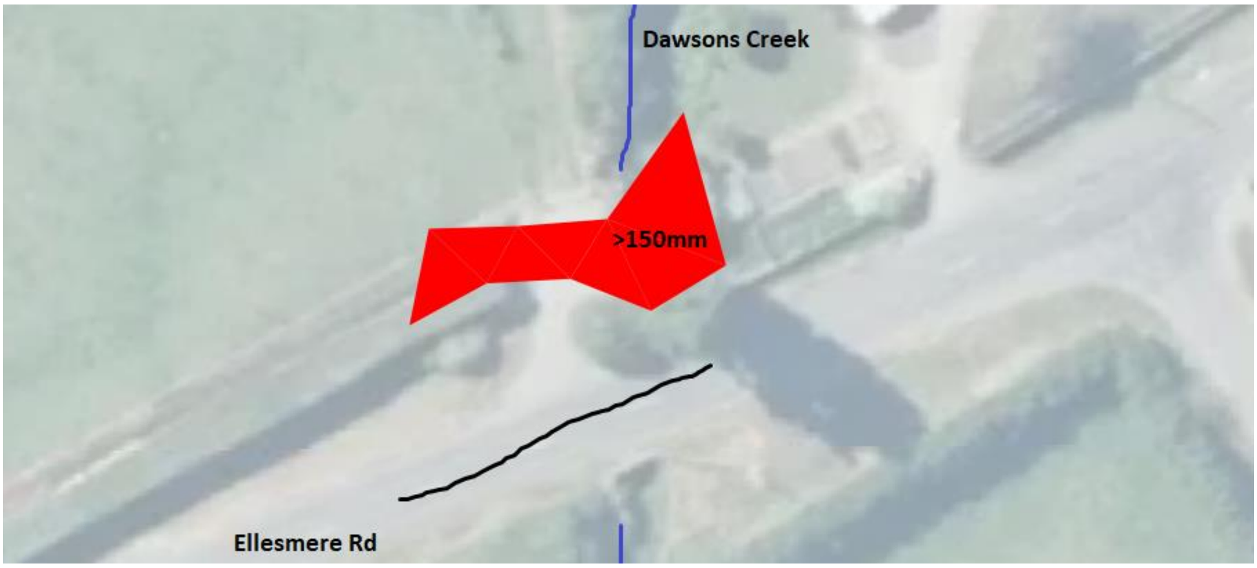



Figure 3-3: Oscillations observed in second-round batch at Dawsons Creek at Ellesmere Road

Oscillations larger than 150mm were observed at the internal dike in Halswell Springs Reserve. The oscillations are less than 250mm.

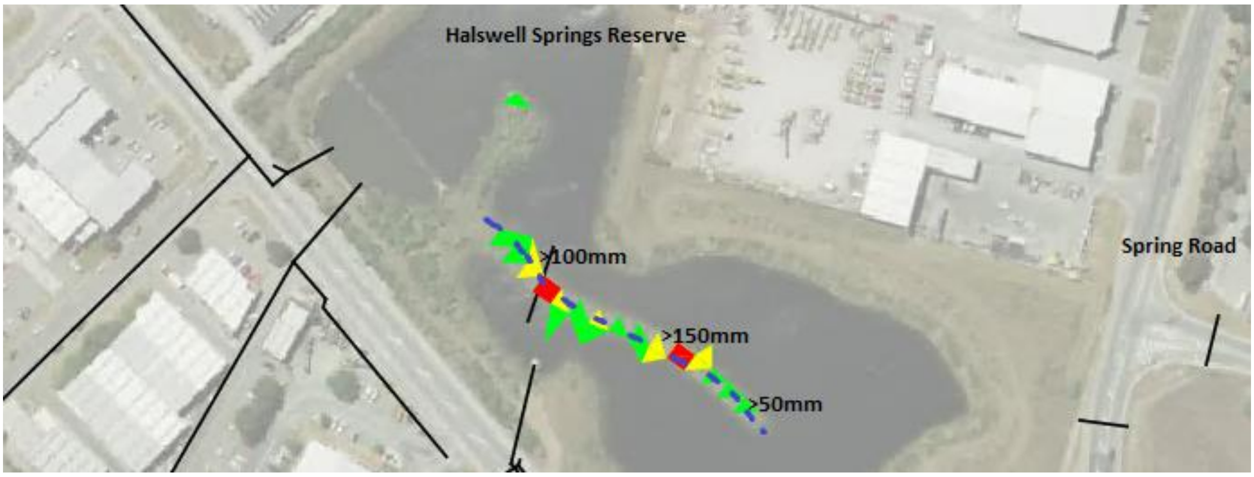



Figure 3-4: MIKE 21 oscillations at Halswell Springs Reserve internal dike in the second-round batch

## 3.2 MIKE Urban oscillations

In round two 231 manholes showed oscillations greater than 150mm (out of 1685 or 13.7%). These were recorded in a shapefile as per the methodology. 31 manholes showed oscillations greater than 300mm (out of 1685 or 1.8%). These were reported as per the methodology.

