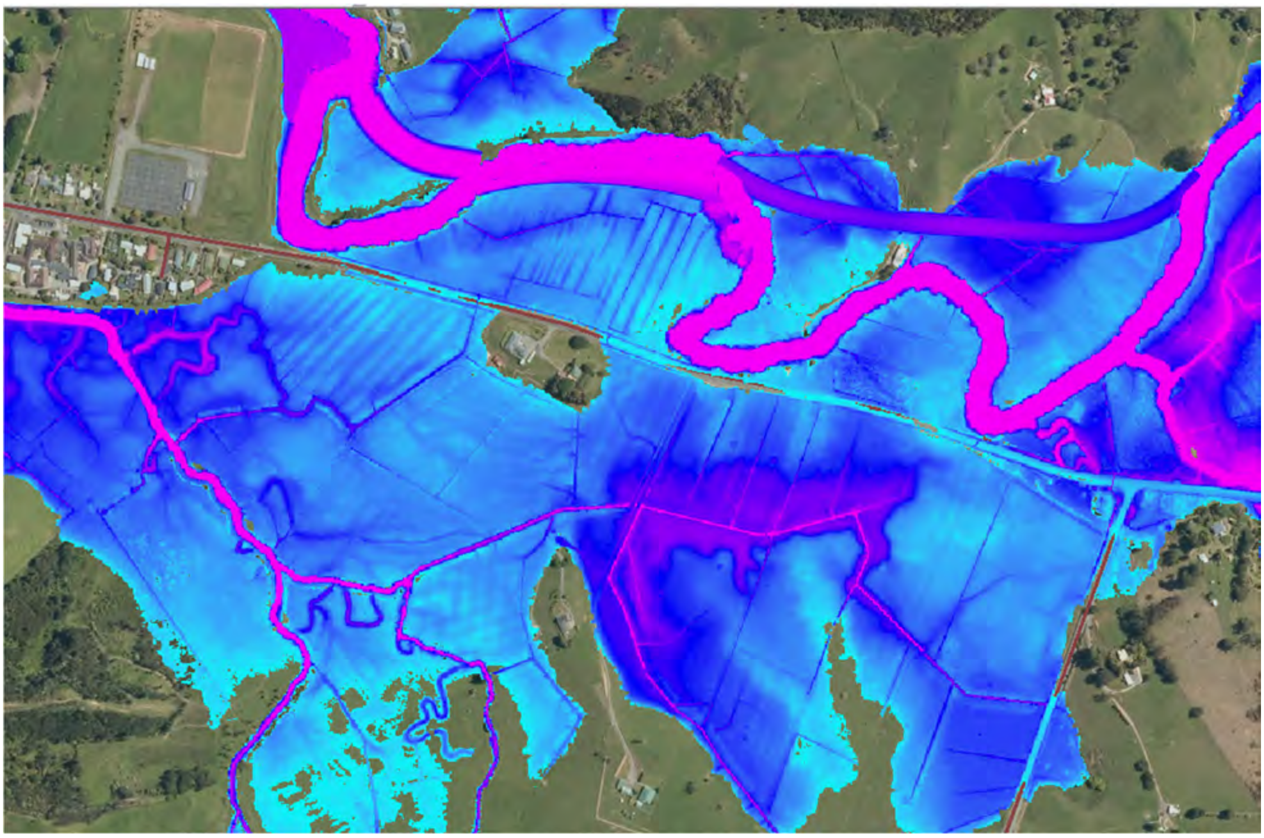
Awanui Flood Model Upgrade

Model Build



Northland Regional Council

Report

January 2021

The expert in **WATER ENVIRONMENTS**

This report has been prepared under the DHI Business Management System certified by Bureau Veritas to comply with ISO 9001 (Quality Management)

Awanui Flood Model Upgrade

Model Build, Calibration and Design

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CONTENTS

1 Introduction ... 1
1.1 Awanui Catchment ... 2
1.2 List of Deliverables ... 3

2 Data ... 4
2.1 GHD model ... 4
2.2 Survey data ... 4
2.2.1 Cross sections ... 4
2.2.2 Stopbank survey ... 5
2.2.3 State Highway 1 survey ... 5
2.2.4 Structure survey ... 5
2.3 Terrain ... 6
2.4 GIS asset and land use data ... 6
2.5 Calibration Data ... 7
2.5.1 Staff Gauge conversions ... 7
2.6 Design Model Data ... 8
2.6.1 Previous estimates of 1% AEP flow ... 8

3 Model build ... 10
3.1 Hydrological model ... 10
3.1.1 Catchment delineation ... 10
3.1.2 Hydrological Parameters ... 10
3.1.3 Rainfall ... 13
3.1.4 Baseflow ... 15
3.2 Hydraulic model ... 16
3.2.1 MIKE 11 model ... 16
3.2.2 Cut-down MIKE 11 model ... 18
3.2.3 MIKE 21 ... 19
3.2.4 MIKE URBAN ... 25
3.2.5 MIKE FLOOD ... 27

4 Calibration and Sensitivity Testing ... 29
4.1 Calibration ... 29
4.1.1 Hydraulic calibration ... 30
4.1.2 Hydraulic calibration: State Highway 1 overflow ... 31
4.1.3 Hydrological calibration ... 31
4.2 Sensitivity testing ... 32

5 Design model ... 35
5.1 Rainfall and Tide ... 35
5.2 Design Scenarios ... 36
5.2.1 Version D updates ... 36
5.2.2 Version F updates ... 40
5.2.3 Version G updates ... 44
5.3 Flood Mapping procedure ... 45
5.4 Design model results ... 47
5.4.1 Validation of design flows ... 47

6 Future work and Conclusions 48
6.1 Conclusions: 48
6.2 Future work 48

7 References 49

FIGURES

Figure 1-1: Awanui Catchment..2
Figure 2-1: State Highway 1 Survey upstream of Kaitaia (blue dots) 5
Figure 3-1: Awanui Catchment delineation 12
Figure 3-2: Location of Calibration Automatic Rainfall Sites..13
Figure 3-3: Design Rainfall Stations..14
Figure 3-4: Example 100yr rainfall profile. “Rainfall depth” axis is depth in mm per half-hour. 15
Figure 3-5: MIKE 11 Network..17
Figure 3-6: Dikes at the downstream end of the Whangatane Spillway 21
Figure 3-7: Dikes located at outfall of Whangatane Spillway and Awanui River 21
Figure 3-8: Dikes at the downstream end of the Waipapakauri Cut 22
Figure 3-9: Dikes around Awanui Township 22
Figure 3-10: Dikes around the SH1 overflow 23
Figure 3-11: MIKE Urban model 26
Figure 3-12: Example of significant link skewing 27
Figure 3-13: Example of sinuous branches, with the green area representing the 2D domain. 28
Figure 4-1: C9 Calibration at School Cut 29
Figure 4-2: C9 Calibration at Puriri Place..30
Figure 5-1: Runoff and Tide coincidence 35
Figure 5-2: School Cut/Te Ahu Centre..36
Figure 5-3: Bells Hill 37
Figure 5-4: Whangatane Spillway 38
Figure 5-5: Gills Rd downstream of Waihoe Channel..39
Figure 5-6: Scheme design upstream of Church Road..40
Figure 5-7: Allen Bell Park on the Awanui..41
Figure 5-8: Whangatane Diversion 42
Figure 5-9: Benching along the Whangatane Spillway 43
Figure 5-10: Stopbanks added to the G version model on the Tarawhataroa 44
Figure 5-11: Stopbanks added to the G version model on the Awanui 45
Figure 5-12: Example 1D/2D mapping results, depth in the 100yr event for version F 46

TABLES

Table 2-1: Cross section survey data..4
Table 2-2: Staff Gauge conversion to OTP 7
Table 2-3: Estimates of 1% Annual Exceedance Probability peak flow, Awanui River approaching Kaitaia. 8
Table 2-4 Summary of Flows Derived using Hilltop Software..8
Table 2-5: Historical Flood Frequency Estimates for the School Cut gauge record 9
Table 3-1: NAM parameters described 11
Table 3-2: Design Rainfall Depths 15
Table 3-3: MIKE 11 roughness by area 18
Table 3-4: MIKE 21 roughness values..23
Table 3-5: MIKE Urban node losses 25
Table 3-6: Urban Infiltration parameters 25
Table 4-1: Design Roughness Values..31

Table 4-2: NAM parameters 32
Table 4-3: Sensitivity Tests 32
Table 5-1: Key thresholds....... 47
Table 5-2: Modelled 100-year ARI peak flow approaching Kaitaia 47

APPENDICES

Appendix A – Model Review

Appendix B – Calibration

Appendix C – Design Rainfall

Appendix D – Model Results Table

1 Introduction

DHI Water and Environment Ltd (DHI), jointly with Macky Fluvial Consulting Ltd (MFC) were commissioned by Northland Regional Council (NRC) to carry out the hydrological and hydraulic flood model upgrade of the Awanui catchment.

The primary objectives of this project are as follows.

1. Objective I - to modify the existing MIKE FLOOD model to improve predictions of the current flood risk; and
2. Objective II - to provide a tool that will be suitable as a basis for computing the effects of proposed upgrades to the Awanui Flood Scheme.

This project includes hydrological and hydraulic calibration of the upgraded model for the February 2007 and January 2011 rain events. This project also includes the production of flood maps to help inform the following:

- strategic and site-specific development decisions;
- the development of designs for the Awanui Flood Scheme upgrade;
- communities and businesses of their flood risk, so that they can become more resilient to flooding; and
- Civil Defence preparedness and response planning.

Significant flood events over the last 15 years, have driven a need for developing mitigation measures within the catchment. Proposed mitigation measures have been incorporated into the model to assess their suitability and effectiveness at reducing flood risk.

The model has been peer reviewed by Hugh MacMurray of Barnett and MacMurray Ltd.

1.1 Awanui Catchment

The Awanui River Catchment (Figure 1-1) is a catchment in the Northland District, with Kaitaia the main township within the catchment. The catchment covers a total area of approximately 456 km^2, where the south east part of the catchment is predominantly steep terrain but becomes noticeably flatter following the flow direction towards the north and west. For example, between Kaitaia and the Ranganuu Harbour, the elevation drops by 14m over a 12km distance, from 15m RL to 1m RL, whereas the highest elevation in the catchment reaches above RL 730m, reflecting steep headwaters.

The Awanui River is fed by a number of tributary streams, including the Takahue River, Victoria River, Karemuhako River and Tarawhataroa Stream. Running adjacent to the Awanui in the flats of the lower catchment are the Waipapakauri Stream and the Whangatane Spillway (which is a constructed diversion of the Awanui, starting at the town of Kaitaia).

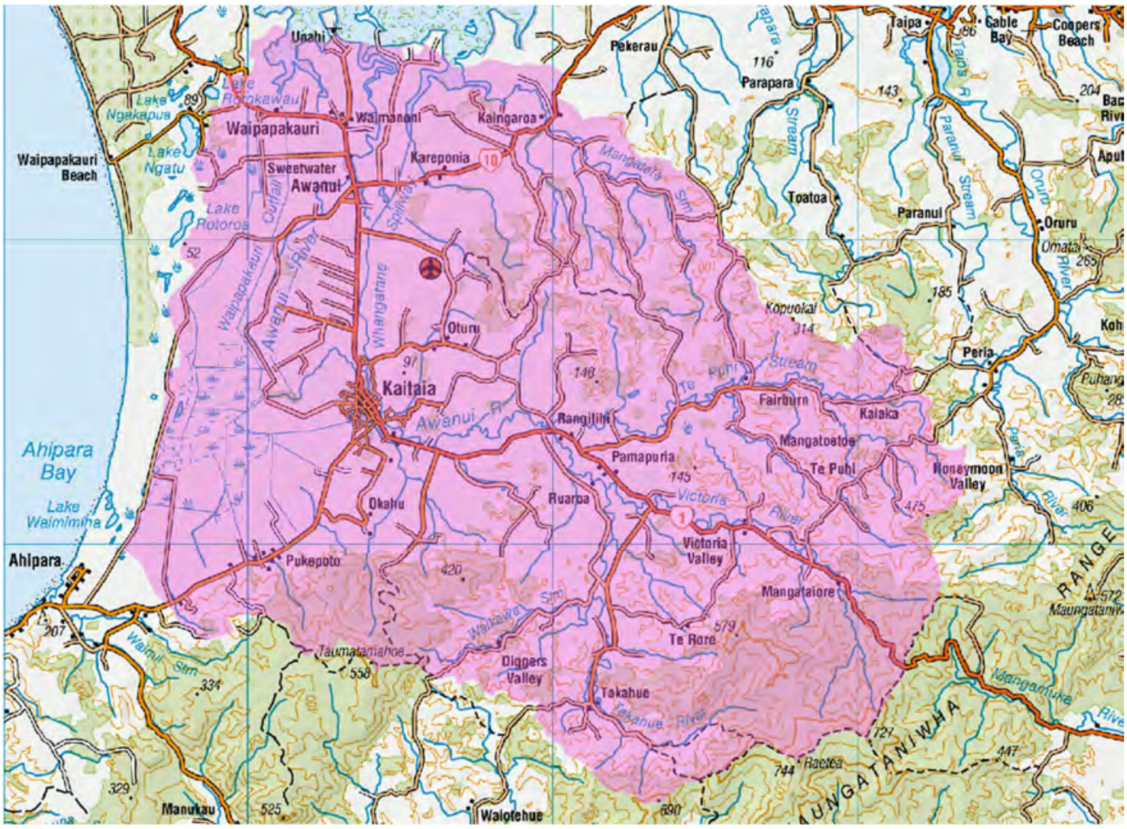


Figure 1-1: Awanui Catchment

1.2 List of Deliverables

The following is a list of deliverables included with this report:

Full MIKE FLOOD models and results for the following scenarios

- Calibration runs
    i. Version C9 Jan 2011
    ii. Version C5 Feb 2011
    iii. Including Calibration spreadsheet summarising the results of each of the MIKE FLOOD calibration simulations
- Version E (pre base)
    i. E6 10yr 24 hr
    ii. E6 20yr 24 hr
    iii. E6 50yr 24 hr
    iv. E6 100yr 24hr
    v. E6 100yr with Climate Change 24hr
- Version D (Current Day)
    i. D6 10yr 24 hr
    ii. D6 20yr 24 hr
    iii. D6 50yr 24 hr
    iv. D6 100yr 24hr
    v. D6 100yr with Climate Change 24hr
- Version F (Scheme Design)
    i. F6 10yr 24 hr
    ii. F6 20yr 24 hr
    iii. F6 50yr 24 hr
- Version G (Updated Scheme Design)
    i. G6 100yr 24hr
    ii. G7 100yr with Climate Change 24hr

Processed model results

- Raster database including of all results for each scenario listed above, 1D/2D combined 2D grids, for Water Level, and Depth
- Difference Rasters

Supporting Files

- Spreadsheet of all NAM catchment parameters
- Shapefile of sub-catchments
- Shapefile of all streams with names

2 Data

2.1 GHD model

The 2015 GHD model (/2/) was provided as a starting point for the model build process. DHI have completed a review of this model as part of this project, with the findings presented in Appendix A.

The GHD model provided the basis for the MIKE 11 model branches and structures, the MIKE URBAN model network and the hydrological catchments. However, many components have been adjusted during the course of the upgrade.

2.2 Survey data

2.2.1 Cross sections

The raw cross section data was provided by NRC, including the following datasets (Table 2-1). In addition to this, further cross sections from the bridge surveys were also used in the modelling. As is described in the model build section (Section 3 below), the surveyed cross sections were used to define the low flow extent of those cross sections where the LiDAR was unable to return good enough level data due to water and vegetation obstructions. Cross sections and survey data were also provided so that the pre-development condition at the Whangatane Spillway inlet could be modelled. Works to change the terrain in this area occurred between 2016 and 2017.

Table 2-1: Cross section survey data

Survey type	Year	Coverage
Von Sturmers	2004, 2012, 2015	Upper Waipapakuri
Estuary Survey	2004	Estuary including the downstream sections of the Awanui, Waipapakuri and Whangatane Spillways.
Boundary Hunter	2010	Various but survey mainly on the Tarawhataroa and tributaries, the Awanui from the SH1 overflow through Kaitaia, and various bridges throughout the catchment.
Awanui Catchment sections	1991-1992 estimated	Covering the reaches of the Awanui, Waipapakuri and Whangatane Spillway downstream of Kaitaia.
Pre-Development Awanui and Whangatane Inlet	2015 or earlier estimated	Awanui near the Whangatane Inlet and the Whangatane Spillway. The survey covers approximately 400m on each branch, with spacing between 10 and 60m.
Post-Development Awanui and Whangatane Inlet	Not provided	As requested by NRC it was assumed the low levels were unchanged and the higher levels were taken from the 2018 LiDAR

2.2.2 Stopbank survey

After analysis of the 2018 LiDAR survey data, it was found that in some areas the high points of the stopbanks either had not been picked up in the survey or had been processed out of the dataset, resulting in some gaps in the stopbank. To supplement the LiDAR data the 2010 Cato Bolam stopbank survey was used.

2.2.3 State Highway 1 survey

A survey of State Highway 1 upstream of Kaitaia was provided by Opus in September 2018, Figure 2-1, to allow for a more accurate representation of the road levels in this area, where the Awanui is known to spill from its left bank and flow over the highway.

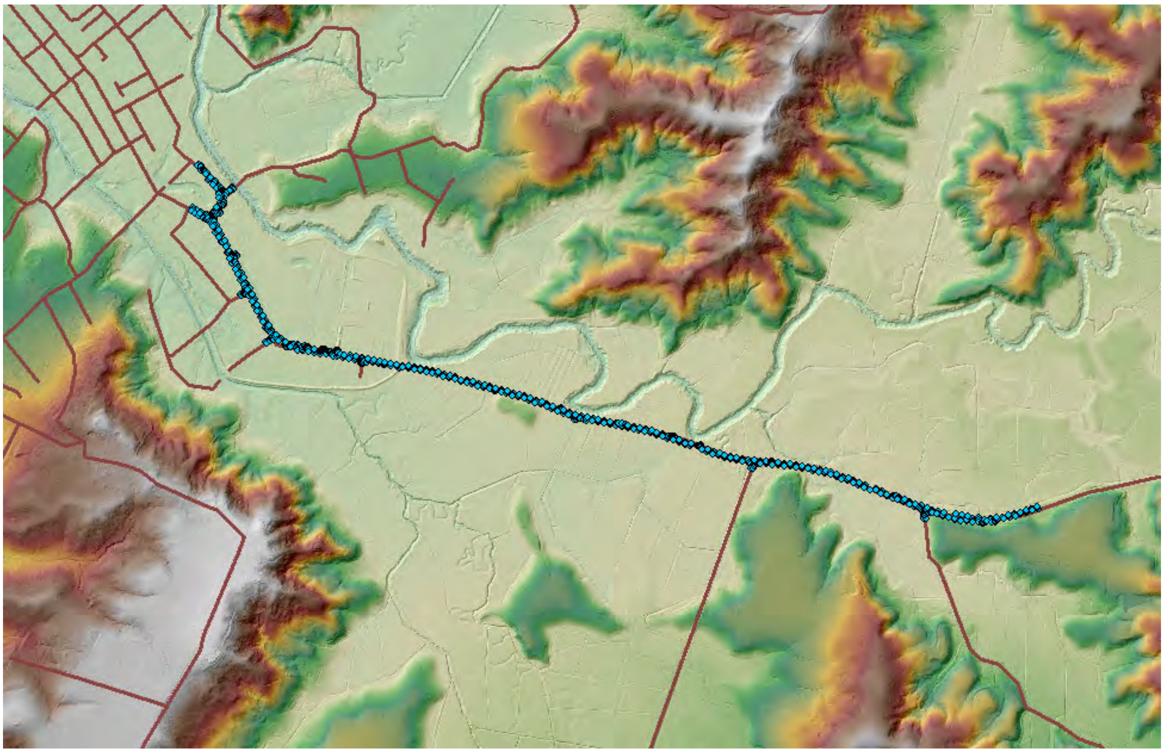


Figure 2-1: State Highway 1 Survey upstream of Kaitaia (blue dots)

2.2.4 Structure survey

A comprehensive survey of bridges, culverts, weirs and floodgates was provided by NRC in October-November 2018. This included additional survey that was completed after the commencement of this project. In some cases the previous GHD model had already been updated with the latest survey data, in which case only a check was done to ensure that the data used in the model appeared correct.

2.3 Terrain

LiDAR data was flown in the first half of 2018. Metadata for this survey was not provided when the survey was delivered, but it was confirmed by NRC that the survey uses the OTP (One Tree Point) datum and the New Zealand Transverse Mercator projection. The dataset included the data in various formats; for most of the modelling the HE_DEM dataset was used. This is the hydraulically enforced DEM which has some culverts burned into the grid. NRC have also checked these cut-outs and provided DHI with some guidelines on how to deal with these in the model. However, this advice only dealt with the cut-outs along SH1 near the overflow: this part of the model would be described by the survey of the road itself and thus the cut-outs would not affect the model in any material way. The HE_DEM contains approximately 400 of these cut-outs through roads or structures.

Checks of the LiDAR data show that for the most part the resolution is good. However, in some vegetated areas, especially in the upstream hilly and forested areas of the catchment, the resolution is not ideal. In addition to this, some high points are not well represented, such as stopbanks along the lower Awanui and Whangatane Spillway, where vegetation is present. Gaps in the data have been supplemented with survey data where possible, or by using sensible assumptions about what the data would look like. For example, some banks on the Awanui appeared to have been almost completely removed by the LiDAR processing, and it was assumed that the bank level would be similar to the levels upstream and downstream of the missing section of bank. The missing sections of bank were detected by carefully checking the LiDAR around the stopbanked areas of the Awanui and Whangatane Spillway. About half a dozen areas, where the stopbank did not appear correct, were detected. To ensure stopbanks were being modelled at the correct heights, where available, the 2010 Cato Bolam survey data was also used to ensure the most accurate stopbank heights were used.

The previous 2003 LiDAR, surveyed by AAM, was also used in a small section of the model to represent the pre-development levels in the area around the Whangatane Spillway inlet. The LiDAR was supplied by NRC in the OTP datum.

2.4 GIS asset and land use data

Land use shapefiles and asset data were provided by NRC, including:

- Roads/Railways
- Soil layers
- Land use layers
- Hydrological catchments
- Proposed additional streams

2.5 Calibration Data

The following data was provided or acquired to assist with the model calibration:

- Continuous rainfall measurements from the Te Rore, Mangakawakawa, School Cut, Observatory and Aero rainfall sites, for the February 2007, July 2007 and January 2011 flood events. Data from additional sites was included but not used in the modelling.
- Locations of all nearby flow and rainfall gauges.
- Data from daily-read rain gauges within the catchment
- Water level and rated flow data, at Donald Rd (Whangatane Spillway), Victoria Valley Rd, Te Puhi at Meffin Rd, Tarawhataroa at Puriri Place, Takahue at Grays and Awanui at School Cut. Each gauge datum is specified relative to the OTP datum.
- NRC's analysis of the flow gauging sites, including derivation of stage-flow rating curves.
- NRC analysis of overflows across State Highway 1 upstream of Kaitaia near Larmer Road.
- Spot level measurements labelled "MAX", for the February 2007, July 2007 and January 2011 flood events. These are surveyed debris levels, which can be taken as peak water level, albeit with an uncertainty of possibly $\pm$0.2 m.
- The water level at Ben Gunn Wharf, to be used as a tidal boundary condition, with the gauge datum specified relative to the OTP datum.
- Previous reports on the model and previous calibrations
- Video footage of the Whangatane intake in the February 2007 event

2.5.1 Staff Gauge conversions

The following Table 2-2 shows the conversion factors used by NRC to convert the staff gauge readings to the OTP datum.

Table 2-2: Staff Gauge conversion to OTP

Gauge Location	River	Conversion Factor SG to mOTP
School Cut	Awanui	8.664
Waikuruki	Lower Awanui	6.814
Donald Road	Whangatane Spillway	8.118
Victoria Valley Rd	Victoria River	61.091
Takahue at Grays	Takahui River	42.718
Tepuhi at Muffin Rd	Te Puhi Stream	30.369
Ben Gun Wharf	Awanui River	-3.530
Puriri Place	TarawhataRoa River	10.142

2.6 Design Model Data

The following data was provided and acquired for use in the Design Event modelling

- Rainfall Hyetographs and Aerial Reduction Factors from NRC was used in generating the design rainfall for the catchment.
- Rainfall depths were extracted from HIRDS v4
- Tidal timeseries representing the downstream boundary provided with the GHD model.
- Baseflow data from the flow gauge recordings and the previous GHD model

2.6.1 Previous estimates of 1% AEP flow

There have been a number of previous studies of Awanui catchment flooding. Various estimates have therefore been made of peak flow rates approaching Kaitaia (noting that the larger Awanui River floods spill over State Highway 1 (SH1) into Tarawhataroa Stream).

Tonkin & Taylor (2014) (/3/) tabulated some of these peak flow estimates, and their Table 2.1 is re-produced below as Table 2-3.

Table 2-3: Estimates of 1% Annual Exceedance Probability peak flow, Awanui River approaching Kaitaia.

Table 2-1 Awanui River upstream of State Highway 1 overflow – summary of 1% AEP peak flood flows

Method	Estimated peak discharge (m^3/s)	
	Lower	Upper
Frequency analysis	380	440
Catchment model (GHD, 2013)		882
Rational method	270	312
Clark Unit Hydrograph	295	328
McKerchar & Pearson 1989	360	380
TM61	350	390
NIWA 2005		368

However, an analysis of annual peak flows by NRC, dated 24th August 2018 (Table 2-4 produced 100-year flows, using five different frequency distributions, ranging from 449 m^3/s to 542 m^3/s, and chose the middle of these (Pearson Type 3), rounded for design purposes to 500 m^3/s.

Table 2-4 Summary of Flows Derived using Hilltop Software

Event (ARI)	Gumbel (EV1)	Generalised Pareto	Pearson Type 3	Generalised Normal	GEV
100	449.35	490.18	501.16	528.13	542.38
50	400.95	435.33	439.45	449.29	452.33
20	336.36	358.66	357.29	353.87	349.94
10	286.46	297.33	294.49	287.55	282.77

In the NRC flood frequency analysis, completed in 2018, there is some significant uncertainty with design flow estimation for the Awanui, and the outputs are sensitive to the Q/h rating assessed for School Cut, and the rating assessed for SH1 overflow. Table 2-5 shows the various assessments done since the 1980's.

Table 2-5: Historical Flood Frequency Estimates for the School Cut gauge record

	NRC 2018	T+T 2014	NIWA 2005	NIWA 1996	NCC 1986
Record assessed	**1958 - 2017**	**1958 - 2012**	**1958 – 2004**	**1958 – 1995**	**1958 – 1985**
20yr ARI	336 - 350		281 +/- 40	248 +/- 39	301
50yr ARI	401 - 452		331 +/- 53	291 +/- 51	366
100yr ARI	449 – 542	380 - 440	368 +/- 62	323 +/- 60	423

The latest estimate done by NRC in 2018 results in an increase in the estimated flood flow. The reasons for this are:

1. Additional large flood events since 2005, however there have not been significant floods since 2012.
2. Recent NRC Q/h rating changes have increased flows of the largest events. For example the March 2003 flood was taken to have a peak flow at School Cut of 232m^3/s in the NIWA 2005 assessment, but following rating adjustments there is now an estimated flow of 268m^3/s at School Cut. Following the recent NRC (hydrology) review of the School Cut ratings, post 2006 flows were increased, resulting in an upwards adjustment to the July 2007 peak flow (now 256 m^3/s).
3. The SH1 overflow rating has been updated, and now gives higher flow. For March 2003, NIWA estimated an overflow of 126m^3/s, whilst for the same event, the new SH1 overflow rating gives an overflow of 173 m^3/s. The 1st ranked event (March 2003) therefore has a total flow of 441 m^3/s (upstream of SH1 overflow), using Pearson Type 3 distribution, the return period of this event is 51 years.

3 Model build

The model uses the MIKE by DHI software version 2017 with Service Pack 2 and associated hotfixes. The model is based on the OTP (One Tree Point) datum and the NZGD 2000 New Zealand Transverse Mercator projection. All input data and previous model data has been adjusted from the Unahi datum where necessary.

3.1 Hydrological model

3.1.1 Catchment delineation

The sub-catchment configuration has been adjusted from the existing GHD model. Sub-catchments were first adjusted based on the instructions in the Request for Tender document NRC (2017). These changes mainly referred to the catchments in or around Kaitaia where the MIKE Urban model is located. Following the extension of the MIKE 11 model, with the addition of several new branches, the hill catchments were further split and adjusted to allow for better allocation of flow to these new branches. During this process all sub-catchments in the model were checked and some improvements to the delineation were made to better represent the most likely flow pattern in the catchment. The delineation process was done in ArcGIS on a sub-catchment by sub-catchment basis using the output from the hydrology tools as a guide. The catchment is delineated into 637 sub-catchments, 353 of these are represented in the MIKE URBAN model and the remainder modelled within the MIKE 11 NAM model. Although there are more MIKE urban catchments these account for only 0.5% of the total catchment area of approximately 456 km^2 or 0.5% of total runoff volume

In the MIKE 11 model a catchment was either assigned to a single point on a branch or distributed between points, depending on whether the catchment discharges to a particular location. Because there were many changes to the catchment delineation and new branches added, all of the MIKE 11 rainfall runoff connections were updated in the model. This was done by first auto-assigning the catchments based on the nearest MIKE 11 h-points and then manually checking and correcting where necessary any mis-assigned catchments.

3.1.2 Hydrological Parameters

The hydrological model applied to the rural sub-catchments (most of the Awanui catchment) is NAM, which is the abbreviation of the Danish "Nedbør-Afstrømnings-Model", meaning precipitation-runoff-model. This model was originally developed by the Department of Hydrodynamics and Water Resources at the Technical University of Denmark (Nielsen and Hansen 1973, /5/).

NAM accounts first for surface storage, which loses water in four ways: evaporation, interflow (horizontal leakage), infiltration into the “lower zone” and infiltration direct to groundwater storage. Once maximum surface storage is reached, some of the excess water therefore enters the streams as overland flow, after being routed through reservoirs to provide the time-of-concentration of the sub-catchment. The fraction of excess water that becomes overland flow varies linearly with the lower zone storage (which can be regarded as the soil moisture content of the root zone).

The key NAM parameters are listed and described in Table 3-1. For more detail, see DHI (2017)

Table 3-1: NAM parameters described

Umax	Upper limit of surface storage (mm)
Lmax	Upper limit of lower zone storage (~ soil moisture)
CQOF	Overland flow runoff coefficient
CKIF	Interflow runoff coefficient
CK1,2	Time constant for reservoir routing for overland flow and interflow
TOF	Soil moisture threshold for overland flow
TIF	Soil moisture threshold for interflow
TG	Soil moisture threshold for groundwater recharge
CKBF	Time constant for reservoir routing for baseflow
Initial Conditions	
U	Surface storage (fraction of Umax)
L	Lower zone (soil mosture) storage (fraction of Lmax)
QOF	Overland flow
QIF	Interflow
BF	Baseflow

The MIKE 11 NAM parameters were derived through the hydrological calibration process, which is described in Appendix B. For this calibration, the Awanui catchment was divided into 4 hydrological zones, Swamp, Te Puhi, Tarawhataroa and Takahue, as illustrated in Figure 3-1. Each of these areas have been assigned different Lmax and Umax parameters and different initial lower zone storage L, all of which have been derived in the calibration process.

The CK 1,2 parameter was assumed to be equivalent to the individual time of concentration for each sub-catchment as estimated by calculating the geometric mean of the Bransby Williams and Ramser – Kirpich time of concentration methods. To calculate the flow length and slope required for these methods, flow lengths were derived for each sub-catchment, and their equal-area slope calculated. For very small catchments a minimum time of concentration of 10 minutes was used, a total of 27 catchments out of 284 were classified as “very small”. Note the unit for the CK 1,2 parameter is hours.

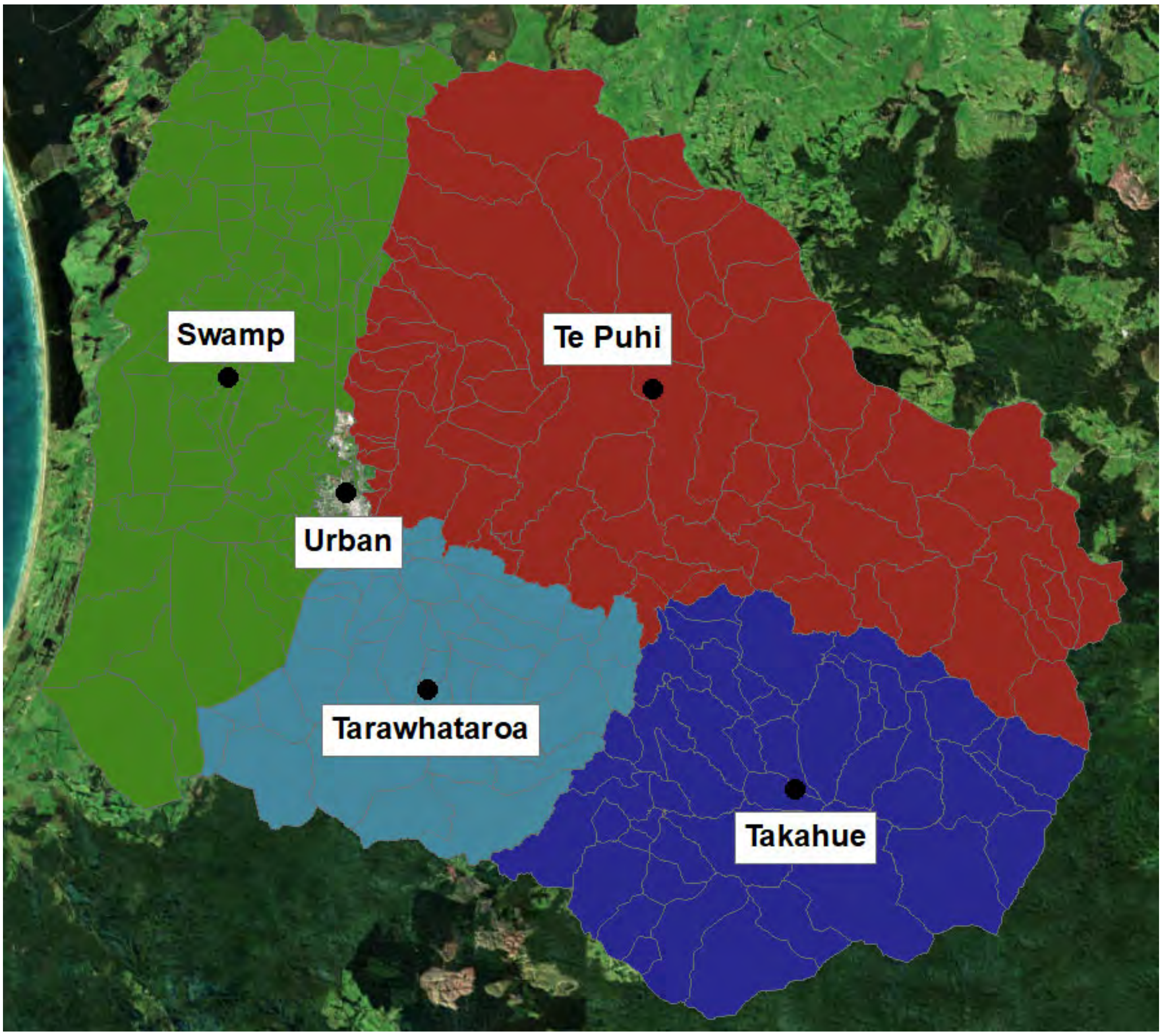


Figure 3-1: Awanui Catchment delineation

The MIKE Urban model uses virtually the same MODEL B setup as was used for the previous GHD modelling. Where some sub-catchments have been merged, the parameters have been recalculated based on the proportion of the separate sub-catchments.

3.1.3 Rainfall

Rainfall is applied differently between the Calibration and Design models. The methodology is described below

3.1.3.1 Calibration event rainfall

In the calibration models, the gauged rainfall is used and distributed to the sub-catchments using Thiessen polygons, Figure 3-2. Where two or more polygons divide a sub-catchment; the polygon with the largest coverage takes priority, a simplification compared to averaging the rainfall gauge records for that sub-catchment. Because of the small size of the catchments relative to the Thiessen polygons it is assumed that using this method of choosing the polygon with the largest coverage will make practically no difference to the model results.

The Thiessen polygon approach is long-established approach and provides a reasonable rainfall distribution for modelling. However, it cannot account for features of the true rainfall distribution that go unrecorded. For this reason, and particularly given recent rain radar observations Northland (/6/), modelled sub-catchment rainfalls have significant uncertainty despite the accuracy of the rain gauges themselves.

Figure 3-2 illustrates the locations of the 5 rainfall sites used in the two calibration events, these are all automatic gauges.

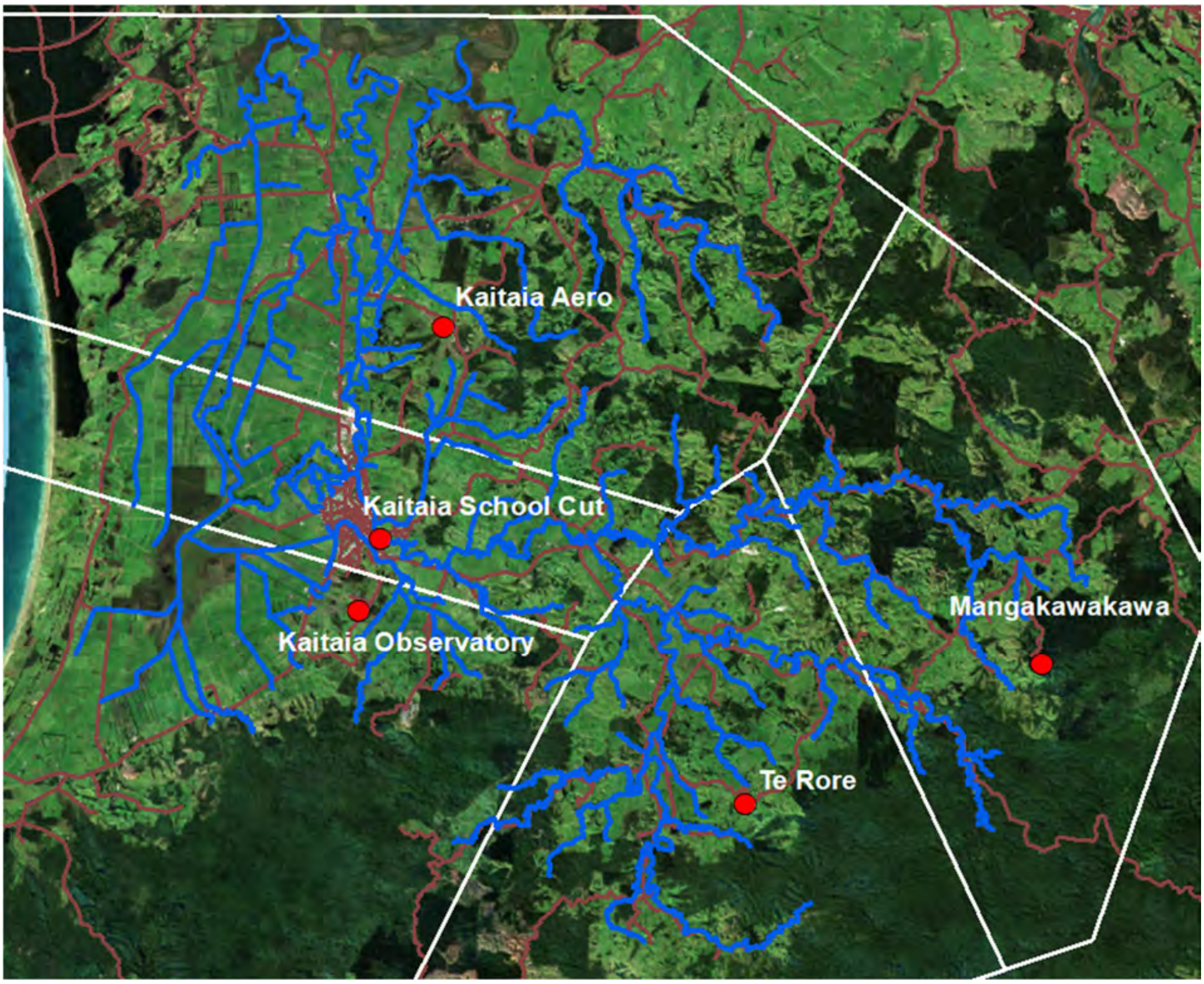


Figure 3-2: Location of Calibration Automatic Rainfall Sites

3.1.3.2 Design event rainfall

A detailed description of the derivation of the design hyetographs is given in Appendix C. It was agreed with NRC that the design rainfall depths should be derived directly from all rain gauge records with a reasonably long record. This included six daily-read gauges, some with very long records, along with the original 5 automatic rainfall stations used in the calibration. For simplicity the two stations near Kaitaia were merged into one location. 24-hour rainfall depths for all the rain gauge sites were analysed using DHI's software EVA. Design rainfall hyetographs for individual sub-catchments were then obtained by applying Thiessen polygons centred on these gauges, Figure 3-3.

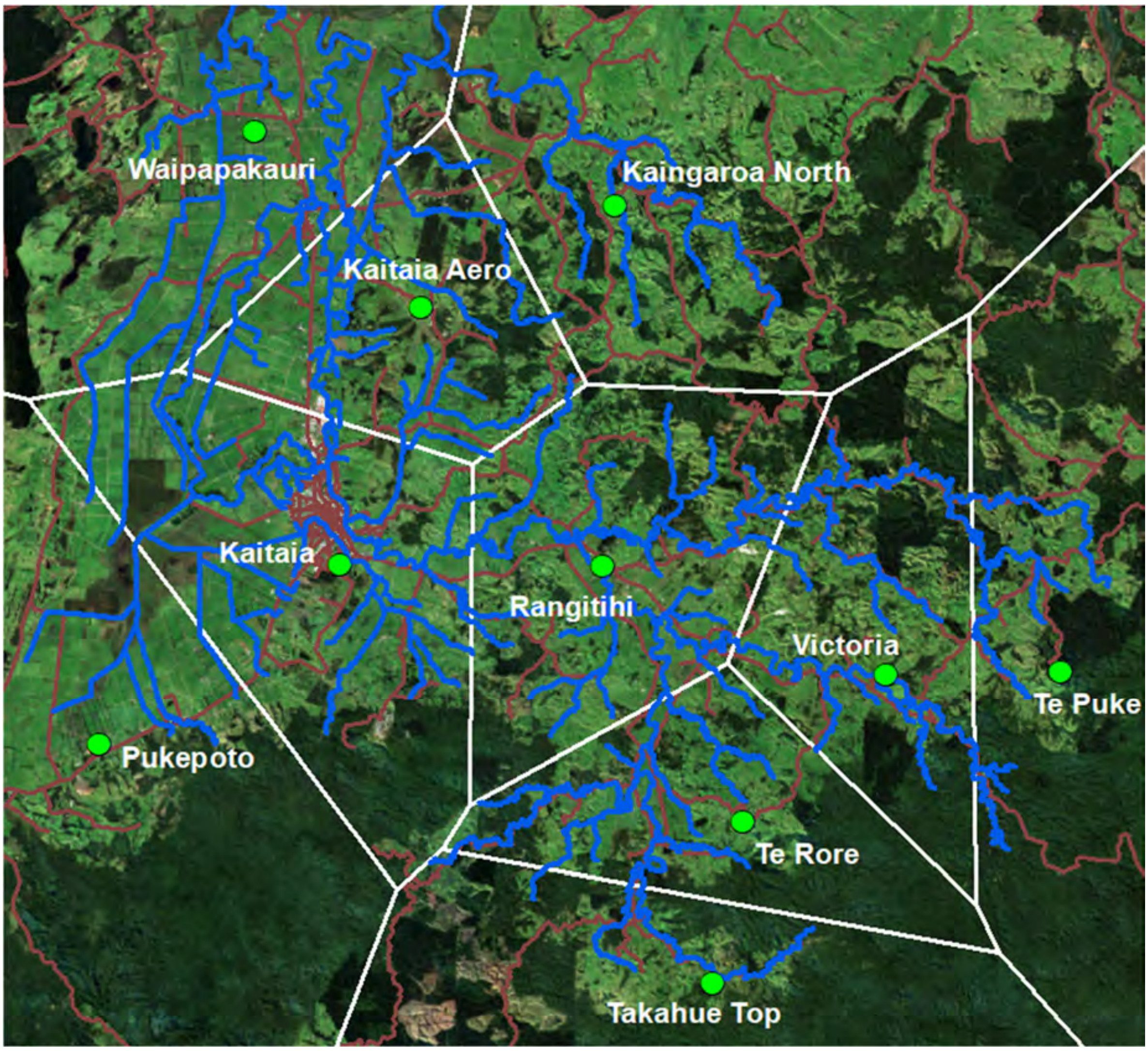


Figure 3-3: Design Rainfall Stations

An Area Reduction Factor (ARF) of 0.94 was applied for all model runs as specified by NRC.

Five design rainfall scenarios were modelled: 24-hour events with Average Recurrence Intervals (ARIs) of 10 years, 20 years, 50 years, and 100 years, including the 100-year ARI event with climate change. The 24-hour rainfall profile specified in HIRDS v4 was used and is illustrated in Figure 3-4. Unlike NRC's Priority Rivers hyetograph and some other design hyetographs used elsewhere in New Zealand, this is not a Chicago-type hyetograph in which 100-year rainfall depths for shorter durations are nested within the 24 hour period.

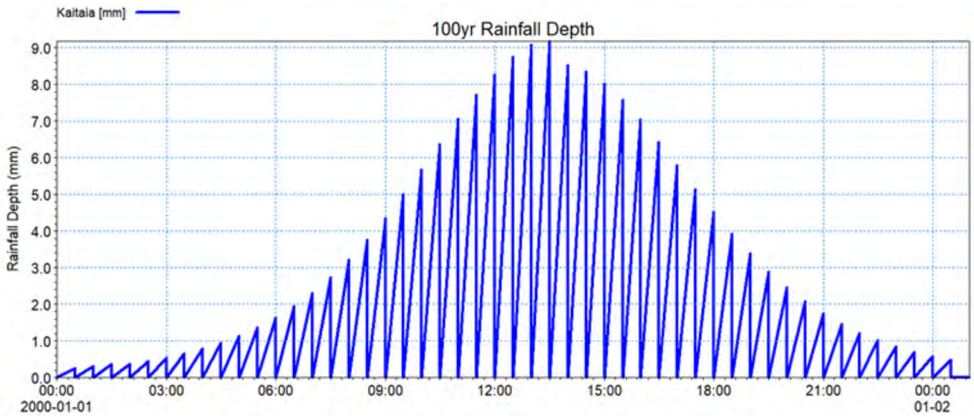


Figure 3-4: Example 100yr rainfall profile. "Rainfall depth" axis is depth in mm per half-hour.

Climate change has been calculated based on a 2.1° increase in temperature. Using the table provided in the HIRDS website help (https://www.niwa.co.nz/information-services/hirds/help), for a 100 year ARI and a 24 hour rainfall duration, the rainfall increase is 8.6% per °C, resulting in an 18.06% increase in rainfall for the 100 year ARI climate change scenario. Note this multiplication factor is used in place of an IPCC scenario. The closest RCP (Representative Concentration Pathway) scenario is the RCP 4.5 which predicts a temperature increase of 2.4° by 2100.

The total 24-hour design rainfall depths applied to each rainfall station area are tabulated in Table 3-2. These depths have an aerial reduction factor of 0.94 applied and an additional +10% scaling factor to the Rangitihi, Takahue top, Te Puhi, Te Rore and Victoria stations. Scaling factors were applied to better match the flood frequency estimates at School Cut. For further information in the derivation of the design rainfall depths and the scaling factors refer to Appendix C.

Table 3-2: Design Rainfall Depths

Station Name/Rainfall Depth (mm)	10yr	20yr	50yr	100yr	100yr + CC
Waipapakauri	115	122	133	147	163
Kaingaroa North	114	128	142	164	182
Aero Kaitaia	123	139	159	186	206
Kaitaia	130	149	165	190	211
Rangitihi	134	152	170	179	219
Pukepoto	133	155	199	237	263
Te Rore	144	160	180	187	228
Takahue top	154	180	219	247	302
Victoria	152	176	209	229	280
Te Puhi	158	183	214	230	281

3.1.4 Baseflow

The initial long-term NAM hydrological calibration (with large sub-catchments) produced modelled time series of baseflow, these flows at the beginning of the calibration events being very minor compared to peak flood flows.

