Wairoa Catchment

Numerical Modelling – MIKE FLOOD Model Build, Validation and Flood Hazard Mapping

Tauranga City Council

Report

August 2018

The expert in **WATER ENVIRONMENTS**

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Numerical Modelling – MIKE FLOOD Model Build, Validation and Flood Hazard Mapping

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1 Executive summary

Tauranga City Council (TCC) has been engaged in a long-term campaign of building flood models of stormwater catchments across its district for use in flood-hazard-mapping as well as for remedial options analysis. Public reaction to flooding during the April 2013 storm event has precipitated an increased urgency for the construction of mitigation measures in a number of TCC's stormwater catchments.

DHI Water and Environment Ltd (DHI) was engaged by TCC to build the MIKE FLOOD model for the lower Wairoa River, floodplain and stormwater reticulation. Following the model build, the model was validated against peak water level data that was available in the form of a good number of debris marks which were picked up by BOPRC along the length of the Wairoa River between the Ruahihi flow gauge and the railway bridge for the 28th – 29th January 2011 storm. Several photographs of the flooding at various locations along the river were made available as well from the report prepared by Tonkin & Taylor (Tonkin & Taylor Ltd, 2013). The calibration indicated that the model reproduces the observations reasonably well.

The sensitivity of the model has been further assessed by simulating variations to tributary inflows and to channel and floodplain resistance.

Following the calibration, 100-year ARI river flow and 100-year ARI storm surge design events were simulated to produce flood maps.

2 Introduction

2.1 Background and Purpose

TCC engaged DHI to build a MIKE FLOOD model for the Wairoa River catchment, as part of a stormwater catchment modelling programme across the TCC district for use in flood-hazard-mapping and remedial options analysis. The flood hazard maps will be incorporated into the TCC GIS for public use.

The initial brief, of September 2015, was to develop a model representing the Wairoa River and floodplain (downstream of Ruahihi), and to simulate design flood events under current climate conditions (TCC, 2015). A subsequent revision to the scope required the stormwater pipe network to be incorporated into the model[1]. All three elements (river, floodplain, pipe networks) were to be dynamically interconnected in the model.

In February 2017, TCC requested that the model be used to assess the flood impacts of the proposed Tauranga Northern Link (TNL) expressway, under design flood conditions incorporating forecast climate change to 2090. The existing situation was to be run. The model was then to be modified to incorporate the TNL (as the design stood at that time) and the results compared to the existing situation.[2]

A further set of model simulations was requested in May 2017, to assess the TNL proposal and initial proposals for infilling areas of the right bank floodplain to accommodate development of the Tauriko West (TW) area[3]. The assessment was to allow for climate change to 2130.

More recently (2018), TCC requested that DHI carry our further assessment of updated proposals for landform changes as part of the TW development.[4] That assessment is ongoing and will be separately reported.

2.2 Description of Study Area

The Wairoa catchment is dominated by a large river but also has limited areas serviced by a stormwater pipe reticulation. It is important to address the flood risk in this catchment, recognising that there are some potential growth areas in the catchment.

The Wairoa catchment covers approximately 450 km^2 and discharges into the Tauranga Harbour (see Figure 2-1). A little under 9 km^2 of the catchment is within the TCC district, consisting of two discrete areas in the lower part of the catchment. It is predominantly rural and semi-rural.

The remainder of the catchment is in the Western Bay of Plenty district, mostly rural in nature, predominantly pastoral and horticultural in the lower catchment and exotic and native forest in the upper catchment.

The entire catchment is in the Bay of Plenty region. Bay of Plenty Regional Council (BOPRC) also has a mandate to plan for and manage flood hazards. Although there is no regionally-managed river scheme associated with the Wairoa River, BOPRC does provide advice on, and

[1] Email from Dayananda Kapugama (TCC) to Philip Wallace (DHI), 17 November 2015

[2] Email from Graeme Jelley (TCC) to Philip Wallace (DHI), 8 February 2017

[3] Email from Graeme Jelley (TCC) to Philip Wallace (DHI), 10 May 2017

[4] Email from Dayananda Kapugama (TCC) to Philip Wallace (DHI), 20 February 2018

assistance with, river works in the catchment and has frequently been asked to provide comment on flood risk for developments within the catchment.