## 3.3 MIKE 11 oscillations

In round two three MIKE 11 branches show oscillations greater than 150mm. These were Hals.MinsonsDrain, Hals.KnightstreamDrain, and Hals.GreenDrain. In at least one simulation in the batch, each of these also showed oscillations greater than 300mm.

# File Note

## 4 Stability Status

As per section 3 Stabilization Round Two, there are persistent oscillations in the MIKE 21, MIKE Urban and MIKE 11 models. The methodology trigger for a second attempt at resolving oscillations is 'any MU [MIKE Urban] sites where the associated M21 [MIKE 21] levels show level oscillations > 100mm' of which there are none. The methodology specifies any oscillations remaining should be reported. Shapefiles were produced showing these locations for all three model components (MIKE 21, MIKE 11, and MIKE Urban). They are included in the Existing Development 2021 peer review package under the folder "Stabilization Files" provided to Christchurch City Council on 29 November 2022.

Kind regards,

**Leif Healy**

(File note Appendix A)

**Natasha Webb**

---

**From:** Elliot Tuck
**Sent:** Tuesday, 10 October 2017 1:42 PM
**To:** Natasha Webb
**Subject:** FW: Halswell Model Stability Standard

FYI some more info on the stability once we get the models up and running.

---

**From:** Pasco, Ben [mailto:Ben.Pasco@ccc.govt.nz]
**Sent:** Tuesday, 10 October 2017 1:06 p.m.
**To:** Elliot Tuck <Elliot.Tuck@beca.com>; Kate Purton <Kate.Purton@beca.com>
**Cc:** Parsons, Tom <Tom.Parsons@ccc.govt.nz>
**Subject:** Halswell Model Stability Standard

Hi Kate / Elliot

Below are some further notes on stability criteria being applied to CWM. This has been prepared by GHD in relation to CWM so application to Halswell may require tweaking for relevance (selection of stabilisation runs?)

**General stability standard methodology for the Citywide stormwater models**

1. Stability assessments will be done in batches to be defined in scoping at the outset. Batches may include various AEP events (eg: 10/50/200yr) but shall not include different families of model runs (ie: pre quake and post-quake)
2. Scoping at the outset will also define which runs to evaluate and/or stabilise – this will typically be a subset of runs within a family with selection based on being representative of the family, with emphasis on priority of runs in the batch and consideration of cost / effort. Typically selected runs will be evaluated in both 1D and 2D results and stability improved but this protocol could also be applied to evaluation based on only 1D or 2D results or evaluation for reporting only with no attempt at improvement.
3. Runs will be typically be evaluated from results saved with the standard results save time steps
    a. M21 save time steps were agreed in tech meeting 28/7/2017 (minutes of which refer back to XLS from TimPreston email sent 20170701_1430)
    b. M11 and MU save time steps are four times the frequency of M21 save time steps (these are often therefore not integer values)
    c. also refer GHD N:\NZ\Christchurch\Projects\51\33275\00.1Job Mgmt\04.Programs_Schedules\20160118 Revisions to future stages\CFM run timesteps 20160701.xlsx
4. For 1D results
    a. Load the 1D result (MU or M11) into Mike View starting with the 25% timestep (ie: for an 80 timestep result file load from timestep 20-80, do not load the initial 25% of results).
    b. Evaluate level oscillations within the MU and M11 results using the Mike View oscillation tool as described below. All sites with oscillation tool amplitude >150mm are to be tabulated, locations mapped (shapefile) and the 1D level timeseries data presented in XLS format (ref 1. Below). All sites will have max amplitude (defined as the smaller of the value changes either side of a local extreme point) calculated in the XLS and tabulated. All MU sites in this set will have the associated M21 timeseries results extracted from the cell above the node if the node is not coupled or the lowest coupled cell if there is any MU-M21 couple (using "Data Extraction FM" tool with a list of named x,y coordinates) and plotted for inspection also.
    c. Work to reduce instabilities shall proceed as follows
        i. For oscillations [150mm-300mm], then we would typically "do nothing" and report them, except for any MU sites where the associated M21 levels show level oscillations > 100mm in which case we would make one attempt to fix them and report thereafter whatever the outcome

        ii. For oscillation >300mm, we would make an initial attempt to fix them. If not solved initially then we would typically "do nothing" and report them, except for any MU sites where the associated M21 levels show level oscillations > 100mm in which case we would make a second attempt to fix them and report thereafter whatever the outcome