For modelling each calibration event with the full model, these baseflows were applied to all sub-catchments as initial conditions, *pro rata* with sub-catchment area.

3.2 Hydraulic model

3.2.1 MIKE 11 model

The MIKE 11 model is made up of 135 branches and 122 structures, Figure 3-5. The basis for the MIKE 11 model was the previous GHD model. However, a number of changes and additions have been made to the model. These include:

- Realignment of branches to better follow the talweg of the streams. This was primarily done to improve the accuracy of the cross-section extraction and 1D flood mapping.
- Addition of new branches in both the upper and lower catchment to allow for flood predictions over a larger area
- Removal of some branches as specified in the RFT
- Adjustments to the Awanui start chainage to align the zero chainage to the confluence between the Victoria and Karemuhako Rivers.
- Addition and update of the hydraulic structures in the model
- Update of the model cross sections based on the 2018 LiDAR survey
- Update of the channel roughness based on the model calibration

Model Cross Section Generation

The methodology for generating the model cross sections was as follows:

1. A MIKE11 model was built with the surveyed cross sections and some selected LiDAR sections from the areas without survey.
2. For the LiDAR sections, the bottom levels were dropped slightly to better estimate the stream bed. This level adjustment was based on an estimate of the water depth. The estimate of water depth was based on comparing the LiDAR versus survey depth for nearby surveyed areas and using this as a guide. A nominal depth of 0.5m was applied where no nearby data was available.
3. Using the MIKE 11 mapping function, a bathymetry was generated out of the 1D cross sections. By interpolating interim cross-sections, the MIKE 11 mapping function follows the stream talweg alignment and allows the full stream planform to be represented in the 1D-2D bathymetry.
4. The output raster was then clipped to the extent of the low flow area (i.e. where there are LiDAR water returns), the clip extent generally being based on a fixed width buffer, for each stream or part of stream.
5. The clipped MIKE 11 grid and a 0.5m grid derived from the LiDAR were merged, choosing the lowest points in the overlapping rasters.
6. The grid was loaded into MIKE Hydro and the cross sections extracted, every 30 meters, resulting in a total of approximately 15,000 cross sections in the model.
7. QA of the cross sections was done, in which each cross section was checked, and some manual edits and deletions were done to ensure the final cross section set contained sensible data. For example, adjustment was needed on some bends because the auto-generated cross section had been created on an alignment that did not represent the actual cross section.

8. Bank markers were set for the cross sections based on the MIKE 21 mesh extent blockout width and the nearest levee in the cross section. The extent of the cross sections was defined as; from between stopbank crests, where these were available. Elsewhere the cross section end points were defined between the width where the channel is incised below the surrounding ground. The location of the boundary between the 1D and 2D models was defined manually by looking at LiDAR data and aerial photography.

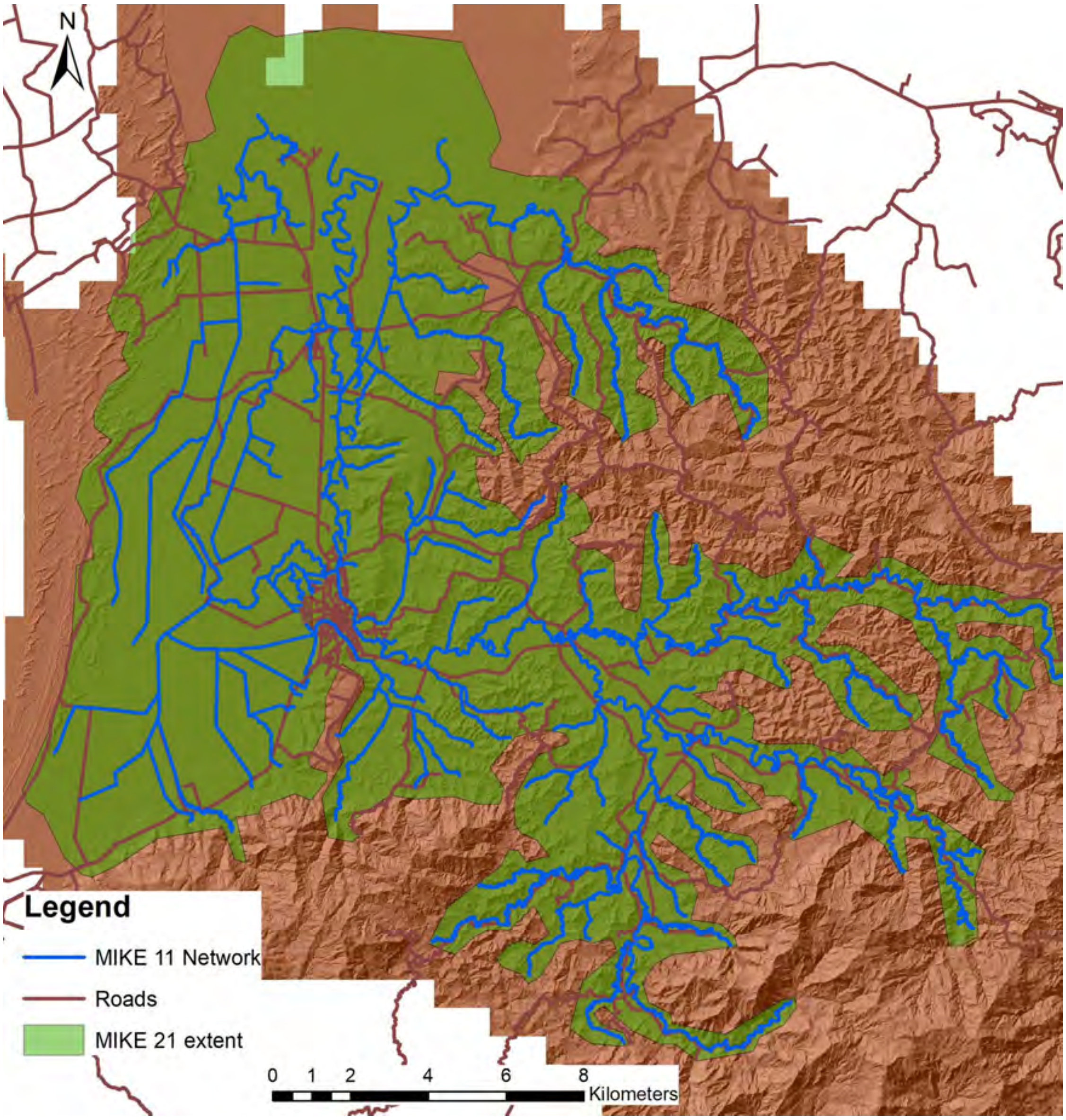


Figure 3-5: MIKE 11 Network

Roughness

The model roughness was calibrated as discussed in Appendix B. For areas that were not able to be explicitly calibrated, general roughness values were used. These comprised:

Table 3-3: MIKE 11 roughness by area

Stream	Manning's n
Lower Catchment (Tidal)	0.03
Upper Catchment (Hills)	0.096
Mid Catchment (Swamp)	0.05

Model parameters

The MIKE 11 model uses all default settings except for using a delta value of 0.85 as is standard for a MIKE FLOOD model, delhs of 0.02 to improve stability of structures in the tidal reaches and a water level exceedance factor of 40m to account for any shallow cross sections. The MIKE 11 model runs at a 1 second timestep in the MIKE FLOOD Simulation.

3.2.2 Cut-down MIKE 11 model

A cut down version of the MIKE 11 model was made for use in the hydraulic calibration. This model contained only the Tarawhataroa Stream downstream of State Highway 1 overflows as far as Lake Tangonge, and (separately) the Awanui downstream of School Cut along with the Whangatane Spillway. This model used the measured flow from the School Cut gauge (on the Awanui) and Puriri Place gauge (on the Tarawhataroa) as the upstream boundary conditions for the model, with no other inflows. This model is discussed in further detail in Appendix B.

3.2.3 MIKE 21

Mesh Build

The MIKE 21 model comprises a flexible mesh terrain with a tidal boundary. Inflow from the rivers and streams spills into the MIKE 21 domain via the MIKE FLOOD lateral links and the MIKE URBAN inlet links. The estuary at the downstream end of the catchment is modelled solely in the MIKE 21 model. The mesh extent was derived by assessing the terrain slope in the eastern hill area, and the streams to be included in the MIKE 11 model. The derivation of the mesh extent was done using GIS with some manual modifications. In order to simplify the mesh outline, the mesh extent was simplified and thus includes some hill areas. However, since all runoff is applied to the 1D river models, these hill areas will not become flooded in the simulation.

The mesh comprises approximately 5 million triangular elements with a minimum element size of 5.5m^2, maximum size of 172m^2 and an average size of 64m^2. Care was taken in the mesh build to avoid the generation of very small elements that would significantly slow down the simulation. To help with this the polygons and polylines used to define the MIKE 11 river area, the main road breaklines and the mesh outline were simplified to prevent vertices from sitting too close to one another. The ArcGIS simplify polygon tool was used to remove unnecessary points from the mesh outline and simplify some of the bends without compromising the overall structure of the mesh blockout. An example of this is if there was a straight segment with 3 points, the middle point is not necessary because it does not add to the definition of the line, so this point can be removed.

The mesh was interpolated using a 1m grid of the HE_DEM (refer Section 2.3). The exception to this was in two areas:

1. The area around the Whangatane Spillway inlet where the terrain had been changed. This area was replaced with the 2003 LiDAR for modelling pre-development conditions.

2. The estuary area where the 2004 bathymetry survey (OTP datum) was incorporated into the HE_DEM data.

The special MIKE 21 feature of depth correction is used in the model to allow a dfsu file to determine the element levels in the simulation. Note that the dfsu file shows negative values, but these are interpreted as positive in the simulation. This depth correction file contains the following modifications from the base mesh:

- Inclusion of survey data at stopbanks at the MIKE 21/ MIKE 11 boundary

- Lowered elements where the MIKE URBAN model links to MIKE 21 using the outlet link. The MIKE 21 element is lowered to match the MIKE URBAN outlet level.

- Depending on the scenario the depth correction file is used to define the differences in terrain for the pre and post development scenarios around the Whangatane Spillway inlet. In addition, benching and spillways for the design scenarios were also represented in the 2D surface this way.

Open boundaries

There is one open boundary at the north end of the model domain near the top end of the estuary. At this location a tidal timeseries is applied along the length of the boundary.

Dikes

In order to more accurately model the obstruction and overflow due to major roads and stopbanks, a total of 61 dike structures were included in the MIKE 21 base model. The benefit of using these structures is that it allows for a good representation of the road or stopbank level and the hydraulic representation of overtopping, without the need to add significant detail into the model mesh. The series of figures, Figure 3-6 to Figure 3-9 indicate the location of the modelled dikes as orange/red lines. Surveyed stopbank data from the 2010 Cato Bolam survey is also plotted, as the dark blue points (note these appear on top of the red lines in Figure 3-6 and Figure 3-8). Where the survey data was available it was used for the Dike structures, otherwise the high points from the 2018 LiDAR survey were extracted to form the dike crests. The LiDAR extraction was done by manually digitising the high point as an ArcGIS polyline, and then converting this line into points spaced at 5m intervals. These points were then allocated the highest value of the LiDAR within a 2.5m radius ensuring that the Dike represents the top of the stopbanks.

Where the Cato Bolam survey data aligned with the lateral links, this data was used to set the 2D mesh level, to ensure that overtopping would occur at the correct level. Because the lateral links are set to use the HGH method, the MIKE 11 cross sections should have picked up the correct bank level already, based on the methodology used to generate the cross sections. This secondary step is used to represent areas where it was found that the stopbanks were missing from the LiDAR survey.

The length of State Highway 1, where the State Highway crosses the floodplain, was also modelled as multiple dike structures, with splits occurring at stream crossings. These dikes used the HE-DEM, except for the main SH1 overflow dike, upstream of Kaitaia, which uses the Opus 2018 Survey instead.

Figure 3-6: Dikes at the downstream end of the Whangatane Spillway

Legend -- Yellow lines: dikes using LiDAR levels, Red lines: dikes using Cato Bolam survey, numbers: river chainage, dark blue points: Cato Bolam survey and blue lines: river branches

Figure 3-7: Dikes located at outfall of Whangatane Spillway and Awanui River

Legend -– Yellow lines: dikes using LiDAR levels, numbers: river chainage, dark blue points: Cato Bolam survey and blue lines: river branches

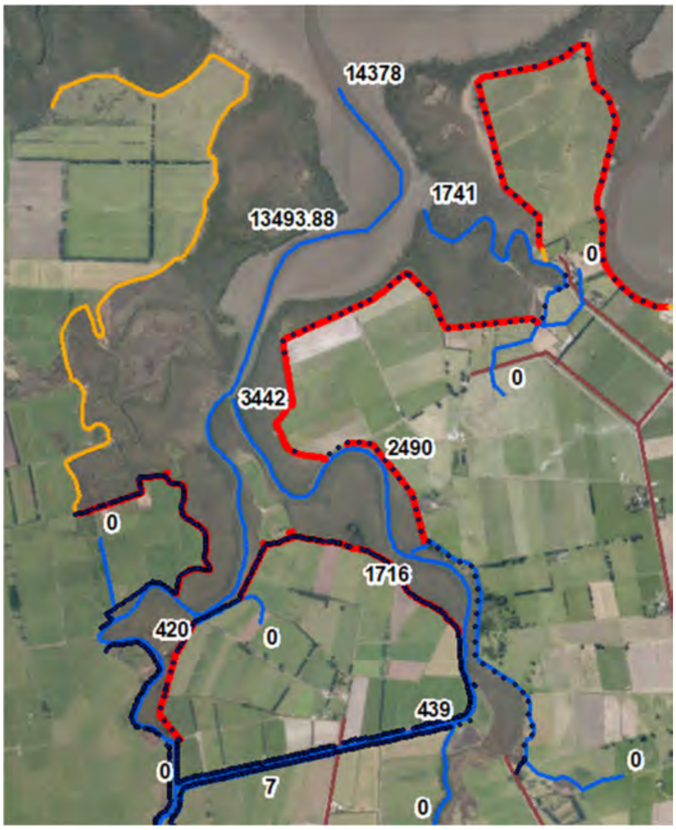


Figure 3-8: Dikes at the downstream end of the Waipapakauri Cut

Legend -– Yellow lines: dikes using LiDAR levels, Red lines: dikes using Cato Bolam survey, numbers: river chainage, dark blue points: Cato Bolam survey and blue lines: river branches

Figure 3-9: Dikes around Awanui Township

Legend – Yellow lines: dike using LiDAR levels, numbers: river chainage, dark blue points: survey and blue lines: river branches

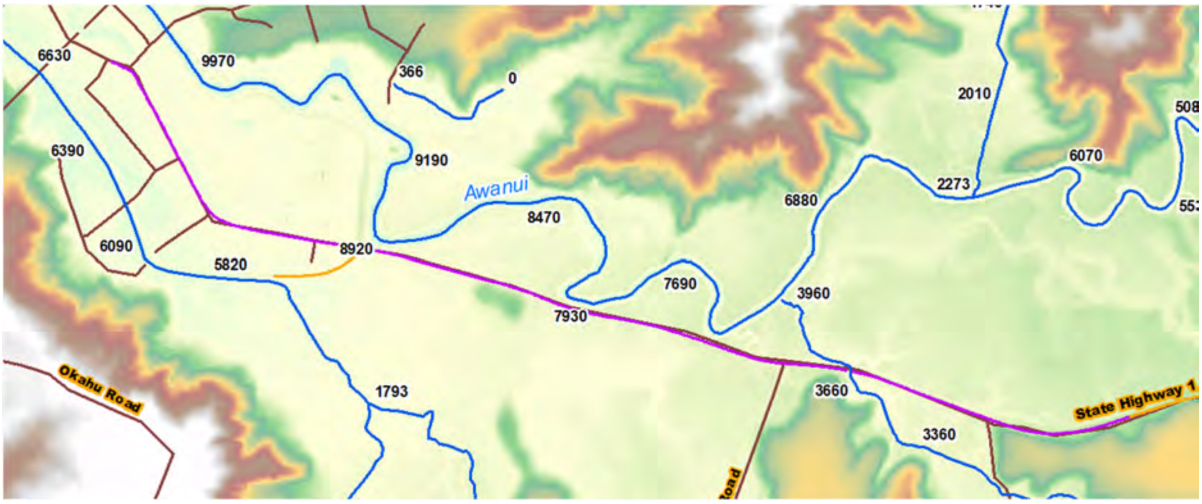


Figure 3-10: Dikes around the SH1 overflow

Legend – Yellow lines: dikes using LiDAR levels, purple line: Opus Survey dike, numbers: river chainage and blue lines: river branches

Figure 3-10 shows the location of the dike structure used to model the State Highway 1 overflow. The level values for this dike (purple line) were taken from the 2018 Opus survey. The small dike in orange between the SH1 and the Tarawhataroa stream is a stopbank, levels for this were taken from the LiDAR.

Roughness

The MIKE 21 roughness was derived from the land use information provided and the previous GHD model, features included in the roughness definition file were: road data, building data and land use type such as vegetation, pasture or urban. The roughness values used are presented in Table 3-4. Building roughness was set to a Manning's n value of 0.2. Various approaches to buildings can be made including blocking the building completely from the mesh. The approach of using a higher roughness was preferred for this project because it allows for water entering a building, which is known to happen in larger flood events.

Table 3-4: MIKE 21 roughness values

Landuse type	Manning's M	Mannings' n
Road	50	0.02
Building	5	0.2
Town	15	0.067
Vegetation	8	0.125
Open Space	20	0.05

Model parameters

The MIKE 21 simulation uses the following parameters:

- Timestep: Minimum 0.1s, Maximum 1s
- Critical CFL number: 0.8
- Flooding and drying depths: Advanced Flood and Dry, Wetting depth 0.05m, Flooding 0.03m and Drying 0.005m.
- Solution technique: The model uses the Low order Solution in space and time. This decision was made after a model run was conducted using the higher order solutions and it was found that there was no significant difference in the model results to warrant using the higher order solution. The Higher Order solution gave a slightly lower flow at School Cut, around 0.5% lower, and the water level differences on the floodplain are in the order of 10mm.
- Eddy Viscosity: Constant formulation using a value of 0.1m^2/s

3.2.4 MIKE URBAN

The MIKE URBAN model consists of 926 manholes, 57 outlets, 656 closed pipes, 276 open channels, 6 weirs and 1 pump. The MIKE URBAN model only covers a small area of the whole Awanui catchment, and represents the town of Kaitaia. The adjustments to the MIKE URBAN network are as per the RFP tender document. Figure 3-11 shows where nodes and pipes have been removed from the MIKE URBAN model as per the RFP.

The Urban network itself is reasonably small relative to the rest of the model, with open channels modelled explicitly in the MIKE Urban model, and the pipe and node detail to sump level. There is one pump in the model in the northern area (Northpark) which pumps at a constant rate of $0.5 m^3/s$ and is controlled by start and stop levels. This pump represents physical pumps on the property which are designed for pumping excess groundwater over the spillway. The size of the pump was assumed to be correct in the GHD modelling.

Pipe roughness ranges between 0.011 and 0.013 Manning's n, and the open channel roughness is set to a Manning's n value of 0.035. Modelled manhole losses per manhole type are presented in Table 3-5.

Table 3-5: MIKE Urban node losses

Node Type	Loss Type	Km
Normal Manhole	Mean Energy approach	0.25
Outlet	Sharp Edge	0.5
CRS node (open channel)	No losses	0

Three sets of infiltration parameters (Table 3-6) were applied in hydrological modelling of different areas (Figure 3-11) using MIKE Urban model B, These parameter sets have not been adjusted from the 2015 GHD model.

Table 3-6: Urban Infiltration parameters

	Start Infiltration rate mm/hr	Final Infiltration rate mm/hr	Hortons exponent (wet) (1/s)
AWA1	6.52	2.25	8.50E-03
AWA2	6.52	4.82	8.50E-03
AWA3	9.50	3.00	8.50E-03

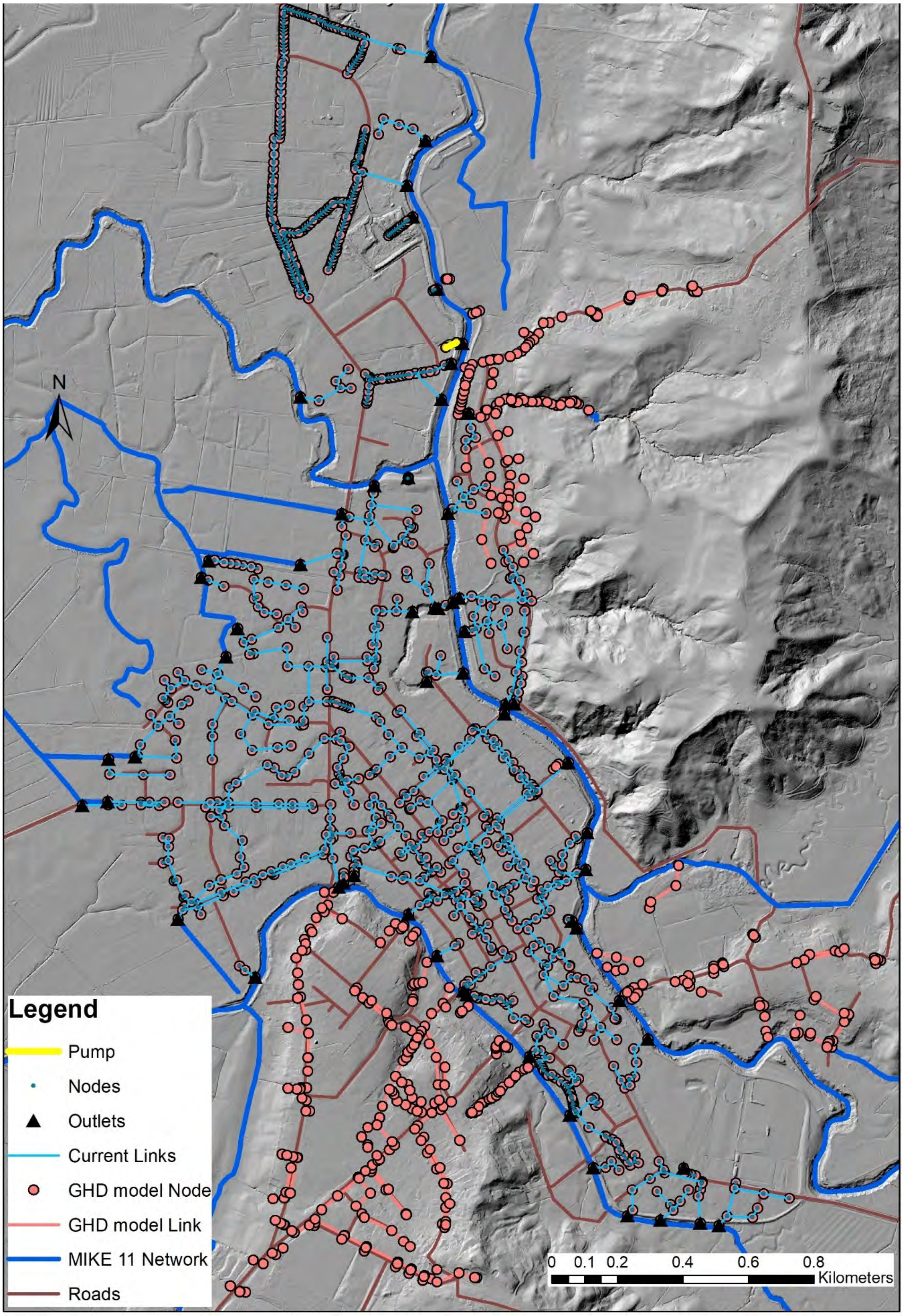


Figure 3-11: MIKE Urban model

3.2.5 MIKE FLOOD

MIKE FLOOD combines the MIKE 11 channel model, the MIKE 21 overland flow model and the MIKE URBAN model into one. The MIKE FLOOD model contains 4726 lateral links, 3 standard links, 862 urban links and 53 urban/river links. The lateral links have been split at regular intervals to reduce link skewing, especially in the winding upper catchment branches. Links have also been split at all hydraulic structures. Link skewing can occur where the links are defined around bends and the MIKE 11 h-point chainage and the MIKE 21 mesh elements become "out of sync" with each other due to the differences in streamline length between the talweg and the bank. An example of link skewing is illustrated in Figure 3-12, where the left picture shows MIKE 11 H-points linking to mesh elements well downstream of where they should be, and the right picture shows the corrected version.

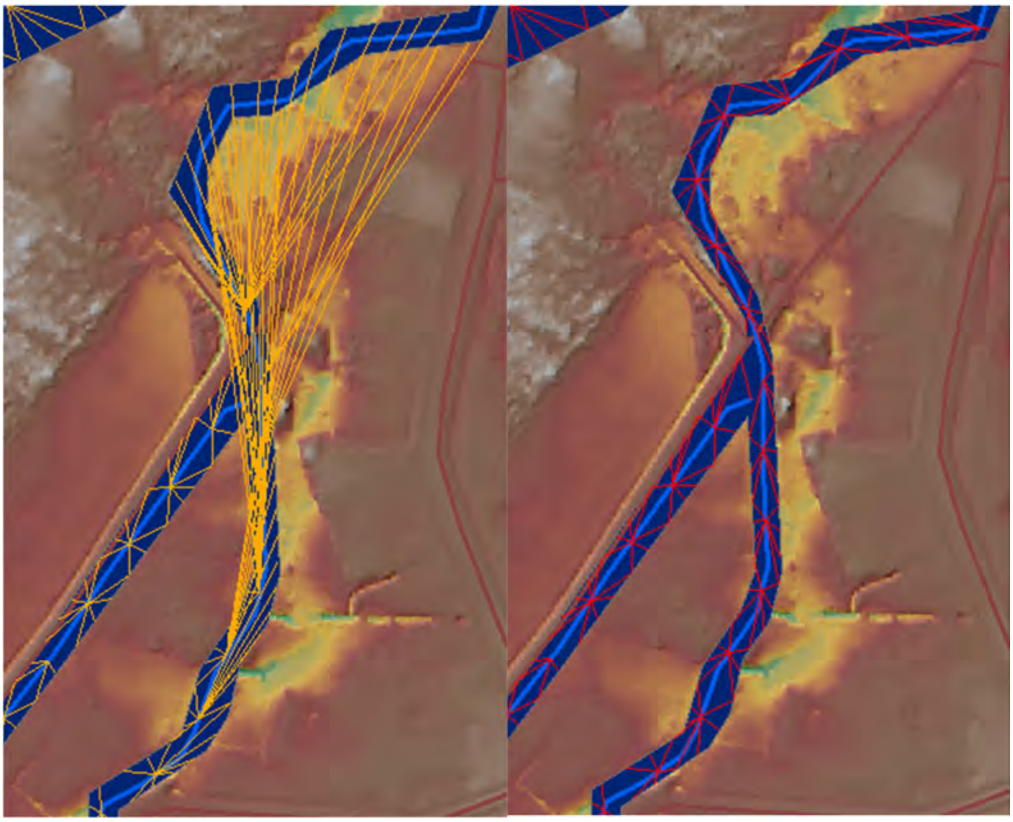


Figure 3-12: Example of significant link skewing

The links have also been snapped to the edge of the mesh area to ensure that they do not overlap with the no-mesh area and cause issues in the flow transfer between MIKE 21 and MIKE 11. The MIKE 11/MIKE 21 link boundary was defined manually, for all MIKE 11 branches, as part of the mesh build and cross section generation, and aligns in most places with the river levees.

The links use the HGH method for determining the lateral link levels. This method takes the higher of the MIKE 11 levee and the MIKE 21 element as the bank level. Given that the MIKE 11 cross sections are spaced at a regular 30m interval using data extracted from the LiDAR terrain, and the MIKE 21 model uses the same terrain, there is a reasonable match between the two datasets. This makes HGH the most sensible source method to use for this study. The urban link parameters from the previous GHD model were retained. The remaining lateral link parameters use the default values, except for the exponential smoothing factor, which was lowered to a small value of 0.01, due to the small timestep of the model. This equates to a lag of approximately 2.5 minutes in the flow hydrograph in the main river, which is not significant in the wider scheme of the model. The depth tolerance in all of the lateral links has been kept at the default value of 0.1m.

Because of the sinuous nature of the upper reaches in the Awanui catchment, some assumptions were needed for the link locations, in order to ensure that the mesh size did not become too small. To avoid generating very small mesh elements, the MIKE 21 domain was not extended into the inside of meander loops where the gap between the two parts of the 1D domain was smaller than 10m. Some examples of this on the Karemuhako Stream are illustrated in Figure 3-13. This simplification potentially reduces the flood conveyance in these areas once the banks are overtopped and may artificially lengthen the flow path. No additional link channels have been added to the 1D channel to represent any overland flows that bypass part of a loop. It is not expected that the model results will be overly sensitive to this assumption, given that model calibration has shown that modelled flow is already arriving downstream earlier than measured, nevertheless. However, the accuracy of this assumption may be worth investigating in future work.

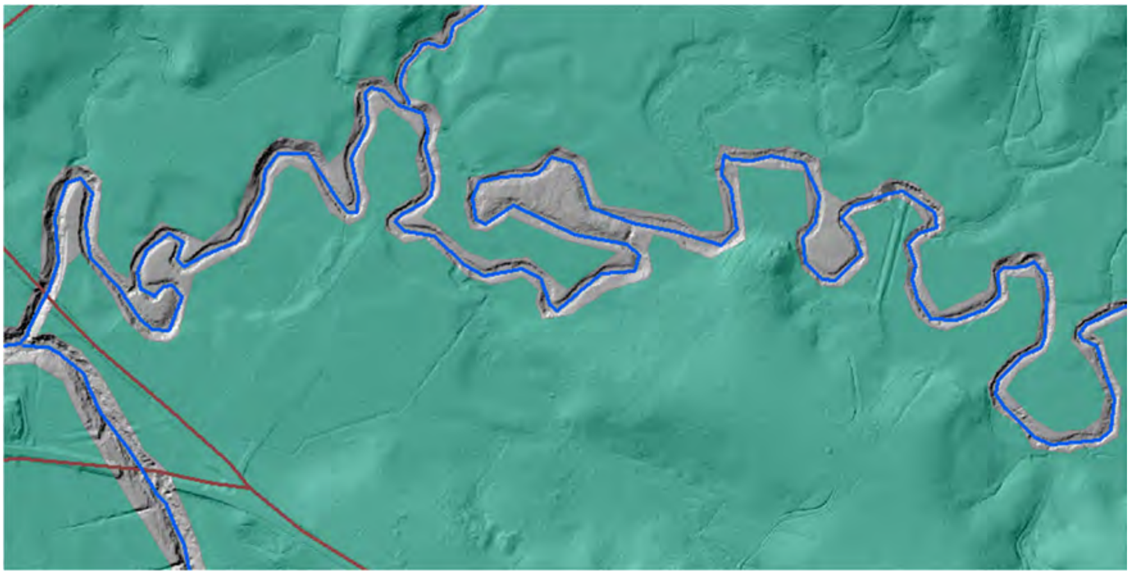


Figure 3-13: Example of sinuous branches, with the green area representing the 2D domain.

The three standard links in the model are at the downstream end of the Whangatane Channel, Awanui River and the Waipapakauri Stream where these three streams flow into the estuary, which is entirely modelled in the MIKE 21 model.

The urban and river links are based on the previous modelling by GHD, with some corrections to the connecting chainages due to changes in the MIKE 11 model. The MU-M21 inlet links, representing sumps, use an inlet area of 3.14m^2 and a maximum flow of 20m^3/s. As both of these values are high, the flow capacity will mainly be restricted by the connecting pipes or open channels in MIKE Urban. In addition, the discharge coefficient and qdh values were set to 0.67 and 0.03 respectively; this helps to better mimic the hydrodynamics of a sump when using the orifice type method. The river urban links are all “outlet” type links.

4 Calibration and Sensitivity Testing

Detail on the process and findings of the model calibration can be found in Appendix B. The following is a brief summary.

4.1 Calibration

The calibration process consisted of four parts:

1. Hydraulic calibration to the February 2007, June 2007 and January 2011 flood events, using the cut-down MIKE 11 model
2. Hydrological calibration to the January 2011 flood event using the full MIKE FLOOD model
3. Validation to the February 2007 FLOOD event using the full MIKE FLOOD model
4. Calibration of the overflow across State Highway 1 upstream of Kaitaia

The final calibration C9 using the full MIKE Flood model, showed a good match to the water level and flow at the School Cut gauge for the 2011 flood event (Figure 4-1) and a reasonable representation of the SH1 overflow, as shown in Puriri Place flows (Figure 4-2).

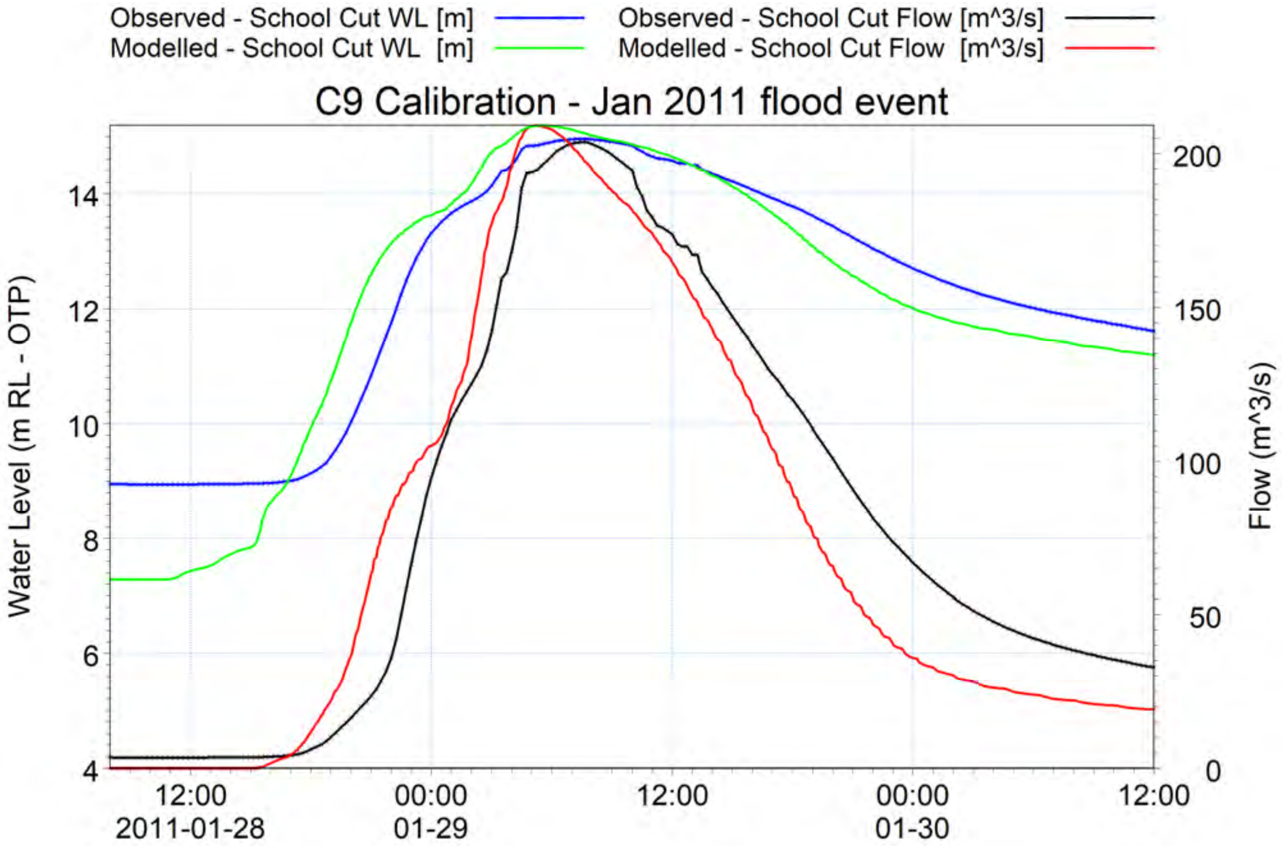


Figure 4-1: C9 Calibration at School Cut

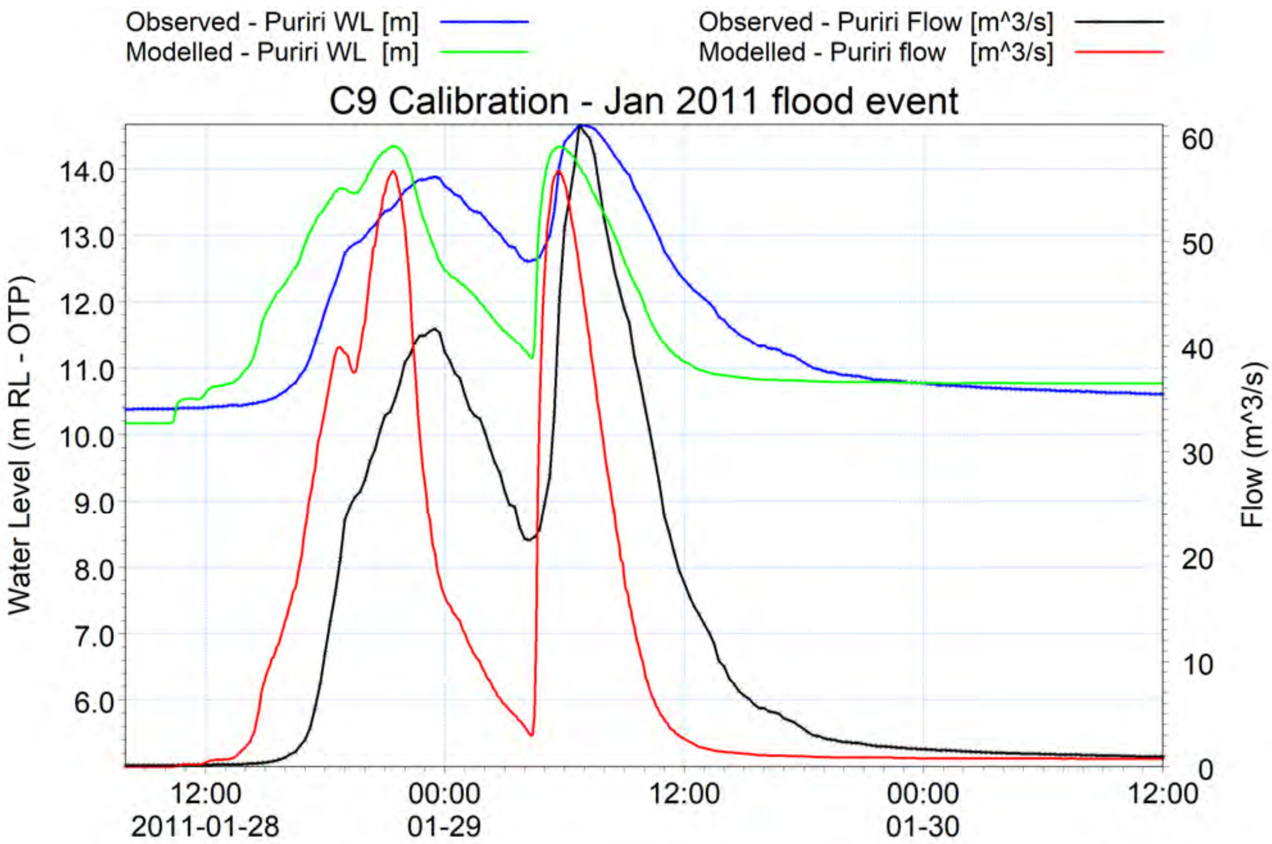


Figure 4-2: C9 Calibration at Puriri Place

4.1.1 Hydraulic calibration

Rated flows at three sites (Puriri Place, School Cut and Donald Road) and the surveyed debris levels for the February 2007, June 2007 and January 2011 flood events were used to calibrate parts of the MIKE 11 hydraulic model: Tarawhataroa Stream at Kaitaia, the Awanui River from School Cut downstream, and the Whangatane Spillway.

Tonkin & Taylor have developed a separate hydraulic model of the main Awanui River channel near Kaitaia, and have used that model to design proposed river improvements both upstream and downstream of Kaitaia. Liaison with Tonkin & Taylor ensured that calibration of the two models is comparable (although not identical: see Appendix B).

A single Manning's n value was applied at each cross-section (rather than attempting to explain differences between the three events by varying n with water level). Some compromise was needed with each flood event to obtain a reasonable representation of all three flood profiles.

From the model calibration, roughness values were derived for the main river channels at and downstream of Kaitaia (Table 4-1, lower 3 rows), for use in the Design Event scenarios. There were no calibration data for other channels, and the first 3 rows in Table 4-1 are assumed values.

Table 4-1: Design Roughness Values

Stream	Manning's n
Lower Catchment (Tidal)	0.03
Upper Catchment (Hills)	0.096
Mid Catchment (Swamp)	0.05
Whangatane Spillway	0.03 – 0.051 (average 0.038)
Awanui	0.027-0.095 (average 0.057)
Tarawhataroa	0.048-0.06 (average 0.05)

4.1.2 Hydraulic calibration: State Highway 1 overflow

This aspect of the hydraulic calibration was carried out on the full model using the January 2011 event. The School Cut flow corresponding to the threshold of overflow was represented almost perfectly without calibration; no adjustments were made to Manning's n values in the adjacent Awanui River. This indicated that the model was a good representation of reality in this area.

An accurate relationship between School Cut flow and overflow (versus Puriri Place and School Cut rated flows) was obtained by applying extremely high flow resistance to land areas north of SH1 (nearer the river) near Larmer Road. The presence of dense vegetation as viewed in aerial photography and on-site visual survey support the use of higher resistance values in this area. There is also a possibility that higher ground or banks were not picked up in the LiDAR survey due to the presence of this vegetation.

4.1.3 Hydrological calibration

Calibration of the NAM hydrological model was carried out in three stages:

Firstly, the NAM model was run over several years, to find the NAM parameters that produced the best match of runoff volume with rated flows for the largest few events.

Secondly, the time-constant NAM parameter (effectively a time of concentration) for each of the many sub-catchments was determined from two empirical formulas, before the values being adjusted *en bloc* to best fit peak flows at Kaitaia and the timing of those peak flows.

Thirdly, some further parameter adjustment was needed for the best fit of both event runoff volume and peak flow. The final "Design" values adopted for the two NAM parameters (other than the time constant) that were varied during this stage are given in Table 4-2.

The Catchment Group "Swamp" comprises sub-catchments downstream of the rated flow sites, so the NAM parameters adopted for those sub-catchments are assumed values; most have been transposed from the Te Puhi catchment group, but a lower value of Lmax (80mm) has been adopted, on the assumption that poor drainage or a high water table may restrict infiltration to ground.

Table 4-2: NAM parameters

Catchment Group	CQOF	Lmax (mm)
TePuhi	0.945	179
Takahue	0.863	0.9
Tarawhataroa	0.6	120
Swamp	0.945	80

4.2 Sensitivity testing

The calibration of the cut-down hydraulic MIKE 11 model was augmented by several sensitivity tests in which Manning's n and the stage-flow rating curve at Donald Road were separately varied. The effect of these variations on water levels and on the fraction of Awanui flow that is diverted down the Whangatane Spillway, provided an indication of the robustness of model calibration and of modelled flood levels.

Comparisons of successive runs of the full model have provided some validation of how sensitive the model is to assumptions about rainfall and runoff. Comparisons of design runs with and without climate change test how far upstream flood levels are sensitive to sea level.

The full model was also run repeatedly to calibrate overflow at SH1 from the Awanui to Tarawhataroa Stream, and successive model runs have provided validation of how sensitive the overflow rate is to local topography and vegetation.

A full list of sensitivity tests is as follows:

Table 4-3: Sensitivity Tests

	Sensitivity Test	Model Used	Conclusion
1	As part of the calibration, two of the NAM parameters were adjusted, Lmax and CQOF.	Full model	The effect of varying Lmax between run C3 and run C4 was negligible. However, the catchment quickflow is notably sensitive to CQOF. Increasing CQOF by 0.38 (for Takahue areas, between run C3 and C5) significantly increases peak runoff, with the peak flow occurring earlier. Further details and plots in Appendix B calibration section.
2	Increase downstream WLs (also represents sea level rise) The sensitivity of the river levels to higher sea levels was assessed using the Climate change scenario which uses a + 1m tide.	Full model	The higher sea level raises peak flood levels by 100mm or more in the Awanui downstream of chainage 25900m (State Highway 1) and in the Whangatane Channel downstream of chainage 8500m. These differences can be viewed by looking at the res11 results from the simulations. This is in spite of the modelled increase in runoff with climate change. For a rigorous comparison, a further model run would be needed with sea

			level rise but present-day 100-year rainfall. However, the difference is expected to be minor.
3	A standard sensitivity test, to increase the Manning's n everywhere downstream of School Cut by 20%.	Cut-down hydraulic model	Peak water levels are increased everywhere in the order of 200-500mm. The increase on the Awanui is more significant upstream of the bifurcation than downstream, because slightly more flow is diverted into the Whangatane Channel. Further details and plots in Appendix B
4	Reduce Awanui main channel Manning's n downstream of the Whangatane bifurcation by 20%, for 2km. It is an important sensitivity test, as the change upsets the balance of flows between the lower Awanui and the Whangatane Channel.	Cut-down hydraulic model	This test results in less flow to the Whangatane Diversion and hence lower water levels. The increased flow in the Awanui raises water levels from chainage 14000 (2km downstream of the bifurcation) by up to 350mm, with the paradox of increased water levels within most of the reach of reduced roughness. The implication of this is that the balance of flow between the lower Awanui and Whangatane Channel is sensitive to the state of the channel, so that channel "improvements" can have undesired and paradoxical outcomes. Further details in Appendix B
5	Increase Manning's n by 20% for the upstream 1km of Whangatane Channel	Cut-down hydraulic model	This test has the same impact as Test 4, where there is reduced flow into the Whangatane Diversion and higher water levels in the Awanui downstream of the diversion. The increase in the Awanui is around 100-200mm. Further details in Appendix B
6	Varying descriptions of the overflow at State Highway 1 The Awanui flow of 180m^3/s has been identified as the threshold of overflow over SH1. It is the amount of overflow above this Awanui flow that is of interest, and is somewhat uncertain because the Puriri Place rating relies on gaugings at relatively low flows.	**Full model**	Runs C5, C6 and C7, carried out during model development and calibration, have provided this sensitivity assessment, which is described in the Calibration – Appendix B. The overflow rating proved to be sensitive to the road height (C6 vs C5) and to the roughness of the area directly between the river and the State Highway (C7 vs C5).
7	Vary the stage-flow rating on the Whangatane Channel @ Donald Road There is some doubt about the best stage-flow rating to apply at Donald Road, and the two alternatives were both trialled. The best comparison is between runs where the model has been calibrated to one rating or the other; interest then lies in the differences not only between the resulting flow hydrographs but also	Cut-down hydraulic model	These ratings diverge significantly only above the highest flow gauging of 94 m^3/s, but result in a difference of about 13% in rated peak flow for the January 2011 event. When sensitivity tests 4 and 5 are also considered, it becomes clear that the choice of rating can have a significant effect on model calibration and/or on flooding predictions in both channels further downstream.

	between the two sets of calibrated Manning's n values.		

Sensitivity tests 1 and 3-6 in particular show that some model adjustments to model parameters have quite marked effects on river flows and water levels. Calibration of these parts of the model then needs to be done with care, but the sensitivity also indicates that anticipated changes in the catchment may have significant effects on flood levels:

Whilst catchment hydrological parameters may remain constant over time, differing rainfall patterns, which could result in a different hydrological calibration, could significantly affect flows.

Changes to flow resistance at a bifurcation or overflow are likely to affect the split of flows. This implies that care would be needed in planning any channel improvements near Waikuruke Bridge. The SH1 overflow rate (upstream of Kaitaia) is determined by the resistance to overland flow across private property, and this could be altered by the way the landowner keeps the property.

5 Design model

The Awanui MIKE FLOOD model has been run for 5 different design rainfall events and four different development scenarios, totalling 20 simulations.

5.1 Rainfall and Tide

The rainfall events simulated were: design events with ARIs of 10 years, 20 years, 50 years and 100 years, including a 100-year ARI event with climate change. The derivation of these rainfall events is explained in Appendix C.

The downstream tidal variation was provided by NRC, the timeseries is based on MfE 2007 guidance. This 2yr ARI tide is above MHWS (Mean High Water Springs) and incorporates a modest storm surge allowance. The timing of the peak tide was shifted to coincide with peak runoff reaching the estuary. Given that all three rivers peak at different times, this timing was a compromise in which the Waipapakauri Cut coincides with one high tide, the Awanui River peak flow coincides with the next high tide, and the Whangatane Spillway peaks just before the second high tide, as shown in Figure 5-1.