Trustpower is the final organisation with a significant interest in the Wairoa River catchment. Trustpower manages the Kaimai Hydro-electric Power Scheme (HEPS), consisting of four hydro-electric power stations fed by the catchment. The largest and most-downstream of these is the Ruahihi power station, discharging into the Wairoa River approximately 12 km from the river mouth. The river flows downstream of the power station are significantly influenced by the operation of the power scheme.

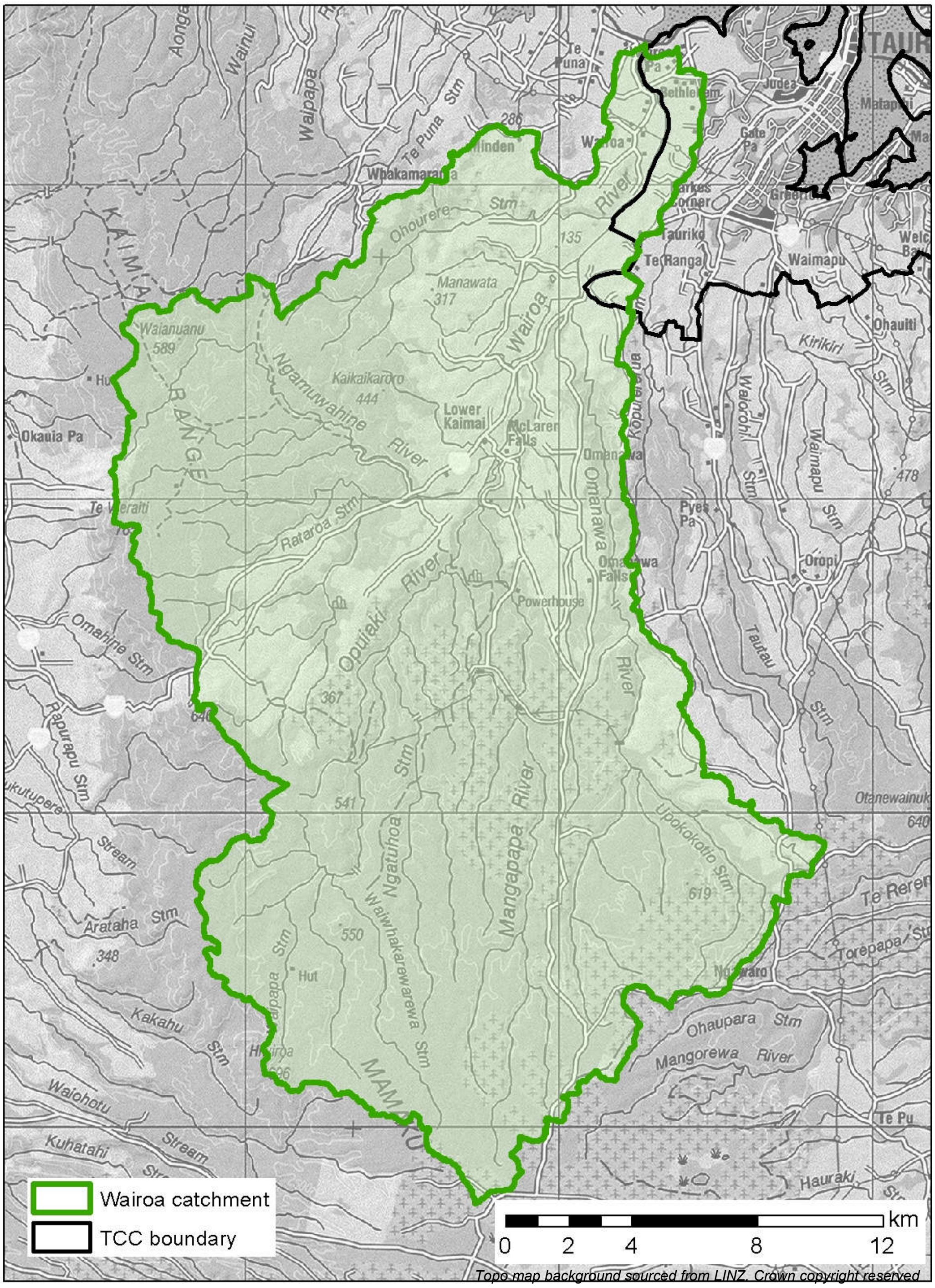

Figure 2-1 Wairoa Catchment

2.3 Previous Modelling and Reports

Number of studies were carried out in the Wairoa catchment over the years. Some of the most relevant studies and reports describing previous modelling of the Wairoa River and catchment investigations include the following:

1. Riley Consultants carried out *Kaimai HEPS Flood Study* (Riley Consultants Ltd, 2005) for Trustpower. A HEC-HMS model of the HEPS was developed and flows were routed through the HEPS. Model outputs include design flows (10-year ARI[5], 100-year ARI, 1000-year ARI, PMF[6]) for the catchment at Ruahihi (including the contribution from the HEPS). The model was calibrated to flows at the Wairoa River recorder just downstream of Ruahihi Power Station.