5. For 2D results
    a. Run the DHI tool using bin values {0, 0.05, 0.15, 0.30, 0.50, 1, 2, 5m}, producing overall oscillation frequency summary data and a DFSU of max oscillation 'map' for each run
    b. For each batch of results being evaluated
        i. Compile the frequency summary data into a single XLS table and present graphically with highlights on any outliers
        ii. convert the DFSU oscillation data into a polygon geodatabase format, remove shapes with less than 50mm max oscillation and compile into a singular shape with an attribute field for each model run result. Add fields of 'max across runs' and 'avg across runs' and 'product of max-avg'.
    c. Work to reduce instabilities shall be at the discretion of the modeller. Further attention shall be directed at runs with outlier oscillation frequencies and locations in the shape with high values in the three summary fields.
6. The deliverables for each batch assessed are;
    a. Transmittal note, covering email, readme file or similar defining the model runs being evaluated and tabulating both their long and abbreviated file names
    b. 1D spreadsheets – one for each model run - that Council can play with (drop down menu for 1D timeseries graphs)
    c. 1D shapefile, per batch of model runs being evaluated, with a separate field for each model run and a summary count of occurrences across the batch
    d. 2D spreadsheet per batch assessed with XLS and graphical 2D statistics
    e. 2D shapefile per batch with summary statistics on each mesh tringle
7. File / field naming: The abbreviated 10 char run naming convention defined below should be used throughout the reporting including; drop down XLS functions, shapefile field names and individual XLS file names. The file names for a batch of the 1D spreadsheets should share a common batch name prefix of not more than 12 char and the abbreviated run name as suffix.
    a. Std Format: ###_##[T]e##h – where [T] means optional character
    b. eg: 079_10e12h translates as Model version 079, 10% exceedance probability rainfall, 12 hour storm duration
    c. eg: 083_20Te01h translates as Model version 083, 20% exceedance probability rainfall [T]idal run, 1 hour storm duration
    d. In both AEP and storm duration the 0.5 values (0.5% AEP or 0.5 hr) will be represented as "00".

**<u>Using the Mike View Oscillation Tool</u>**

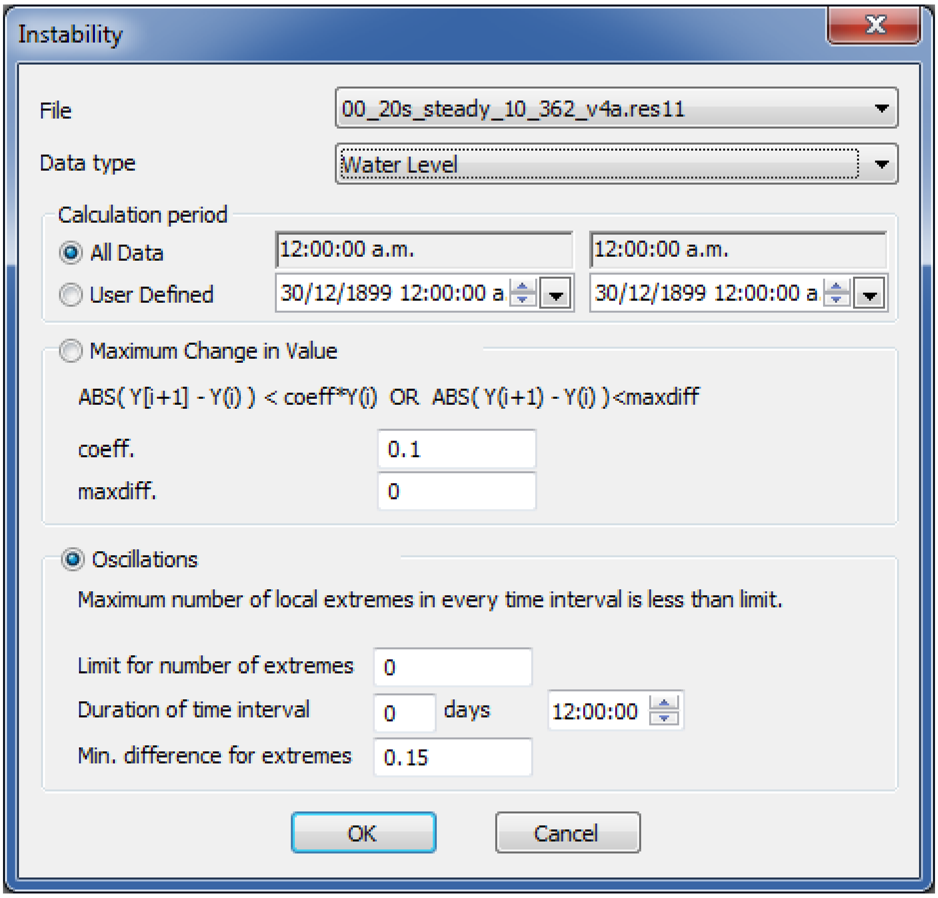
This tool evaluates and provides summary report on instabilities in the Mike Urban and Mike 11 model results.