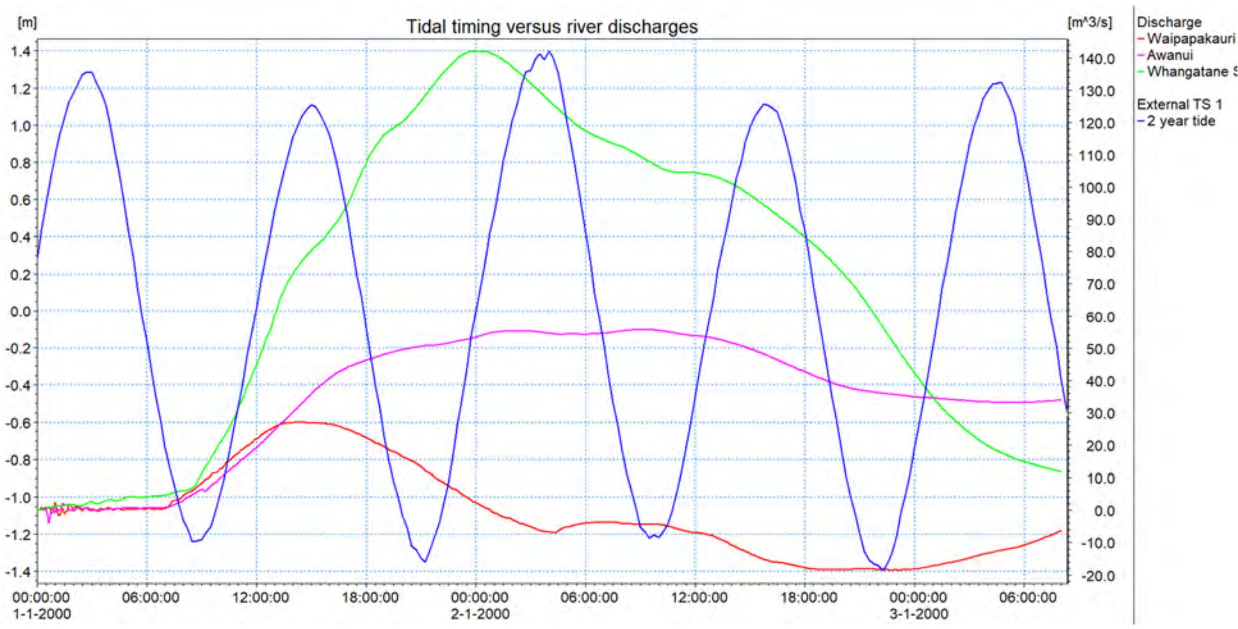


Figure 5-1: Runoff and Tide coincidence

5.2 Design Scenarios

The four design Scenarios modelled are:

Version E - 2011 Catchment condition, as used by the Calibration modelling.

Version D - Present day condition, where works have been completed along the Awanui and the inlet to the Whangatane spillway. Model updates were made to the MIKE 21 depth correction file, MIKE 11 cross sections, and the Awanui bed roughness via the cross section relative resistance.

Version F - The Scheme design, including proposed works around Kaitaia and along the Whangatane spillway designed to reduce flooding. Model updates were made to the model mesh structure, M21 depth correction file, MIKE 11 cross sections, 1D bed roughness and 1D structures.

Version G – This version has some additional modifications to the scheme design in Version F to prevent flooding in some key areas around Tarawhataroa stream and the Whangatane Spillway.

5.2.1 Version D updates

School Cut/Te Ahu Centre on the Awanui River (Figure 5-2). Left bank of MIKE 11 cross sections updated. The coloured raster represents the grid data provided by NRC representing left bank changes in the area.

Figure 5-2: School Cut/Te Ahu Centre

Bells Hill Spillway, and a new channel section between Bells Hill and Church Gully Road drain (Figure 5-3). Both MIKE 11 cross sections and the 2D surface levels were updated.

Figure 5-3: Bells Hill

Matthews Park and the Whangatane Spillway intake (Figure 5-4). These are updated based on the 2018 LiDAR dataset, to replace the earlier LiDAR survey and surveyed cross sections used in the calibration model. The low-flow section of the Awanui channel was retained from the earlier cross section survey, whereas the spillway uses LiDAR only, as this area is dry in low flows.

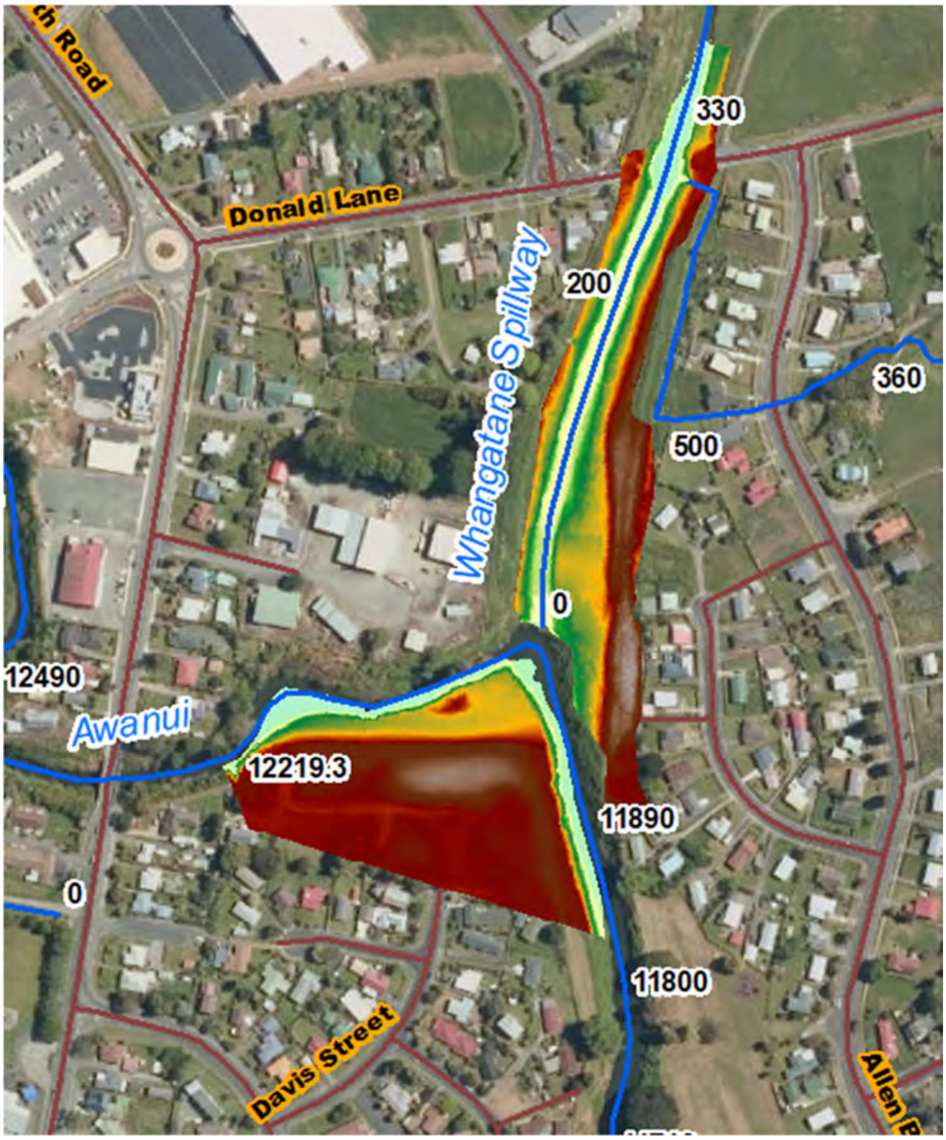


Figure 5-4: Whangatane Spillway

Gills Road floodplain and stopbanks downstream of Waihoe Channel (Figure 5-5). The full width between left and right stopbanks was included in the MIKE 11 model, as the stopbanks will not be overtopped in the flood events modelled. The cross sections were derived from 5 surveyed cross sections (shown in the figure). The data from this survey was integrated into the existing model data, via a grid format, and interpolated further downstream to ensure there would be no breaks in the right stopbank. This method also allowed the cross sections to be extracted at the same locations as in the previous model version, to maintain consistency of the MIKE 11 model grid.

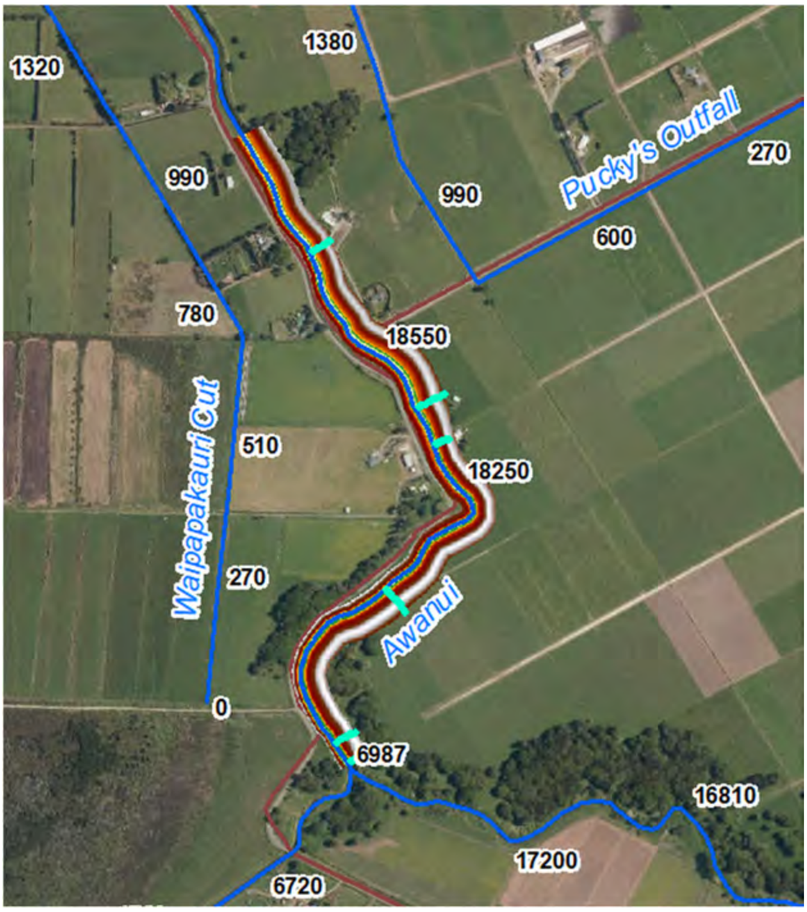


Figure 5-5: Gills Rd downstream of Waihoe Channel

5.2.2 Version F updates

Version F represents the proposed works for the Awanui River and Whangatane spillway. The terrain was provided as a 0.25m-resolution .tiff grid of ground levels all in the OTP datum. The scheme design was provided by NRC on the 19/11/2019 in a .geotiff format and an additional update including the Rugby park area was provided on the 3/12/2019. The data provided were incorporated into the model terrain or cross sections or, where necessary, added as dikes or weir structures.

Awanui upstream of Church Road:

This river reach includes a number of spillways which shorten the flow path, directing flow away from SH1. Some areas are modelled in the MIKE 11 model and others in the MIKE 21 model (Figure 5-6). Flow was modelled in 2D where there was a significant lateral component to the flow path that diverged significantly from the 1D channel assumptions, and also where the total volume would not be able to be correctly represented in the 1D model due to significant bends in the river. The mesh structure was also updated in this reach to better model the spillway hydrodynamics in 2D, by aligning the mesh with the spillway path. Note also the floodplain at chainage 9280m was lowered an additional 1m from the design surface provided as per the instruction from NRC via phone call and email on the 28/11/2019.

Two stopbanks were modelled as M21 dikes (Figure 5-6), and on the Rongopai Place branch (indicated as the small disconnected branch north of the Awanui) a weir structure was included at chainage 127m to represent the splitting of the flow path by the stopbank.

In the terrain provided by NRC, some of the floodplain overlaps directly with the open channel, creating artificially high values in the channel bed levels. As confirmed with NRC, the floodplains should not be raising the natural channel, thus cross sections in these areas were manually modified to ensure that the low spill level of the floodplain was included at the bank level but with the main channel cross section still maintaining integrity.

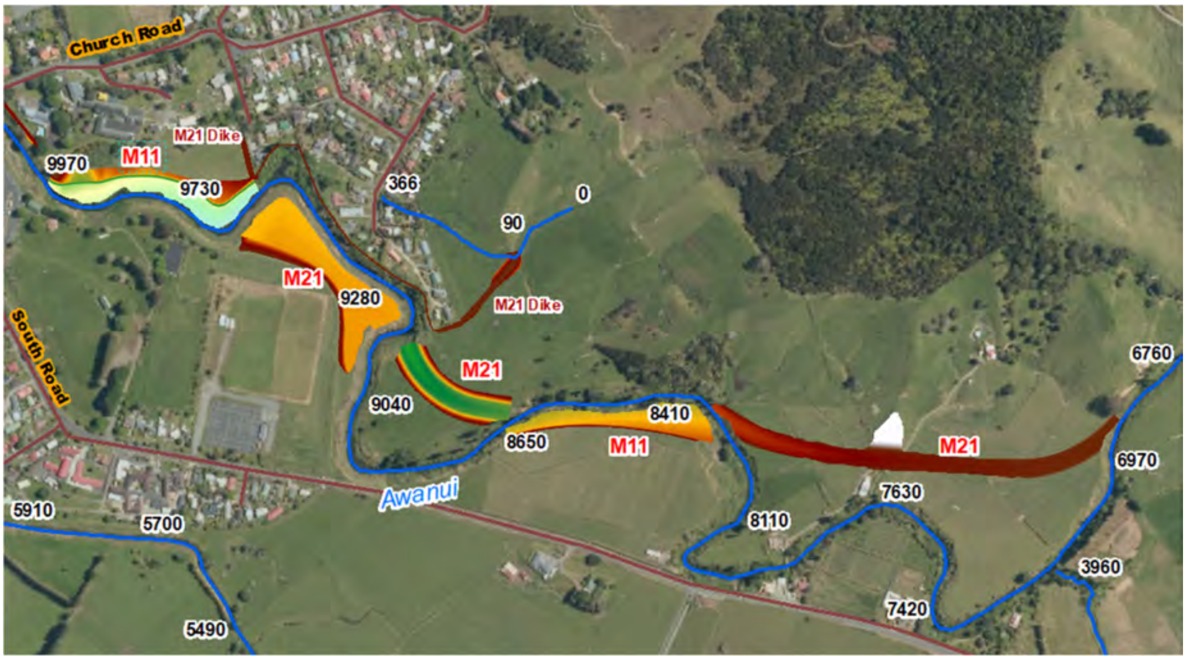


Figure 5-6: Scheme design upstream of Church Road

Allen Bell Park:

This area has a proposed modification to the right bank of the main channel (Figure 5-7), which is modelled in the 1D model, and an extended stopbank which is modelled as a dike in the MIKE 21 model.

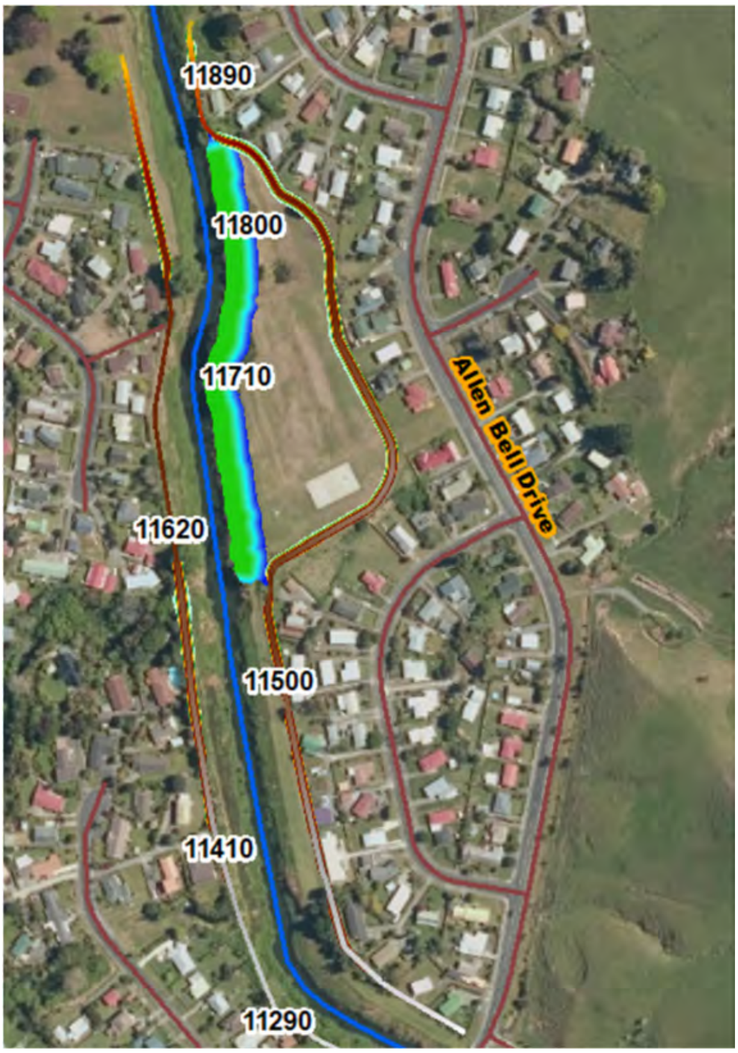


Figure 5-7: Allen Bell Park on the Awanui

Whangatane Diversion

About 900m from the upstream end of the Whangatane Spillway a channel diversion is proposed. This diversion was modelled by including a weir on the Whangatane Spillway at chainage 830m at a level of 10.75m RL, which will block the majority of the flow, while the main flow will be diverted into a new MIKE 11 channel, *Whangatane Diversion1*. The small stream (*whangatane branch9*) crossing this diversion was also adjusted in the 1D model so that it will instead connect to the new diversion channel. A one-way culvert structure was added to reduce backflow from *Whangatane Diversion1* upstream into *whangatane branch9*.

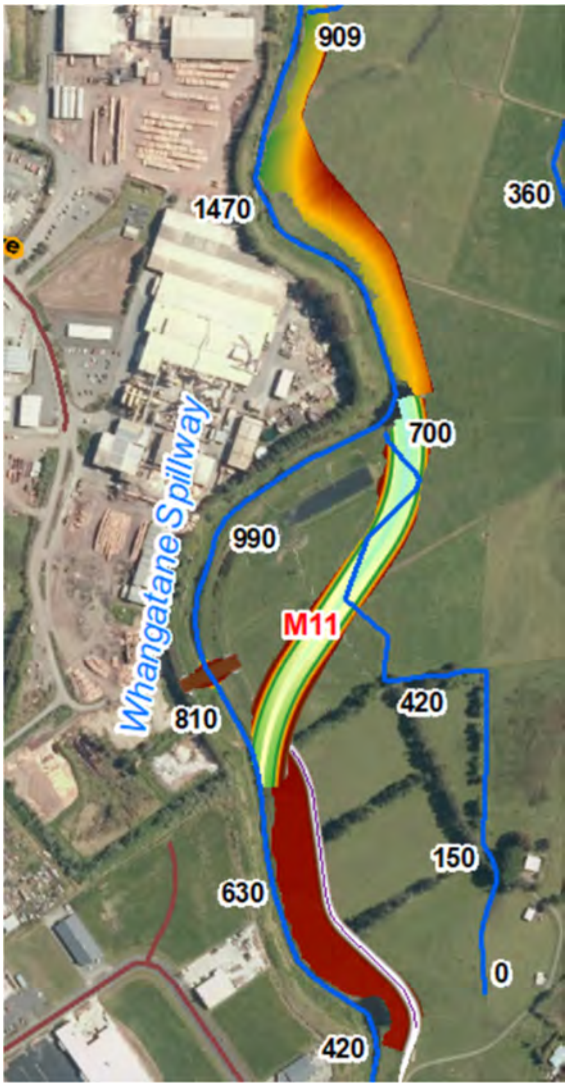


Figure 5-8: Whangatane Diversion

Benching along the Whangatane Spillway:

From Whangatane Spillway chainage 1300m down to 8700m, benching and stopbanking information was provided in the supplied terrain. These proposed changes were all modelled in the MIKE 11 model, and in some locations, where the benching resulted in a widening of the channel, the lateral links were moved out to match the new channel width. Because the lateral links act as a barrier to flow, this method ensures that there is no double counting of flow between the 1D and 2D model. An example of the benching data provided is illustrated in Figure 5-9.

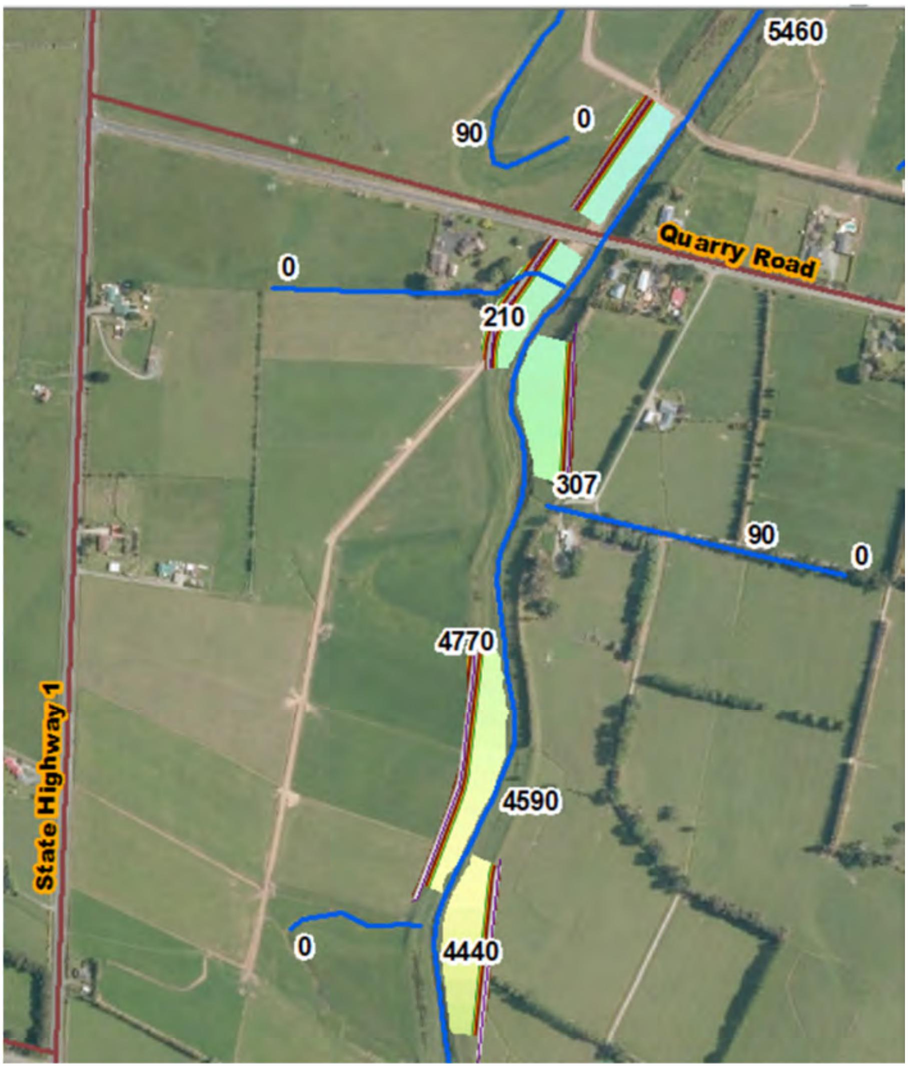


Figure 5-9: Benching along the Whangatane Spillway

5.2.3 Version G updates

The version G update is an additional update to the main scheme design and is intended as the final design. The purpose of the update was to prevent overflows in some key locations for which the scheme is intended to protect. The overflows occurred due to either LiDAR levels for the banks not being picked up well, or the existing banks being too low. Version G was run with the 100yr event with and without climate change, as the prevented overflow volumes were relatively small.

The updates consisted of works to prevent flooding from the Tarawhataroa stream between chainages 5610 to 7720, the Awanui between chainages 12219 to 12666 and the Whangatane spillway at chainage 4530-4770 (right bank). The flooding was prevented by essentially glass-walling the banks, not allowing any flow into or out of these sections of the river. The stopbanked areas are shown as red in Figure 5-10 and Figure 5-11.

In addition to the stopbanking some of the urban network was adjusted around the Northpark area, directly north of the Whangatane/Awanui confluence. Two changes were made in this area. The first was to adjust the pump station outlet into the MIKE 11 model to prevent any circular flow. The second change was to add a non-return flow on the pipe with MUID “New Culvert” which connects the North Park area to the Whangatane Spillway. The addition of the non-return regulation significantly reduces local ponding in the North Park area.

An additional version of the G model, labelled as G8 includes an additional adjustment where the right bank of the Whangatane Spillway, directly downstream of State Highway 10, was reverted back to the option D levels by using the external bank level definition option (In MIKE FLOOD). In addition, the adjacent Mangatakawere stream bank levels were adjusted slightly to better reflect the actual spill levels (downstream of chainage 8670).

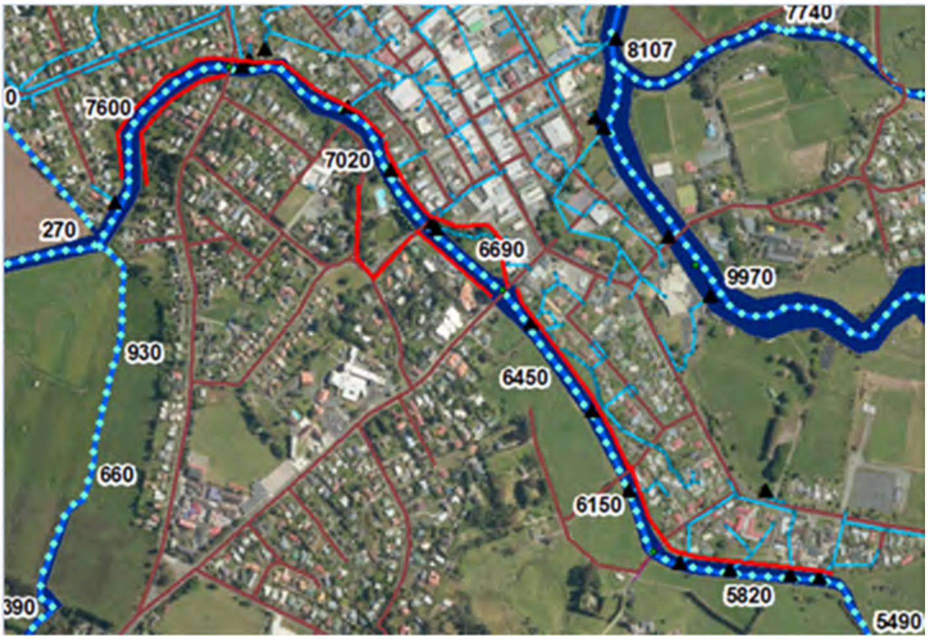


Figure 5-10: Stopbanks added to the G version model on the Tarawhataroa

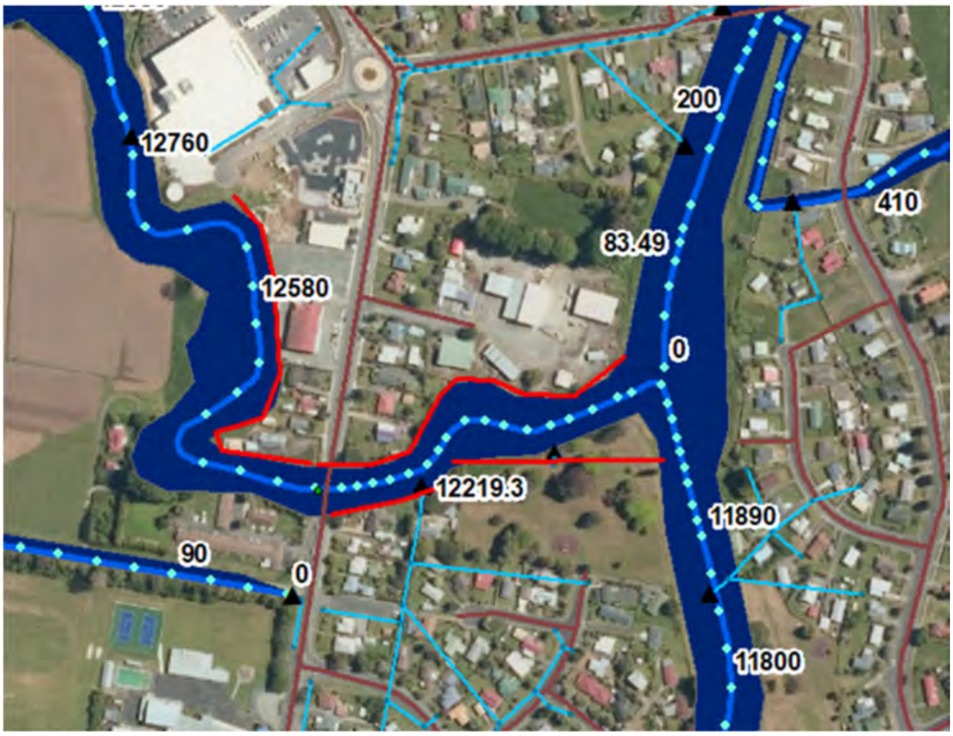


Figure 5-11: Stopbanks added to the G version model on the Awanui

5.3 Flood Mapping procedure

The model results from the 1D and 2D models were combined into a seamless 2D grid using the following procedure:

1. Created a combined terrain grid of the low flow MIKE 11 channel, the LiDAR surface and the Estuary survey (1m grid)
2. Saved the MIKE 11 results (level) as a dfs2 for each simulation (using a 2m grid)
3. Converted the MIKE 21 dfsu (mesh) results to a 2m grid (level only)
4. Merged the MIKE 11 and MIKE 21 results, using interpolation to fill in any small gaps between the two extents. If the two extents overlap then the MIKE 21 is used by preference.
5. Created the flood depth at 2m using the combined water level and the terrain generated in step 1.
6. Clipped the combined water level to the depth extent to produce a water level raster.
7. A depth filter of 0.05m was applied to clip the final flood extents. So that flooding was not shown below these depths.

This methodology results in a combined output similar to the one shown in Figure 5-12. The output formats are maximum depth and water level rasters, contained in a geodatabase which can be accessed like a folder from ArcGIS. In addition to this the velocity was also exported as velocity vectors and difference rasters calculated for specific scenarios.

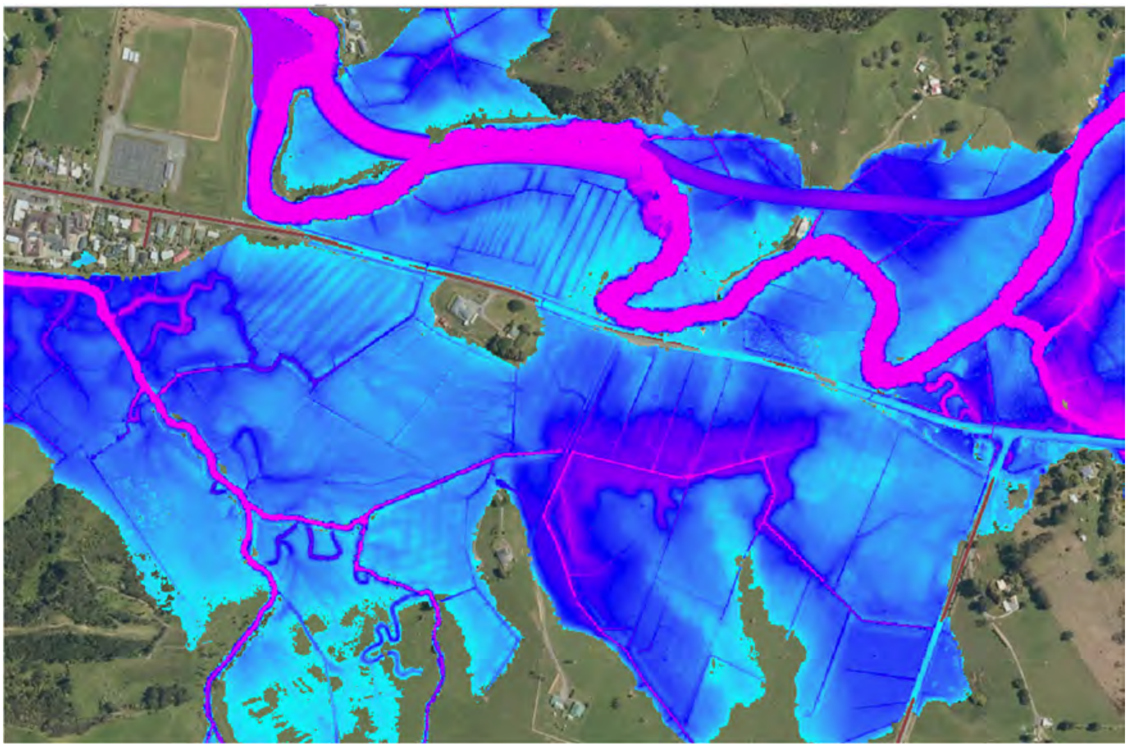


Figure 5-12: Example 1D/2D mapping results, depth in the 100yr event for version F

Legend: Depth colour gradient ranging from 0-3m, light blue to pink.

The MIKE 21 results were converted into a 2m grid, which is a finer resolution than the element areas, thus some triangle edges are seen in the final result. This could be remedied by converting to a 5m grid instead, which would further smooth the water surface. However, this technique may lose some resolution in other areas. The processing was not done using a 1m MIKE 21 grid because the file sizes were becoming prohibitively large and difficult to work with. As with most methodologies for combining 1D and 2D results, there are some small inconsistencies, where either the LiDAR has inconsistent values or the 1D mapping of the bathymetry has produced an unexpected result. These inconsistencies have been kept to a minimum by using the above methodology. As an example, inconsistencies can occur at branch connections where the water levels in the two branches do not blend together well.

5.4 Design model results

In general, the design model results appear reasonable and as expected. The scheme design in version F appears to reduce the risk of flooding to the Kaitaia township by reducing the peak SH1 overflow by 64m^3/s in the 100 year design event. Further reduction in flooding is obtained by preventing overflows from the Tarawhataroa stream in the version G model.

The following key thresholds, Table 5-1, have been identified by NRC to understand and confirm the operation of the model around Kaitaia, these are:

a. Flow bifurcation at peak flow between the lower Awanui and Whangatane Spillway.
b. The flood level at School Cut at which the SH1 overflow starts
c. The flood level at School Cut at which the Whangatane Spillway starts operating.
d. Peak SH1 overflow (m^3/s)

Table 5-1: Key thresholds

	E 100yr	F6 100yr	G8 100yr
a	Awanui = 74m^3/s, Spillway = 168m^3/s	Awanui 96m^3/s, Spillway = 225m^3/s	Awanui 92m^3/s, Spillway = 229m^3/s
b	14.3m RL	13.9m RL	13.9m RL
c	11.3m RL	9.7m RL	9.7m RL
d	236m^3/s	172 m^3/s	172 m^3/s

A table of the MIKE Flood peak flows and water levels at key locations is presented in Appendix D.

5.4.1 Validation of design flows

To assure that the design flows were within the ranges expected the design hyetographs were confirmed in discussion with NRC and only after preliminary model runs with other assumptions. The total Awanui River flow approaching Kaitaia (i.e. before any SH1 overflow) is the most important flow parameter, and successive values obtained from model runs are set out in Table 5-2. The first modelled flow of 800m^3/s appears much too high compared to other estimates to be realistic. However, the second value of 430 m^3/s is comparable to rated peak flows from the largest recorded events, and therefore appears to be too low for an ARI of 100 years.

The final choice of design rainfall pattern is based on NIWA's best estimate of a typical hyetograph. This pattern appears credible and provides an equally credible peak flow that aligns with NRC's estimated flow for river design works.

Table 5-2: Modelled 100-year ARI peak flow approaching Kaitaia

Rainfall assumptions	Peak flow (m3/s)
HIRDS version 4 rainfall depths, NRC Priority Rivers hyetograph shape	800
Rainfall depths from rain gauge sites only, HIRDS hyetograph shape	430
Adopted for Design Events: Rainfall depths from rain gauge sites only, plus 10%, HIRDS hyetograph shape	502

For comparison: Frequency Analysis (Table 2-3) (Tonkin & Taylor 2014)	440
For comparison: NRC's peak flow for design of river works, obtained from a frequency **analysis of** rated flow records	500

6 Future work and Conclusions

6.1 Conclusions:

A new numerical model of the Awanui catchment has been produced, by upgrading the earlier (2015) GHD model, including the addition of more river channels. The model, which uses DHI's MIKE FLOOD software, combines a NAM hydrological model, a 1-dimensional hydraulic model of the river and stream channels, and a 2-dimensional hydraulic model of overland floodplain flow.

The new model has been calibrated against the peak river levels at and downstream of Kaitaia in the flood event of January 2011 and validated against the July 2007 flood event. In upstream river reaches, minor channels and on the floodplains, the model provides flood flow and water level estimates that are largely uncalibrated but are very credible and consistent with anecdotal information, including aerial photos of widespread flooding of the lower catchment, confirmation from NRC on flooding across SH10 from the Whangatane Channel and flooding of the Tarawhataroa flats upstream of Kaitaia.

Design rainfall events, including an event with a 100-year Average Recurrence Interval both with and without future climate change, have been applied to the model to produce maps of peak water level. These maps have been rendered in GIS format suitable for application in NRC planning documents.

Some modelling uncertainty remains and is indeed inevitable, in particular, knowledge of rainfall events (both historical events and design events) is somewhat uncertain. In addition, the fraction of river flow that overflows State Highway 1 upstream of Kaitaia and the fraction that flows into the Whangatane Spillway both appear to be sensitive to local vegetation. Nevertheless, we consider this catchment model to provide the best description of flows and water levels that is possible with present knowledge. We consider the model to be fit for its main purposes of flood risk management and the planning and design of river improvements.

6.2 Future work

The Awanui model is expected to prove valuable beyond its present application to produce flooding maps and in testing the effectiveness of further flood mitigation measures that might be considered. These measures may well include:

- Flood detention storage in the upper catchment
- Flood detention storage in Lake Tangonge, possibly including real-time control of the Waihoe Gate
- Real-time control of flow into the Whangatane Spillway
- Further river channel improvements and/or bypass channels.

Re-analysis of rainfall records, and re-specification of design rainfall events, may be warranted once the rain radar station, now in place, has captured a reasonably long record. This would be likely to be followed by a re-calibration of the model for more accurate flood maps.

The hydrological and river channel model components in MIKE 11 are suitable to use for flood forecasting using forecast rainfall, which was a desired additional outcome of this project. This would require some additional work specifying “off-line” detention storage to replicate the floodplain storage now modelled by the MIKE 21 component.

7 References

/1/ NRC (2017) Northland Regional Council, *Hydrological and hydraulic model upgrade for the Awanui catchment flood model.* November 2017.

/2/ GHD (2013) *Awanui River Catchment Flood Model Upgrade Report*, April 2013

/3/ Tonkin & Taylor (2014) “*Awanui Flood Scheme Preliminary Design*” Report, T&T Ref. 29154.100 September 2014

/4/ DHI (2017) *MIKE 11: A Modelling System for Rivers and Channels: Reference Manual*.

/5/ Nielsen, S.A. and Hansen, E. (1973) *Numerical Simulation of the Rainfall Runoff Process on a Daily Basis*. Nordic Hydrology, 4, 171-190.

/6/ Sutherland-Stacey, L., G. Austin, J. Nicol and B. Reboredo (2018) “*Potential for Use of Rainfall Radar in Northland*”, report for Northland Regional Council, Weather Radar New Zealand Ltd.

APPENDICES

APPENDIX A – Model Review

Awanui Model Review Report

Technical Memo

To:	Toby Kay
Cc:	Dragan Tutulic; Joe Camuso
From:	Nancy Zhang, Antoinette Tan, Dragan Tutulić and Graham Macky
Date:	1st August 2018
Subject:	Review and Quality Assessment of Existing Awanui Model

1 Introduction

This project comprises the upgrade of an existing model rather than creating a new one, and it is important to understand the existing model and to be aware of any aspects that could be improved. It is therefore required to carry out a review of the present hydrological and hydraulic models to assess their adequacy for the upgrade that this project will provide.

We have looked at the models' performance by running one design scenario, i.e. 100-year ARI, with the existing model setup, using the 2016 Service Pack 3 (SP3) version of MIKE FLOOD. As indicated in our proposal, we have carried out our usual performance checks of a model review: output flow and velocity hydrographs, mass balance, numerical instabilities, Froude Numbers, and the link flows between the MIKE 11, MIKE 21 and MIKE Urban models.

We have also reviewed the models' components to assess how well catchment and channel properties have been represented.

In reviewing the hydraulic model, we have checked channel roughness (Manning's n values) and considered whether channels are realistically represented by the choice of cross-section locations.

We have also looked at the linking between the MIKE 21 domain and the two 1D networks: the MIKE 11 channel and that part of the MIKE URBAN network which is to be retained. We have checked that the channel area represented by the MIKE 11 model is fully blocked out from the MIKE 21 domain without leaving areas that are not modelled at all.

This technical memo outlines the status of the 2015 Awanui MIKE FLOOD model last used and modified by GHD.

2 Modelling software

The Awanui model has been developed using version (v) 2014 SP2. There are two main components of the Awanui model; the hydrology (i.e. rainfall –runoff) and the hydraulics (i.e. channel flow, pipe flow and overland flow).

2.1 Hydrology

For the Awanui model, MIKE URBAN Model B hydrological model is used to determine the stormwater runoff from sub-catchment. Urban Runoff Model B of MIKE11 Rainfall Runoff (RR) module has been used to determine runoff from sub-catchments represented in MIKE 11 river model. Details on the hydrological models adopted are discussed in Section 4.2 and 4.4.

Note that the catchment geometry is not included in the urban sub catchments, which will make it difficult to adjust the existing catchment delineation, unless shapefiles of the catchment delineation are made available.

2.2 Hydraulics

The hydraulic model of the Awanui River Catchment was developed using a 3-way coupled MIKE FLOOD model incorporating MIKE 11 (1-Dimensional river model), MIKE 21 (2-Dimensional overland flow model) and MIKE URBAN (1-Dimensional reticulation pipe model). All three models were dynamically linked using MIKE FLOOD.

2.3 Software versions adopted within model

The MIKE software is updated from time to time with new service packs and occasionally a new version of software. These updates generally solve small errors or add a new feature to the software.

Table 2-2 outlines the software used for the Awanui model development and the newest software now available.

Table 2-2 Software Versions

Model Component	Software version used	Newest software available
MIKE ZERO (MIKE 11, MIKE 21 and MIKE FLOOD)	v.2014 SP2	v.2017 SP2
MIKE URBAN	v.2014 SP2	v.2017 SP2

3 Model coverage

The Awanui River Catchment covers a total land area of 45,509 hectares (455 km^2) and has its headwaters located to in the South East of the catchment. The four major upstream rivers namely Takahue River, Victoria River, Karemuhako River and Tarawhataroa Stream meet to become the Awanui River. The Awanui River then passes through the alluvial foothills and down to the lowlands ultimately discharging into the Rangaunu Harbour near Ben Gunn.

In low and moderate flows all stormwater discharges to the Rangaunu Harbour at the Awanui mouth, with no flow passing to adjacent catchments. However, in high flows Awanui River water is diverted into the artificial Whangatane Channel, and in the largest flood flows water can overtop the Awanui stopbanks to enter Waipapakauri Stream and other minor drains. All these channels flow into Rangaunu Harbour.

The catchment boundary is shown in Figure 3-1; the Awanui MIKE 11 and MIKE 21 model extents are shown in Figure 3-2.

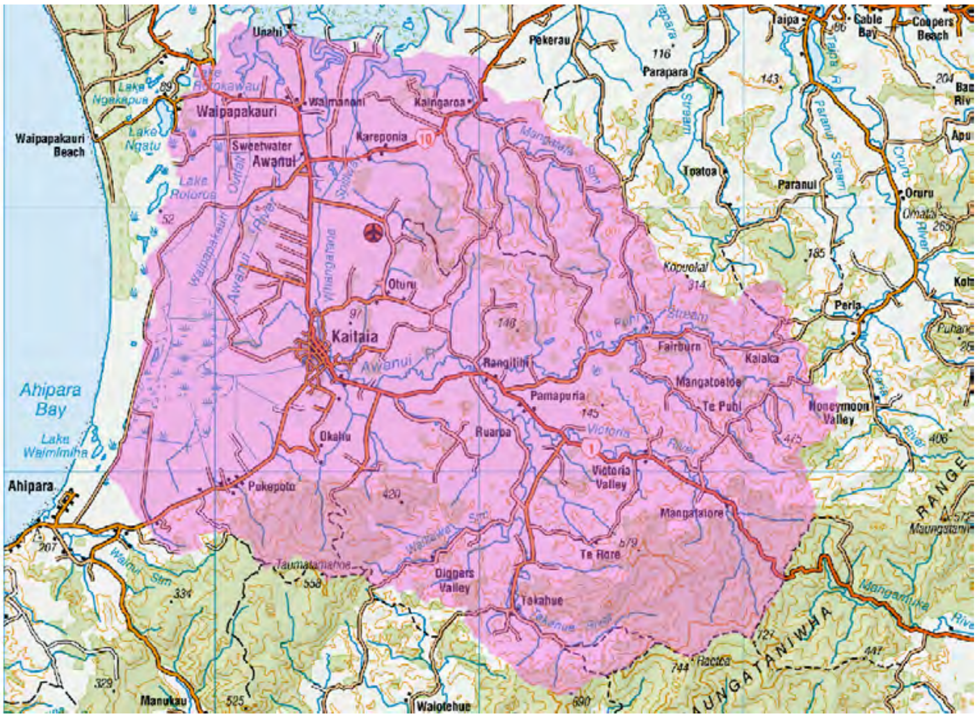


Figure 3-1 Awanui catchment

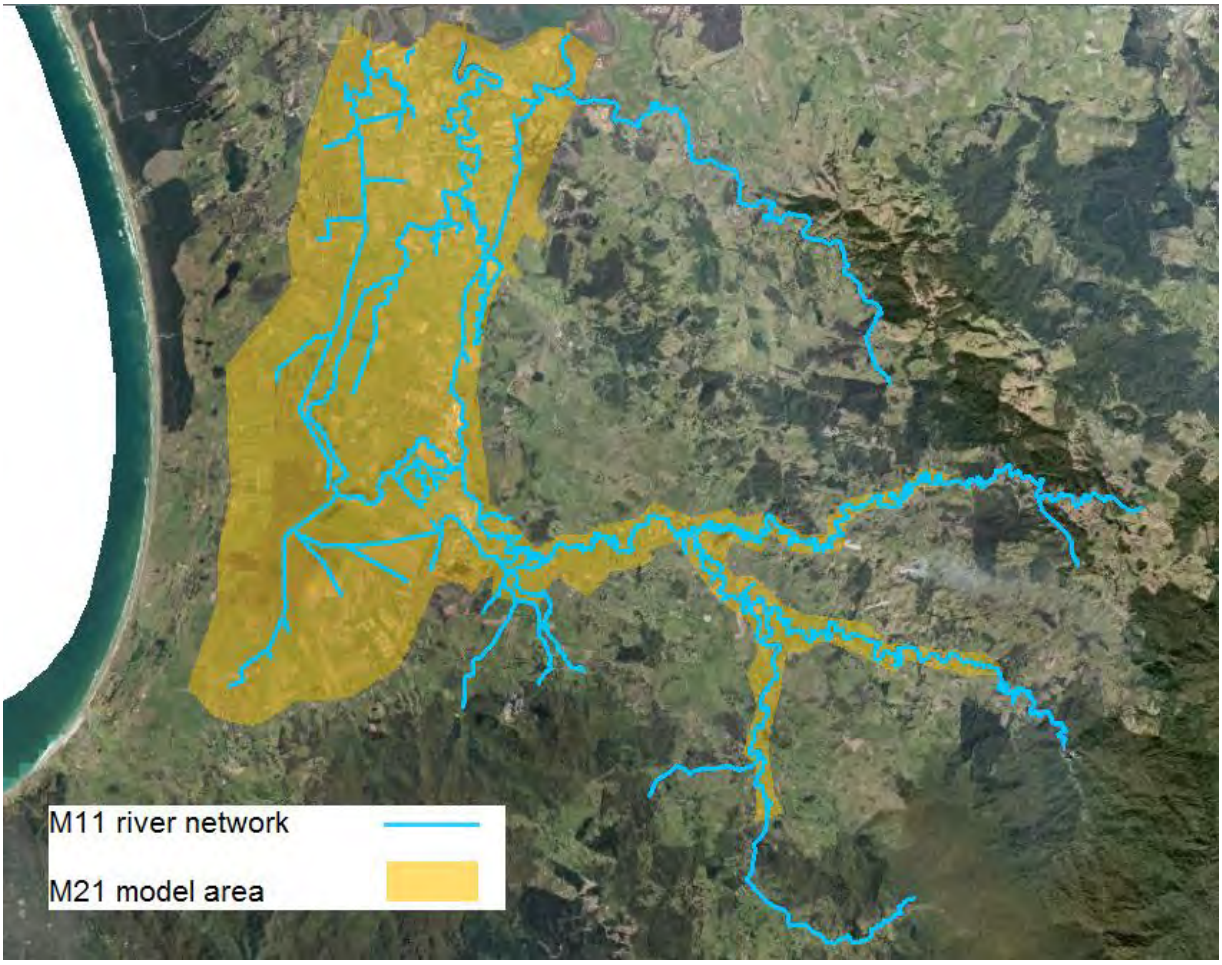


Figure 3-2 MIKE 11 and MIKE 21 Model Extents

3.1 MIKE 11

The Awanui MIKE 11 model is made up of 131 branches in total; 40 of which are Linked channels. All the major rivers namely Awanui, Takahue, Victoria, Karemuhako, Tarawhataroa, Whangatane, Mangatete, and Waipapakauri Cut and their tributaries are modelled.