2. Tonkin & Taylor carried out *Kaimai Hydro Electric Power Scheme: PIC Assessment* (Tonkin & Taylor Ltd, 2013) for Trustpower. Dam break hydrographs (derived from HEC-HMS) were routed from the Mangaonui Dam to the Tauranga harbour, via the Opuaiki and Wairoa Rivers, using Mike 11. LiDAR and photogrammetry were used to derive river cross-sections, supplemented in lower reaches by manual adjustment to invert levels with the aid of three actual cross-sections near the power station.

3. Ryder Consulting undertook a river modelling exercise for a proposed development near Tauriko, *Wairoa River at Tauriko – Floodplain Assessment* (Ryder Consulting Ltd, 2010). River cross-sections were surveyed over a 2.6km reach adjacent to Tauriko, while cross sections downstream to the railway (near the mouth) were based on 1m contours over the remaining reach to the railway near the mouth. The exercise was carried out using HEC-RAS.

2.4 Scope of Work

The focus of this study is to build a MIKE FLOOD model of the Wairoa River and floodplain, downstream of Ruahihi Power station. The model area covers that shown in Figure 2-2. It should be noted that the upstream extent of the model is upstream of tidal effects.

The scope included the following tasks:

- Validate an existing HEC-HMS model of the upper catchment and use it to prepare design Wairoa River flow hydrographs;
- Build the 2-D surface model using MIKE 21 FM (quadrilateral elements);
- Build a MIKE URBAN model of the Wairoa stormwater catchment reticulation;
- Build a MIKE 11 model of the Wairoa river;
- Dynamically link the 1-D pipe network model, 1-D river model and the 2-D surface model;
- Validate the model by comparing the outputs to the existing observed and anecdotal information for the January 28th-29th 2011 event;
- Carry out sensitivity simulations to understand the model predictions sensitivity to changes in hydrologic input and surface roughness; and
- Carry out production runs to produce flood hazard maps.

[5] Average Recurrence Interval. A 10-year ARI event is also referred to as a 10% AEP (Annual Exceedance Probability) event. Likewise, a 100-year ARI event is also referred to as a 1% AEP event.