<u>How to Access:</u>

1. Open Mike Urban or Mike 11 model results (limit results to load according to other specification).
2. While the 'Horizontal Plan' is active, Go to 'Tools' -> 'Compute' -> 'Instability'

<u>Tool settings</u>

- 'Data type' - Node level (Mike Urban) and Water Level (Mike 11)
    - Note the flow data is not required to be assessed due the limited practicality, high level of effort required and the lower importance of flows in relation to flood levels
- 'Calculation period' - All Data (control data period for analysis when loading the result)
- 'Oscillations' option selection
- 'Limit for number of extremes' – zero
- 'Duration of time interval' field – leave as default
- 'Min. difference for extremes' – ie: set to 0.15 or 0.30m or otherwise in accordance with the general specification




**References:**


1. Tim Preston email 20170209-1315, especially notes in the attached XLS and tech meeting minutes thereafter
2. DHI M21 oscillation evaluation tool received 17/2/2017: N:\NZ\Christchurch\Projects\51\33275\99.Automation tools\00_MikeFloodFM_DHITools\DHI M21 oscillation tool
3. Background discussion on how to use 1D oscillation tool - emails with Tim / Chamila 20170922_1215


**Further Notes (Follow up Email from GHD):**

1. Stability standards will be done in two batches – post quake and pre quake
2. Work on 'stability standards' for either batch will commence once run completion has been achieved across all the specified model runs in that family
3. In the post quake batch we will evaluate and stabilise a set of 10 runs as follows;
   i. 10y 1hr *
   ii. 10y 6hr
   iii. 50y 1hr
   iv. 50y 6hr *
   v. 200y 2hr
   vi. 200y 3hr *
   vii. 200y 6hr
   viii. 200y 24hr *
   ix. 200y 24hr tidal pair
   x. March 2014 calibration event (this list is as reported to CCC 20170209-1315 but with two runs removed)
4. In the pre quake batch we will evaluate and stabilise a set of 4 runs (those above marked with '*')
5. We will initially evaluate and report the model results 'as supplied to Council' which includes some now quite old model versions. This reporting is to set a benchmark and to educate by inference the stability qualities of the set of existing deliverables. This evaluation and reporting will be limited to GIS deliverables and M21 frequency analyses but will exclude XLS 1D evaluation (timeseries inspection)

6. We will re-run this set of design results on the then current model version and evaluate those results as the basis for identifying locations stability improvement work
    a. The Mar 2014 event will have been re-run as part of VO56 scope prior to stability standards and so will not require re-running for stability standard purposes.
7. We will follow the general method below for one attempt to fix all issues reported >150mm, and re-evaluate
8. If such problems remain we will attempt the second fix for issues that have >300mm oscillations and with M21 oscillations however our expected and budgeted effort for this is nil. If such work is required then a third cycle of evaluation will be required
9. We will replicate fixes in both directions between the post / pre quake models
10. We will supply to Council in XLS and GIS format for each batch, both the initial evaluation and final evaluation

We have a PCG in 2 weeks so I would hope that this can be interpreted by then, and ideally application started so that we will have an idea of the scale of any stability issues before the meeting. Have you had any update on delivery of the sump couple file?

Kind regards,


**Ben Pasco**

**Project Manager**
Land Drainage Recovery Programme
City Services Group - 3 Waters and Waste

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