3.2 MIKE 21

The MIKE 21 component of the Awanui model is made up of an **8m x 8m quadrilateral flexible mesh** covering an area of 21,120 m by 20,688 m.

The MIKE 21 component covers all the lower catchment and encompasses all the branches within the network. However, the headwaters of the Tarawhataroa River, Takahue River, Karemuhako River and Mangatete River, mostly steep terrain in bush, are not included.

3.3 Co-ordinate reference system

The model has been developed in the New Zealand Transverse Mercator (NZGD2000) coordinate reference system.

4 Model parameters

The key parameters used within each model component are discussed in the following sections. These parameters are critical to the model behaviour and results.

4.1 MIKE 11 river hydraulics

The MIKE 11 hydraulics component is made up from 4 separate files;

- Network File (.nwk11);
- Cross Section File (.xns11);
- HD Parameters File (.hd11); and and
- Boundary Data File (.bnd11).

The remainder of this section outlines the inputs and parameters specified for all 4 files.

4.1.1 Network file

A short summary of the network is provided in Table 4-1 below. The network is made up of 131 branches in total; 90 branches are “Regular” branches with cross sections at specified chainages; the remaining 41 branches are “Link channel” branches which do not require cross sections to be specified. More details about findings on the link channels in Awanui model will be discussed in Section 4.1.1.2.

Table 4-1 Network file breakdown

Input/parameter			**Number**
Network	Branches	No.	131
	Points	No.	12,960
Structures	Weirs	No.	1
	Culverts	No.	33
	Bridges	No.	34
	Control Structures	No.	0
	Energy Losses	No.	0

4.1.1.1 Points

All points within the network have been specified with x and y coordinates. The chainage types adopted within the model are a mix of both “user and system defined”.

4.1.1.2 Branches

91 “Regular” branches are used for the natural rivers within the Awanui model network. This is the normal branch type used within MIKE 11 and is composed of a number of calculation points defined by cross sections specified in a cross-section file.

40 “Link Channel” branches are used to represent the geometry of higher grounds (e.g. roads) between adjacent waterways and floodplains within the Awanui model network. Link channels do not require cross sections to be specified and are consequently simpler to use than regular channels. The link is modelled as a single structure branch of only three computation calculation points (h-Q-h).

Note that it is not common practice to use link channels between rivers and floodplains while also using MIKE FLOOD lateral and standard link coupling at the same location. Using both together may produce unexpected results and potential double counting of floodplain volume and conveyance. It is recommended that these link channels are assessed further and most likely removed when updating the model.

4.1.1.3 Weirs

The weir specified within the model (a dummy weir located on Tarawhataroa_1 branch) has been modelled using the Broad Crested Weir formulation, which calculates a Q/h relationship based on the levels and widths specified for each weir.

The weir does not have any valve regulation; therefore, depending on up and down stream conditions, flow may travel in either direction.

The head loss factors adopted for the weir are all default values.

4.1.1.4 Culverts

A range of culvert geometries have been used within the Awanui model. They are all “Regular” structures; which means flow passes through the culvert based on up and down stream flow conditions.

Valve regulation is set as “only positive flow” for 25 culverts (understood to be fitted with flap-gates) and “none” (where flow is permitted in both directions) for the other 8 culverts.

Manning’s n varies from 0.013 up to 0.03 for the culverts specified.

The head loss factors adopted for the culverts are all default values.

4.1.1.5 Bridges

34 bridge structures have been modelled for the Awanui catchment.

All bridges have been modelled using the Federal Highway Administration (FHWA) WSPRO method. This method considers contraction loss by the calculation of an effective length and expansion loss via experimentally based tables.

If the bridge is submerged, the model uses the Pressure Flow FHWA method. Two different equations are used for this method depending on whether the downstream orifice is partially or fully submerged.

If overflow occurs, the behaviour is modelled using the FHWA method. This method models overflow using a weir equation taking tail water submergence into account.

Note that three bridges (Clough Road Bridge, Unnamed Road2 Bridge, Unnamed Road3 Bridge) have a shift in the x geometry data – which causes errors when running the model with newer versions of the software. Modifications have been made (i.e. shifting the entire bridge cross sections to the left by subtracting the start X values from each X value) to the model simulated using the MIKE release 2016 SP3.

4.1.2 Cross sections

For every cross section, the following information is required:

- River (branch) name;
- Chainage;
- Cross Section ID (optional);
- Topo ID;
- Section Type;
- Radius Type;
- Coordinates (optional); and,
- Resistance.

Cross sections are defined for every regular branch. The Topo ID's and key cross section properties have been specified below.

4.1.2.1 TopoID and Cross section ID

There are a range of different Topo ID's used within the Awanui model, with additional Topo ID's present for all the rivers (especially the main rivers) within the cross-section database. No specific information about the Topo ID's has been found.

4.1.2.2 Cross section properties

Spot checks have been carried out to summarize the cross-section properties.

The section type of the cross sections used within the model is set to be open (for open channels).

The radius type of the cross sections is set to Total Area, Hydraulic Radius, which means that the total area is equal to the physical cross-sectional area.

Resistance numbers of the cross sections are set to be using the distributed distribution and a relative resistance approach. Higher values of relative resistance are adopted for high flow zones; a relative resistance value of 1 is adopted for low flow zone.

Note that for some cross sections on Whangatane_Branch5/6/7/8, a relative resistance value of 0.01 has been most likely mistakenly adopted.

4.1.3 HD parameters

The Hydrodynamics (HD) editor is used for setting supplementary data used for the model runs. Most of the parameters are set default values upon creating a new HD Parameters file (.hd11). For many parameters global values are used for the entire model, with local values specified in particular locations if required.

The following information is included within the HD Parameters file:

- Default Computational Parameters;
- Initial Conditions;
- Wind Factors;
- Bed Resistance;
- Wave Approximation;
- Quasi Steady Parameters;
- Heat Exchange Parameters;
- Stratification Parameters;
- Groundwater Leakage;
- Flood Plain Resistance; and,
- Encroachment Simulations.

Some of these parameters are summarised in the remainder of this section. Default settings have been adopted for other parameters not mentioned.

4.1.3.1 Default computational parameters

These parameters are essential for the computational scheme. Table 4-2 outlines the values adopted within the model.

Table 4-2 Default computational parameters

Parameter	Value
Delta	0.85*
Delhs	0.01
Delh	0.1
Alpha	1
Theta	1
Eps	0.0001
Dh Node	0.01
Zeta Min	0.1
Struc Fac	0

Inter1Max	10
NoIter	1
MaxIterSteady	200*
FroudeMax	-1
FroudeExp	-1

Note: * indicates non-default values

Default values have generally been used for most computational parameters. The exceptions to this are Delta, and MaxIterSteady.

Delta has a value of 0.85 (default value is 0.5). Delta changes the time-cantering of the gravity term in the momentum equation; a high value of Delta (with a maximum of 1.0) has a dampening effect which can significantly influence model dynamics, whilst the lowest value (0.5) will produce the most accurate calculations (provided the model is stable). The MIKE 11 user manual recommends a Delta of 0.85 for MIKE FLOOD models with small time steps.

MaxIterSteady has a value of 200 (default value of 100). This factor defines the maximum number of iterations used to obtain a steady state water level profile at the start of a simulation.

4.1.3.2 Initial conditions

Initial water depths and discharge have been set globally as 0 meter and 0 m^3/s, respectively, for the simulation.

4.1.3.3 Bed resistance

Table 4-3 summarises the global bed resistance settings for the MIKE 11 component. A global Manning's number of 0.05 has been set for the network. This value is adjusted within individual cross sections using a relative resistance approach (see Section 4.1.2.2) which scales this value up or down. 272 Local values of Manning's n between 0.013 and 0.12, which supersedes the global value, have been set within the model, located in various branches including the Awanui, Tarawhataroa, Mangatete, Karemuhako branches etc.

Table 4-3 Bed resistance

Parameter	Value
Approach	Uniform Section
Resistance Formula	Manning (n)
Resistance Number	0.05
Bed Resistance Toolbox	Not used

The Manning's n values used by GHD in their model for those locations we visited during our site visit in April 2018 were reviewed more closely.

Some of the values will have been determined as part of model calibration. The calibration proposed in the present study will also involve adjusting Manning's n values to match flow and stage records. We have therefore taken a broad-brush approach at this stage:

- assessing Manning's n from site photographs, and
- identifying those locations where the calibrated values in the GHD model differ from these by more than 40%.

It may turn out that somewhat unrealistic Manning's n values still must be assumed to get a well-calibrated model. However, the assessment below provides a starting point for examining the reliability of the flow and stage data used for model calibration, and for considering whether the calibration process can be improved, to modify the more unrealistic assumptions of Manning's n whilst still achieving reasonable agreement between modelled and gauged water levels and flow rates.

Takahue River

In two parts of the river, in the headwaters of the Awanui catchment, GHD assign a Manning's n value of 0.11, which in our assessment is probably too high.

Figure 4-1 Takahue_1 2800m – 4100m

Figure 4-2 Takahue ~320m

Tarawhataroa

GHD assigned the Tarawhataroa a Manning's n value of 0.013 where it flows past Kaitaia town, where our immediate estimate is 0.055 – 0.07. This value is implausibly low and must be presumed to have been obtained by model calibration. The channel is generally straight with a generally uniform cross-section. The small n value therefore is likely to reflect an over-estimate of flow rates during the calibration events.

Figure 4-3 Tarawhataroa 3736m

Figure 4-4 Tarawhataroa 4232m

Awanui River

A little distance upstream of Kaitaia, one bank is heavily vegetated, the other in long grass. Our rough assessment of Manning's n is 0.06 whereas the model has n=0.10. For comparison, Manning's n on the Awanui at School Cut, the same location and then quite heavily vegetated on both sides, was measured by Hicks & Mason as 0.06.

Figure 4-5 Awanui River within Kaiataia 9329m

In contrast, a reach upstream and at Church Road was assigned n values of 0.026. It is in dense grass with some bushes and trees, and our assessed Manning's n is 0.05.

Figure 4-6 Awanui River 11254m

At and downstream of the SH1 bridge at the northern end of Kaitaia, the model assigns an average n of 0.06, whereas our estimate is 0.09. It is difficult to estimate conveyance in this reach ("the Throat") and both channels resistance and effective cross-section area are likely to change over time with channel maintenance.

This reach is just downstream of the Whangatane Channel diversion and may have required some calibration against estimated flows in both the river and the Whangatane Channel.

Figure 4-7 Awanui River: looking downstream to SH1 12368m

Figure 4-8 Awanui River from SH1 12418m

Downstream from this reach (12700m to 13900m) the channel appears more open (our estimate n=0.07), but GHD assigned n=0.12.

Figure 4-9 Awanui River 12760m

Further downstream, NRC has widened the channel and replaced rampant shrubs and trees with new grass. GHD's model assigns n=0.08, which will be applicable to past data, but for modelling future scenarios the new cross-sections will be needed and n~0.035 is suggested.

Figure 4-10 Awanui River 19798m

Whangatane Channel

From its intake to Donald Road, the channel now has a rip-rap rock bed and sides in rough grass. It has been quite overgrown in the past. A further complication is that at the intake the cross-section has been altered, to divert more flow when most needed.

Downstream of Donald Road, the channel is grassed.

GHD's model assigns n=0.025 to this reach. This value appears too low either for the present condition or the past. It is understood that this model value was determined during calibration. Manning's n of about 0.4 appears likely.

Figure 4-11 Whangatane Channel, upstream from Donald Rd 255m

Figure 4-12 Whangatane Channel, downstream from Donald Rd 275m

4.1.3.4 Wave approximation

The wave approximation method has been set to the Fully Dynamic approximation for the entire network.

4.1.4 Boundary data

Boundary conditions for the MIKE 11 component are held in a .bnd11 file.

A total of 68 boundary conditions have been specified as follows:

- Open water level boundaries: A 2-year tidal boundary condition has been adopted at the downstream ends of the Awanui, Waipapakauri_cut, Waipapakauri_Branch11 and WHANGATANE_SPILLWAY channels where they flow into the Rangaunu Bay. A constant water level of 14.78 meters has been adopted at the downstream end of the RongopaiPlace branch.
- Open inflow: Non-zero constant inflow (from 0.01m^3/s to 1.5 m^3/s) boundaries have been adopted at the upstream end of 8 branches; no specific information about the derivation of these values has been found. The remaining 55 open inflow boundaries have a constant inflow value of 1e-005 so that the branches do not dry out.

In summary, the model consists of 2 types of downstream water level boundaries and various upstream inflow values.

4.2 MIKE 11 rainfall runoff

Hydrology for the sub-catchments discharging into MIKE 11 branches and onto MIKE 21 is modelled using the Rainfall Runoff component (RR) of MIKE 11. The approach adopted is Urban Runoff Model B. This approach is founded on the non-linear reservoir with kinematic wave routing.

The area's, lengths, slopes and percentage of pervious surfaces vary among 283 catchments being delineated. Different Model B parameter values are used for each catchment.

Infiltration to groundwater is calculated using a modified Horton Equation.

Runoff results of 235 catchments are input as lateral inflows to the MIKE 11 hydrodynamic module by setting up rainfall-runoff links. Runoff results of 48 catchments are input as source points to the MIKE 21 hydrodynamic module.

4.3 MIKE 21

The Awanui MIKE 21 model uses a 2-dimensional flexible mesh engine to model the floodplain for the Awanui catchment. It considers the topography of the land and determines overland flow paths and water depths during flood events.

The key parameters within the MIKE 21 model are presented in Table 4-4.

Table 4-4 MIKE 21 parameters

Parameter	Values
Module Selection	Hydrodynamic
Orientation of Grid	North (0°)
Map Projection	NZGD_2000_New_Zealand_Transverse_Mercaor
Grid Size	8 m
Grid Dimensions	21,120m * 20,688m
Surface Elevations	Unahi datum (0.186 m below the One Tree Point (1964) datum.)
Using depth correction	Yes
Initial Condition	Constant – 0 water level, 0 velocity
Resistance	From file
Infiltration	No infiltration
Eddy Viscosity	Constant Eddy Formulation, constant value 1.2 m^2/s
Start Type	Cold Start

Time Step	0.5 second (min. 0.3 second; max. 0.5 second)
Solution techniques	Lower order, fast algorithm
Critical CFL number	0.95
Depth correction	From file
Flood and dry type	Advanced flood and dry (floodplain)
Drying depth	0.01m
Flooding depth	0.03m
Wetting depth	0.1m
Boundary	Specified tidal boundary from file at the downstream end of the Awanui catchment; land boundary for all other locations.
Modelling Duration	5 days
Inland flooding applied	yes

4.4 MIKE URBAN

The Awanui MIKE URBAN one-dimensional model is used to model the stormwater pipe network within Kaitaia Township.

A summary of model components of the Awanui MIKE is given in Table 4-5 below.

Table 4-5 MIKE URBAN components

Item	Number
Manholes	1342
Basins	0
Outlets	95
Storage Nodes	0
Circular Pipes	974
Rectangular pipes	1
CRS defined pipes	359
Pumps	1 (Pump_Northpark, pumping from M21. Start/stop level 10.0m/9.5m. constant flow 0.5m3/s)
Controlled Pumps	0
Weirs/Orifices	22

Controlled Weirs/Gates	0
Valves	0
Controlled Valves	0

4.5 MIKE URBAN rainfall runoff

Hydrology for the sub-catchments within the urban zones is modelled using MOUSE kinematic wave (B) hydrology model of MIKE URBAN.

522 sub catchments are modelled. The area's and X/Y coordinates, lengths vary among these catchments; different sets of Model B parameters are adopted to the sub catchments. Note that the geometry is not included in urban catchments, i.e. may affect the accuracy of rainfall runoff calculation.

All these sub catchments are connected to a nearby node so that computed storm water runoff is input to the MIKE URBAN pipe flow network.

4.6 MIKE FLOOD

The three model components are coupled using MIKE FLOOD, allowing the dynamic exchange of water between the 1D engines (MIKE 11 and MIKE URBAN) and the 2D engine (MIKE 21) to predict the flooding during various storm events.

Lateral links have been used within the Awanui model to link the MIKE 11 and MIKE 21 model components. Table 4-7 summarises the parameters adopted for lateral links.

Table 4-7 Lateral link parameters

Lateral Link Parameters	
Parameter	**Value**
Number of links	224
Coupling Type	HD Only
Method	Cell to cell
Structure Type	Weir 1
Source	HGH or M21
Depth tolerance	0.1 (0.2 for RongopaiPlace branch)
Weir coefficient	1.838
Manning's n	0.05

Spot checks on the linking between the MIKE 21 domain and the MIKE 11 river network has been carried out by plotting the lateral link locations, the link connections and comparing the position of Marker 1 and Marker 3 for all the H-points with MIKE 21 grid. It was found that the 1D channel width and the 2D blocked-out width matched up reasonably well. However, some other issues were found with the lateral linking that can be improved:

- The block-out in the 2D domain has gaps where the stream is narrow. This causes the left and right bank linking to overlap, which can create errors in the model results.
- Some skewing is occurring in the lateral links, and the lateral links are crossing over the MIKE 11 structures at some locations. This will cause inaccuracies in the model results and an underestimation of head loss at the structures.

90 River/Urban links have been used to link the MIKE URBAN outlets and MIKE 11 branches. The coupling type is “MIKE URBAN Outlet to MIKE 11” for all the River/Urban links.

Urban links have been used to connect the MIKE URBAN nodes and one or more MIKE 21 cells. Table 4-8 summarises the parameters adopted for urban links.

Table 4-8 Urban link parameters

Urban Link Parameters	
Parameter	**Value**
Number of links	1211
Coupling Type	Mostly M21 to inlet; M21 to outlet for 10 links
Max flow	10 or 20
Inlet area	Varies from 0.84 to 64 m^2
Inlet method	Orifice equation
Discharge coefficient	0.98
QdH factor	0 (0.2 for node KT_SWP1074)

5 Model calibration

The January 2011 event has been used for the calibration of the Awanui model. The differences between model results and measured data at river gauge locations are quantified. The model was also calibrated against observed debris levels and bank spilling. The calibration results were discussed and agreed by NRC.

Please note that the calibration assessment was based on the provided GHD report. However, as mentioned earlier, there is a concern that the report doesn’t apply to the actual model provided.

6 Results analysis

A test run of The Awanui model with MIKE software release 2016 SP3 and basic performance checks have been carried out. Details are discussed in the sections below.

6.1 Mass balance

An overall continuity balance check has been conducted. Table 6-1 summarises the detailed information of the continuity balance check.

Table 6-1 Continuity balance check details

Item	Value (m^3)		
MIKE 21			
A: Initial volume in model area			491965.95
Final volume in wet area			16978692.31
Final volume in dry area			6748.36
B: Final volume in model area			16985440.67
X: Source inflow	1517031.36		
MIKE 11 inflow target	2634866825		
MIKE 11 inflow correction	-2.89		
MIKE Urban inflow target	27919207.05		
MIKE Urban inflow correction	0		
Source outflow		0	
MIKE 11 outflow target		2620297747	
MIKE 11 outflow correction		-13777.65	
MIKE Urban outflow target		29773975.48	
MIKE Urban outflow correction		-2251792.73	
C: Total volume from source			15738264.52
D: Total volume from precipitation/evaporation			0
E: Total volume from boundaries			755210.14

MIKE 11			
F: Initial volume in model area			0
G: Final volume in model area			3050206.91
MIKE 21 lateral inflow	2620297770		
MIKE 21 boundary inflow	0		
MIKE URBAN CS lateral inflow	3511513.16		
MIKE URBAN CS boundary inflow	12519.61		
Y: Lateral sources inflow	0		
Lateral correction	362.47		
Z: Open boundaries inflow	27104732.2		
W: Rainfall runoff inflow	48285054.7		
H: Total inflow			2699211952
MIKE 21 lateral outflow		2634866960	
MIKE 21 boundary outflow		0	
MIKE URBAN CS lateral outflow		792856.52	
MIKE URBAN CS boundary outflow		55936.41	
Lateral sinks outflow		0	
U: Open boundaries outflow		62320966.46	
I: Total outflow			2698036746
MIKE URBAN			
J: Start volume in Pipes, Manholes and Structures			5.7
K: End volume in Pipes, Manholes and Structures			5732.1
L: Total inflow volume			
M: Specified inflows			
V: Rainfall runoff:	602608		

N: Non-specified inflows			
Outlets (inflow):	807250.2		
Inflow from 2D overland:	29775174.7		
Total	31185032.9	-->	31185032.9
O: Total diverted volume			
Volume not possible to extract:	-449.3		
P: Operational, non-specified outflows			
Outlets:	3453863.2		
Weirs:	0		
Pumps:	0		
Flow to 2D overland:	27730916.4		
Total	31184330.3	-->	31184330.3
Q: Water generated in empty parts of the system			36751.5
R: Total change of volume= (A-B)+(G-F)+()K-J)= 19549408 m^3			
S: Total inflows + runoff= X+E+Y+Z+W+V= 78264636 m^3			
T: Total outflows= U= 62320966 m^3			
Overall balance=R-S+T= 3605738.09 m^3/s			

A deviation in volume balance is shown in the above table (4.6%), i.e. Overall balance compared to the total inflow + rainfall. This may be caused by many factors, e.g. different bed levels of rivers meeting at junctions (which has been found at various locations in MIKE 11 network file); lateral extraction of water from the MIKE 11 river network to MIKE 21 dry cells, etc. Note that the MIKE URBAN outflow correction value is very high, hence further assessment when updating the model is strongly recommended.

6.2 Numerical instabilities

Spot stability checks on MIKE 11 flow and velocity result hydrographs have been carried out. The model is generally stable except for a few branches, such as Waihoe_1 and Tarawhataroa. An example of instabilities on Tarawhataroa branch is shown in Fig. 6-1.

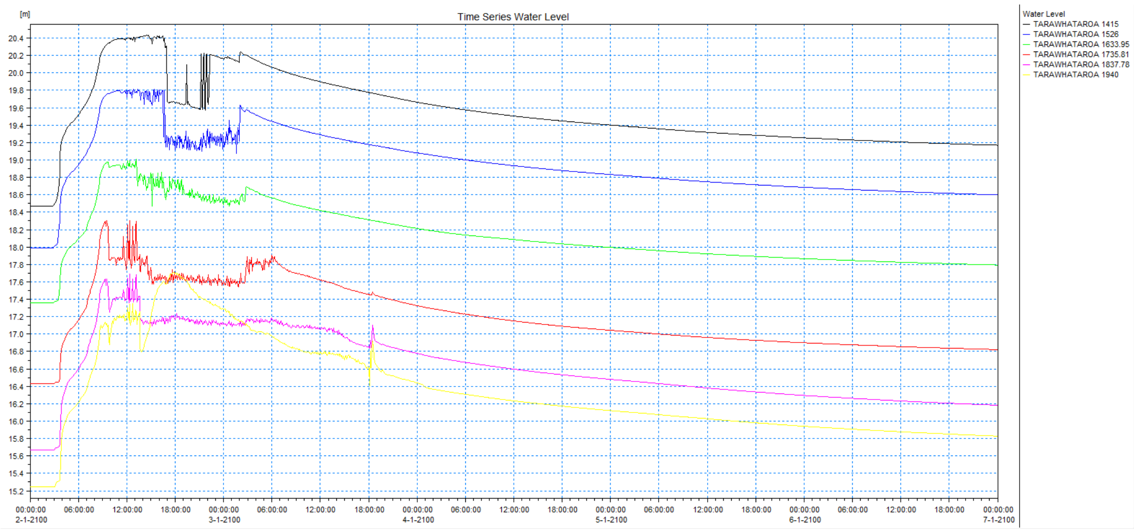
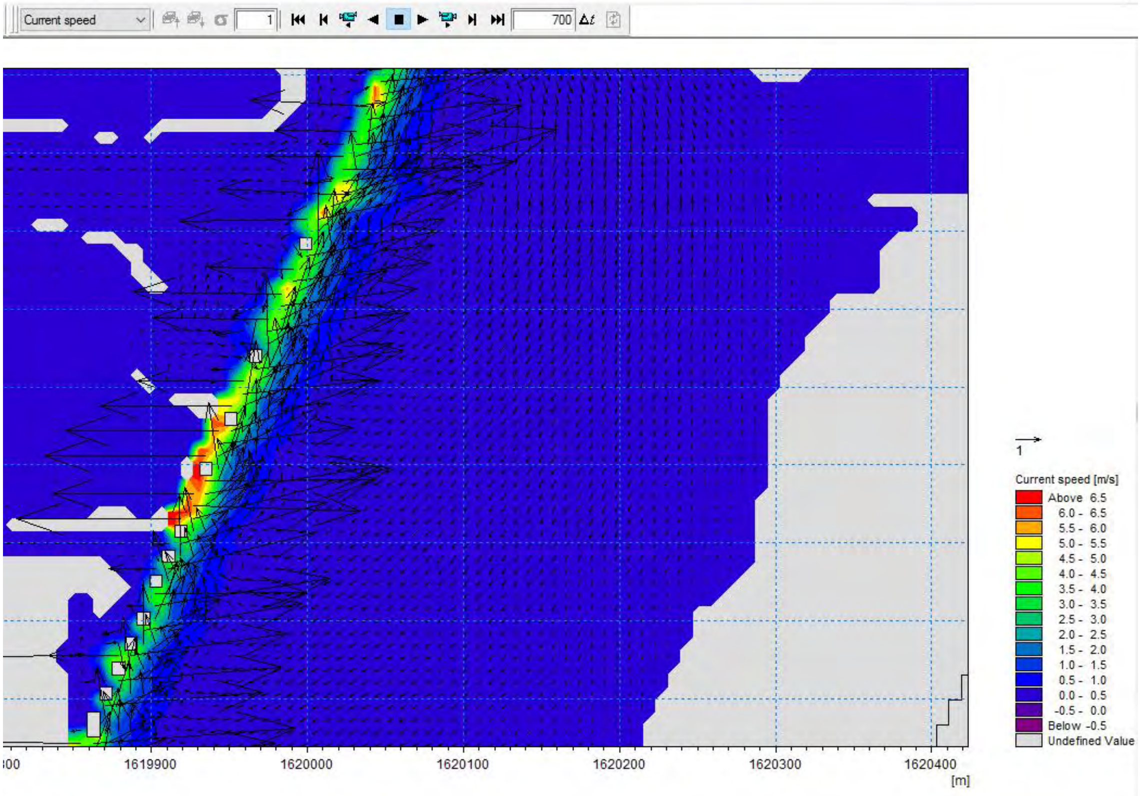


Figure 6-1 Instabilities on Tarawhataroa branch

The MIKE 21 water level, depth and velocity results have been checked. They are generally stable, but show questionable results at the locations where the 2D block-out has gaps. Fig.6-2 shows an example of abnormal velocity result.



7 Conclusions and Recommendations

Existing model runs reasonably well, but mass balance is tolerable rather than ideal, and there are some parameters and model details that ought to be corrected for more accurate and more stable model runs.

The following actions are recommended to be addressed during the model update.:

1. Further assess the geometry of the MIKE URBAN sub-catchments to revise the catchment parameters in Model B;
2. Further assess the Link channels in the MIKE 11 setup; alternatively, these should be removed, and the floodplain included in the MIKE 21 model;
3. Update the relative resistance values in the model, as part of the calibration process;
4. Fix the skewness occurring in the lateral links; and
5. Lower the values of Critical CFL number and wetting depth in MIKE 21 settings to more reasonable values.

APPENDIX B–Calibration

Calibration and Sensitivity

Calibration of Awanui catchment numerical model

Contents

Hydraulic Calibration of river channel downstream of School Cut ... 3

Awanui River and Whangatane Channel ... 3

Flow hydrographs at Donald Road ... 5

Flow-stage rating ... 8

Peak water level profiles ... 8

Tarawhataroa Stream ... 14

Flow-stage rating ... 14

Peak water level profiles ... 15

Calibration outcome: proposed Manning's n ... 16

Hydraulic calibration: State Highway 1 overflow ... 18

Attempted hydraulic calibration of upper Tarawhataroa Stream ... 22

Hydrological Calibration of the full model upstream of School Cut ... 24

Sensitivity to 2 NAM hydrological parameters ... 28

Validation against February 2007 event ... 29

Calibration: Conclusions ... 31

Sensitivity tests ... 33

Increase in downstream (Rangaunu Harbour) sea levels ... 34

Variation in flow resistance (Manning's n): Tests 3-5 ... 36

Rating curve uncertainty (Sensitivity Test 7) ... 38

Figure 1 Rated flows at School Cut and Donald Road compared ... 4
Figure 2 Modelled flows at School Cut and Donald Road compared ... 4
Figure 3 Rated and modelled flow hydrographs at Whangatane Channel @ Donald Road, February 2007 Also shown: Rated flow at School Cut, and Awanui flow downstream of bifurcation ... 5
Figure 4 Rated and modelled flow hydrographs at Whangatane Channel @ Donald Road, July 2007 Also shown: Rated flow at School Cut, and Awanui flow downstream of bifurcation ... 6
Figure 5 Rated and modelled flow hydrographs at Whangatane Channel @ Donald Road, January 2011 Rated flow at School Cut shown for comparison ... 7
Figure 6 Whangatane Channel @Donald Road (chainage 295m): Stage-flow rating ... 8
Figure 7 Peak observed and modelled water levels, February 2007 ... 9
Figure 8 Peak observed and modelled water levels, July 2007 ... 9
Figure 9 Peak observed and modelled water levels, January 2011 ... 10
Figure 10 Peak observed and modelled water levels, all 3 events ... 10
Figure 11 Peak observed and modelled water levels, February 2007 ... 11
Figure 12 Peak observed and modelled water levels, February 2007 ... 11
Figure 13 Peak observed and modelled water levels, July 2007 ... 12

Figure 14 Peak observed and modelled water levels, January 2011 ... 12
Figure 15 Peak observed and modelled water levels, all 3 events ... 13
Figure 16 Tarawhataroa @ Puriri Place (chainage 6006m): Stage-flow rating ... 14
Figure 17 Peak observed and modelled water levels, all 3 events ... 15
Figure 18 Awanui River Manning's n: Calibration values from each event and proposed values ... 16
Figure 19 Whangatane Channel Manning's n: Calibration values from each event and proposed values 17
Figure 20 Tarawhataroa Stream Manning's n: Calibration values from each event and proposed values 17
Figure 21 "Rating" between Awanui flow at School Cut and overflow over State Highway 1 ... 19
Figure 22 Modelled peak overland flow, State Highway 1 upstream of Kaitaia, Run C5. River flow is from right to left. ... 20
Figure 23 Modelled peak overland flow, State Highway 1 near Larmers Rd, Run C7. River flow is from right to left. ... 21
Figure 24 Modelled flow resistance at the SH1 overflow, Calibration run C7. The legend specifies M=1/Mannings n ... 22
Figure 25 Flow rates at Tarawhataroa @ Puriri Place, January 2011: rated and modelled (runs C7 and C9) 23
Figure 26 Hydrological areas for NAM modelling ... 24
Figure 27 Rated flow approaching Kaitaia compared with successive model runs ... 26
Figure 28 Awanui River @ School Cut: rated and modelled flows, January 2011 ... 27
Figure 29 Tarawhataroa @ Puriri Place: rated and modelled flows, January 2011 ... 27
Figure 30 February 2007 event: Combined flow hydrograph, Awanui @ School Cut + Tarawhataroa @ Puriri Place. ... 30
Figure 31 Effect of climate change including sea level on peak water levels in lower Awanui River, 100-year flood ... 35
Figure 32 Effect of climate change including sea level on peak water levels in lower Whangatane Channel, 100-year flood ... 35
Figure 34 Awanui River peak water level differences, January 2011, Sensitivity tests 3-5 ... 36
Figure 35 Whangatane Channel peak water level differences, January 2011, Sensitivity tests 3-5 37
Figure 36 Flow hydrographs downstream of the Whangatane bifurcation, January 2011 event, sensitivity tests 3-5. The higher flow peaks are in the Whangatane Channel, the lower flow peaks in the Awanui River. ... 37
Figure 37 Alternative ratings for Whangatane Channel at Donald Road. Rating A was adopted for model calibration. ... 38
Figure 38 Modelled and rated hydrographs, January 2011, Whangatane @ Donald Rd, alternative ratings 39
Figure 39 Manning's n values calibrated for Rating A (Run 190207h) and Rating B (Run 190403y) 40

Hydraulic Calibration of river channel downstream of School Cut

Calibration of the MIKE 11 river channel model was originally intended to be against two events in 2007. NRC subsequently added the event in January 2011, so that three separate calibrations were effected for 3 events in February 2007, July 2007 and January 2011.

To speed the process, a “cut-down” model was created comprising just the Awanui River channel downstream of School Cut, the Whangatane Channel, and Tarawhataroa Stream downstream of the State Highway 1 overflow. Consultation with NRC had established that the only significant tributary to these channels, Church Road Gully, does not contribute to peak Awanui flows at all.

It was hoped that it proves appropriate to average these calibration results to obtain a single description of the channels’ flow resistance for use in modelling the design scenarios. It was also intended that, following liaison with Tonkin & Taylor, this flow resistance would be similar or identical to the calibration they obtain for their numerical model covering much of the same river network.

Calibration has in the end been carried out solely by adjusting Manning’s n. This approach means that the adjustments to n values not only represent variations in channel roughness but are also a surrogate for any variations from the modelled cross-sections in the effective channel area. At an early stage of the calibration, point head losses were inserted near the bifurcation. However, their effect was minor, and for simplicity point losses were therefore not included in the model.

Awanui River and Whangatane Channel

The hydraulic calibration has included the Awanui River downstream of School Cut, and the Whangatane Channel. The data available for calibration was:

- Rated flow and gauged water level at Whangatane Channel @ Donald Road
- Peak water levels inferred from bankside debris in both channels.

The upstream boundary condition was the rated flow at School Cut, and the downstream boundary condition for both channels was the tidal water level in Rangaunu Harbour.

Before proceeding to the calibration model runs, the rated flows at Awanui @ School Cut and Whangatane @ Donald Road were compared for all three events (Figure 1). As agreed with NRC in January 2019, the rating for Donald Road shown here, and adopted in this study, is the earlier of two alternative ratings which differ in the extrapolation above the largest gauged flow of about 90 m^3/s (Rating A in Sensitivity Test 7, see below). Above about 120 m^3/s School Cut flow (and 70 m^3/s in the Whangatane Channel) the relationship between the sites varies for the 3 events: proportionally less flow appears to have been diverted down the Whangatane Channel in the July 2007 event. This small difference is in spite of known debris accumulation in the Awanui River near Waikuruke Bridge in the July 2007 event.

Figure 1 therefore provides a reminder that the Manning’s n values that make up the model calibration can be expected to vary from time to time as the condition of both channels changes.

It was agreed with NRC that the peak of the rated Donald Road flow record in July 2007 is suspect, due to flow reaching the bridge soffit. Figure 1 shows the modelled flows for this hydrograph peak, which have been assumed to approximate the true flows.

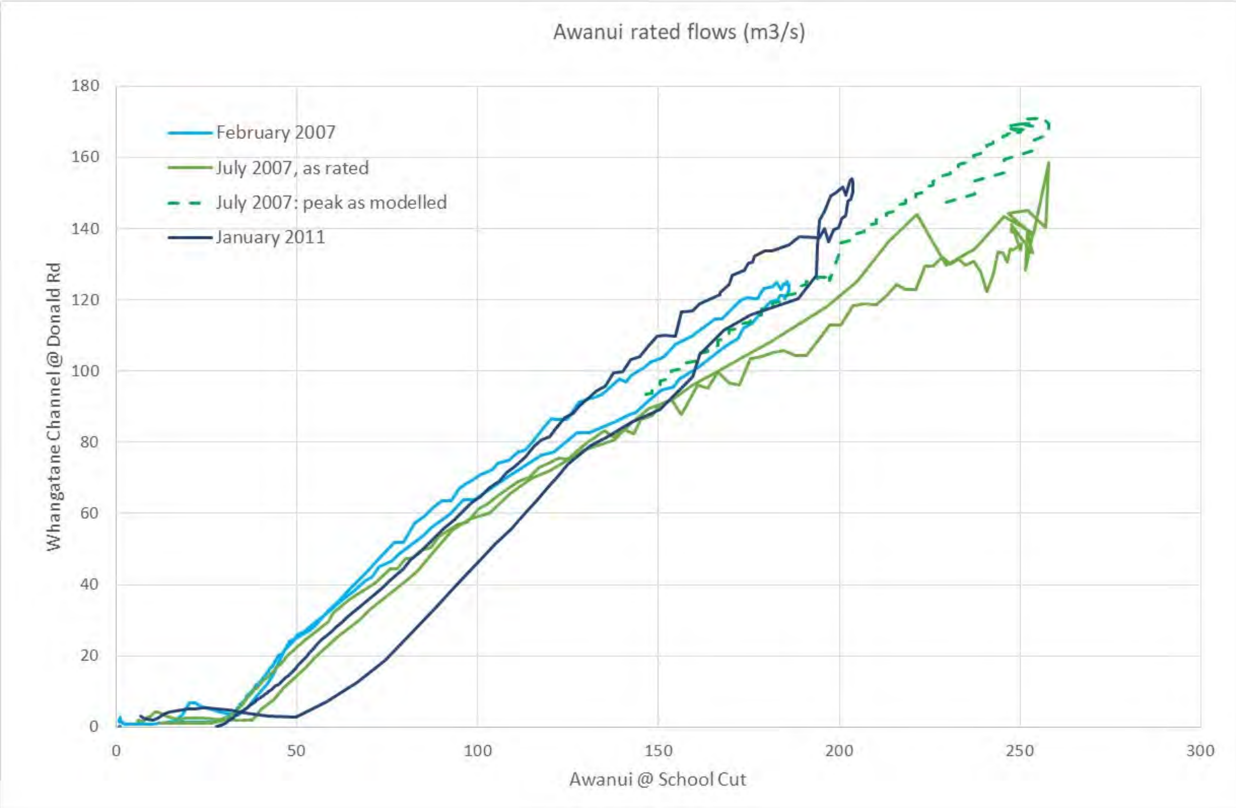


Figure 1 Rated flows at School Cut and Donald Road compared

For comparison, the equivalent modelled flows are shown in Figure 1Figure 2. This graph shows that any looped rating effects are minor.

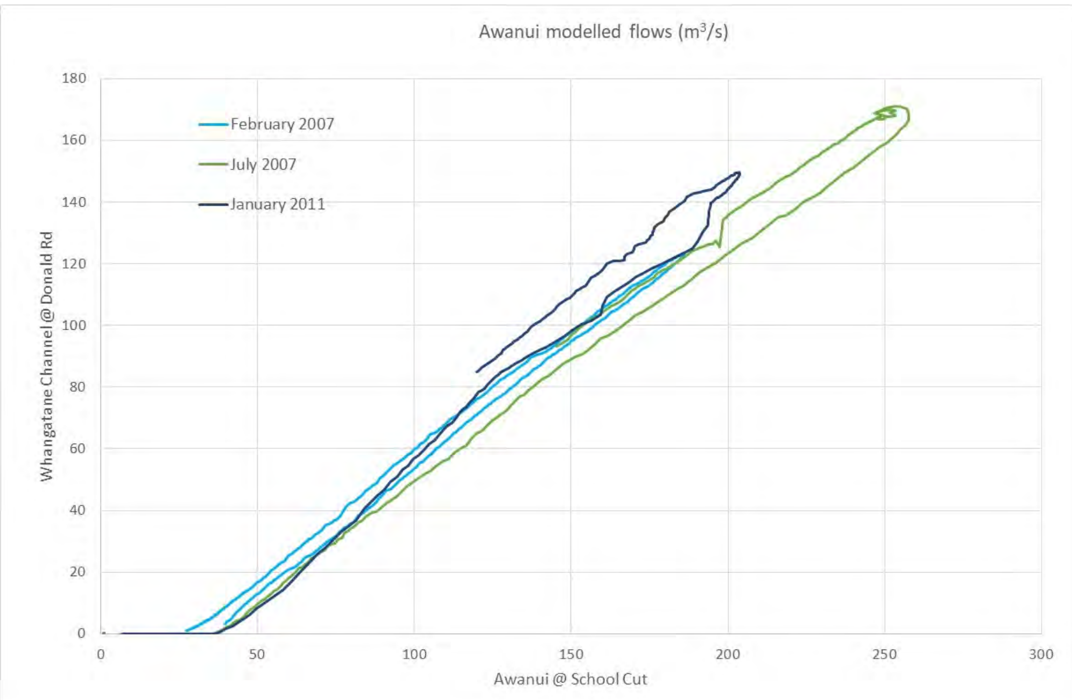


Figure 2 Modelled flows at School Cut and Donald Road compared

Calibrating this part of the model has required many iterations. It is important to replicate the flow rate at Donald Road as well as possible. This requires adjusting flow resistance immediately downstream of the bifurcation in both channels to get the flow split as accurate as practicable. Only then can the flow resistance further downstream be adjusted to match peak modelled flow with measured debris levels.

It has been assumed that any form losses at or near the inlet to the Whangatane Spillway are minor and can be accommodated by the choice of Manning's n. Should evidence become available that these had losses are significant, a revision of the calibration including a head loss at a point would be warranted, although the overall effect on modelled water levels ought to be minor.

Flow hydrographs at Donald Road

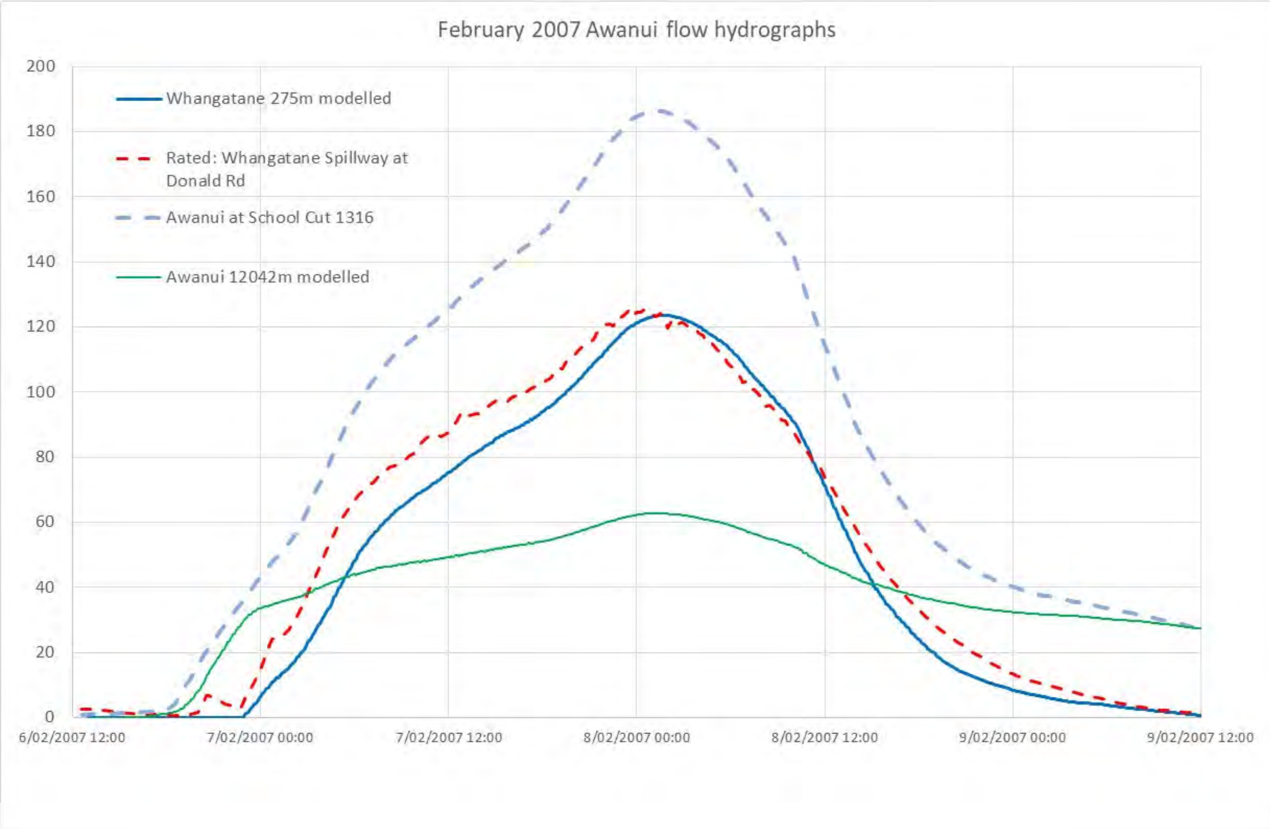


Figure 3 Rated and modelled flow hydrographs at Whangatane Channel @ Donald Road, February 2007
Also shown: Rated flow at School Cut, and Awanui flow downstream of bifurcation.