[6] Probable Maximum Flood

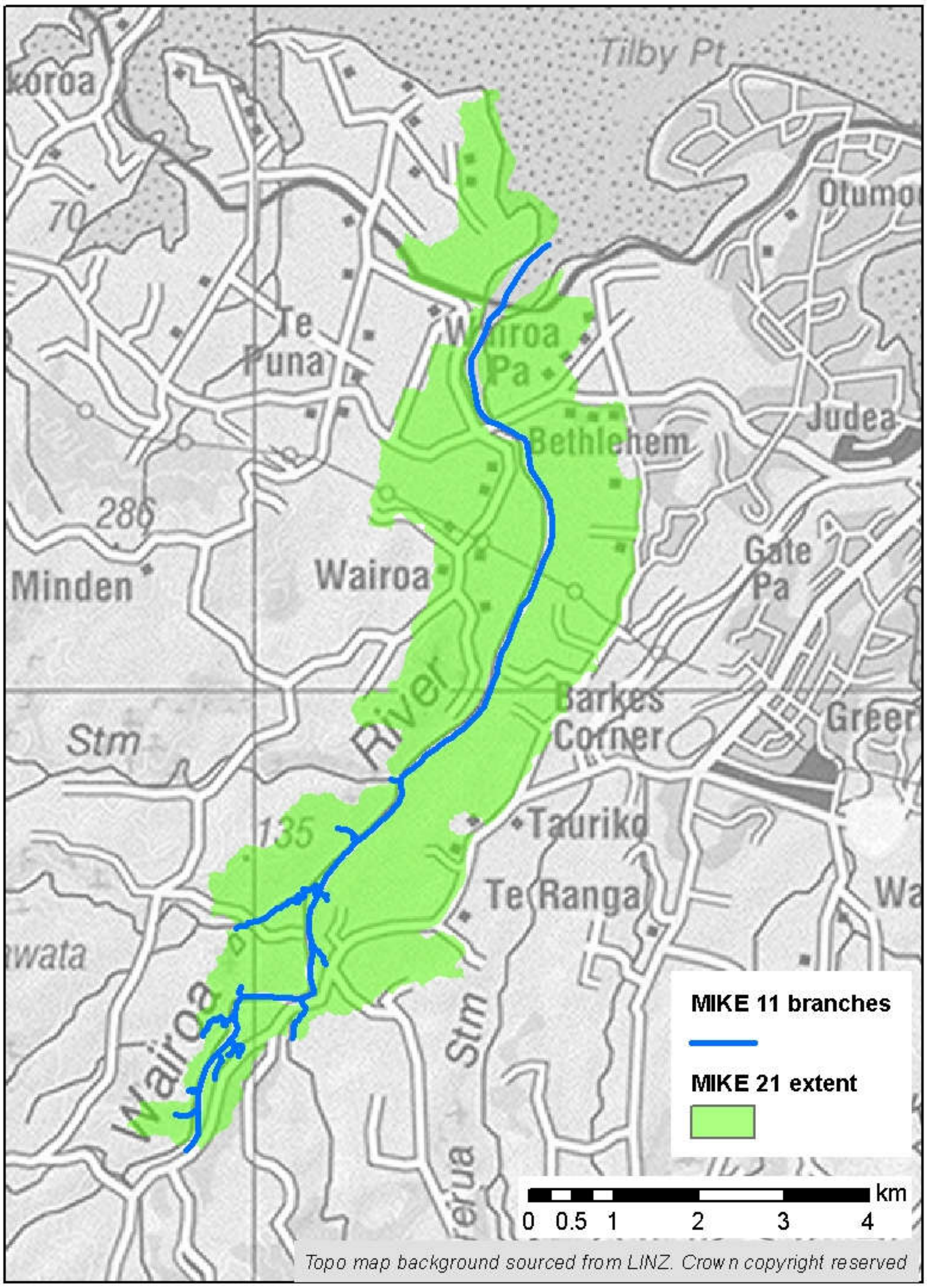

Figure 2-2 Model Extent

2.5 Approach to the Study

A fully 3-way coupled MIKE FLOOD model was built and it consists of:

- a 2-D model that uses LiDAR data converted into a 4 m grid to represent the overland 2-D terrain (MIKE 21 FM model);
- a 1-D river model that uses surveyed stream cross sections (MIKE 11 model); and
- a pipe network model that is based on the stormwater asset data provided by TCC (MIKE URBAN model).

The completed model with its various hydrological scenarios was compared against flood debris levels (i.e. peak water levels) down the river collected by BOPRC during a flood event from 28th-29th January 2011 as well as some flood photographs from the same event presented in *Kaimai Hydro Electric Power Scheme: PIC Assessment* report (Tonkin & Taylor Ltd, 2013).

The modelling work was carried out in accordance with TCC's Guidelines for Stormwater Modelling using MIKE FLOOD (DHI, 2017). That document presents guidelines and general advice on best practice concerning schematisation and model build using MIKE FLOOD. It also specifies the format for the deliverables, including the table of contents for the model build report.

2.6 Task Outline

The following steps were undertaken to complete the project:

1. Undertake a site visit;
2. Review available data for the model build, identify data gaps and present the extent of missing data to TCC;
3. Identify what, if any, flood observations are available for model calibration and verification;
4. Prepare a river cross-section survey specification, structures survey specification and required format; survey of verifiable flood marks from actual events (e.g. identified in photographs or described by residents);
5. Confirm hydrological inputs;
6. Prepare a MIKE 11 model of the Wairoa River from Ruahihi to the Tauranga Harbour;
7. Build a MIKE URBAN model of the stormwater reticulation in the catchment;
8. Construct a MIKE 21 FM model of the floodplain surface;
9. Couple the MIKE 11, MIKE URBAN and MIKE 21 FM models using MIKE FLOOD (MIKE 2016 SP3 was used for final simulations);
10. Verify model parameters by assessing the model's capability to reproduce flooding observed during the storm events of January 2011;
11. Produce flood maps for the existing-climate, 100-year ARI design event using catchment and boundary condition parameters relevant for the existing case; and
12. Produce a model build and flood-hazard mapping report to accompany the hand-over of the final model and associated results to TCC.