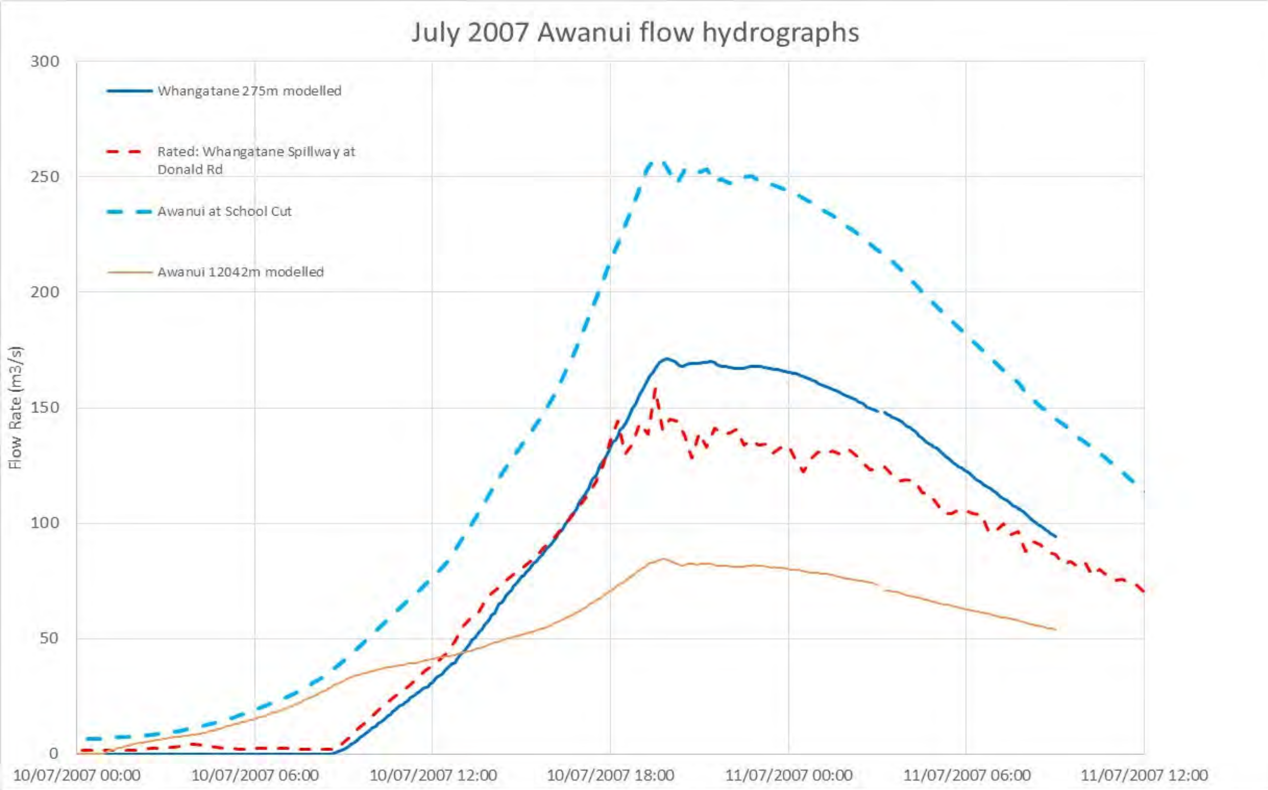


Figure 4 Rated and modelled flow hydrographs at Whangatane Channel @ Donald Road, July 2007
Also shown: Rated flow at School Cut, and Awanui flow downstream of bifurcation

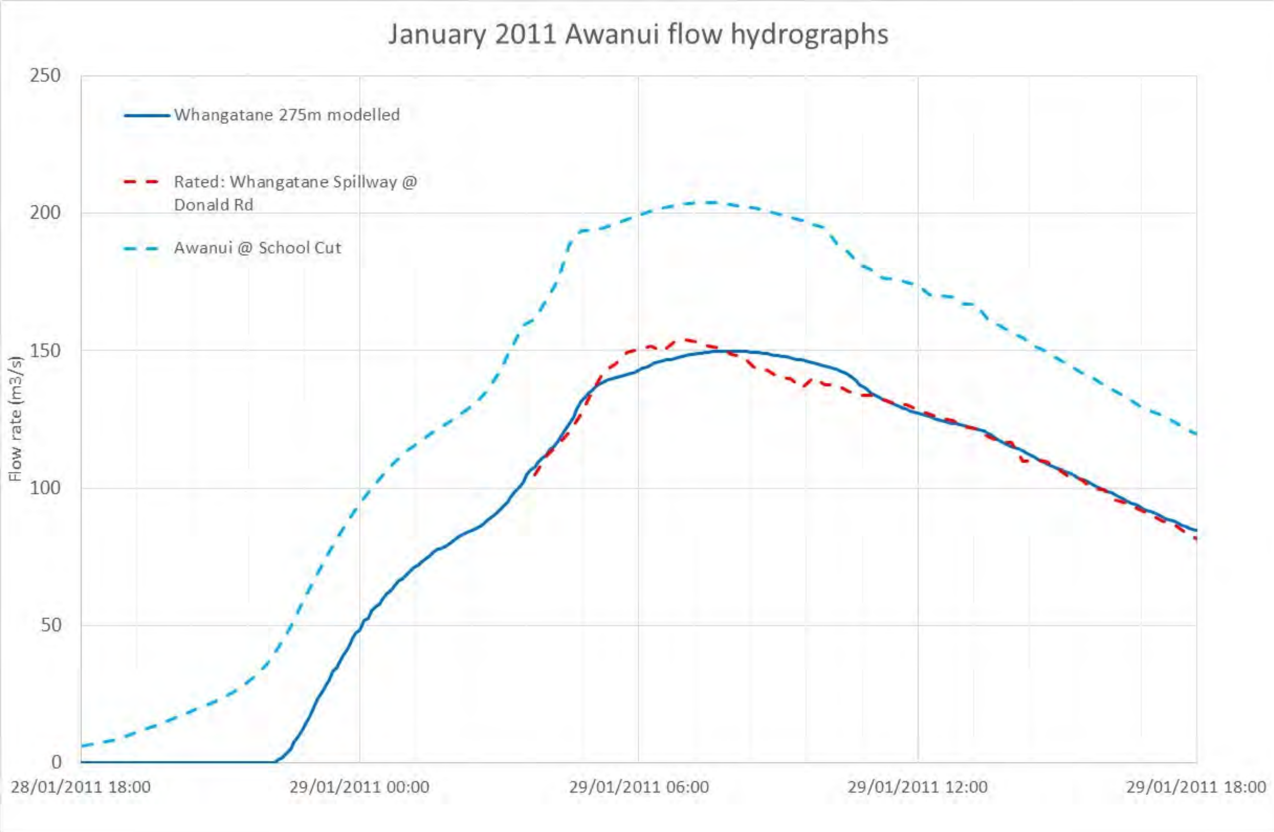


Figure 5 Rated and modelled flow hydrographs at Whangatane Channel @ Donald Road, January 2011 Rated flow at School Cut shown for comparison

The hydrograph shape at Donald Road has been reasonably well modelled in Figure 3 and Figure 5. The July event is a little more difficult to assess, because it is presumed that the rated record has under-recorded the peak of the event (when flow was obstructed by the bridge deck).

Flow-stage rating

Whangatane Channel @ Donald Road

Stage/flow pairs of values from the model runs for the Donald Road gauging site are compared in Figure 6 with NRC's rating. For flows above 40 m^3/s, the model rating agrees well with the actual rating. This suggests that application of a single Manning's n value for the channel is particularly appropriate in this case.

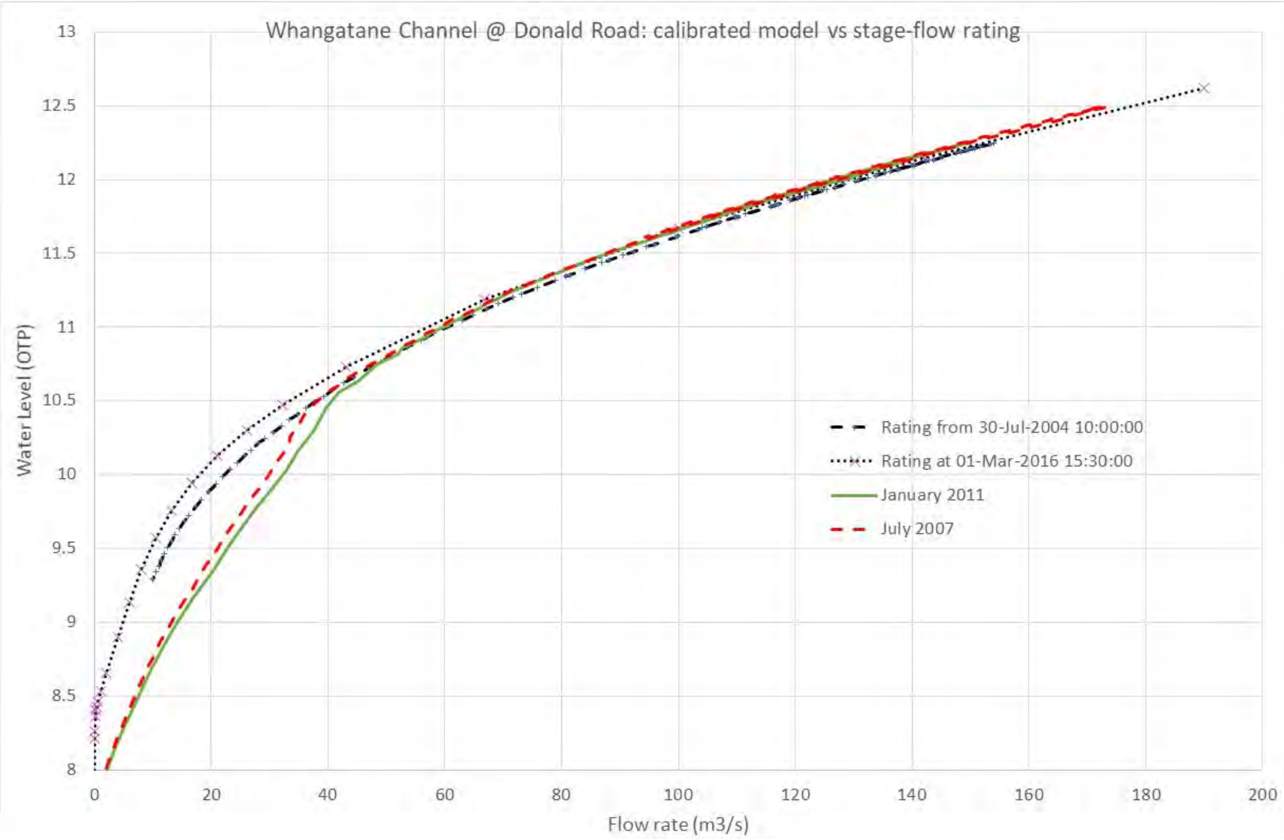


Figure 6 Whangatane Channel @Donald Road (chainage 295m): Stage-flow rating

Peak water level profiles

In Figure 7 to Figure 15 the modelled peak water level profiles in both the Awanui River and the Whangatane Channel are plotted against measured levels (which include the Donald Road record as well as surveyed debris levels).

The January 2011 event proved difficult to calibrate perfectly. After consultation with NRC, the debris level adjacent to the bifurcation has been given less weight. However, the profiles in both channels (Figure 9 and Figure 14) remain a little low downstream of the bifurcation (albeit within the like uncertainty of the debris levels as peak water levels).

Awanui River

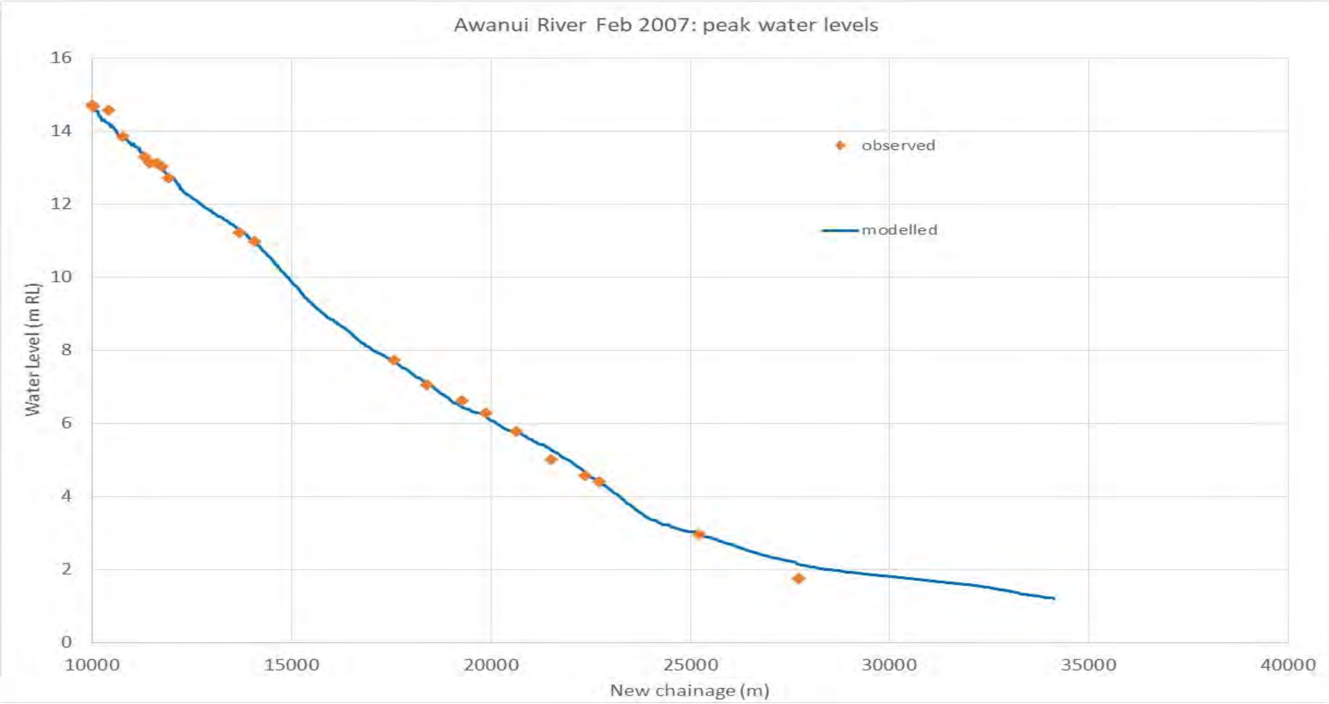


Figure 7 Peak observed and modelled water levels, February 2007

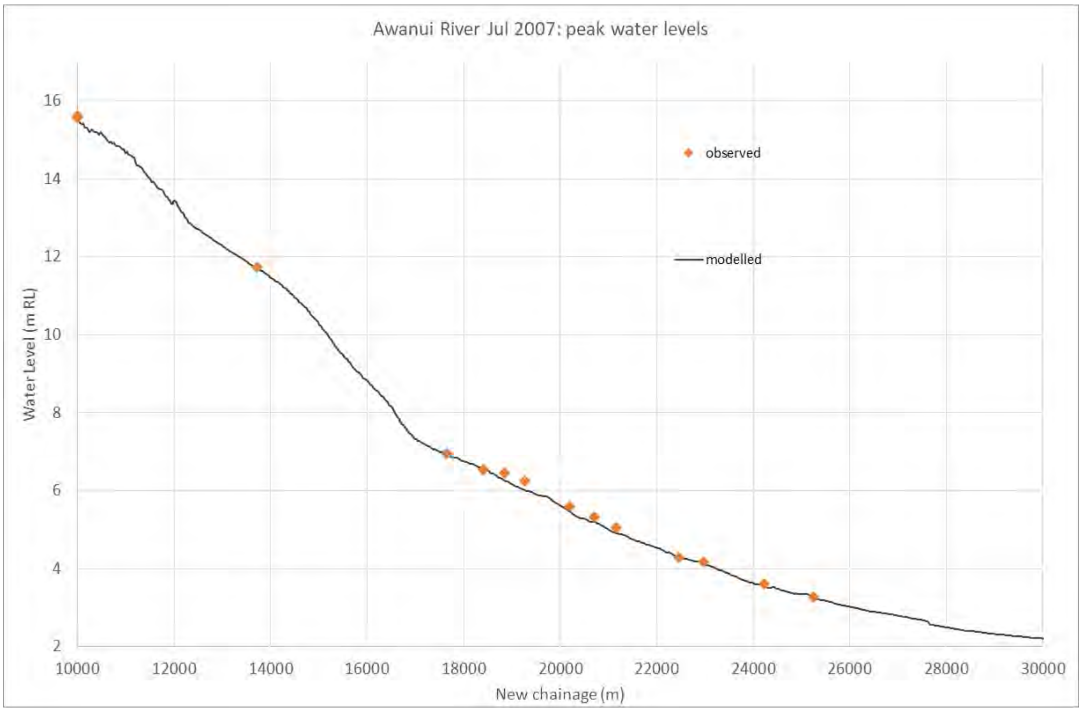


Figure 8 Peak observed and modelled water levels, July 2007

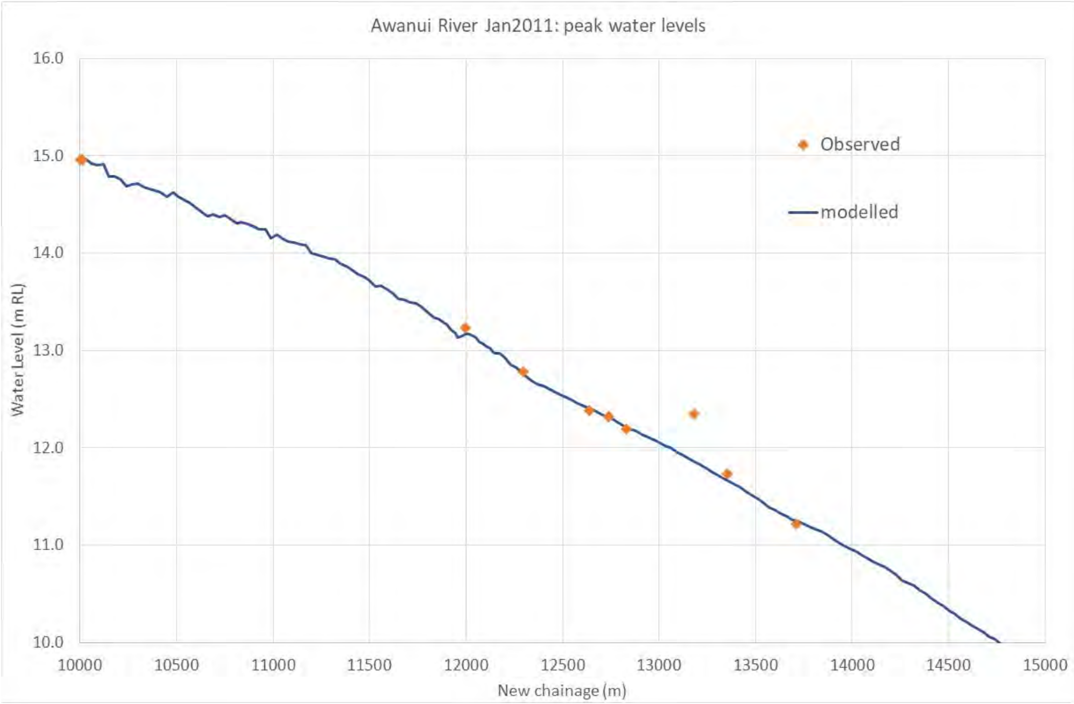


Figure 9 Peak observed and modelled water levels, January 2011

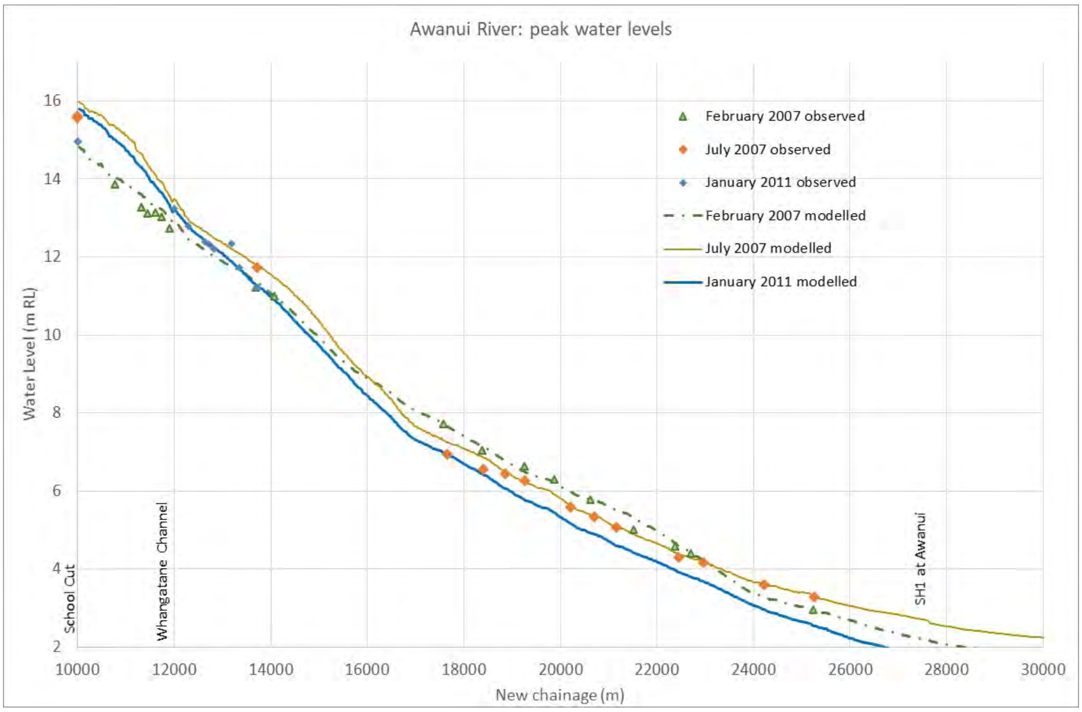


Figure 10 Peak observed and modelled water levels, all 3 events

Whangatane Channel

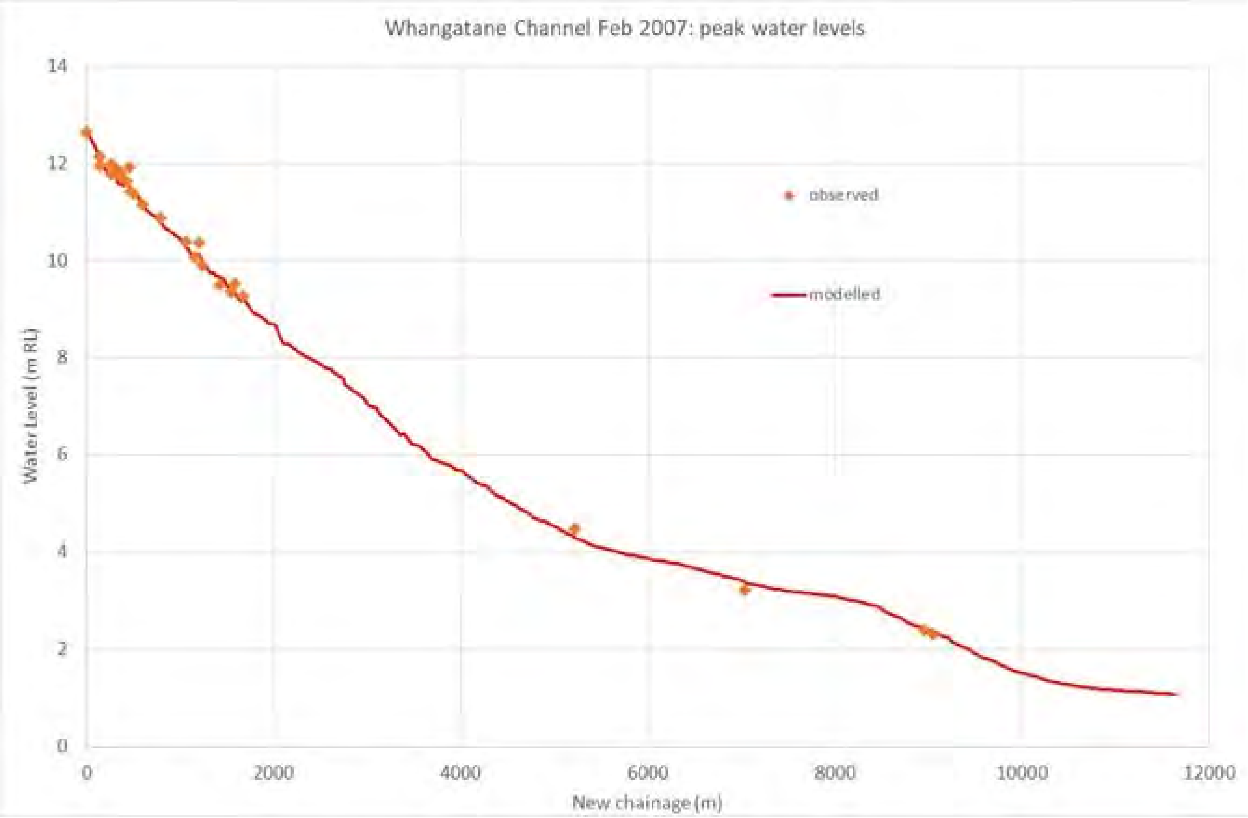


Figure 11 Peak observed and modelled water levels, February 2007

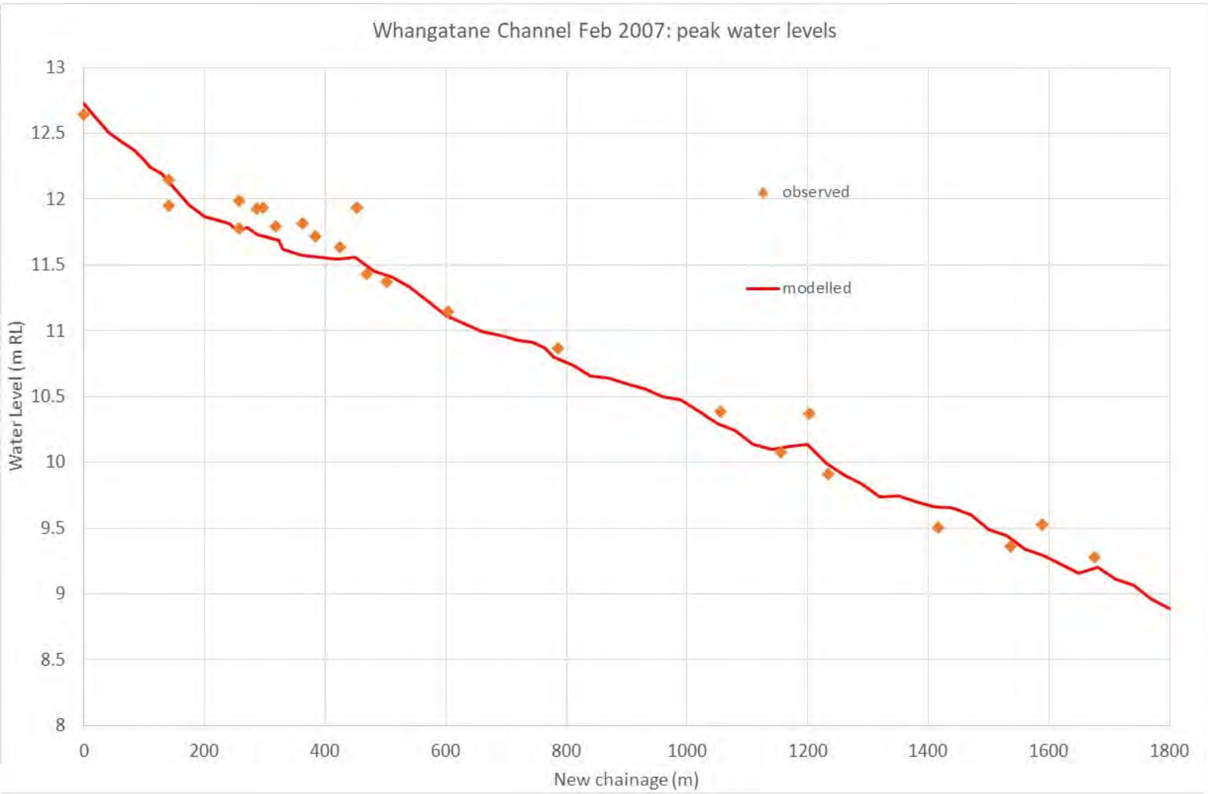


Figure 12 Peak observed and modelled water levels, February 2007

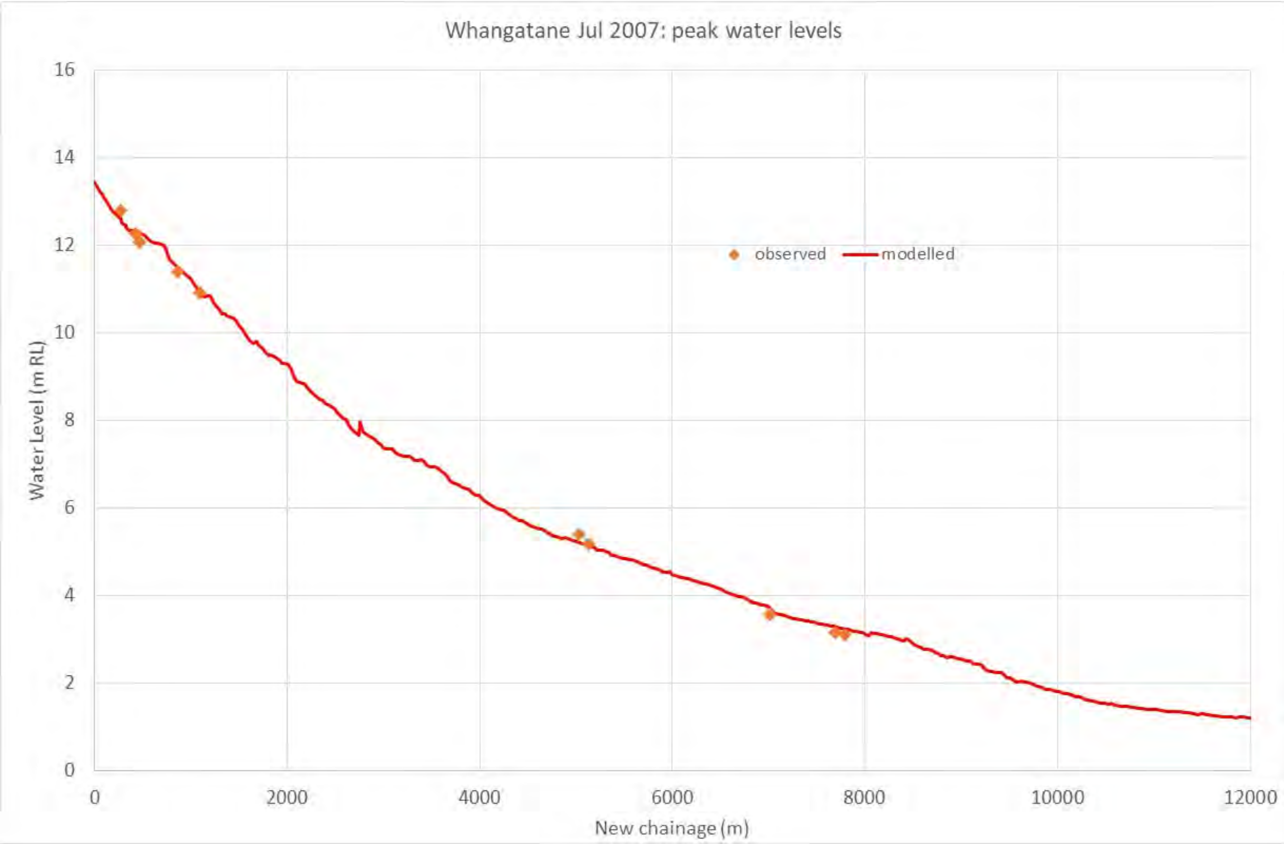


Figure 13 Peak observed and modelled water levels, July 2007

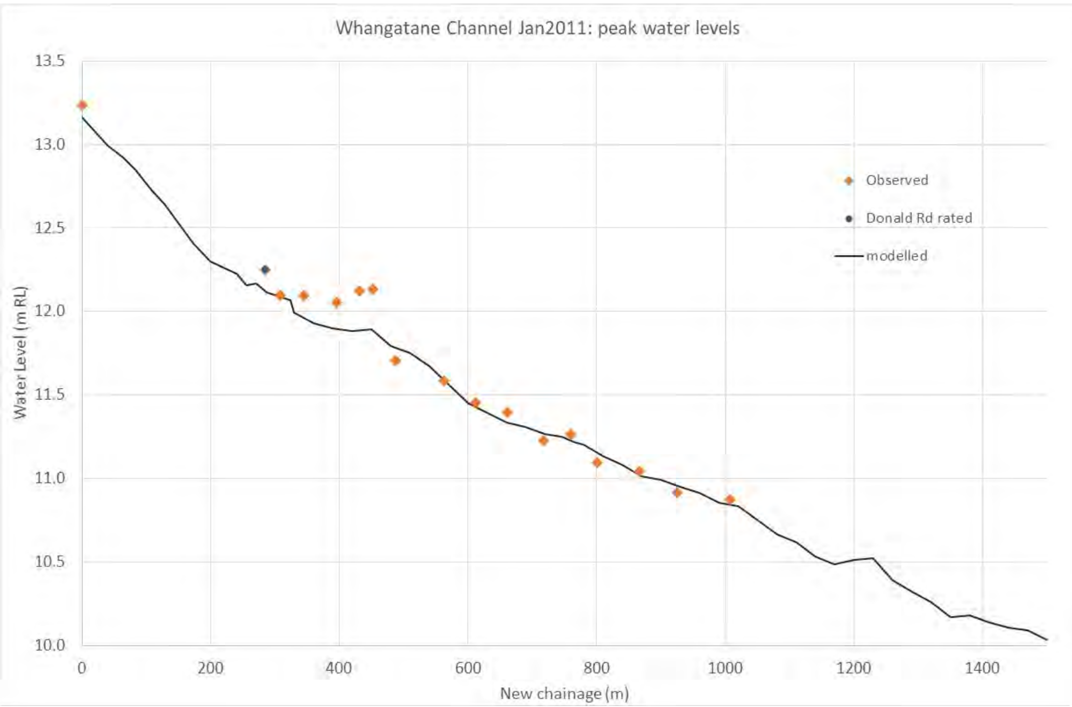


Figure 14 Peak observed and modelled water levels, January 2011

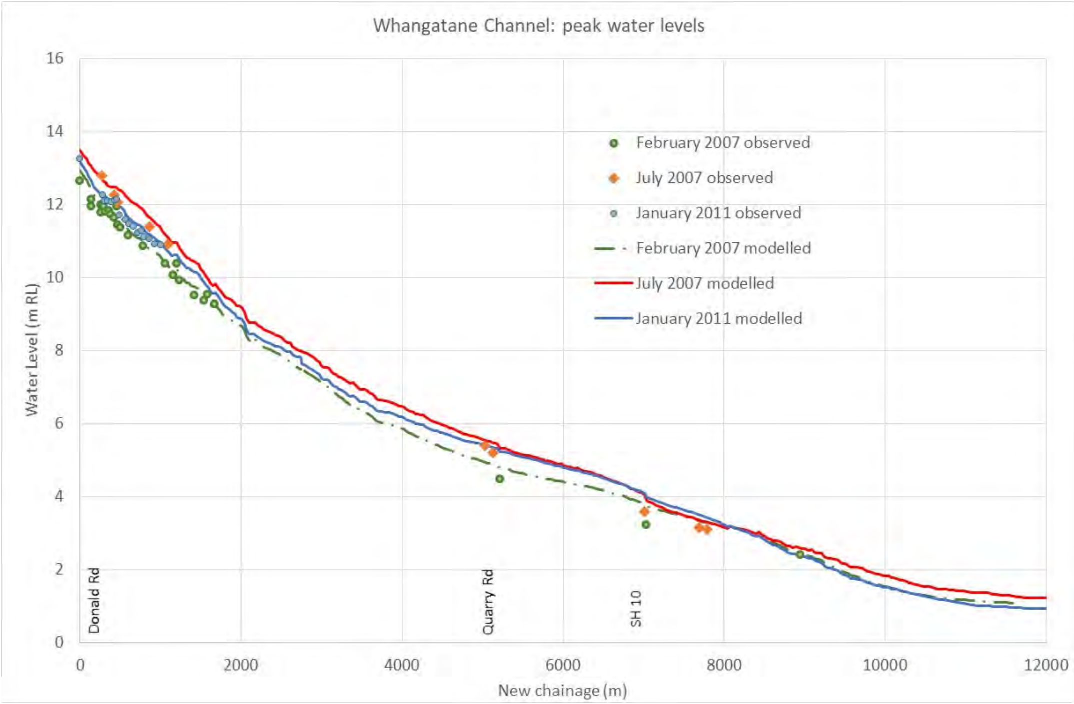


Figure 15 Peak observed and modelled water levels, all 3 events

Tarawhataroa Stream

The cut-down model includes Tarawhataroa Stream downstream of the SH1 overflow, and has been calibrated against gauged water level at Puriri Place and peak water levels inferred from bankside debris. The boundary conditions were:

- Upstream boundary condition: rated flow at Tarawhataroa Stream @ Puriri Place.
- Downstream boundary condition: a representative water level in Lake Tangonge

Flow-stage rating

In Figure 16 stage/flow pairs of values from the model runs for the Puriri Place gauging site are compared with NRC's rating. These data show the effect of the different calibration for each event, with moderate variation between events and with the NRC rating. The rating itself is somewhat untested; the highest gauged flow has been 25 m^3/s, in the February 2007 event.

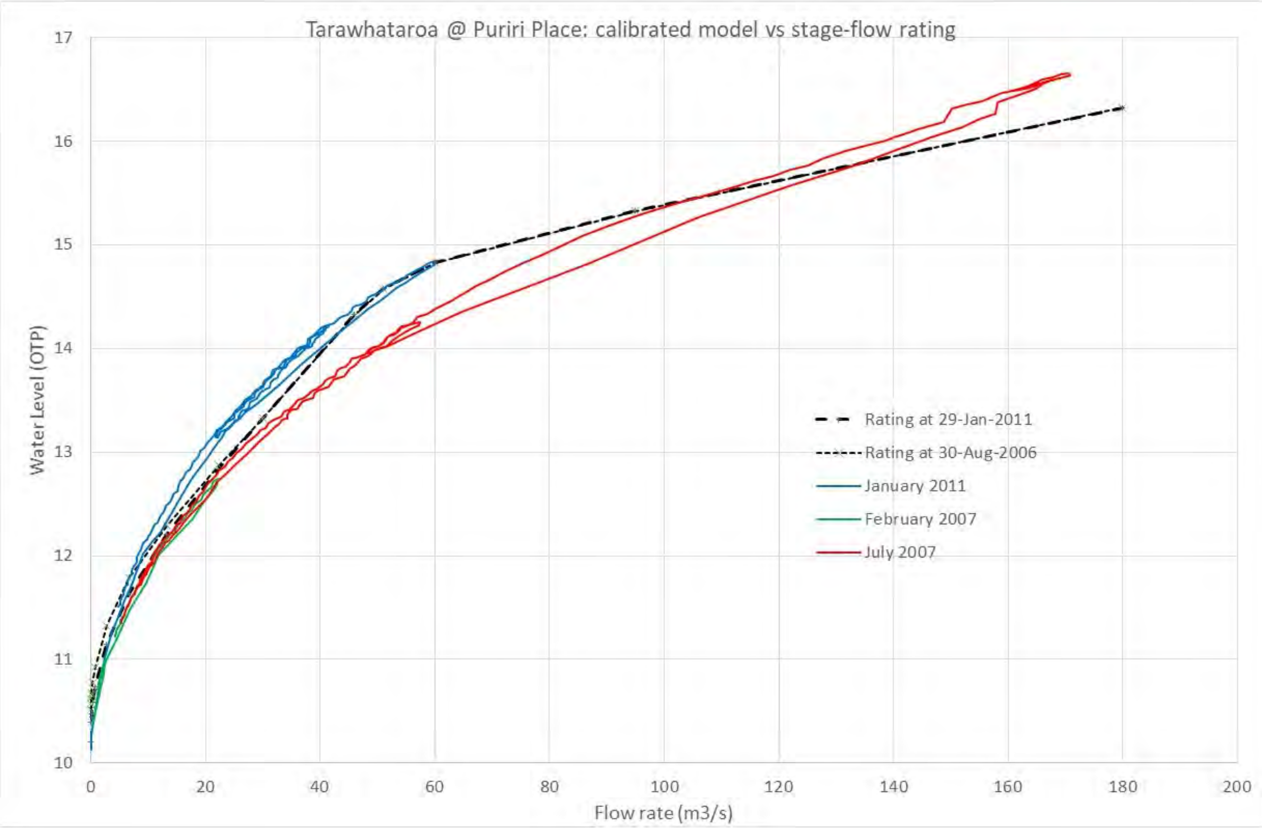


Figure 16 Tarawhataroa @ Puriri Place (chainage 6006m): Stage-flow rating

Modelled peak water surface profiles are plotted, along with measured levels, in Figure 17.

Peak water level profiles

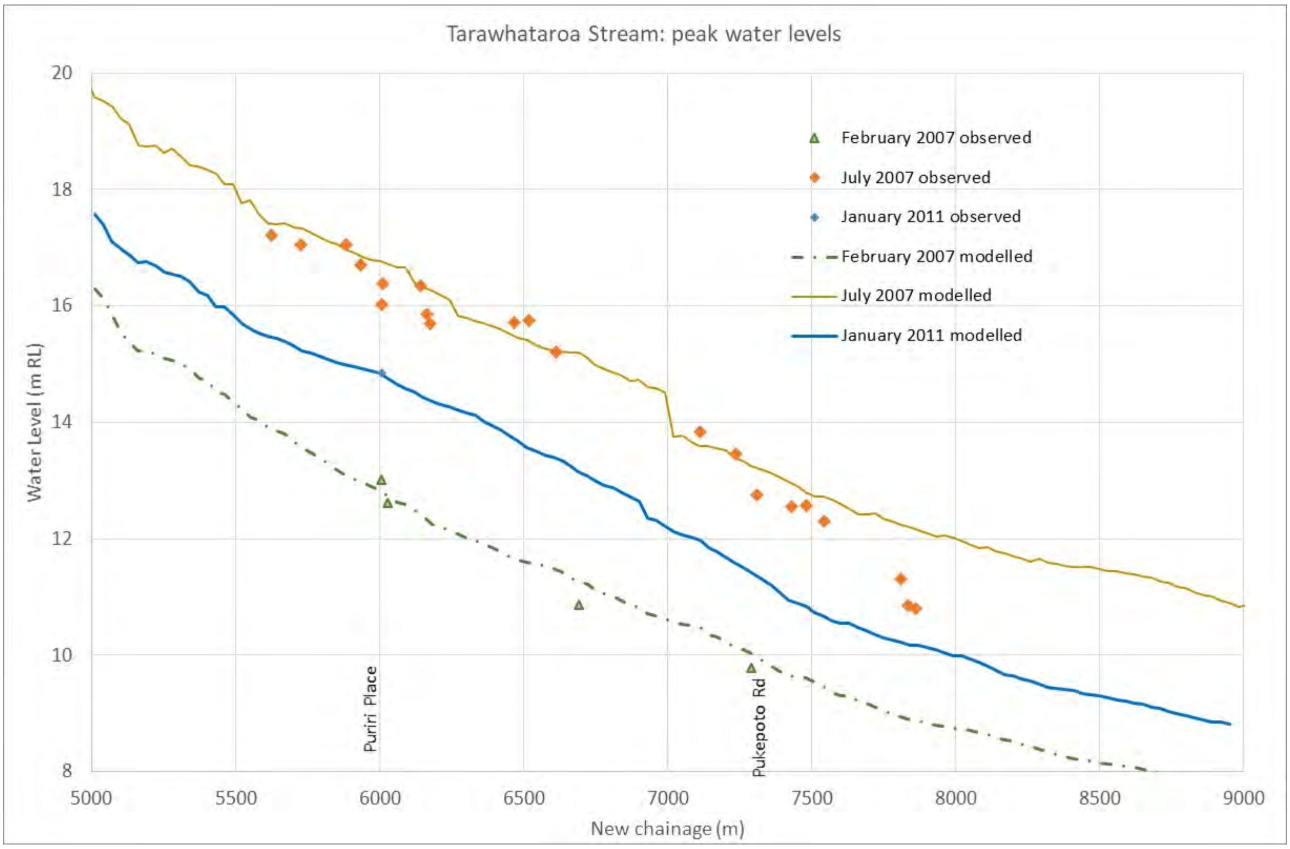


Figure 17 Peak observed and modelled water levels, all 3 events

Calibration outcome: proposed Manning's n

The values of Manning's n determined by the calibration process for each of the 3 events are plotted in Figure 18 to Figure 20, leaving gaps in reaches without any measured peak water level. These graphs show some variation between events, but also allow an average calibration to be chosen that is reasonably close to those for each of the individual events.

Figure 18 to Figure 20 include an estimate of this average calibration (black dashed line labelled "3 events") and also the calibration values obtained by Tonkin & Taylor and sent to Graham Macky on 17th January. These two lines are not identical but appear to be close enough for there to be no major implications for choosing one over the other.

Following further consultation with Tonkin & Taylor on 26th February 2019 (G. Macky /M. Tailby) our joint position was that in general either line represents a suitable calibration against the 3 events. The final choice was a matter for NRC review, but - upstream of any outflows to overland flow – it was thought the following could be adopted with reasonable confidence:

- In the Awanui River, Tonkin & Taylor's values where provided;
- In the Whangatane Channel, G. Macky's values; and
- In Tarawhataroa Stream, a uniform Manning's n of 0.05 (although 0.055 would also provide a reasonable calibration.

It is understood that the Tonkin & Taylor model doesn't incorporate any outflows to overland flow. In that case, downstream of the outflows the calibrated Manning's n values for that model should be retained. In contrast, the calibration described in this report does allow for outflows, and is therefore more suitable for the full 1d/2d model. The average of Manning's n values obtained from the three events (the dashed black line in Figure 18 to Figure 20) was therefore adopted for the full Awanui catchment model.