4 Model components

4.1 MIKE URBAN

The model has been built using stormwater asset data extracted on 18th of February 2016. The following layers with numbers of elements were used to build the model:

- SWManhole: 101
- SWNode: 4
- SWSoakHole: 19
- SWStructure: 46
- SWSump: 86
- SWServiceLine: 97
- SWMain: 125

Most imported pipes have diameter and material information. However, many of imported nodes have either ground level or invert levels missing, as seen in Table 4-1 and Table 4-2. Missing invert levels were replaced by interpolated (source 'Inserted') values or in terms of sumps and soak holes by standard configurations described later in this report. Missing ground levels were extracted from LiDAR information.

Table 4-1 Source of information for invert levels

Invert Level Source	
GIS	79
Inserted	72
Sump Configuration	86
Soak Hole Configuration	19

Table 4-2 Source of information for ground levels

Ground Level Source	
GIS	78
LIDAR	178

4.1.1 Sumps

There are 96 sumps in the model. Eighty-six of these were imported asset data and 10 are dummy sumps. These dummy sumps were added for pipes that do not have an upstream node in the defined in the asset data.

The sumps are set up using TCC's Guidelines for Stormwater Modelling using MIKE FLOOD (DHI, 2013):

- Sump depth 1.2 m;
- Lead diameter 0.35 m if unknown;
- Lead upstream level sump invert + 0.45 m;
- Maximum lead slope 10% if possible; and
- Minimum lead slope 1% if possible.

Sumps are coupled to MIKE 21 FM 2-D overland model and the coupling parameters were adopted as per the guidelines (DHI, 2013), i.e. the inlet area, discharge coefficient and QdH factor were based on the physical properties of the sump.

4.1.2 Cesspit Manholes

Cesspit manholes, from Asset layer SWManhole type Cesspit Manhole, are modelled as sumps. There are two cesspit manholes in the model.

4.1.3 Inlets

Inlets, from Asset layer SWStructure type Inlet, are modelled as manholes which are coupled to MIKE 21 FM. The inlet areas in the couple file are set to match the cross section area of the connecting pipe. There are 18 inlets in total in the model.

4.1.4 Outlets

Outlets, from Asset layer SWStructure type Outlet (28 elements in total) and SWNode type End (3 elements in total), are modelled as manholes which are coupled to MIKE 21 FM. The inlet areas in the couple file are set to match the cross section area of the connecting pipe. One exception is manhole id 57997 which discharges into the river. This outlet was modelled as an outlet structure and it was coupled to MIKE 11. The model has 35 outlets of which four are dummy outlets placed at pipes with no downstream nodes defined in the asset data.

4.1.5 Soak holes

A total of 19 soak holes in the model were set up as per TCC guidelines for soak holes in mainland catchments. There are no as-builds or asset information on soak holes apart from their location, hence standard assumptions were used as per Modelling Approach for Soak Holes (DHI, 2014):

- 3 m deep;
- 1 m in diameter; and
- Soakage rate 157.1 l/h when ground level is above 3 m (which was the case for all 19 soak holes).

Each soak hole is modelled as a manhole with a dummy pipe with a dummy outlet on the downstream end. Dummy pipes are assigned a passive flow regulation of 157.1 l/h; the dummy outlets were not linked to MIKE 21 FM, thereby the water is removed from the model. This approach represents the water soaking into the ground at the bottom of the soak hole. The soak hole itself is linked to MIKE 21 FM to enable spilling onto the ground if the water level in the soak hole itself reaches the ground level.

4.2 MIKE 11

4.2.1 Model branches

The MIKE 11 model consists of the main Wairoa stream downstream of the Ruahihi flow gauge and 12 smaller tributary streams. In the model the Wairoa stream branch has been split into sections to allow for the full length of the two bridges, SH2 and the Railway Bridge, to be modelled while maintaining the smaller maximum dx of 20 m in the remainder of the Wairoa Stream.

4.2.2 Cross-sections

Survey cross sections from February 2016 were used for the length of the Wairoa. Where necessary, these cross sections have been extended using a 2 x 2 m raster of the LiDAR data. These survey sections are spaced at approximately 400 m intervals (Appendix C).

A second survey was undertaken to check the very deep cross section measured at the bed upstream of the SH2 bridge (model chainage Wairoa_US 10252) (Figure 4-1); it was confirmed that there is a deep hole that drops below -17 m RL (Appendix C).

Additional tributary streams were included in the model to ensure that the floodplain could correctly drain into the Wairoa and to facilitate the connection of some of the external catchments into the MIKE FLOOD model. The cross sections for all of the tributary streams were extracted from the 2 x 2 m LiDAR raster except for the Omanawa Stream where survey cross sections were available.

As part of a separate exercise on behalf of the New Zealand Transport Agency (NZTA), Bloxam, Burnett & Olliver (BBO) commissioned a detailed riverbed survey near the proposed TNL in 2017. The extent of the survey is shown in Appendix C, along with a map of the riverbed bathymetry derived from the BBO data. Selected points along the thalweg are plotted in Figure 4-1. The bathymetry map suggests that the river bed varies between cross-sections 8 and 10b and the model may be improved with additional cross-sections in that reach. However, given the differences between observed and predicted flood levels for the calibration event, highlighting other uncertainties as discussed in section 5.2.3, the work to insert extra sections and rerun all models is not considered worthwhile at this stage.

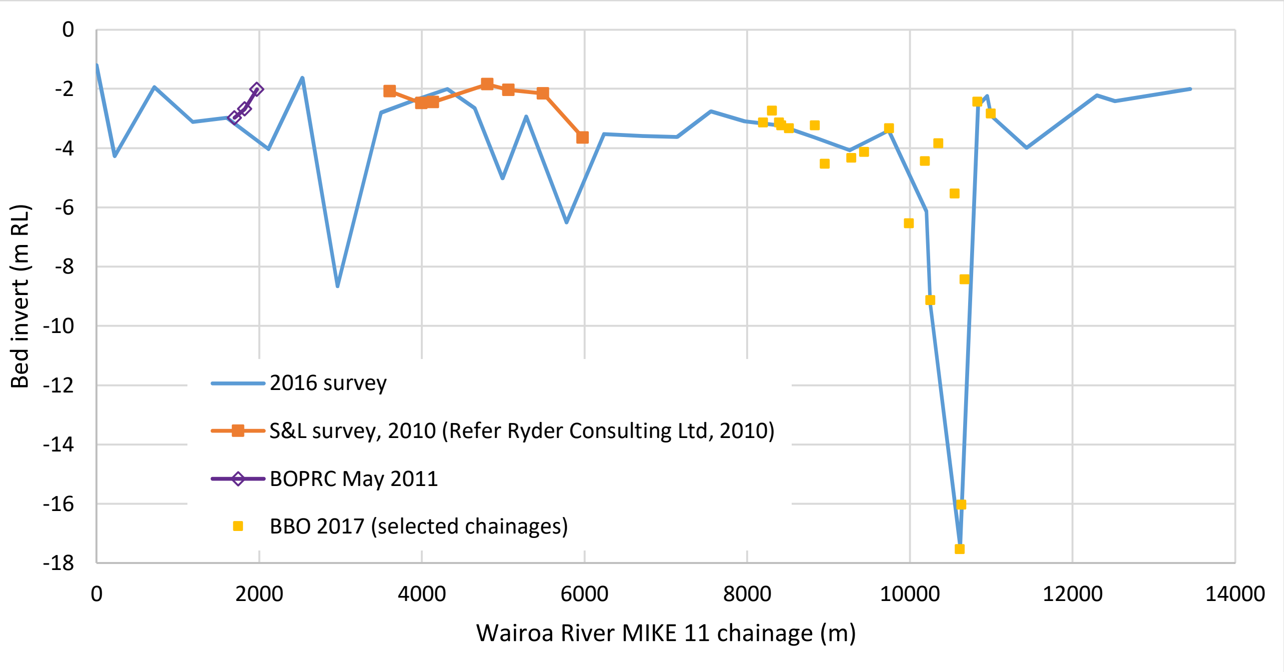

Figure 4-1 Long-section plot of Wairoa River bed thalweg levels

4.2.3 Structures

Five structures were included into the MIKE 11 model. The properties of these structures are provided in Table 4-3.