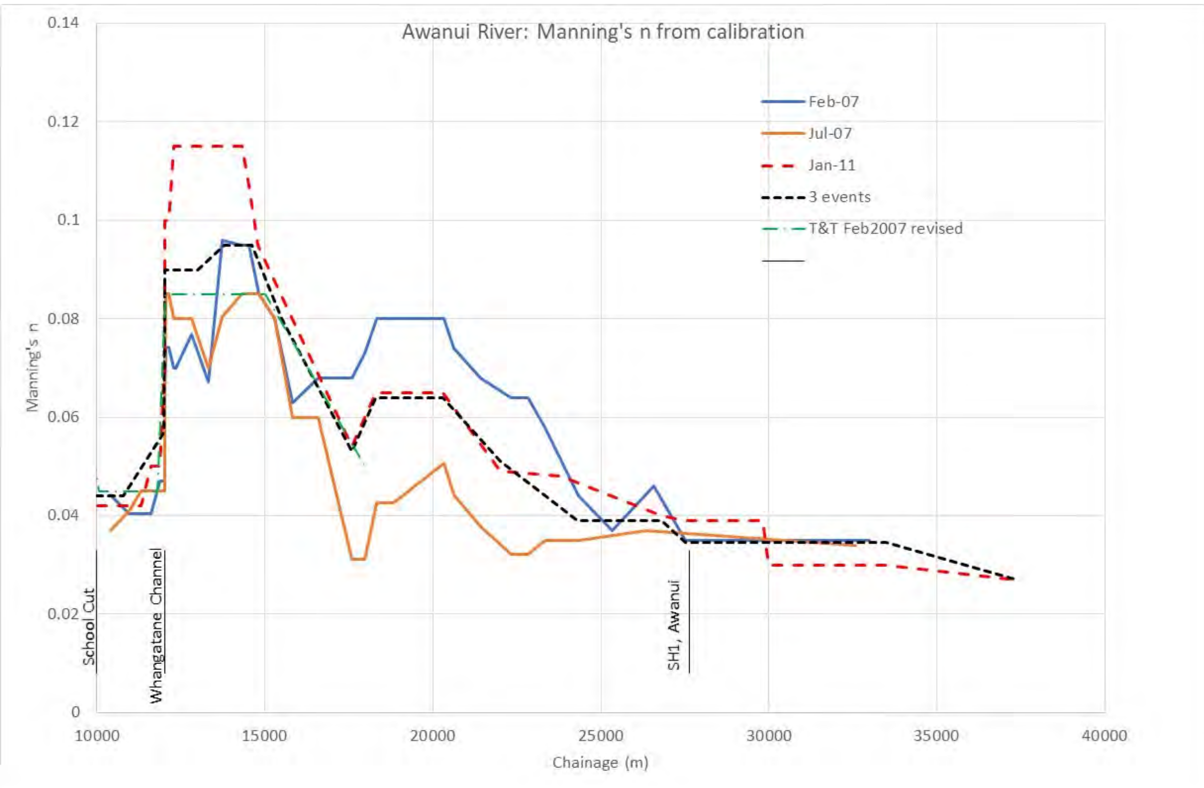


Figure 18 Awanui River Manning's n: Calibration values from each event and proposed values

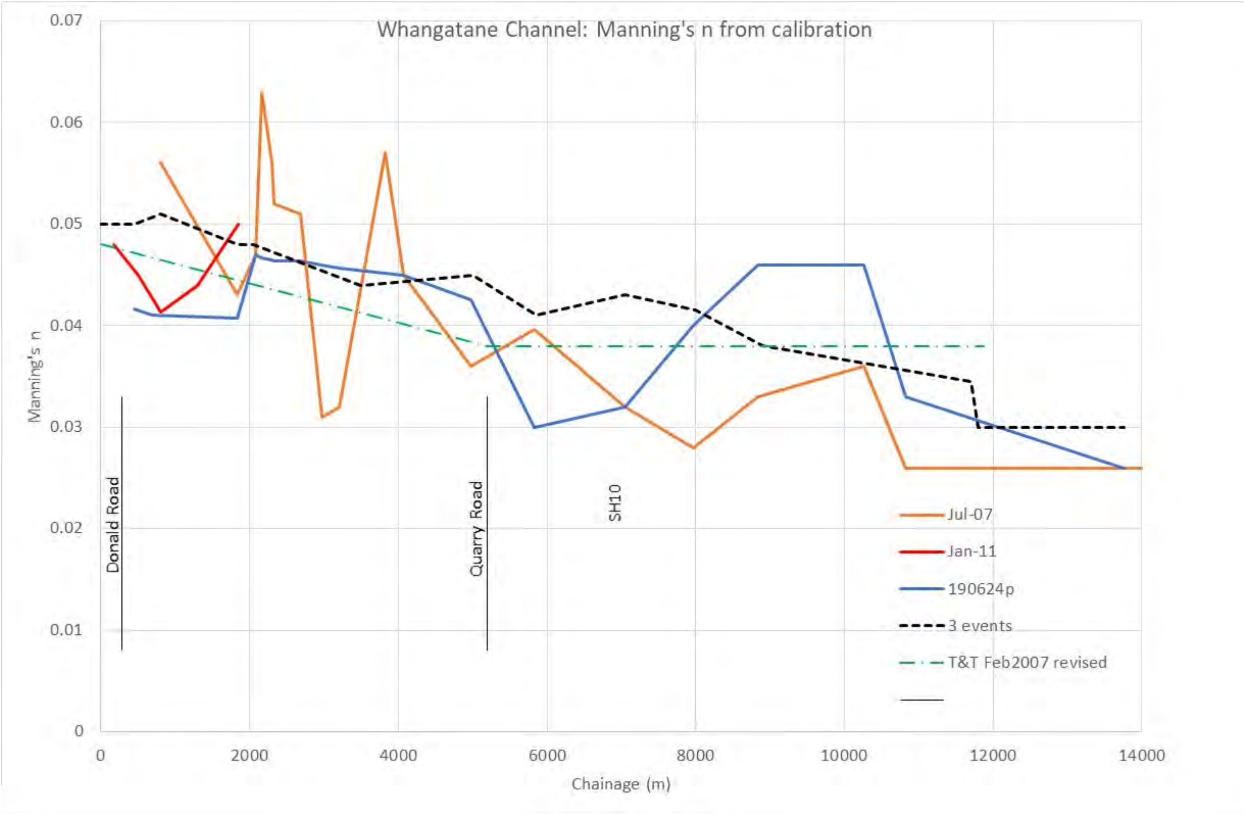


Figure 19 Whangatane Channel Manning's n: Calibration values from each event and proposed values

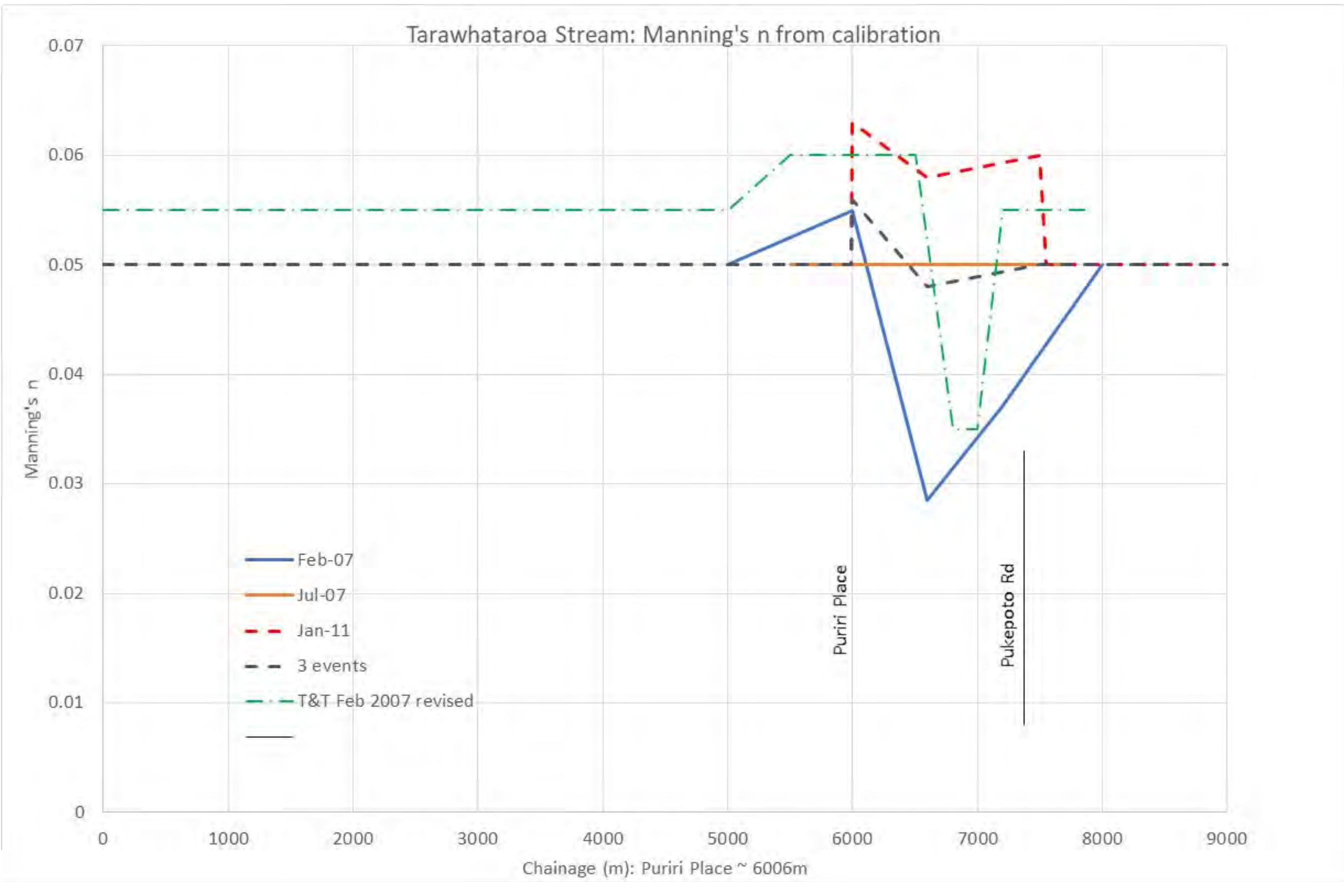


Figure 20 Tarawhataroa Stream Manning's n: Calibration values from each event and proposed values

Hydraulic calibration: State Highway 1 overflow

This aspect of the hydraulic calibration has been carried out on the full model using the January 2011 event.

As originally assembled, the model included specification of a weir along the State Highway 1 (SH1)_ centreline, but otherwise applied the LiDAR ground levels in the same manner as the rest of the overland flow, using the same assumptions for flow resistance (Manning's n). Calibration runs up to C5 revealed that almost all the SH1 overflow occurs over a length of road of about 150 m near Larmers Road, at the most upstream of the three riverbends (Figure 22).

A hydrograph was extracted from run C5 of the modelled total overflow across SH1 south of Kaitaia. The flow rates are plotted against modelled School Cut flow in Figure 21, with a lag applied from the SH1 hydrograph to the School Cut hydrograph. The particular lag has been chosen to reduce the looped nature of the "rating" as much as possible, 40 minutes in the case of Run C5. Calibration runs C6 and C7, detailed below, have been processed in the same way, with the "optimum" lag found to be 25 minutes. Figure 21 also plots the NRC as well as the equivalent ratings from earlier modelling by NRC and by Tonkin & Taylor.

The line of lagged data from run C5 shows reasonable agreement with the NRC line for the onset of the overflow (177 versus 180 m^3/s at School Cut). However, above that flow the modelled overflow increases too sharply with increasing Awanui flow.

An improved rating between School Cut flow and the SH1 overflow was then achieved by artificially elevating part of the modelled weir along the road centreline (calibration run C6 in Figure 21).

However, at NRC request, that approach was abandoned in favour of applying extremely high flow resistance to areas north of SH1 (nearer the river) (Calibration run C7). Aerial photographs of that area, and photographs from the SH1, show dense vegetation in places; there may also be a length of constructed berm parallel to the road (too narrow to be represented in the Lidar data). The areas of high resistance were defined from reference to the photographic evidence, but the choice of Manning's n was largely subjective (Figure 24).

An attempt at refining the flow resistance values for a yet better fit (run C8) gave a comparable result but with a slightly erratic hydrograph, and is not presented here. Calibration run C7 is therefore the most successful at replicating the NRC rating, and its description of the overflow area has therefore been adopted for all future runs. The flow pattern at the overflow location is shown in Figure 23.

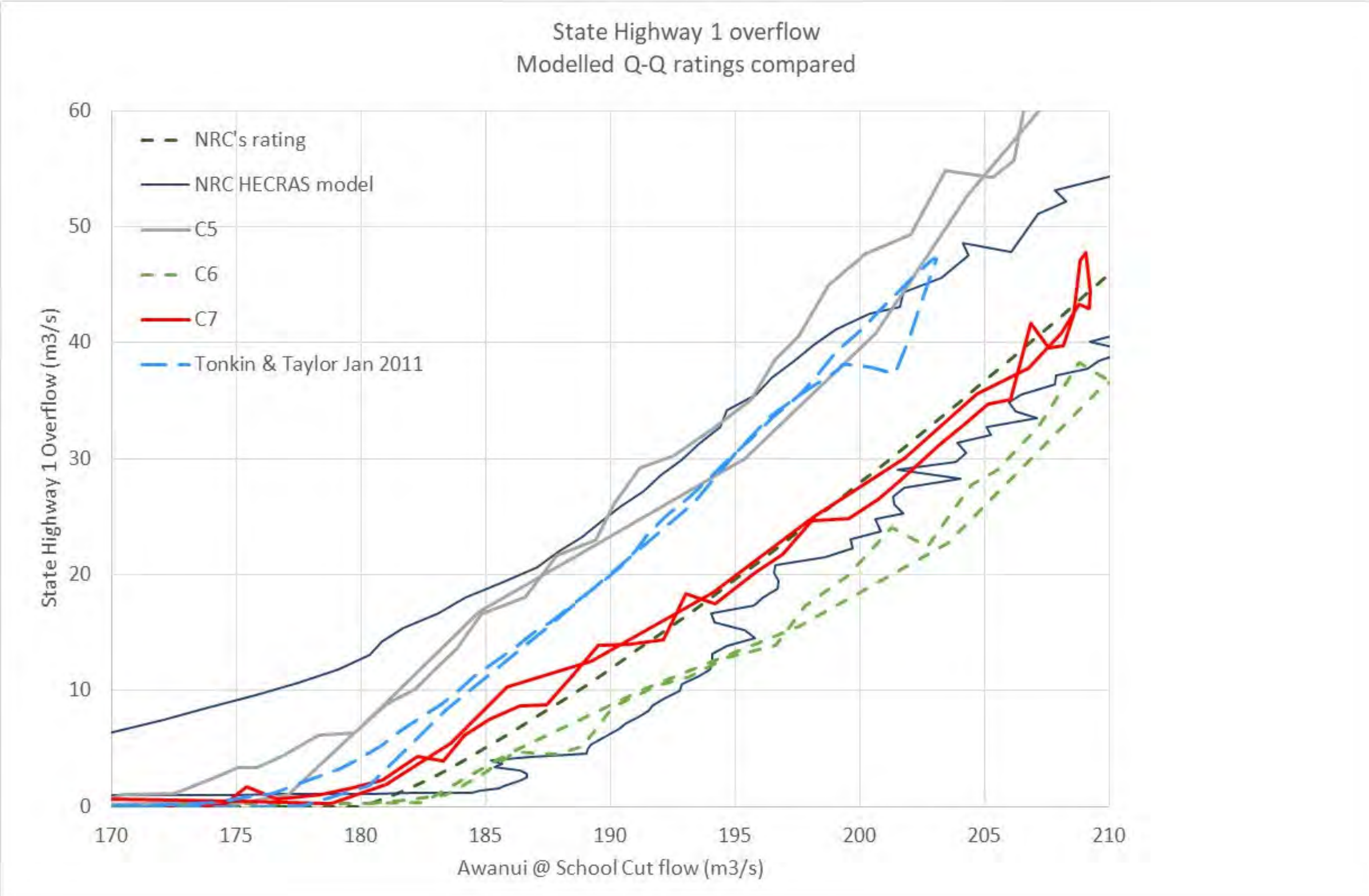


Figure 21 "Rating" between Awanui flow at School Cut and overflow over State Highway 1

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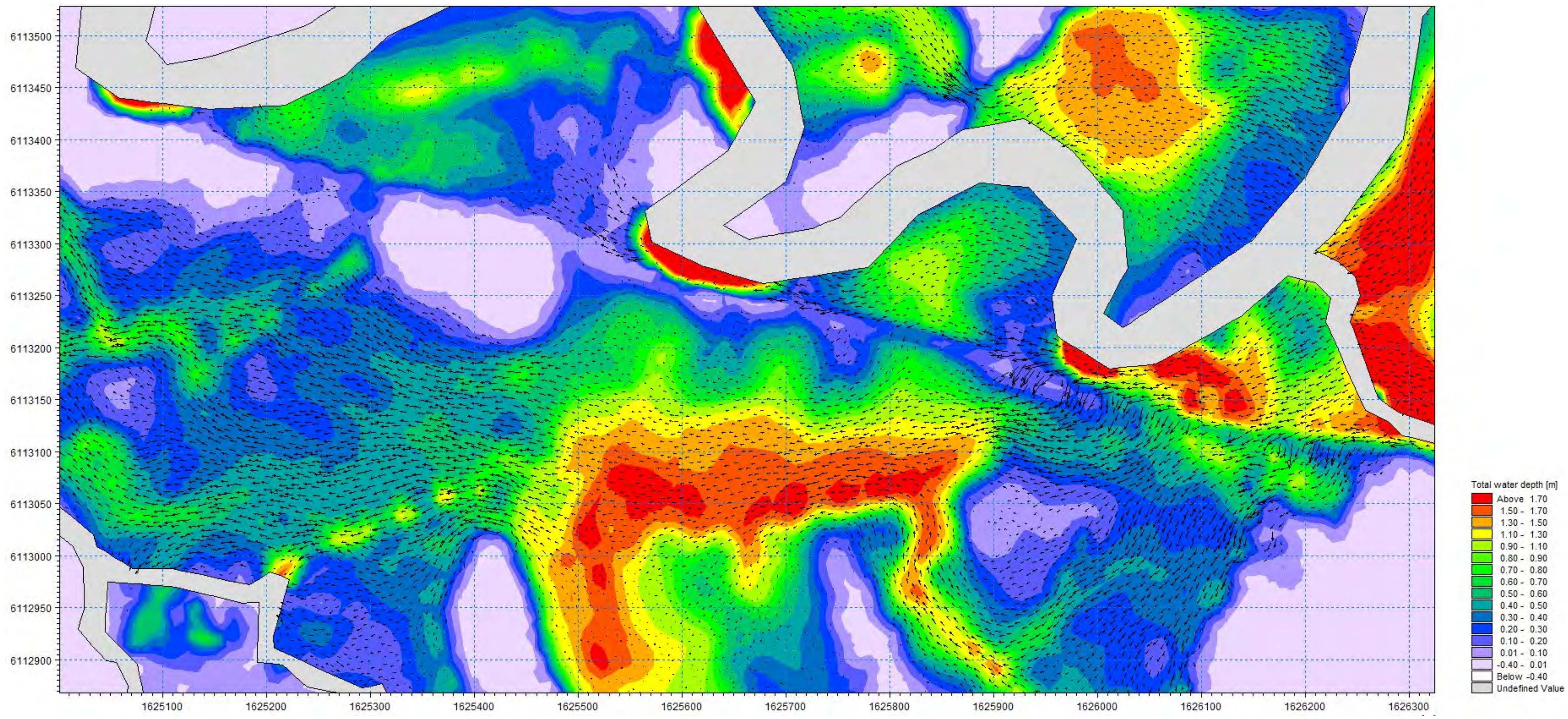


Figure 22 Modelled peak overland flow, State Highway 1 upstream of Kaitaia, Run C5. River flow is from right to left.

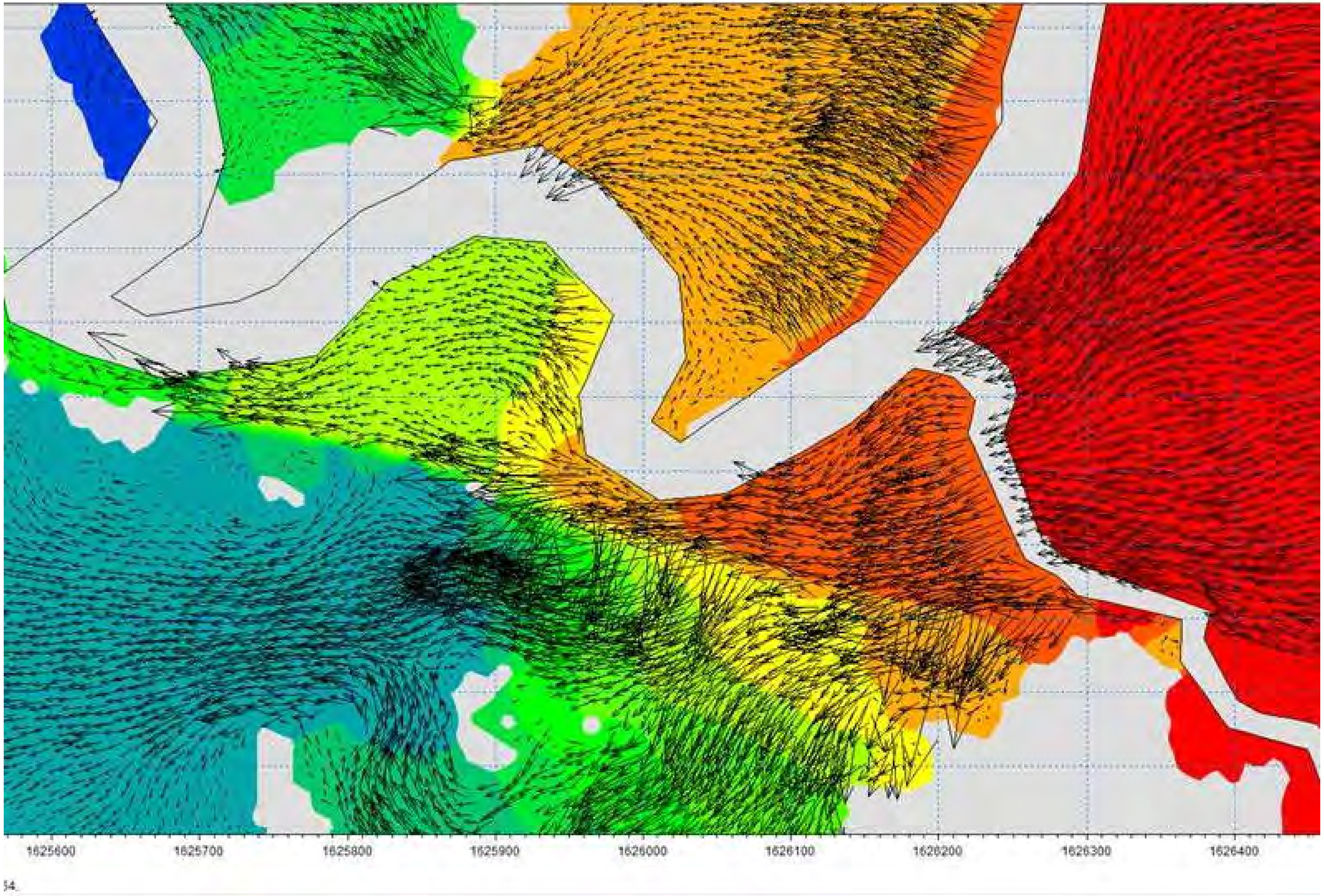


Figure 23 Modelled peak overland flow, State Highway 1 near Larmers Rd, Run C7. River flow is from right to left.

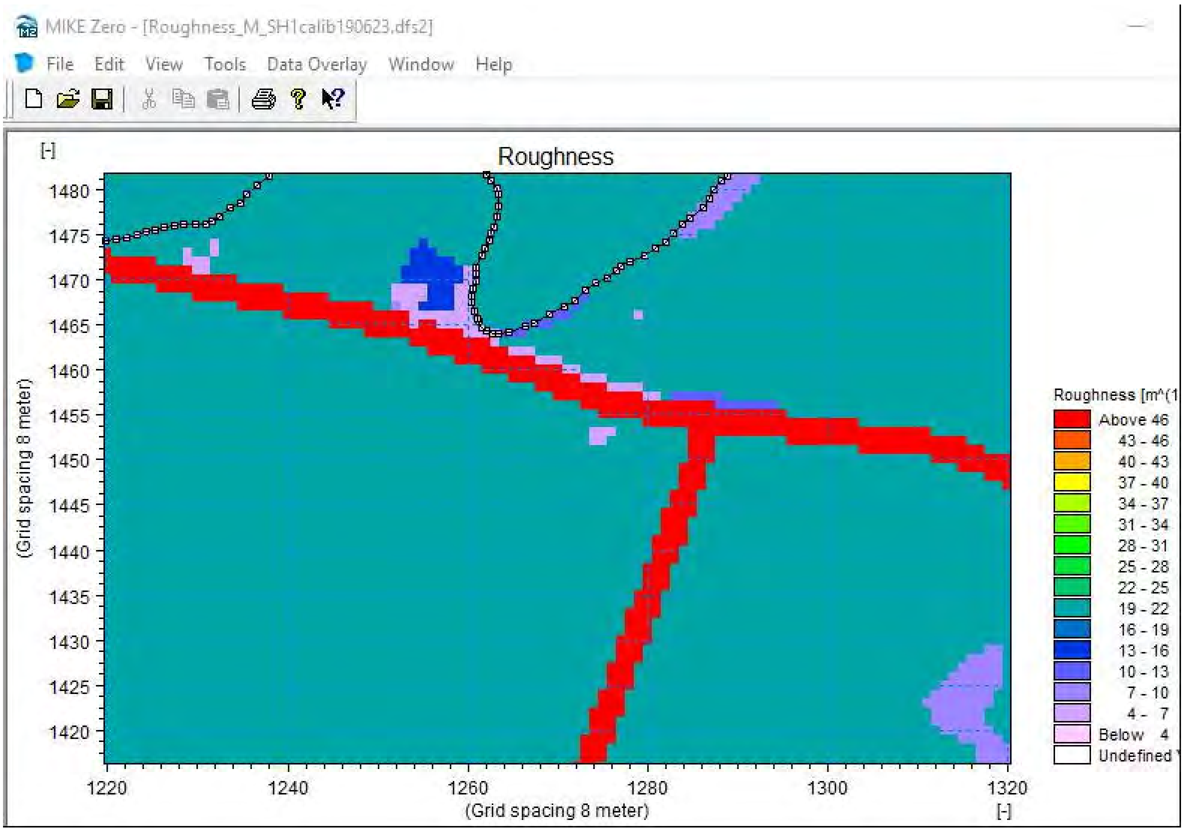


Figure 24 Modelled flow resistance at the SH1 overflow, Calibration run C7. The legend specifies M=1/Mannings n

Attempted hydraulic calibration of upper Tarawhataroa Stream

Further modifications to the full model were made in an attempt using the January 2011 event to calibrate the hydrograph shape and peak flow of the Tarawhataroa Stream from its own catchment (calibration run C9). This runoff is easily distinguishable from the SH1 overflow from the Awanui River, which arrives later. The hydrological model had already been calibrated to replicate the volume of flow rated at Puriri Place (as described below). However, the modelled hydrograph begins and peaks significantly earlier than the rated hydrograph (Figure 25) and the peak is significantly higher. Changes were made to the MIKE 11 channel model to force more water onto the floodplains:

- The bridge and culvert passing flow under Larmers Road were both partially blocked off. It is understood that he floodplain upstream of Larmers Road floods frequently, and this blockage therefore appears very plausible.
- From the aerial photograph, the main Tarawhataroa Stream appears to pass through a farm culvert on the floodplain, and this was assumed to have a quite limited capacity.

The effect of these changes on the Puriri Place modelled flow was negligible (Figure 25). Inspection of channel flow rates at various locations indicates that the model generated floodplain flow as intended but not the detention for several hours that would be needed to match the rated hydrograph.

These model changes have been left in place, and they are believed to represent likely features of the catchment's state during a flood. However, a closer examination of the catchment, including its hydrological response, would be needed to explain and model the reasons for the observed lag in flows.

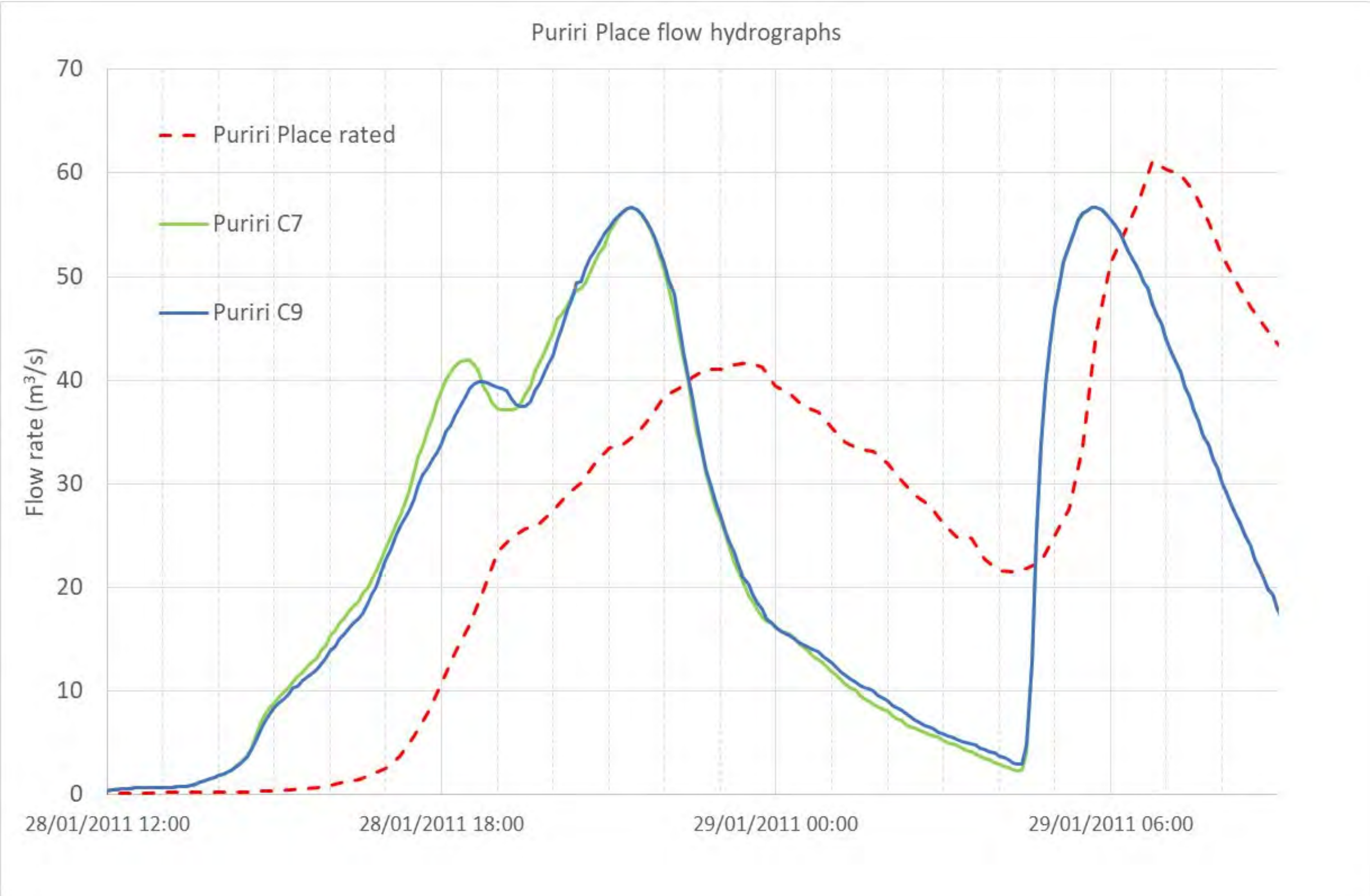


Figure 25 Flow rates at Tarawhataroa @ Puriri Place, January 2011: rated and modelled (runs C7 and C9)

Hydrological Calibration of the full model upstream of School Cut

Hydrological calibration has been carried out on the full model **using the January 2011 event**.

The entire Awanui catchment has been modelled hydrologically as many small sub-catchments, modelled in NAM. NAM comprises a set of linear equations describing water movement within a simple catchment-averaged model of rainfall, evaporation, soil moisture, surface runoff and loss to groundwater. Calibration of this hydrological model has proceeded in two steps.

First, the auto-calibration feature of MIKE NAM was used to calibrate the catchments of the Te Puhi and Takahue gauged flow sites. (The Victoria site was not used as its record appeared rather erratic, and the Tarawhataroa does not have a very long rainfall record for calibration.) This auto-calibration was run for the full available record of several years, and was set to optimised high flows.

The process provided a set of NAM parameters to be applied to the many sub-catchments of the full model, with two exceptions: the initial flows required scaling to sub-catchment size, and the parameter CK1,2 (which can be equated to the time-of-concentration) was calculated for each sub-catchment using the sub-catchment geometry and two well-known empirical equations.

To simplify this process, the Awanui catchment was divided into four hydrological areas (Figure 26): Te Puhi, Takahue, Tarawhataroa and “Swamp”, the first three comprising not only the gauged catchment but reasonably adjacent sub-catchments with apparently similar physical attributes. Any subsequent adjustment of NAM parameters was then applied *en bloc* within a hydrological area.

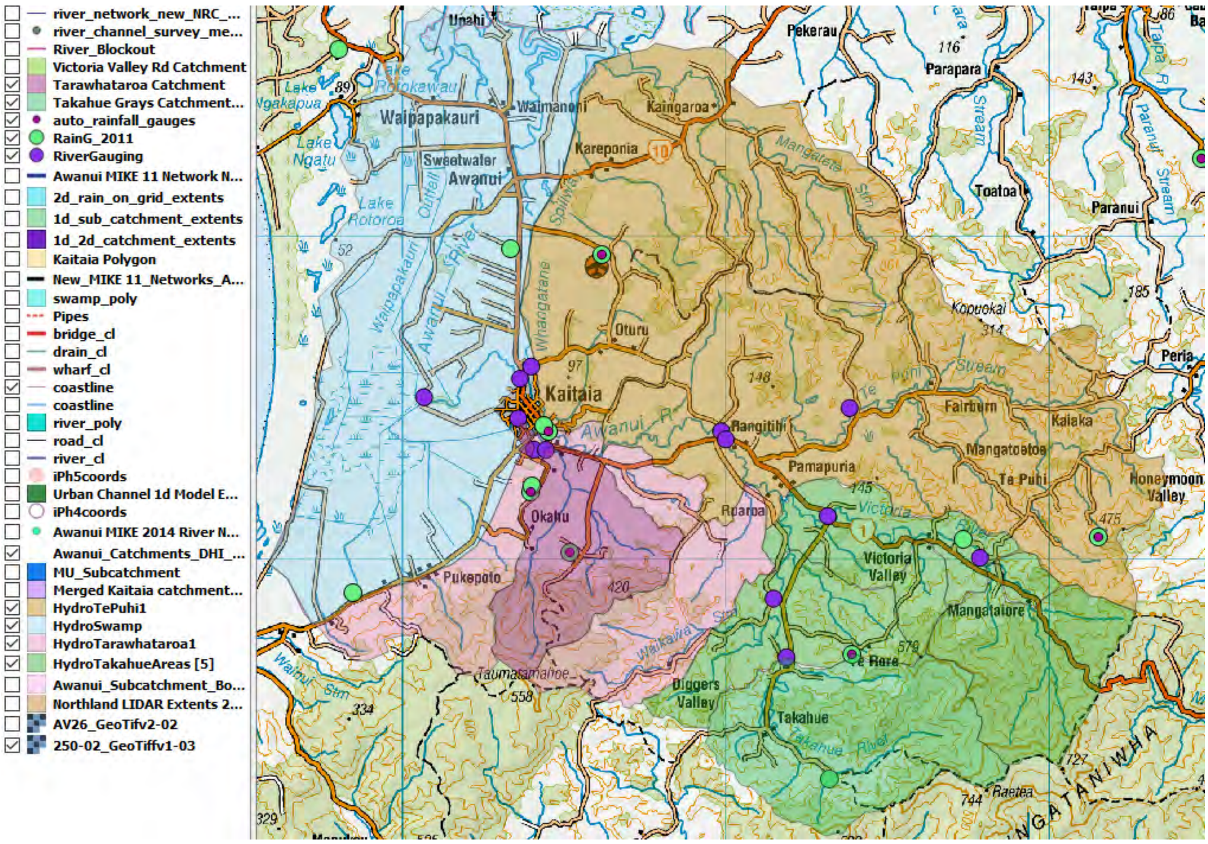


Figure 26 Hydrological areas for NAM modelling

The NAM catchments were incorporated into the full MIKE Flood model, which covers the entire Awanui Catchment. Preliminary model runs were carried out primarily to ensure the model was robust, but also allowed some model adjustment. One part of that adjustment was to the initial value of L/Lmax, which can be regarded as the percentage saturation of the soil. The values of L/Lmax adopted (Table 1) needed to be higher than the values extracted for the beginning of the event from the Te Puhi and Takahue auto-calibrated runs.

This initial calibration process was followed by three runs (C3-C5) conducted solely for hydrological calibration, with C5 then superseded by Run C7, which calibrated the overflow across State Highway 1 (see the separate section above). The prime goal of the hydrological calibration was to replicate measured flows at Kaitaia: the sum of Awanui River flow at School Cut and any overflow across State Highway 1 into Tarawhataroa Stream. Flow hydrographs at the gauged flow sites in the tributaries (including Takahue and Te Puhi) has not matched the rated flows there particularly well, and calibration at those sites was not progressed.

Table 1 sets out most of the NAM parameters used in Runs C5 and C7. The two parameters adjusted in the calibration process were the CQOF value and Lmax. Within the NAM equations, CQOF is the fraction of excess rainfall that becomes overland flow rather than infiltrating to groundwater, and Lmax, expressed in millimetres, is the maximum soil moisture depth.

Table 1 NAM parameters for the 4 hydrological areas

	Te Puhi	Takahue	Tarawhataroa	Swamp
Umax	14.5	10.9	10.8	7
Lmax	179	240	120	80
CQOF	0.945	0.863	0.6	0.945
CKIF	775.4	917.6	502.8	775.4
CK1,2	various	various	various	various
TOF	0.397	0.05	0.0965	0.397
TIF	0.0639	0.763	0.673	0.0639
iTG	0.3	0.604	0.619	0.3
CKBF	3004	1636	1709	3004
Carea	1	1	1	1
Sy	0.1	0.1	0.1	0.1
GWLBF0	10	10	10	10
GWLBF1	0	0	0	0
Cqlow	0	0	0	0
Cklow	10000	10000	10000	10000
Initial Conditions				
U	0	0	0	*0*
L (fraction of Lmax)	0.724	0.842	0.758	*0.63*
QOF	various	various	various	various
QIF	various	various	various	various
BF	various	various	various	various

Figure 27 shows rated and modelled hydrographs for the combined flow from catchments upstream of Kaitaia (Tarawhataroa @ Puriri Place and Awanui @ School Cut). Four model runs (C3 to C7) are shown, of which C3 – C5 show the later stages of hydrological calibration of the model. The first of these calibration runs (C3) results in a peak combined flow rate more than 20% below the rated flow. Two attempts were made to further refine the NAM parameters: Run C4 made little difference, but Run C5 did, and the adjustment to the SH1 overflow for run C7 has resulted in very good agreement of the total peak flow. However, the modelled peak does occur about two hours earlier than was measured.

(For Run C7, the only change from C5 was to the terrain at the State Highway 1 overflow. This change has a minor effect on the combined hydrograph, but its primary effect is on flows in each of the two channels and is discussed above.)

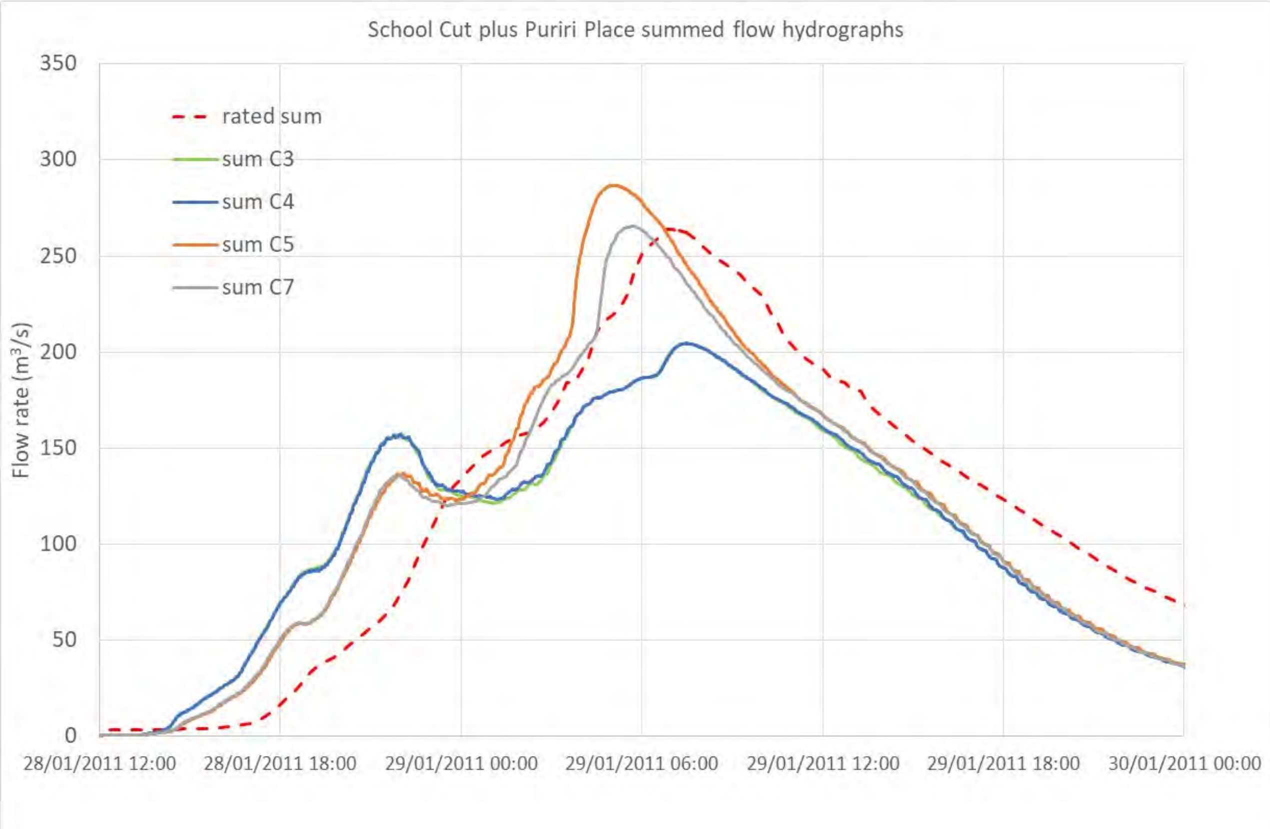


Figure 27 Rated flow approaching Kaitaia compared with successive model runs

The model produced quicker runoff than actually occurred. Further calibration and validation results are presented in the "*Awanui_calibration.xlsx*" file provided with this report.

Modelled Individual flow hydrographs (Runs C3-C7) in the Tarawhataroa Stream @ Puriri Place and the Awanui River @ School Cut are compared with the rated records in Figure 28 and Figure 29. The model produced quicker runoff than actually occurred, particularly runoff from the Tarawhataroa catchment. In run C7, modelled and rated peak flow in the Awanui River agree well, and the slightly lower Tarawhataroa peak flow can be attributed to the model's over-quick Tarawhataroa catchment response rather than any mismatch of the overflow into the Tarawhataroa across State Highway 1.

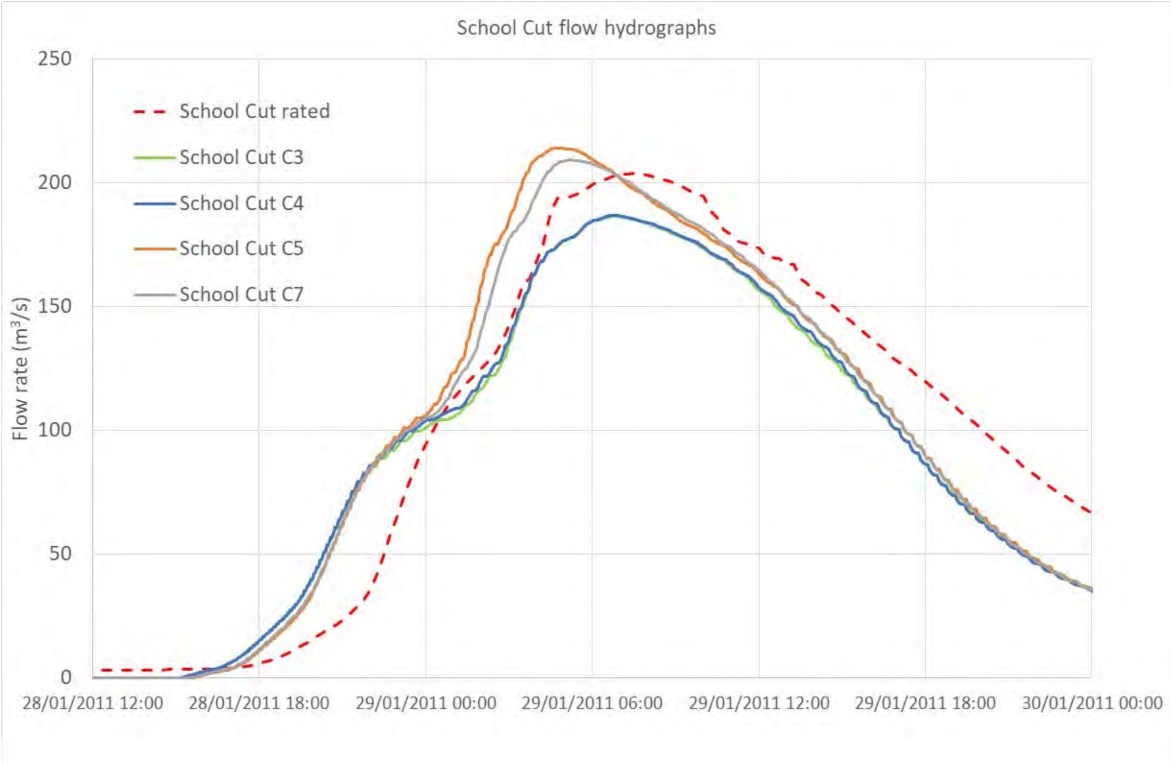


Figure 28 Awanui River @ School Cut: rated and modelled flows, January 2011

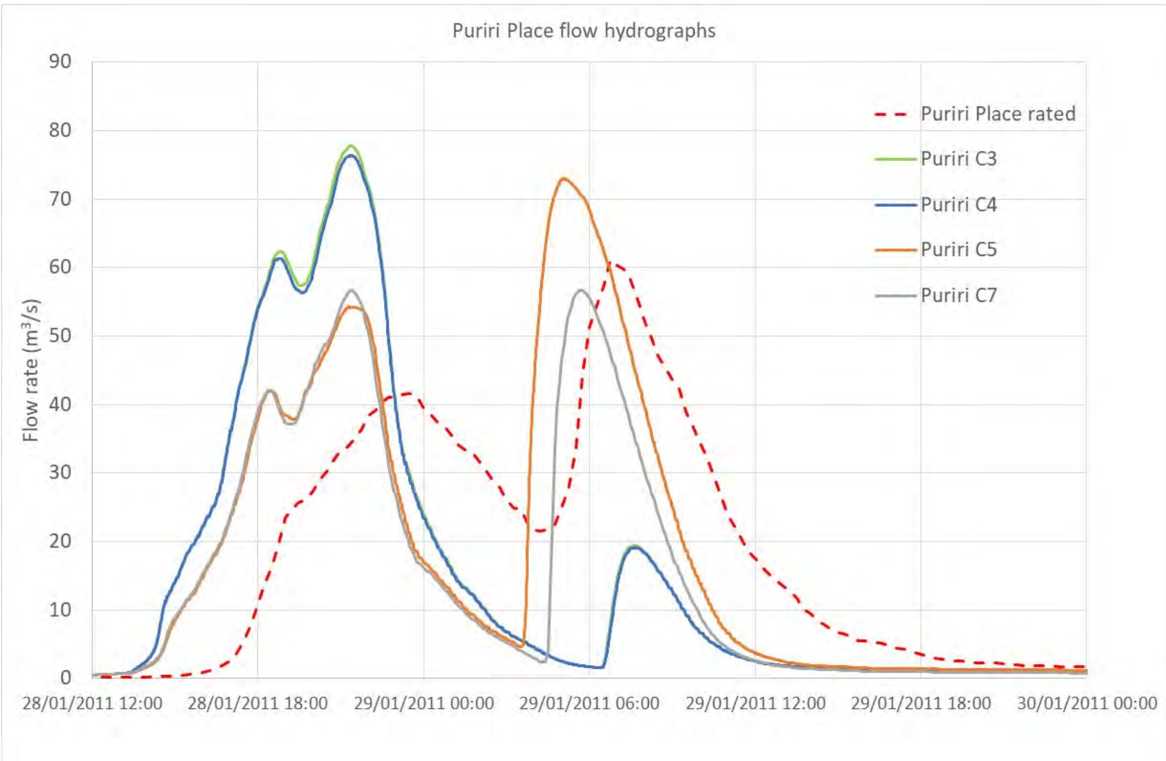


Figure 29 Tarawhataroa @ Puriri Place: rated and modelled flows, January 2011

Modelled volumes and peak flows for Run C7 are compared with the equivalent rated data in Table 2, which shows very good agreement (1% difference) in the total volume.

The model is diverting too much water across SH1 to Tarawhataroa Stream, in spite of successful calibration of the overflow rating against School Cut flow. However, the effect on Awanui River flows is negligible.

Table 2 Comparison of modelled and measured flow peaks and volumes

	Peak flow	**Volume (m^3)**
Modelled	**(m^3/s)**	up to 30/01/2011 00:00:00
School Cut	209.3	1.30E+07
Overflow component of Puriri Place (estimated from hydrograph)	55.4	2.03E+06
total	264.7	1.44E+07
Rated		
School Cut alone	204	1.33E+07
Overflow component of Puriri Place (estimated from hydrograph)	50.4	1.30E+06
total	254	1.46E+07
Modelled - rated		
School Cut alone	2.7%	-2%
Overflow component of Puriri Place	9.9%	56%
total	4.1%	-1%

Sensitivity to 2 NAM hydrological parameters

Just two NAM parameters (Table 3) were varied in this final and formal calibration of the hydrological model. Runs C3-C5 provide a useful sensitivity test for these parameters.

Lmax represent the maximum soil moisture content, in mm of rainfall. CQOF is the fraction of rainfall converted to runoff rather than sent to groundwater. Lmax was decreased in the Te Puhi and Takahue hydrological areas, and CQOF was increased in the Takahue area, in separate attempts to increase modelled peak runoff.

Table 3 NAM parameters varied between model runs C3-C7

C3	Te Puhi	Takahue	Tarawhataroa	Swamp
Lmax	179	240	120	80
CQOF	0.945	0.609	0.949	0.945
C4	Te Puhi	Takahue	Tarawhataroa	Swamp
Lmax	143.2	144	156	80
CQOF	0.945	0.609	0.949	0.945
C5,C7	Te Puhi	Takahue	Tarawhataroa	Swamp
Lmax	179	240	120	80
CQOF	0.945	0.99	0.6	0.945

In Figure 27, the effect of varying Lmax between run C3 and run C4 is clearly negligible. However, the catchment quickflow is notably sensitive to CQOF. Increasing CQOF (for the Takahue area, between run C3 and C5/C7) significantly increases peak runoff, with the peak flow occurring earlier.

Validation against February 2007 event

Using the information gained from the hydrological calibration of the January 2011 event, and the roughness values from the hydrological calibration, a validation run of the February 2007 event was completed using the full MIKE Flood model. The only change to the hydrology applied for the February event was in the initial conditions, i.e. the NAM baseflow, and the initial L/Lmax. The chosen values of initial L/Lmax (Table 4) reflect a finding from preliminary long-term modelling that the catchment was significantly drier before the February 2007 event than just before the January 2011 event.

Table 4 Initial L/Lmax modelled for the 4 hydrological areas for the February 2007 event

	Te Puhi	Takahue	Tarawhataroa	Swamp
L (fraction of Lmax)	0.425	0.557	0.375	0.319

The model results show a reasonable match with both observed peak flow and volume, underestimating the peak flow by 7%, and the volume by 4%.

The shape and timing of the flow hydrograph are not as accurate as with the 2011 event, however. Part of this discrepancy may be attributable to the double-peaked rainfall hyetograph, with two peaks separated by several hours. Other factors acting against an accurate modelling might include the channel condition in the upper catchment, and spatial variation of rainfall not detected by the rain gauge network.