Table 4-3 MIKE 11 structure parameters

Name	Branch	Chainage	Geometry	Size	Length
Flapgate	Wairoa_US	10007	Circular, Side Structure	0.6m	6m
SH 29 Culvert	Branch 20	682	Circular	1.2m[7]	15m
Rail Bridge	Wairoa_Rail	12414	Bridge with Piers (7% blocked)	Soffit @ 3.87m	24m
SH2 Bridge	Wairoa_SH2	10883	Bridge with Piers (5% blocked)	Soffit @ 4.975m	30m
SH29 Bridge	Omanawa_Bridge	462	Bridge with Piers (9% blocked)	Soffit @ 6.2m	15m

4.2.4 Boundary conditions

Catchment inflows into the MIKE 11 model were applied at the upstream end of the model (Wairoa @ Ruahihi) and at the upstream end of six of the 12 tributary streams (Table 4-4 and Figure 4-2). With the exception of the Wairoa @ Ruahihi site for the calibration event (where the recorded flow was used), these inflows were first generated in the HEC-HMS rainfall-runoff model (refer section, 5.1). A tidal boundary condition is applied at the downstream end of the Wairoa River.

Table 4-4 Hydrological inputs to hydraulic model

Site (Figure 4-2)	Sub-catchment	Applied as	Hydrograph source (Model validation)	Hydrograph source (Design)
1	Wairoa @ Ruahihi	MIKE 11 point inflow	Recorder	HEC-HMS
2	Wairoa6	MIKE 11 point inflow	HEC-HMS	HEC-HMS
3	Wairoa5	MIKE 11 point inflow	HEC-HMS	HEC-HMS
4	Omanawa	MIKE 11 point inflow	HEC-HMS	HEC-HMS
5	East Omanawa	MIKE 21 source	HEC-HMS	HEC-HMS
6	Waireia (Wairoa3)	MIKE 11 point inflow	HEC-HMS	HEC-HMS
7	Ohourere (Wairoa4)	MIKE 11 point inflow	HEC-HMS	HEC-HMS
8	Vernon Rd (Wairoa1)	MIKE 21 FM source	HEC-HMS	HEC-HMS
9	Wairoa2	MIKE 11 point inflow	HEC-HMS	HEC-HMS

[7] In the calibration event, this has been modelled as twin culverts, on the basis of information provided at that time. Design runs modelled as a single 1200mm culvert. Results for the calibration not expected to be significantly affected.