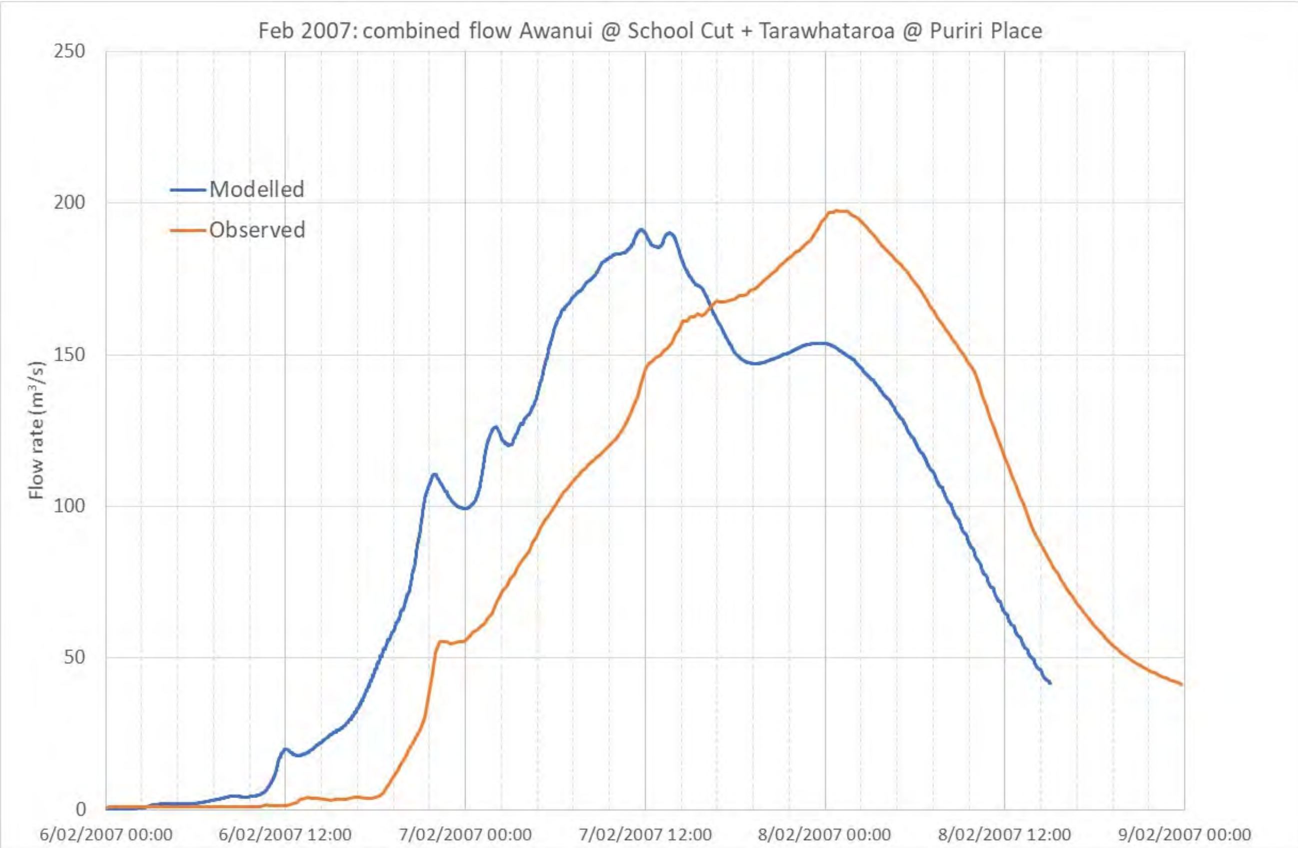


Figure 30 February 2007 event: Combined flow hydrograph, Awanui @ School Cut + Tarawhataroa @ Puriri Place.

Calibration: Conclusions

The 1-dimensional hydraulic model of the Awanui River and Whangatane Channel downstream of School Cut has been exhaustively calibrated against 3 different flood events. We believe that the resulting model calibration is a reasonable reflection of typical conditions in these channels.

Some uncertainty remains, partly due to the approximate nature of both the calibration process and the peak water levels and flow ratings upon which the calibration is based. Furthermore, Figure 1 indicates clearly that channel conditions do change from time to time. As well as directly affecting peak water levels, any such changes may well result in variation in what fraction of Awanui River flow is diverted into the Whangatane Channel.

Nevertheless, we believe that the averaged model calibration in Figure 18 to Figure 20, or a very similar set of Manning's n values agreed with Tonkin & Taylor, will be suitable for application in design runs with the model. For the present modelling, we have chosen the former.

The full model has been calibrated against rated flows in the January 2011 event, to provide a very good match of peak flow rate approaching Kaitaia (as measured by combining the flow at School Cut with calculated overflow at State Highway 1). The model has been validated against the February 2007 flood event, and on present information the calibrated hydrological parameters should be applied to the design events.

The final January 2011 calibration runs (C7/C9) have shown that the overflow across State Highway 1, and the resulting peak flow in Tarawhataroa Stream, are now well-estimated by the model, following calibration to agree with NRC's stage-overflow rating for the overflow. However, the first peak flow in the Tarawhataroa Stream, which is due to the Tarawhataroa catchment rather than Awanui overflows, is significantly too high, occurring several hours too early. If accuracy in this aspect of the model is needed, a more detailed examination of the sub-catchment will be needed.

Sensitivity tests

Seven different sensitivity tests were identified (Table 5), model runs to test how output flows and water levels vary with variations in the input parameters. These tests provide an indication of the uncertainty in model results, and also an indication of how much those results would be affected by real changes to the modelled conditions.

Table 5 Model sensitivity tests

1	Varying the rainfall or one of the catchment hydrological parameters	**Full model**	Three of the later runs carried out during model development and calibration have provided this sensitivity assessment, which is described above in the Calibration section. Even after calibration, there is uncertainty in our modelling of the hydrological response of the catchment. Increasing runoff by a plausible amount (either by increasing rainfall or decreasing infiltration) will help in assessing the resulting uncertainty in flood levels.
2	Increase downstream WLs (also represents sea level rise)	**Full model**	Useful not only for climate change but also for sensitivity to storm surge in Rangaunu Harbour. There will also be uncertainty in our modelling of the channel mouths at Rangaunu Harbour, although the effect of that on water levels is expected to be minor. The 100-year ARI full model runs with and without climate change have been accessed for this test (disregarding the relatively minor effect of the difference in rainfall between these two runs).
3	Vary Manning's n in all the main channels downstream of School Cut	Cut-down hydraulic model	A standard sensitivity test, to vary Manning's n by 20%. This might represent the uncertainty still inherent in n after calibration, or changes in n with changes in channel vegetation and bed conditions, or changes in cross-section area. As the sensitivity tests will be considered in setting freeboard, this test will be an **increase** in n.
4	Just vary Awanui main channel Manning's n downstream of the Whangatane bifurcation, perhaps for 2km.	Cut-down hydraulic model	A 20% reduction in n might be the better version of this test, as it could represent channel improvement from the overgrown state understood to have existed during the calibration events. It is an important sensitivity test, as the change upsets the balance of flows between the lower Awanui and the Whangatane Channel.

5	Just vary Manning's n for the upstream 1km of Whangatane Manning's n	Cut-down hydraulic model	A 20% increase in n could represent the effect of overlooking channel maintenance. Similar to varying n in the Awanui only, the change upsets the balance of flows between the lower Awanui and the Whangatane Channel, but the degree of this effect may be different.
6	Varying descriptions of the overflow at State Highway 1	**Full model**	The Awanui flow of 180m^3/s has been identified as the threshold of overflow over SH1. It is the amount of overflow above this Awanui flow that is of interest, and is somewhat uncertain because the Puriri Place rating relies on gaugings at relatively low flows. Runs C5 and C6, carried out during model development and calibration, have provided this sensitivity assessment, which is described above in the Calibration section.
7	Vary the stage-flow rating on the Whangatane Channel @ Donald Road	Cut-down hydraulic model	There is some doubt about the best stage-flow rating to apply at Donald Road, and the two alternatives were both trialled. The best comparison is between runs where the model has been calibrated to one rating or the other; interest then lies in the differences not only between the resulting flow hydrographs but also between the two sets of calibrated Manning's n values.

Sensitivity tests 1 and 6 are comparison of different calibration runs and are described above. Test 2 is a comparison of two model runs with design events. Tests 2-5 and 7 are presented here.

Increase in downstream (Rangaunu Harbour) sea levels

The sensitivity of model output to sea level rise has been assessed from design scenario results. The 100-year peak water levels in downstream reaches of the Awanui River, modelled with and without climate change, are compared in Figure 31 , and those for the Whangatane Channel in Figure 32. Modelled climate change includes both increased rainfall and increased sea levels. However, a trial pair of model runs were available in which increased sea level was omitted from the climate change scenario, and these runs have been compared with Figure 31 and Figure 32 to estimate the limit of influence of increased sea level.

For all practical purposes, the effect of increased sea level in the Awanui River is felt roughly as far upstream as State Highway 1, 8.67km upstream of Unahi Wharf, where the difference in peak water level attributable to sea level rise is less than 100mm. An increasingly minor effect is felt further upstream as far as Gill Road.

In the Whangatane Channel, there is a more clear-cut limit to this sea-level effect at about chainage 8500m, which is the downstream end of the straight excavated channel, 1500m downstream of State Highway 10 and about 3.4km upstream of the tidal confluence with Mangatete River.

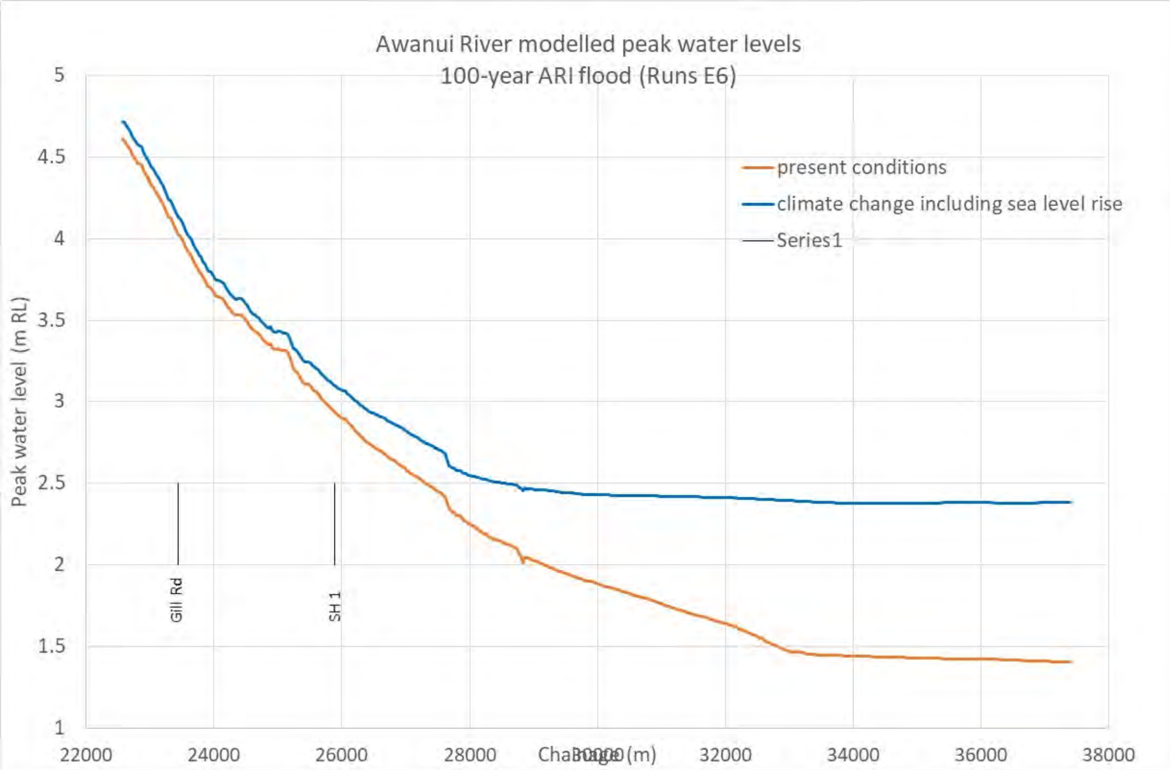


Figure 31 Effect of climate change including sea level on peak water levels in lower Awanui River, 100-year flood

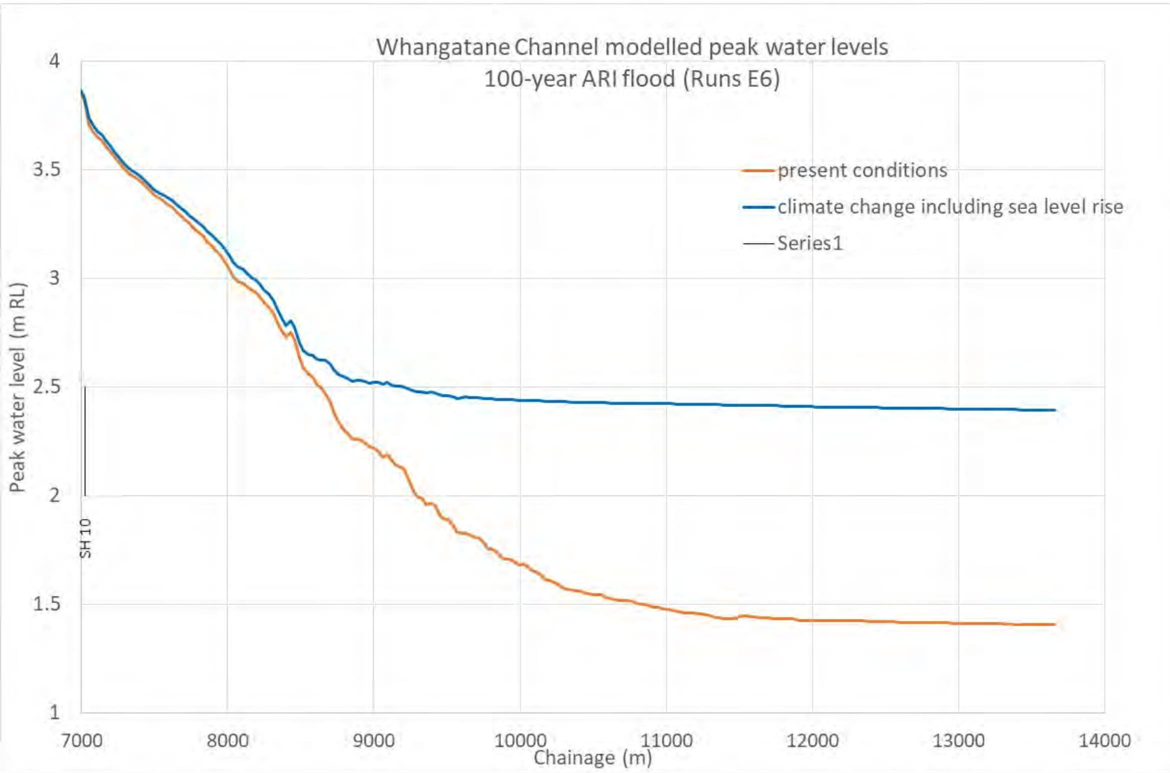


Figure 32 Effect of climate change including sea level on peak water levels in lower Whangatane Channel, 100-year flood

Locations downstream of these chainages, including flooded land, are to varying degrees vulnerable to sea level rise and to storm surge in Rangaunu Harbour.

Variation in flow resistance (Manning’s n): Tests 3-5

In these three tests Manning’s n has been varied from the “base case” by 20%. Figure 33 and Figure 34 plot the difference these changes have on peak water levels, and the hydrographs in Figure 35 shows the effect on the distribution of flow between the Awanui River and the Whangatane Channel.

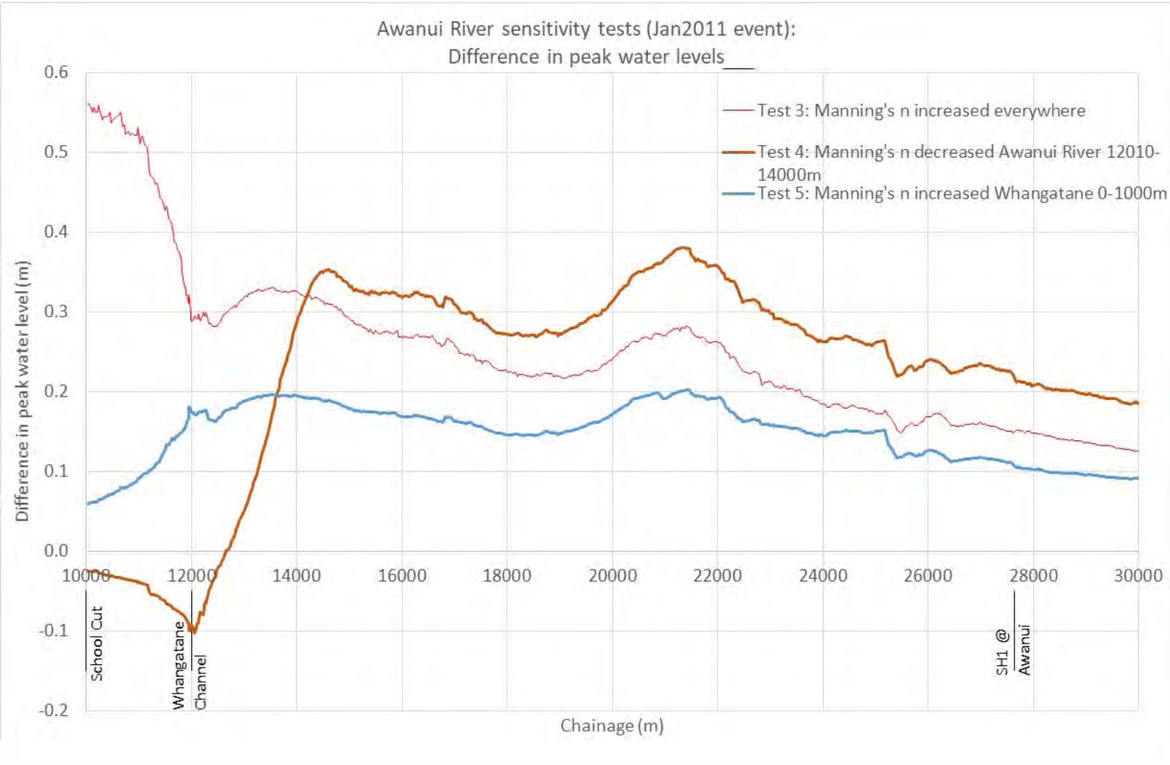


Figure 33 Awanui River peak water level differences, January 2011, Sensitivity tests 3-5

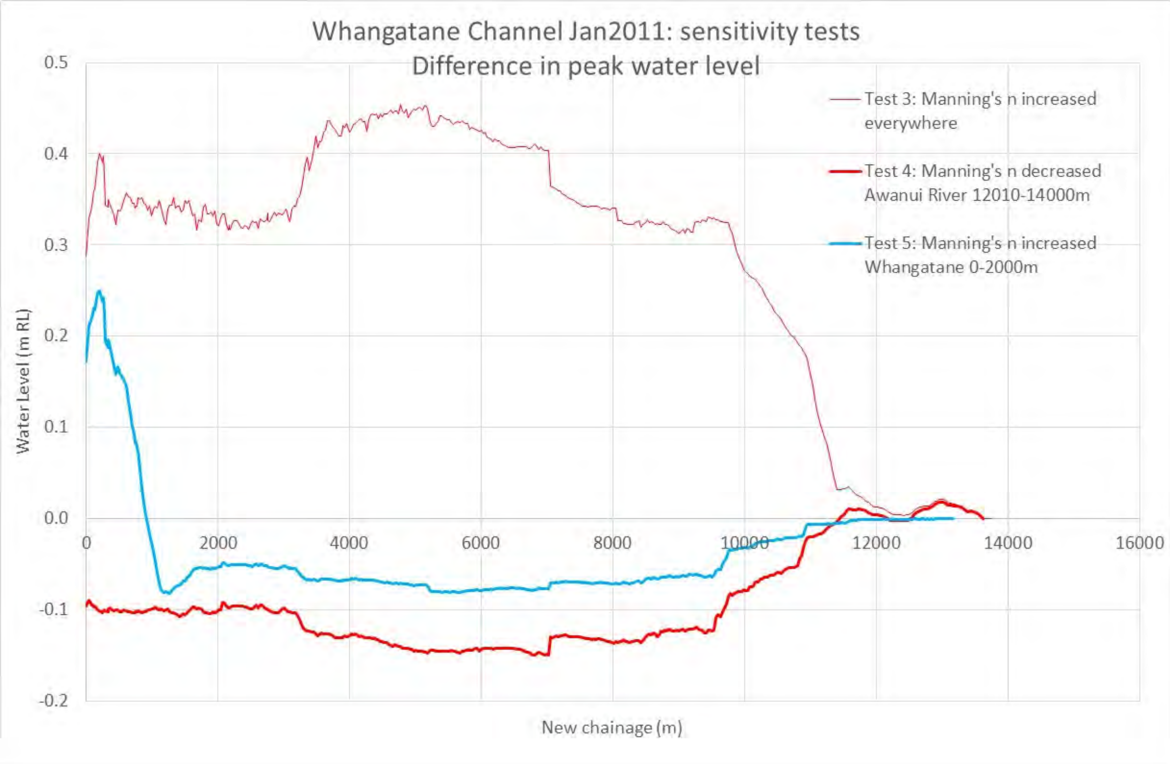


Figure 34 Whangatane Channel peak water level differences, January 2011, Sensitivity tests 3-5

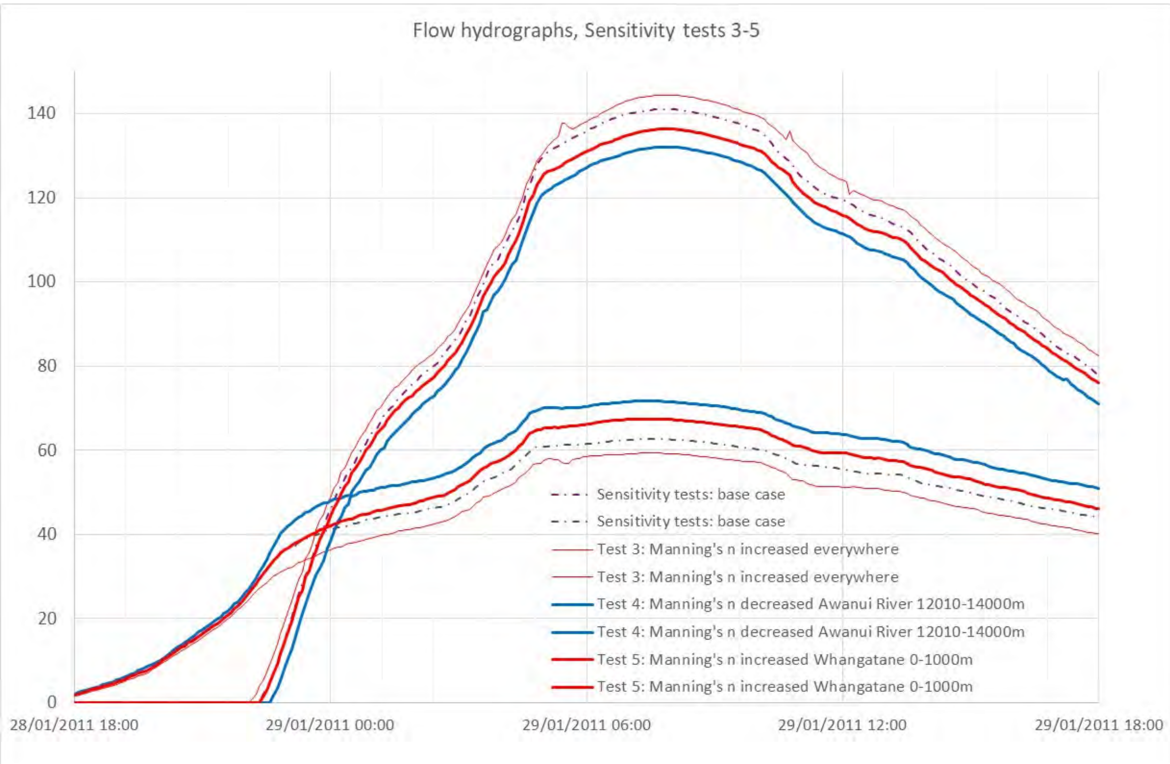


Figure 35 Flow hydrographs downstream of the Whangatane bifurcation, January 2011 event, sensitivity tests 3-5. The higher flow peaks are in the Whangatane Channel, the lower flow peaks in the Awanui River.

With a general increase in flow resistance (Test 3) peak water levels are higher everywhere. However, the effect is not the same everywhere: in the Awanui 0.5m in the Awanui upstream of the bifurcation, 0.3m immediately downstream and decreasing gradually with distance downstream; 0.3-0.4 m in the Whangatane Channel. Slightly more flow is diverted into the Whangatane Channel.

In Test 4, flow resistance is reduced in the Awanui River immediately downstream of the bifurcation, whereas in Test 5 flow resistance is increased in the Whangatane Channel immediately downstream of the bifurcation. Both Test 4 and Test 5 therefore result in less diversion of flow into the Whangatane Channel, and so a modest reduction in peak water levels along most of its length. There are higher flows and therefore higher peak water levels in the Awanui downstream of the bifurcation. In Test 4, because of this effect and in spite of the reduced flow resistance, peak water levels are increased within chainages 12500m-14000m.

These tests, particularly Test 4, show the challenge in getting the desired outcome when managing river channels near a bifurcation.

Rating curve uncertainty (Sensitivity Test 7)

The two alternative ratings that have been proposed for the Whangatane Channel at Donald Road are compared in Figure 36. These ratings diverge significantly only above the highest flow gauging of 94 m^3/s, but result in a difference of about 13% in rated peak flow for the January 2011 event.

Figure 37 compares the alternative rated hydrographs with the model runs that came closest to matching each rated hydrograph. Both model runs gave peak water levels (not presented) that were reasonably close to the observed debris levels but not a perfect match. In particular, Run 190207h did not achieve an ideal match with either Rating B or observed peak water levels observed from debris lines, but is close enough to doing so to demonstrate the sensitivity of the solution to the choice of rating.

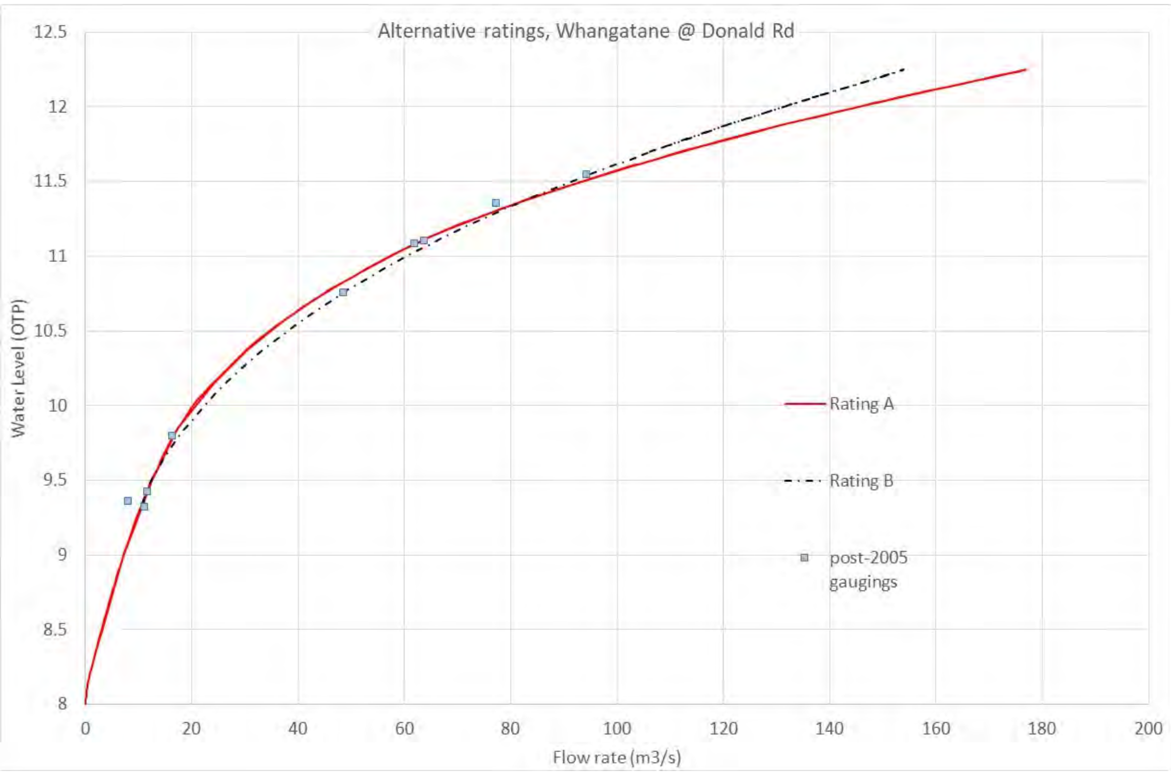


Figure 36 Alternative ratings for Whangatane Channel at Donald Road. Rating A was adopted for model calibration.

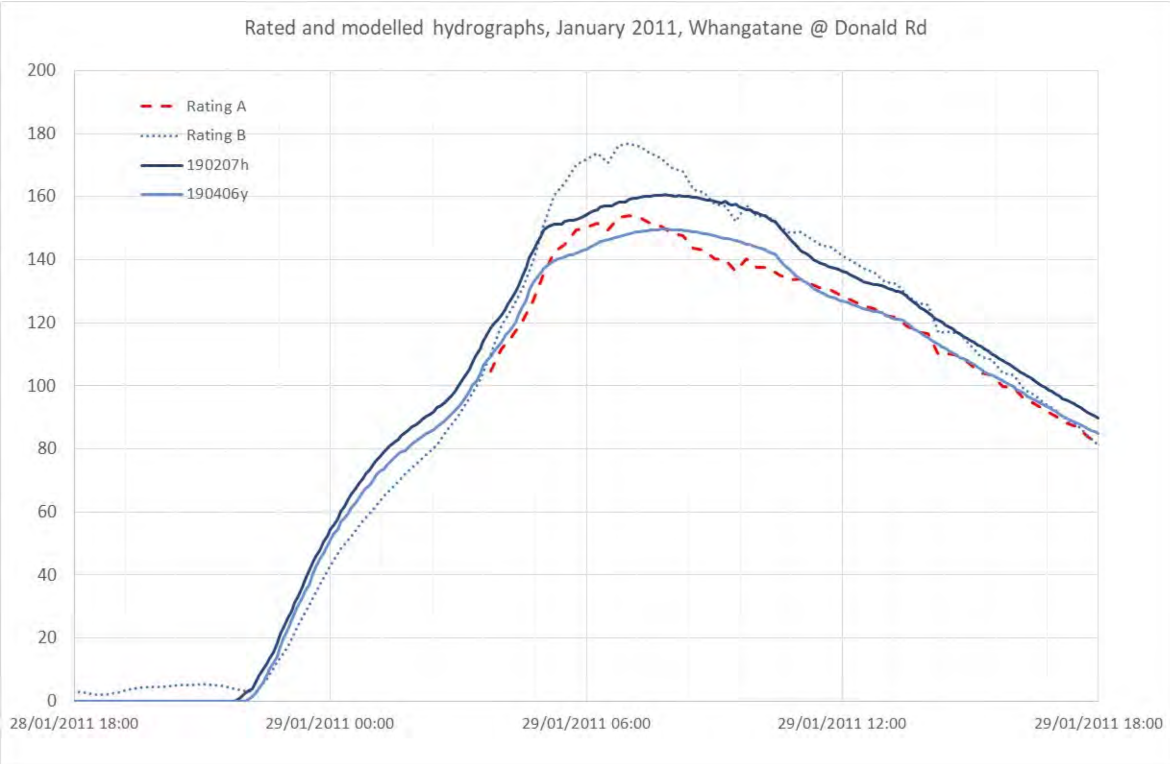


Figure 37 Modelled and rated hydrographs, January 2011, Whangatane @ Donald Rd, alternative ratings

The Manning's n values modelled in the two runs are graphed in Figure 38, for relevant reaches of both channels. To match the higher Whangatane flows determined by Rating B, significantly lower n vales were adopted in the upstream 2km of the Whangatane Channel, and lower n values in the adjacent Awanui River. When sensitivity tests 4 and 5 are also considered, it becomes clear that the choice of rating can have a significant effect on model calibration and/or on flooding predictions in both channels further downstream.

Following discussion of the ratings and their derivation, NRC nominated Rating A to be adopted for model calibration and deployment. It is understood that derivation of both ratings followed established best practice, but with different choices of representative cross-section. Although the choice of Rating A is believed to be a sound one, and has resulted in very plausible hydrographs, this particular rating therefore appears to warrant further investigation at some stage.

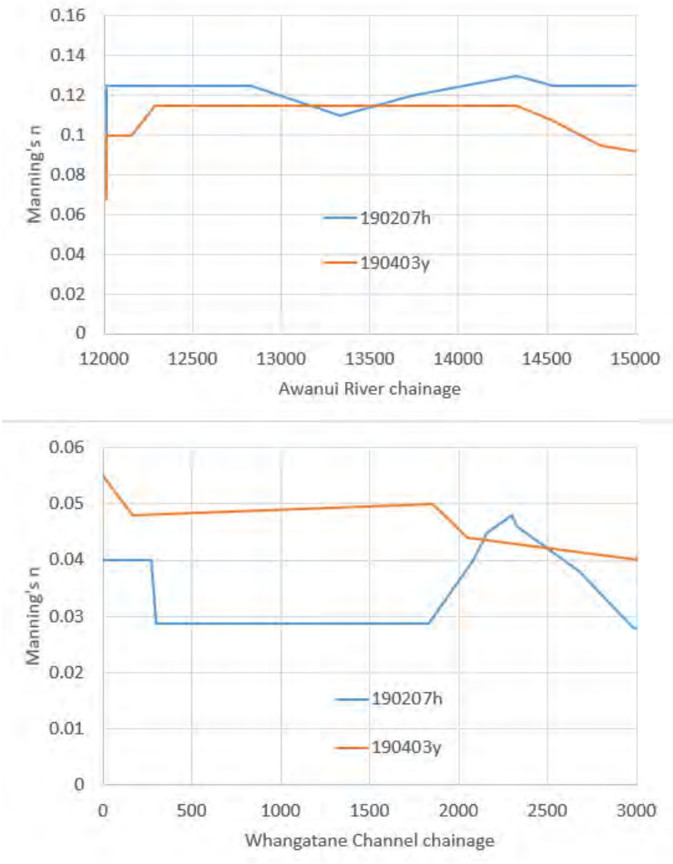


Figure 38 Manning's n values calibrated for Rating A (Run 190207h) and Rating B (Run 190403y)

APPENDIX C–Design Rainfall

Detail on the derivation of Design Rainfall

C Design Rainfall

C.1 Derivation of design rainfall depths from long-term rain gauge sites 1

C.1.1 Preliminary

The recently-available version 4 of HIRDS was applied to obtain rainfall depths in the Awanui catchment. These were applied in preliminary modelling of a 100-year ARI 24-hour rainfall event in the Awanui catchment. This produced a peak modelled flow approaching Kaitaia of 800 m^3/s. This is nearly double recent estimates of the 100-year ARI flow obtained from rated flow records; these estimated flows being 500 m^3/s.

It has become apparent that the rainfall depths produced by HIRDS include values between the automatic gauges that are much higher than the depths at the gauges (about 50% in places). This appears to be due to the analysis method used to derive the HIRDS v4 depths. From e-mail correspondence with NIWA, and from informal discussion with other users of HIRDS, it appears that the HIRDS data incorporate a variation with altitude that is determined from the available rain gauges.

In some locations the variation with altitude might not be realistic. Perhaps more important, this variation was not applied in the present study to historical events including the January 2011 event used for hydrological calibration. Applying the variation with altitude to the design events may therefore be inconsistent with the calibrated model.

C.1.2 Data Analysis: Three automatic rainfall gauges

To address this apparent anomaly, a standard analysis was first carried out of the rainfall data from the three automatic rain gauges within the Awanui catchment upstream of Kaitaia (Table C 1) These were processed to produce 24-hour and 36-hour rainfall depths. These depths were then subject to an extreme value analysis using DHI's software EVA; three statistical distributions were tested: Gumbel, log-normal and GEV. The graphical output from EVA is shown in Figure C 1 to Figure C 3 (24-hour depths) and Figure C 4 to Figure C 6 (36-hour depths), in which individual annual maxima are shown as black dots, and the three statistical estimates of the depth-frequency curve are shown as solid lines.

Table C 1: Rain gauges sites covered by this analysis

Gauge	sub-catchment	NZTM E	NZTM N
Kaitaia EWS (observatory) (MET)	Tarawhataroa / Tangonge	1623959	6112108
Te Rore NRC	Takahue	1633900	6107090
Mangakawakawa Trig NRC	Te Puhi	1641523	6110696

Rainfall depth-frequency estimates provided by NIWA's web-based method HIRDS (both version 3 and version 4) were extracted for the same three sites. HIRDS v3 estimates for the 36-hour depths were not available, and have therefore been estimated from the 24-hour depths (using the ration between 24-hour and 36-hour depths found in the version 4 estimates). These data have been manually over-plotted onto Figure C 1 to Figure C 6 for return periods of 10, 20, 50 and 100 years.

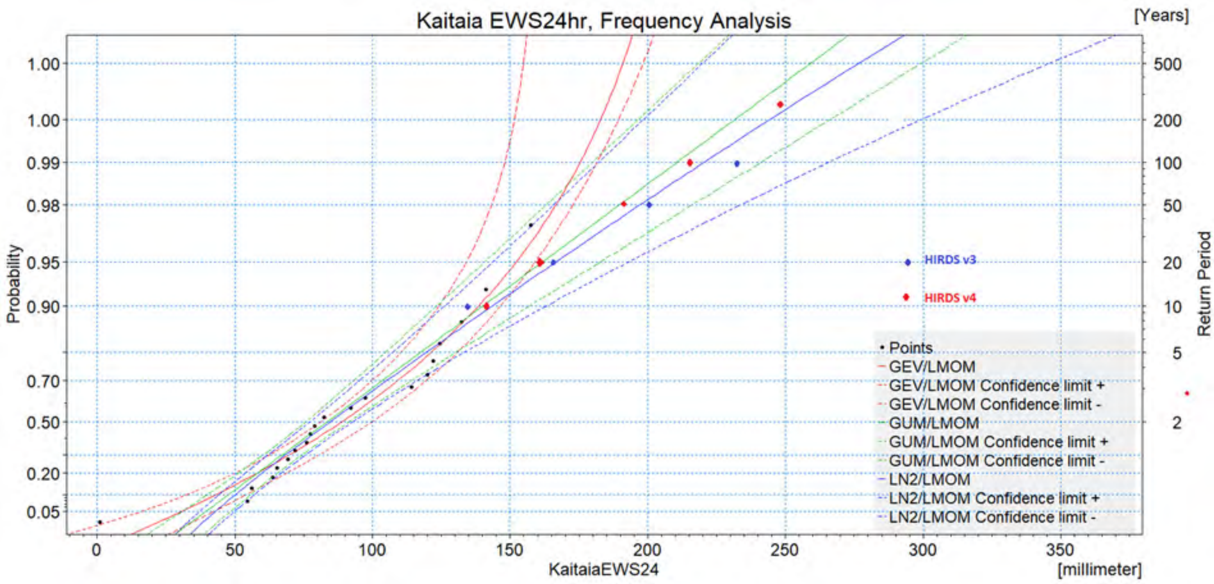


Figure C 1: Extreme value analyses for 24-hour rainfall depths, Kaitaia EWS

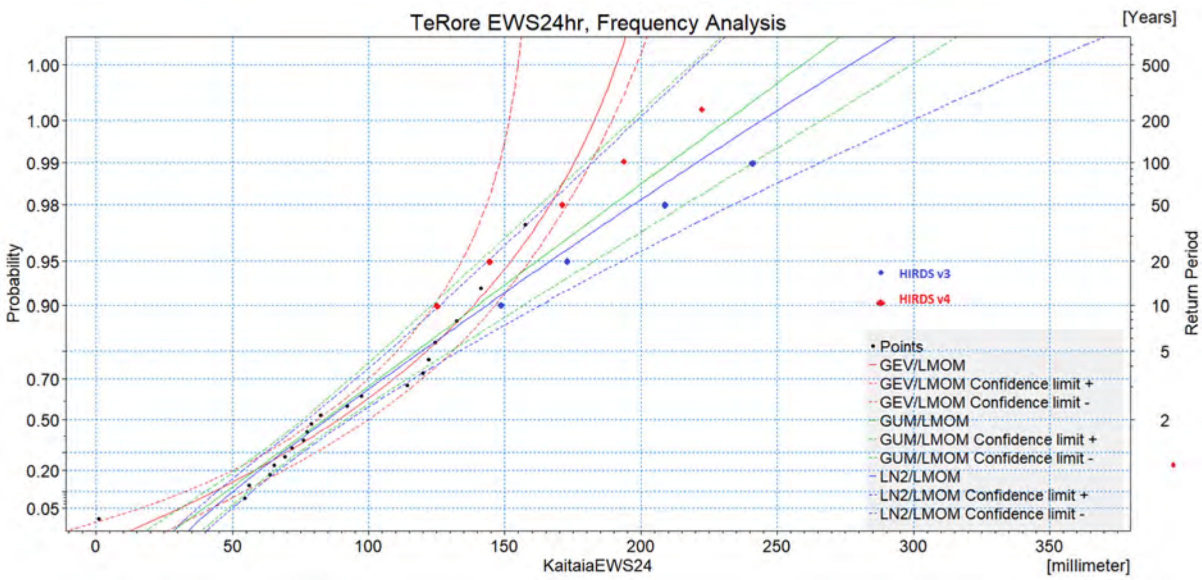


Figure C 2: Extreme value analyses for 24-hour rainfall depths, Takahue @ Te Rore

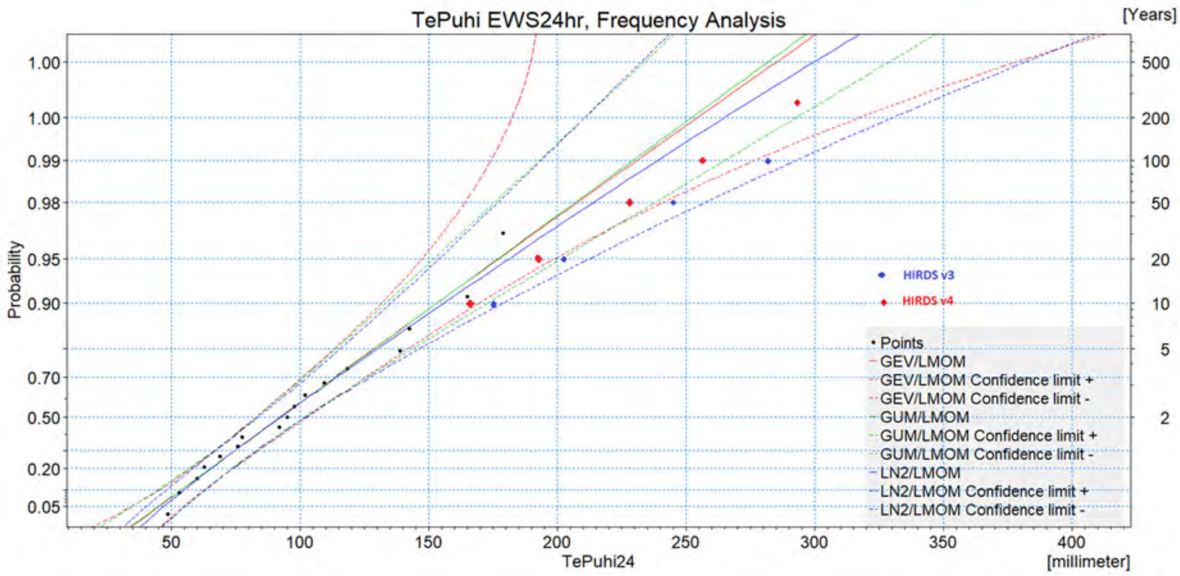


Figure C 3: Extreme value analyses for 24-hour rainfall depths, Te Puhi @ Mangakawakawa Trig

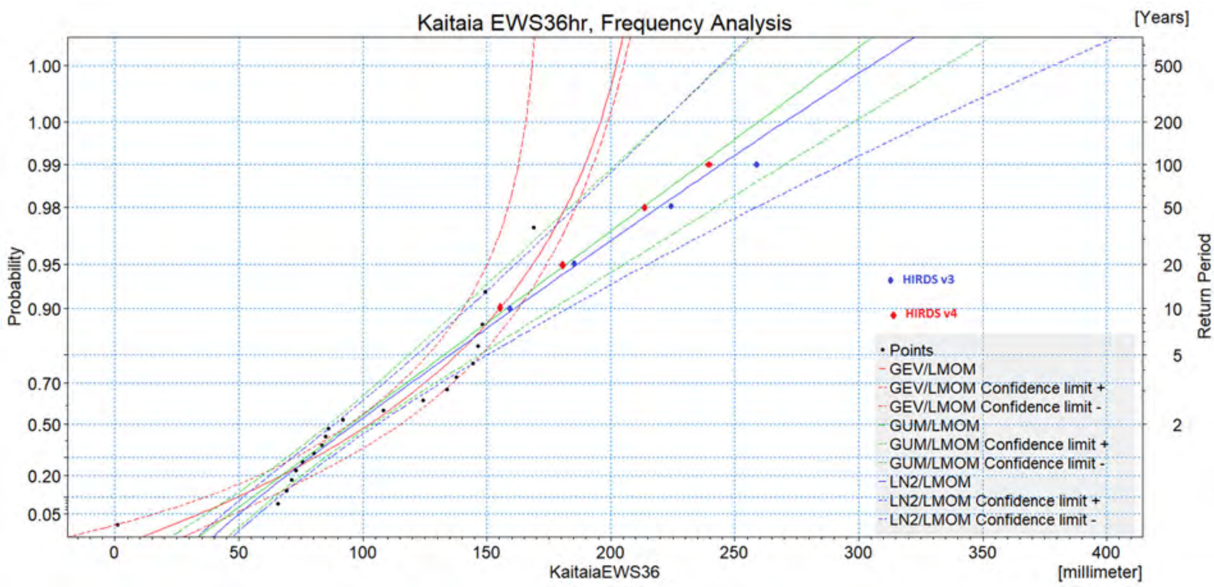


Figure C 4: Extreme value analyses for 36-hour rainfall depths, Kaitaia EWS

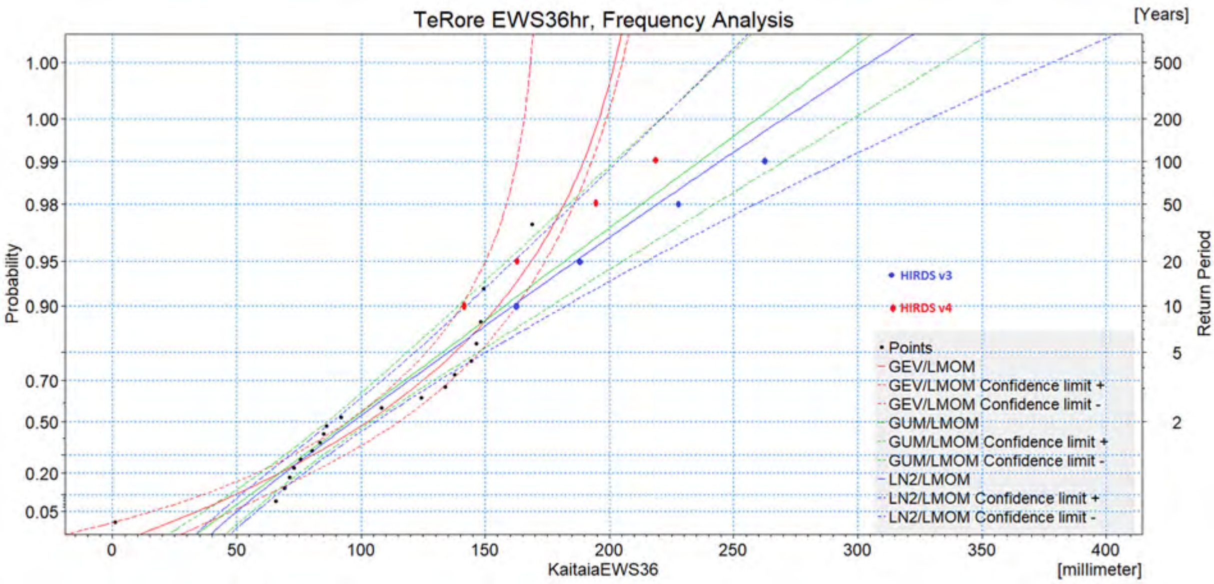


Figure C 5: Extreme value analyses for 36-hour rainfall depths, Takahue @ Te Rore

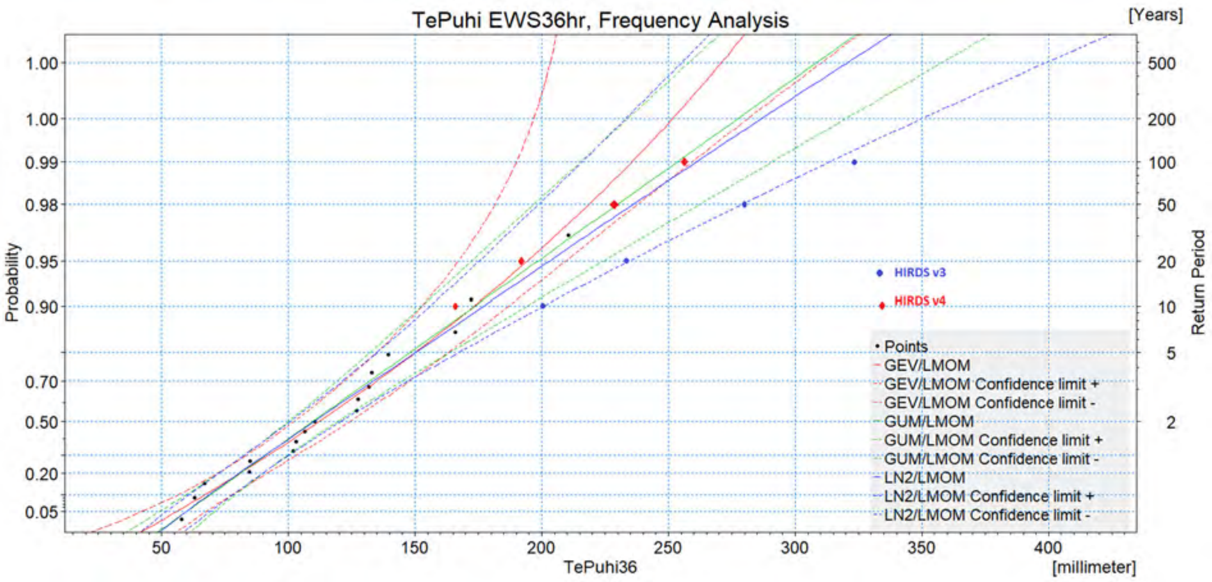


Figure C 6: Extreme value analyses for 36-hour rainfall depths, Te Puhi @ Mangakawakawa Trig

C.1.2.1 Assessment of Figure 1 to Figure 6

All the plots show that both versions of HIRDS produce data for these three sites that plot reasonably consistently with the Gumbel and log-normal distributions, but not generally agree as well with the GEV distribution, which from inspection generally represents the annual maxima better.

The HIRDS version 4 estimated depths are generally lower than the HIRDS version 3 equivalents.

For the Te Puhi site, the GEV distribution produces close to a straight-line frequency distribution, comparable to the other two distributions. However, this is not true for the Te Rore and Kaitaia EWS gauges, where the GEV differs significantly from the Gumbel distribution, indicating an EV2 distribution. For the 100-year event, the GEV produces lower estimated depths for all three sites (Table C 2).

Table C 2: Estimates of 100-year ARI 24-hour rainfall depths

	HIRDS v3	HIRDS v4	GEV (present analysis)	*GEV /HIRDS v4*
Kaitaia EWS	232	215	175	81%
Te Rore	241	193	175	91%
Te Puhi	282	256	230	90%

C.1.3 Data Analysis: other rainfall gauges including daily-read gauges

From consideration of the various rainfall depth estimates in Table C 2, it was agreed with NRC that the design rainfall depths should be derived directly from all the rain gauge records with a reasonably long record. This included six daily-read gauges, some with very long records.

24-hour rainfall depths for these sites were analysed using DHI's software EVA. For the daily-read gauges, a factor of 1.13 was then applied to correct for their records containing only depths falling between 9am one day and 9am the next. This factor, a standard correction, was confirmed for the Awanui catchment from analysis of two of the automatic rain gauges sites.

The graphs produced by EVA are included below as Section C.2. A somewhat subjective decision was made about which frequency distribution to apply at each site. The GEV was preferred where it appeared more realistic than the Gumbel and log-normal distributions, but at some sites an average was used of the depths derived with the three different distributions.

The resulting rainfall depths are set out in Table C 3. In this table, the "Kaitaia" location is midway between two gauge sites 1.9km apart. Design rainfall hyetographs for individual sub-catchments have then been obtained by applying Thiessen polygons centred on these 10 locations

Table C 3: Design rainfall depths at rain gauges sites, for ARIs of 10-100 years

	Easting	Northing	10	20	50	100
Waipapakauri	1622170	6123709	122	130	141	147
Kaingaroa North	1630827	6121914	121	136	151	164
Aero Kaitaia	1626174	6119483	131	148	169	186
Kaitaia	1624200	6113300	138	158	176	190
Rangitihi	1630515	6113261	130	147	164	179
Pukepoto	1618417	6108950	142	165	212	237
Te Rore	1633900	6107090	139	155	174	187
Takahue top	1633179	6103238	149	174	212	247
Victoria	1637333	6110648	147	170	202	229
Te Puhi	1641523	6110696	153	177	207	230

C.1.4 Design hyetograph shape

Following a preliminary model run with the 100-year 24-hour event, it was also decided in consultation with NRC to replace the NRC Priority Rivers design hyetograph shape with that specified by HIRDS version 4. Both of these hyetograph shapes are shown in Figure C 7. The hyetograph change reduces peak flows, notably peak flows approaching Kaitaia.

C.1.5 Subsequent adjustment: validation against peak flow frequency analysis

With the two above changes to the assumed 100-year rainfall (use of all available rain gauges and the HIRDS hyetograph) the modelled peak flow arriving at Kaitaia was about 430 m3/s. This flow is comparable to rated peak flows from the largest recorded events, and therefore appears to be too low for an ARI of 100 years.

Rainfall depths for sub-catchments upstream of Kaitaia were therefore increased by 10%, an amount considered to be within the uncertainty of the estimated depths. The 'stations' that were modified were Rangitihi, Takahue top, Te Puhi, Te Rore and Victoria. With this adjustment, the peak flow approaching Kaitaia is approximately 502 m^3/s, comprising 266 m^3/s in the Awanui River at School Cut and 236 m^3/s overflowing State Highway 1. This peak flow approximately equals the average of various estimates obtained from the frequency analysis of annual maxima. This distribution of rainfall depths has therefore been chosen for subsequent modelling of the 100-year event.

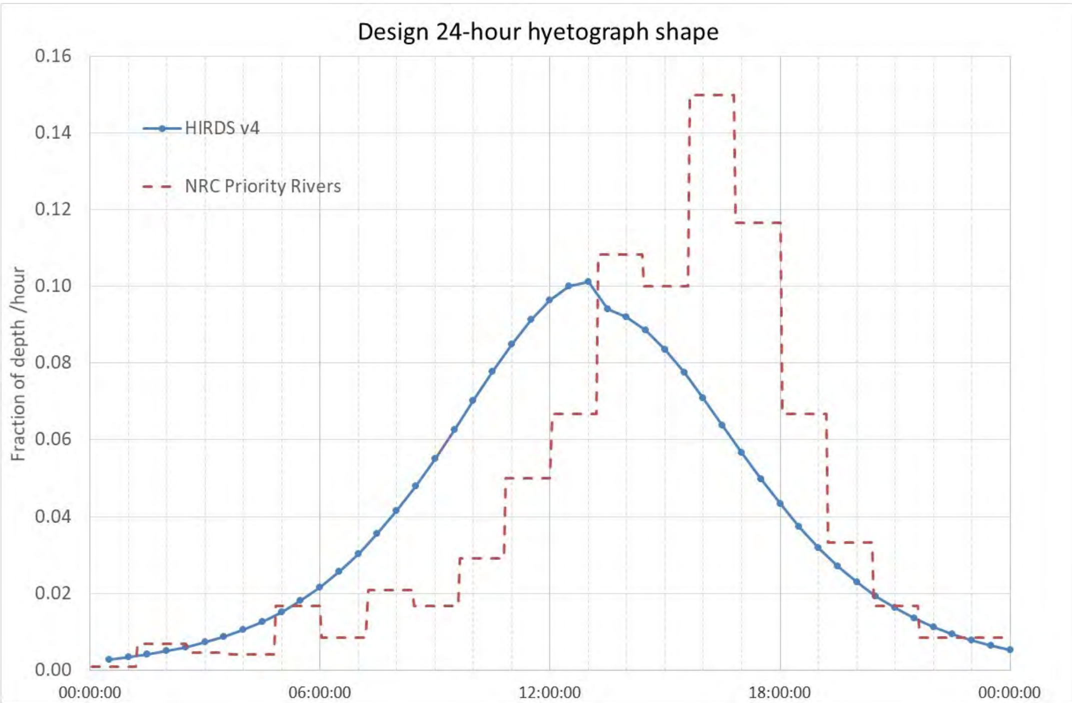


Figure C 7: Design 24-hour hyetograph shapes. The HIRDS v4 shape has been adopted for the Awanui design events.

C.1.6 12-hour event

Following consultation on alternative rainfall events, NRC decided that a 12-hour event should also be modelled. The rationale is that it is likely that some parts of the Awanui catchment – headwaters and sub-catchments in particular – may experience more flooding from the 12-hour event than the 24-hour event.

HIRDS rainfall predictions for the automatic rain gauge sites were therefore accessed to obtain the 12-hour depths as a percentage of 24-hour depths. This percentage is quite consistent between gauges, and comparable values were therefore adopted for the other gauge sites to obtain estimated 12-hour rainfall depths. These rainfall depths are presented in Table C 4, with the left-hand column containing the percentages used to scale them from the 24-hour depths.

Table C 4: Design rainfall depths at rain gauges sites, for ARIs of 10-100 years

% of 24-hour depth		Easting	Northing	10	20	50	100
82.7%	Waipapakauri	1622170	6123709	101	107	117	122
82.7%	Kaingaroa North	1630827	6121914	100	112	125	136
82.7%	Aero Kaitaia	1626174	6119483	108	122	140	154
81.3%	Kaitaia	1624200	6113300	112	128	143	154
81.3%	Rangitihi	1630515	6113261	106	120	133	146
81.3%	Pukepoto	1618417	6108950	115	134	172	193
80.3%	Te Rore	1633900	6107090	112	124	140	150
80.3%	Takahue top	1633179	6103238	120	140	170	198
80.3%	Victoria	1637333	6110648	118	137	162	184
77.7%	Te Puhi	1641523	6110696	119	138	161	179

Only the 100-year event has been modelled. For consistency with treatment of the 24-hour event, the HIRDS version 4 hyetograph (Figure C 8) was applied. The modelling results showed the risk from the 12-hour event to be either less than or comparable to that from the 24-hour event, depending on location. NRC therefore decided that the modelling of design cases need include only the 24-hour event.

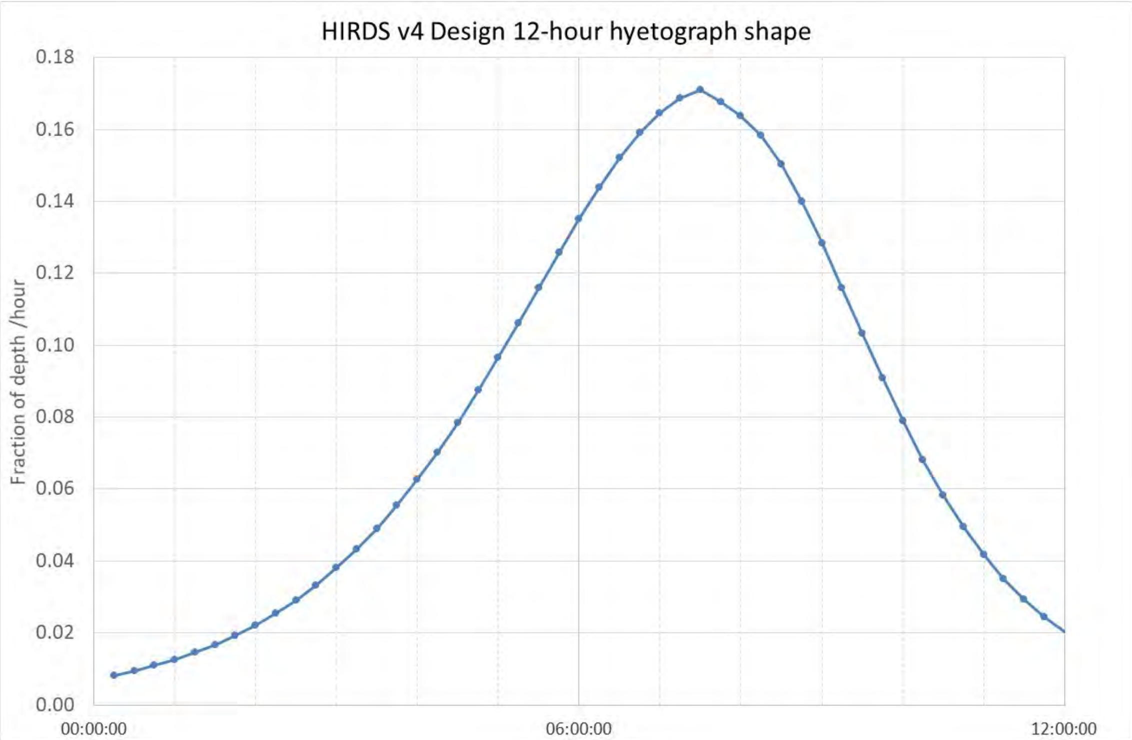
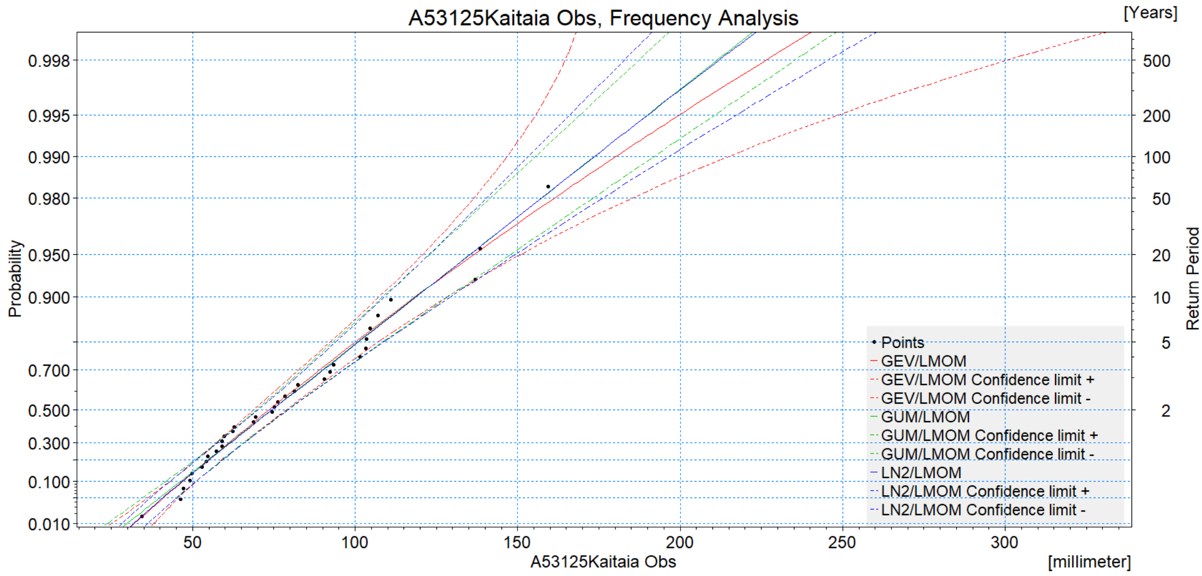
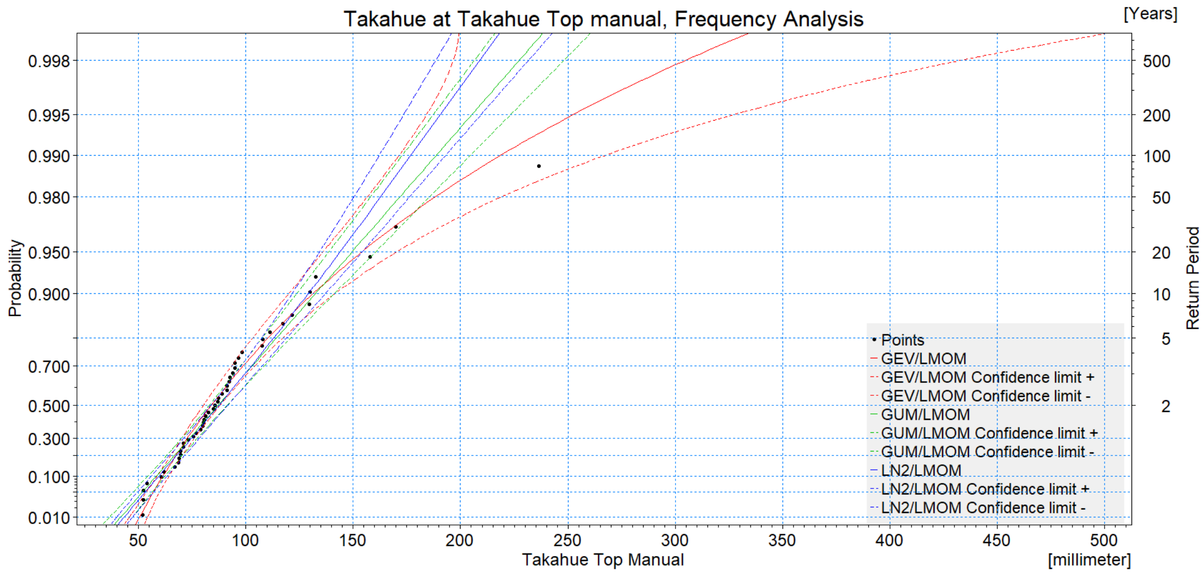
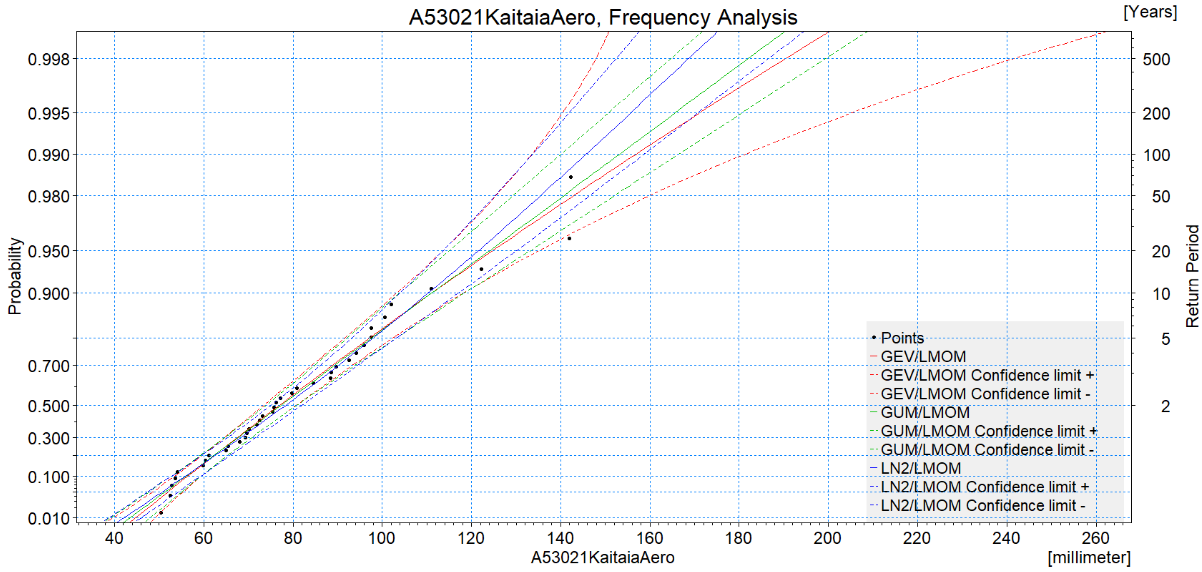


Figure C 8: HIRDS version 4 design 12-hour hyetograph shape

C.2 Graphical output from EVA:
Analysis of Awanui catchment rain gauge records



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Return Period
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Points
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LN2/LMOM
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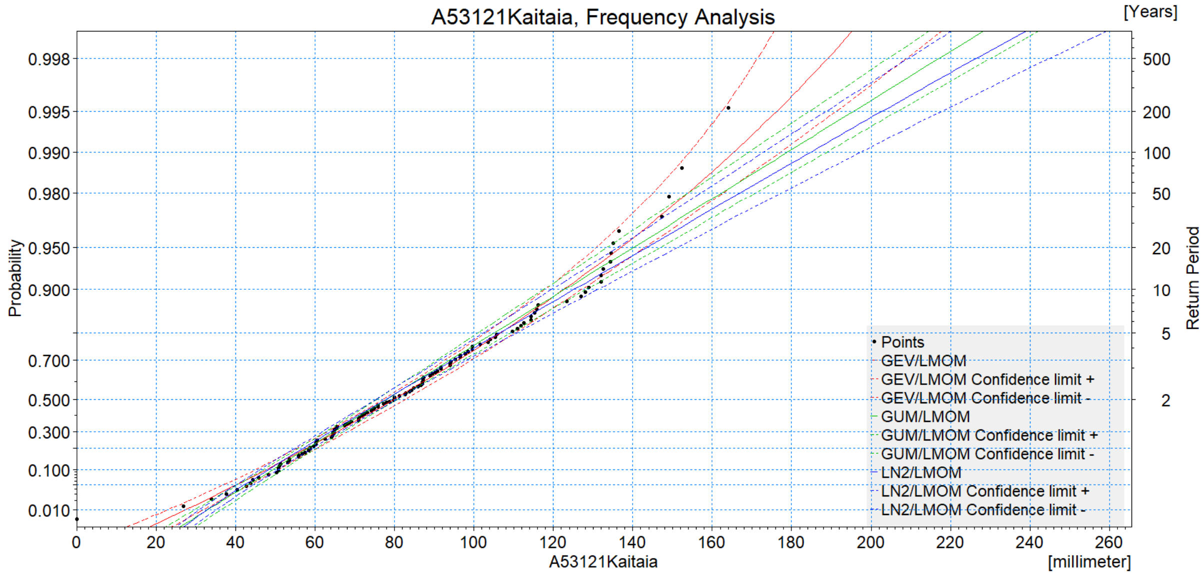
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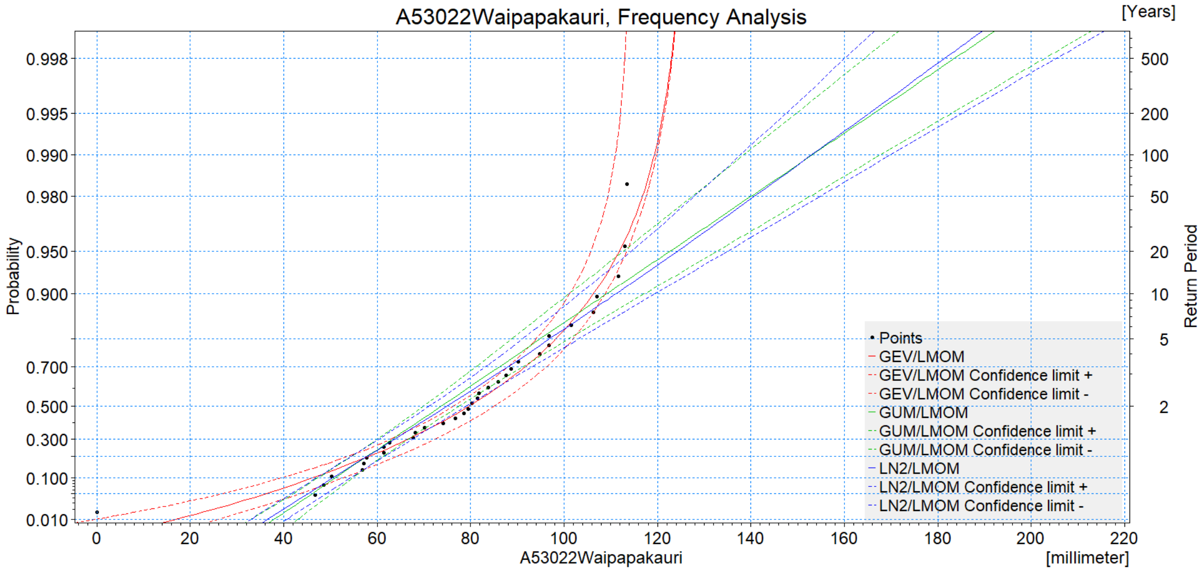

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Probability
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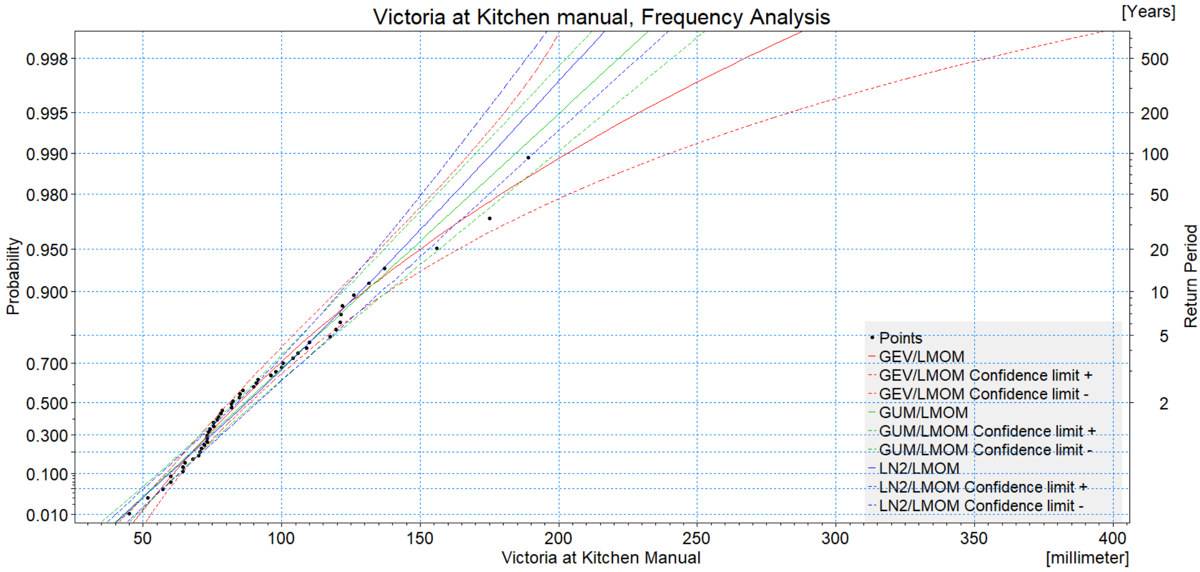
0.700
0.500
0.300
0.100
0.010
Return Period
500
200
100
50
20
10
5
2
0
20
40
60
80
100
120
140
160
180
200
220
A53022Waipapakauri
[millimeter]
Points
GEV/LMOM
GEV/LMOM Confidence limit +
GEV/LMOM Confidence limit -
GUM/LMOM
GUM/LMOM Confidence limit +
GUM/LMOM Confidence limit -
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -



Victoria at Kitchen manual, Frequency Analysis
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Probability
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0.900
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0.300
0.100
0.010
Return Period
500
200
100
50
20
10
5
2
50
100
150
200
250
300
350
400
Victoria at Kitchen Manual
[millimeter]
Points
GEV/LMOM
GEV/LMOM Confidence limit +
GEV/LMOM Confidence limit -
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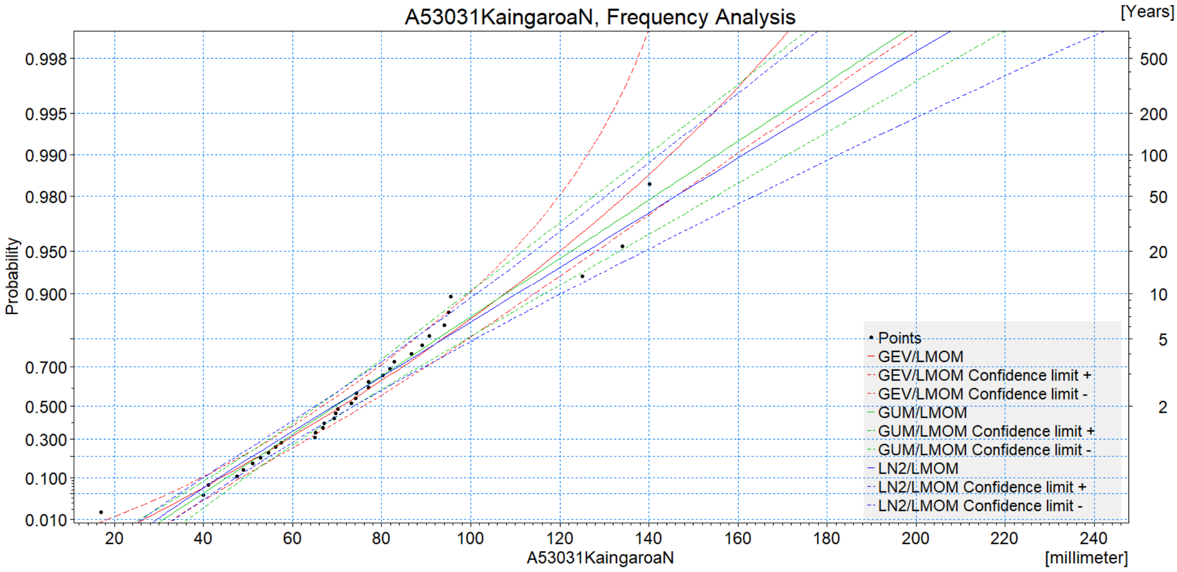
GUM/LMOM Confidence limit +
GUM/LMOM Confidence limit -
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -


A53031KaingaroaN, Frequency Analysis
[Years]
Probability
Return Period
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0.990
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0.100
0.010
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2
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80
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140
160
180
200
220
240
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[millimeter]
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GEV/LMOM Confidence limit -
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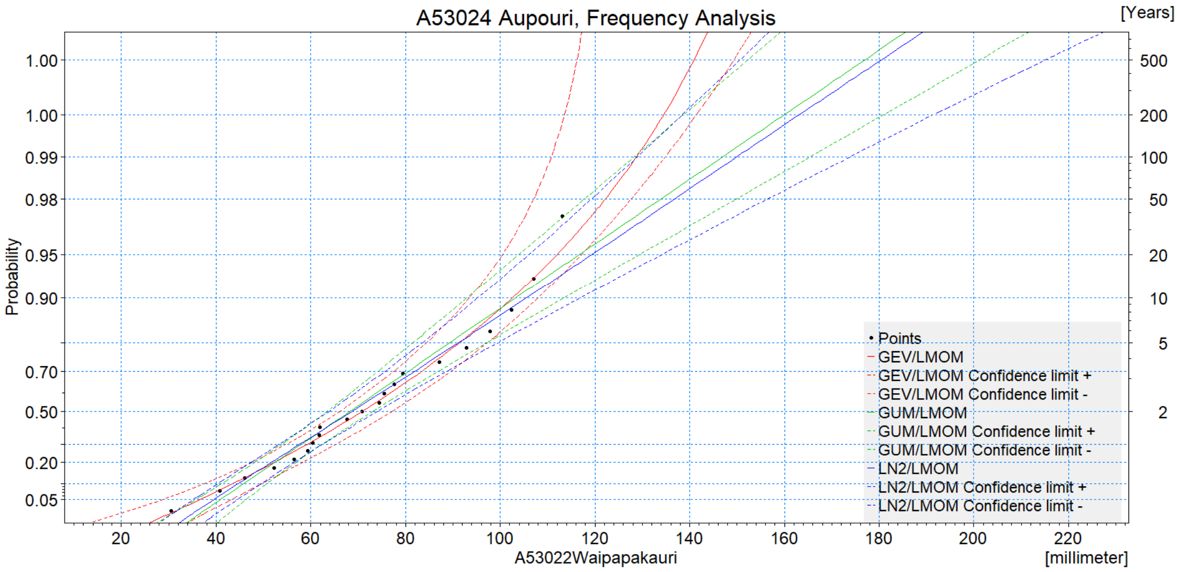
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -


A53024 Aupouri, Frequency Analysis
[Years]
Probability
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160
180
200
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A53022Waipapakauri
[millimeter]
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GEV/LMOM Confidence limit +
GEV/LMOM Confidence limit -
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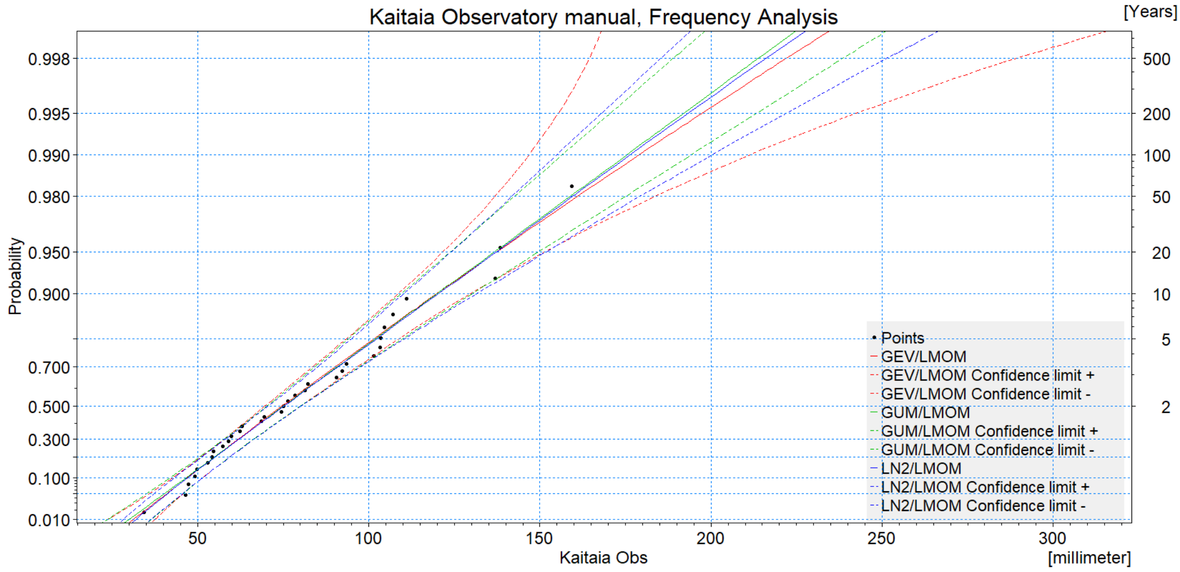
GUM/LMOM Confidence limit +
GUM/LMOM Confidence limit -
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -


Kaitaia Observatory manual, Frequency Analysis
[Years]
Probability
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0.990
0.980
0.950
0.900
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0.300
0.100
0.010
Return Period
500
200
100
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20
10
5
2
50
100
150
200
250
300
Kaitaia Obs
[millimeter]
Points
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GEV/LMOM Confidence limit +
GEV/LMOM Confidence limit -
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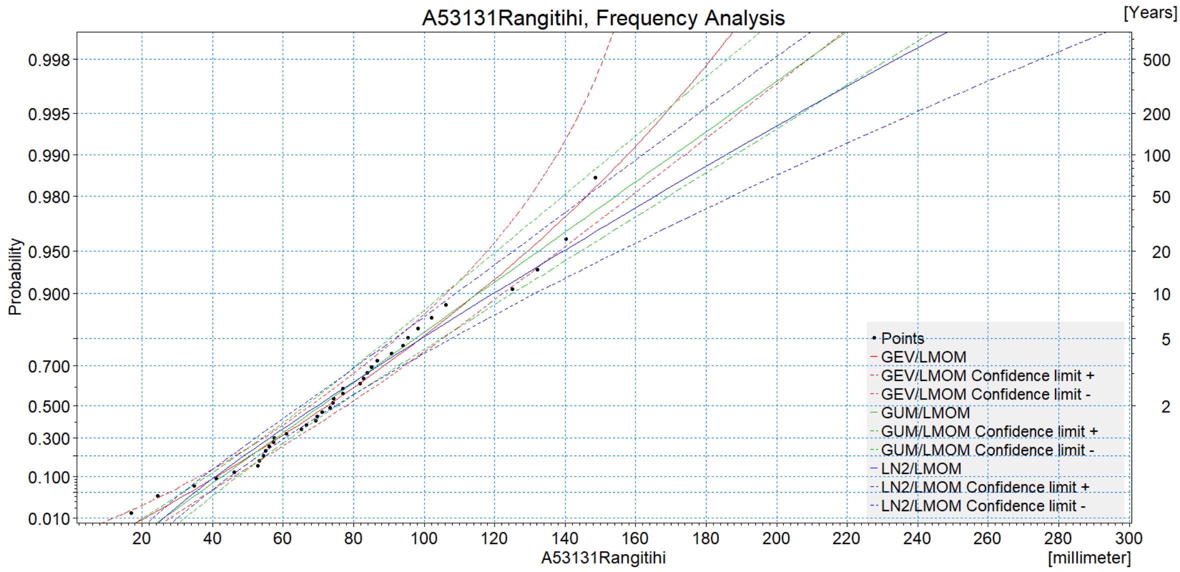
GUM/LMOM Confidence limit +
GUM/LMOM Confidence limit -
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -


A53131Rangitihi, Frequency Analysis
[Years]
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0.010
Return Period
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[millimeter]
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GEV/LMOM Confidence limit -
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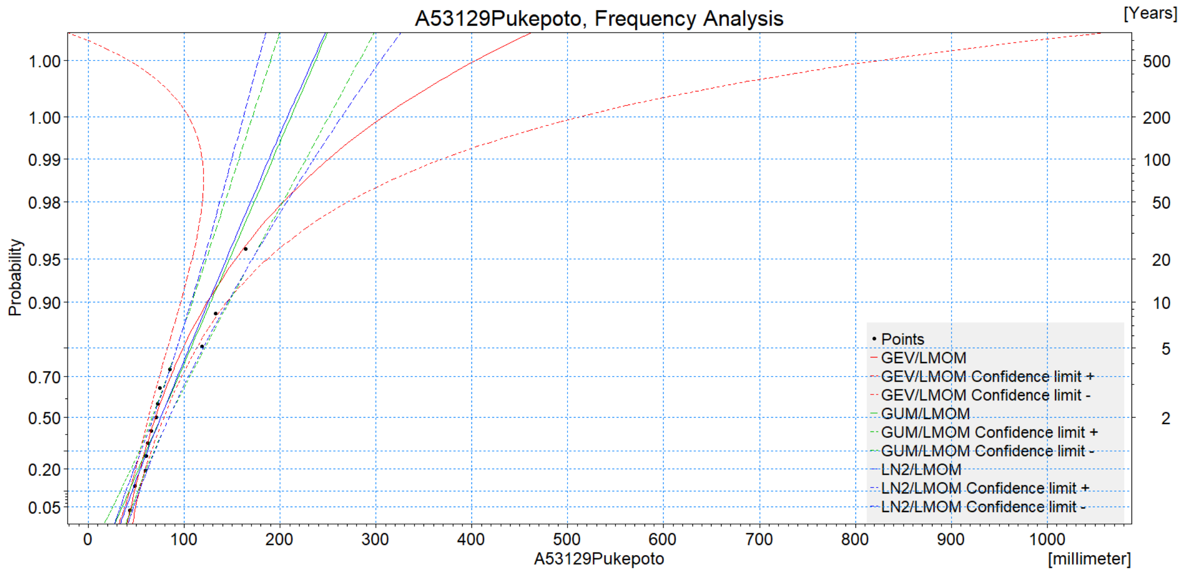
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -


A53129Pukepoto, Frequency Analysis
[Years]
Probability
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0.20
0.05
Return Period
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20
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5
2
0
100
200
300
400
500
600
700
800
900
1000
A53129Pukepoto
[millimeter]
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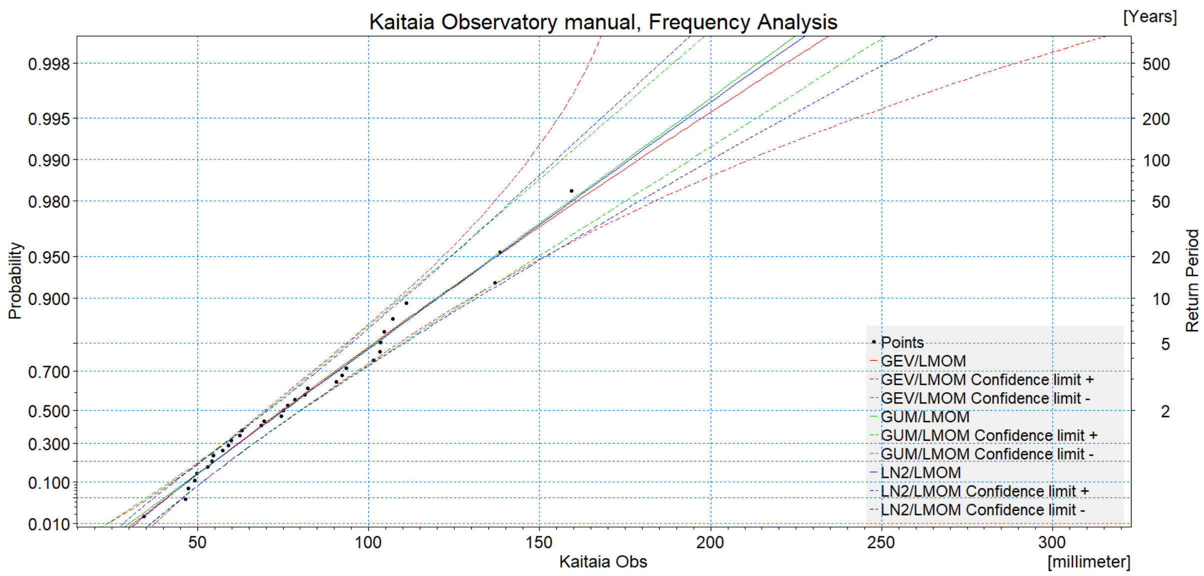
GEV/LMOM Confidence limit +
GEV/LMOM Confidence limit -
GUM/LMOM
GUM/LMOM Confidence limit +
GUM/LMOM Confidence limit -
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -


Kaitaia Observatory manual, Frequency Analysis
[Years]
Probability
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0.980
0.950
0.900
0.700
0.500
0.300
0.100
0.010
500
200
100
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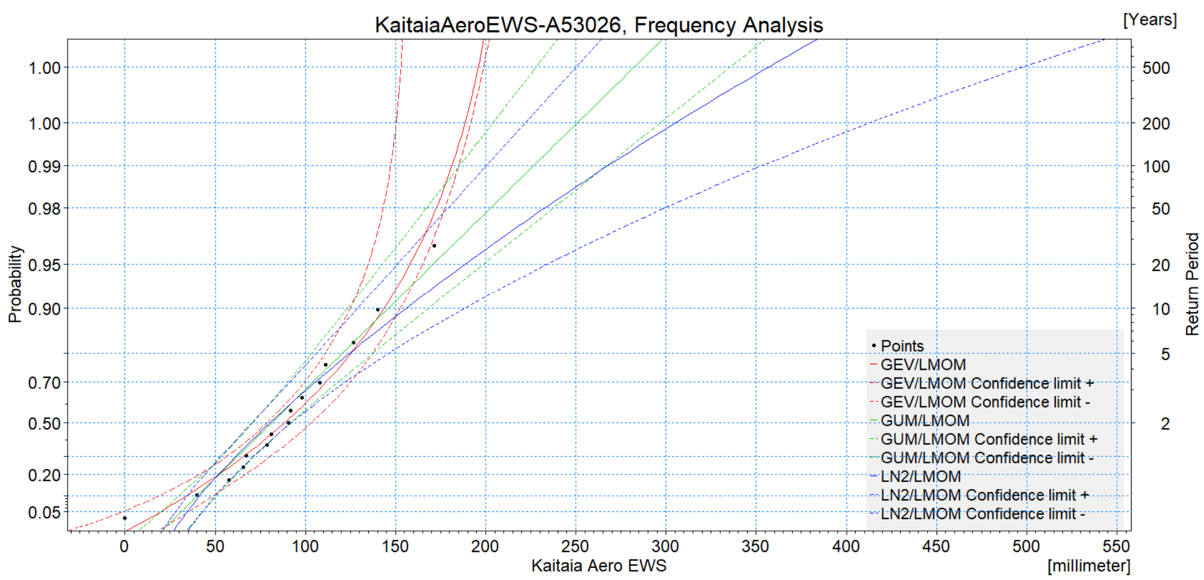
2
Return Period
50
100
150
200
250
300
Kaitaia Obs
[millimeter]
Points
GEV/LMOM
GEV/LMOM Confidence limit +
GEV/LMOM Confidence limit -
GUM/LMOM
GUM/LMOM Confidence limit +
GUM/LMOM Confidence limit -
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -


KaitaiaAeroEWS-A53026, Frequency Analysis
[Years]
Probability
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0.20
0.05
500
200
100
50
20
10
5
2
Return Period
0
50
100
150
200
250
300
350
400
450
500
550
Kaitaia Aero EWS
[millimeter]
Points
GEV/LMOM
GEV/LMOM Confidence limit +
GEV/LMOM Confidence limit -
GUM/LMOM
GUM/LMOM Confidence limit +
GUM/LMOM Confidence limit -
LN2/LMOM
LN2/LMOM Confidence limit +
LN2/LMOM Confidence limit -

APPENDIX D–Results

Model Results Table

Models/flood events	SH1		School Cut		Waikuruki - Lower Awanui		Tepuhi at Muffin Rd		Takahue at Grays		Puriri Place		Victoria Valley Rd		Donalds Rd - Whangatane Spillway		Ben Gun Wharf
Aprox. Chainage	AWANUI 7180		AWANUI 10030		AWANUI 12310		KAREMUHAKO 18750		TAKAHUE 11280		TARAWHATAROA 5970		VICTORIA 8130		WHANGATANE 271.088		AWANUI 37395
	Flow	WL	Flow	WL	Flow	WL	Flow	WL	Flow	WL	Flow	WL	Flow	WL	Flow	WL	WL
Feb-07	158	19.18	173	14.83	56.3	12.22	43.3	33.85	147	46.76	29.0	13.20	133	66.00	118	11.38	1.23
Jan-11	260	19.56	209	15.19	64.9	12.68	55.4	34.01	124	46.48	56.7	14.34	91.6	65.36	143	12.27	1.24
E10	299	19.64	225	15.38	67.9	12.78	57.6	34.04	114	46.39	95.7	15.26	79.4	65.13	152	12.42	1.40
E20	358	19.75	238	15.53	69.9	12.85	64.4	34.15	130	46.54	159	16.04	91.6	65.36	158	12.49	1.40
E50	432	19.87	254	15.64	72.6	12.95	73.1	34.28	152	46.75	233	16.66	107	65.62	167	12.59	1.41
E100	492	19.95	267	15.77	74.3	13.01	79.6	34.38	173	46.92	290	16.99	120	65.82	174	12.64	1.41
E100+CC	604	20.11	293	16.02	77.4	13.12	89.0	34.54	207	47.17	364	17.37	144	66.15	186	12.75	2.38
D10	299	19.64	227	15.17	70.6	12.86	57.6	34.04	114	46.39	93.5	15.23	79.4	65.13	154	12.52	1.40
D20	358	19.75	240	15.35	73.6	12.95	64.4	34.15	130	46.54	156	16.02	91.6	65.36	162	12.60	1.40
D50	432	19.87	256	15.53	76.1	13.04	73.1	34.28	152	46.75	230	16.65	107	65.62	171	12.68	1.41
D100	492	19.95	270	15.66	78.2	13.13	79.6	34.38	173	46.92	288	16.98	120	65.82	178	12.75	1.41
D100+CC	604	20.11	296	15.94	81.8	13.24	89.0	34.54	207	47.17	363	17.35	144	66.15	190	12.86	2.38
F10	306	19.55	259	15.33	73.6	12.97	57.6	34.04	114	46.39	59.9	14.45	79.4	65.13	182	12.28	1.40
F20	366	19.64	296	15.68	81.1	13.19	64.4	34.15	130	46.54	95.3	15.26	91.6	65.36	204	12.55	1.41
F50	439	19.78	317	15.90	84.0	13.28	73.1	34.28	151	46.75	168	16.16	107	65.62	216	12.66	1.41
G8 100	500	19.86	335	16.06	91.6	13.42	79.6	34.38	173	46.92	222	16.72	120	65.82	229	12.77	1.41
G7 100+CC	612	20.01	375	16.38	102	13.56	88.9	34.54	207	47.17	289	17.86	144	66.15	250	12.95	2.39

This table represents the flows and water levels extracted from the MIKE Flood model results. Flow is measured in m^3/s and the water level is RL in the OTP datum. Where floodplain flow was occurring at the flow gauges this additional flow was included. The exception to this was at at Takahue @ Grays where flows were estimated in the lower flow events based on a point 120m downstream where the water was better contained in the MIKE 11 model. The same approach was taken at Puriri place, where a point 600m downstream was used for the events smaller than the 50yr, and in the F and G models where there was less overflow in this area.