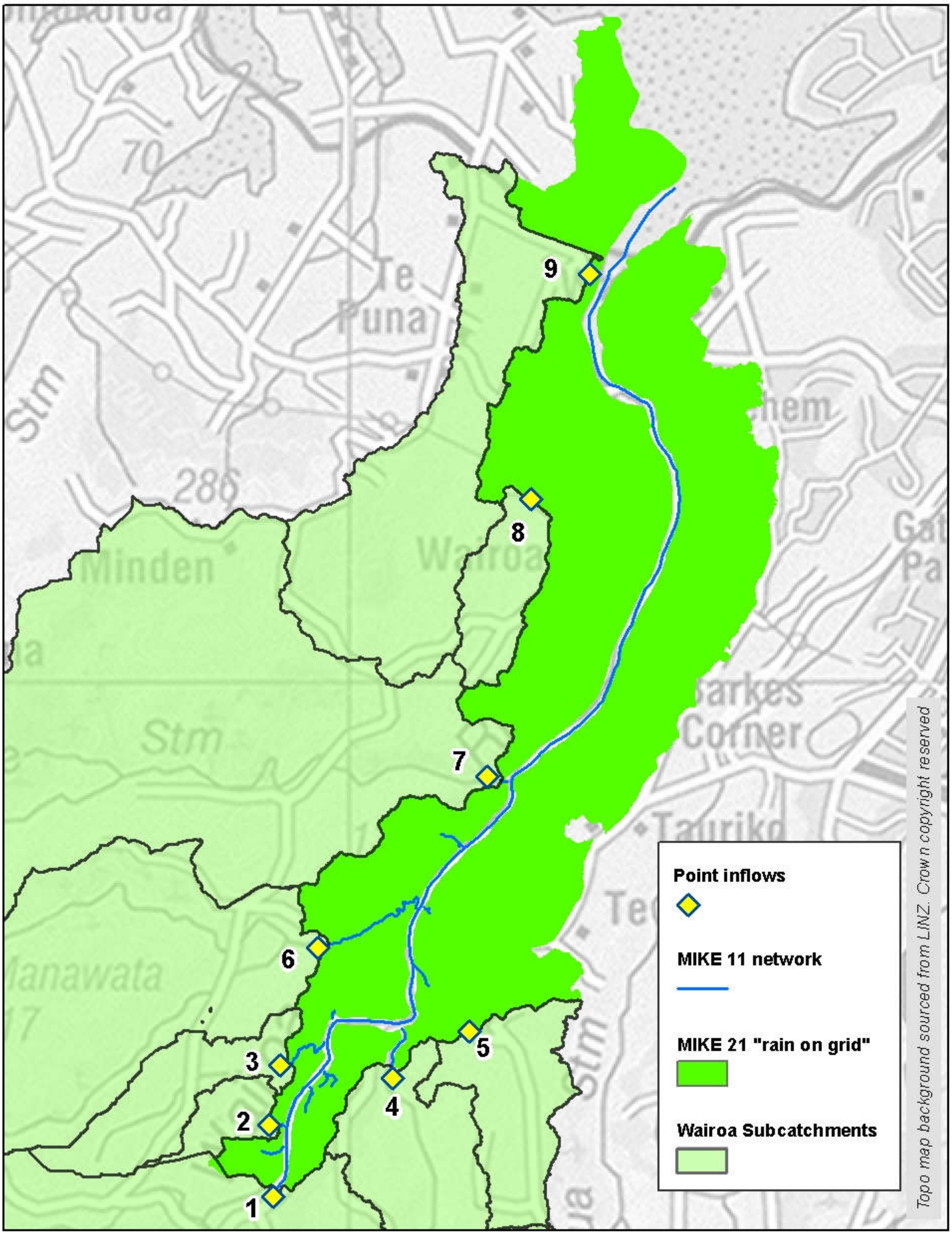

Figure 4-2 Hydrological inputs to hydraulic model

4.2.5 Channel resistance, initial conditions and other .HD11 file parameters

The 1-D bed roughness was applied as uniform in the transverse distribution and varied along the length of the Wairoa River (as part of the model validation process, see Figure 5-6). The roughness in the tributary streams was set to a constant of 0.037.

The initial water level was set to that of the starting level of the tidal boundary condition. The initial flow was set to zero.

Both the 'delta' and 'delhs' values were adjusted in the MIKE 11 HD setup. The 'delta' value was set to 0.85 which is standard for a MIKE FLOOD simulation with a small time step. The 'delhs' value was increased from 0.01 to 0.02 to stabilise the flow calculation through the downstream bridges where the tidal effects are significant.

1-D mapping was setup for the simulation to allow for the 1-D and 2-D results to be more easily merged into a single flood map.

4.3 MIKE 21

The MIKE 21 FM topography was derived from the 2011 and 2014 LiDAR surveys. These LiDAR surveys were combined, prioritising the newer 2014 dataset. Open drains were burned into the resulting dataset by lowering the cells by 500 mm. The outline of the MIKE 21 FM mesh was derived by performing a catchment delineation on the 4 x 4 m grid; the delineation was assisted by digitising additional sink points and trench lines to encourage the flow accumulation to coincide with the existing open channels in the floodplain. The downstream harbour edge was manually edited to match the curve of the coast. Areas beyond the extent of the delineated catchment and within the extent of the MIKE 11 cross sections, were set to land in the 2-D grid. Once the 2-D grid was complete it was converted into a mesh with rectilinear elements and the harbour boundary was set to code 2. The complete MIKE 21 FM grid consists of areas of steep terrain combined with relatively flat floodplain as seen in .

A depth correction file was used to replace the MIKE 21 FM mesh values; this allows for easier editing of mesh values and better accuracy in bed levels. The following modifications to the model grid were made using the depth correction file:

- Areas where high velocities were occurring due to steep slopes were smoothed using the filter tool;
- Elements at the Harbour boundary were lowered so that the boundary would never dry out throughout the tide cycle;
- Element levels were adjusted at the location of standard and side structure links so that the MIKE 21 level matched the connecting MIKE 11 cross section; and
- Element levels were lowered to allow flow to pass through the MIKE 21 FM culverts.

Three culverts and one weir were included directly into the MIKE 21 FM model to allow flow to pass through road obstructions away from the MIKE 11 model. The location of these culverts is shown in and the properties of these are presented in Table 4-5.

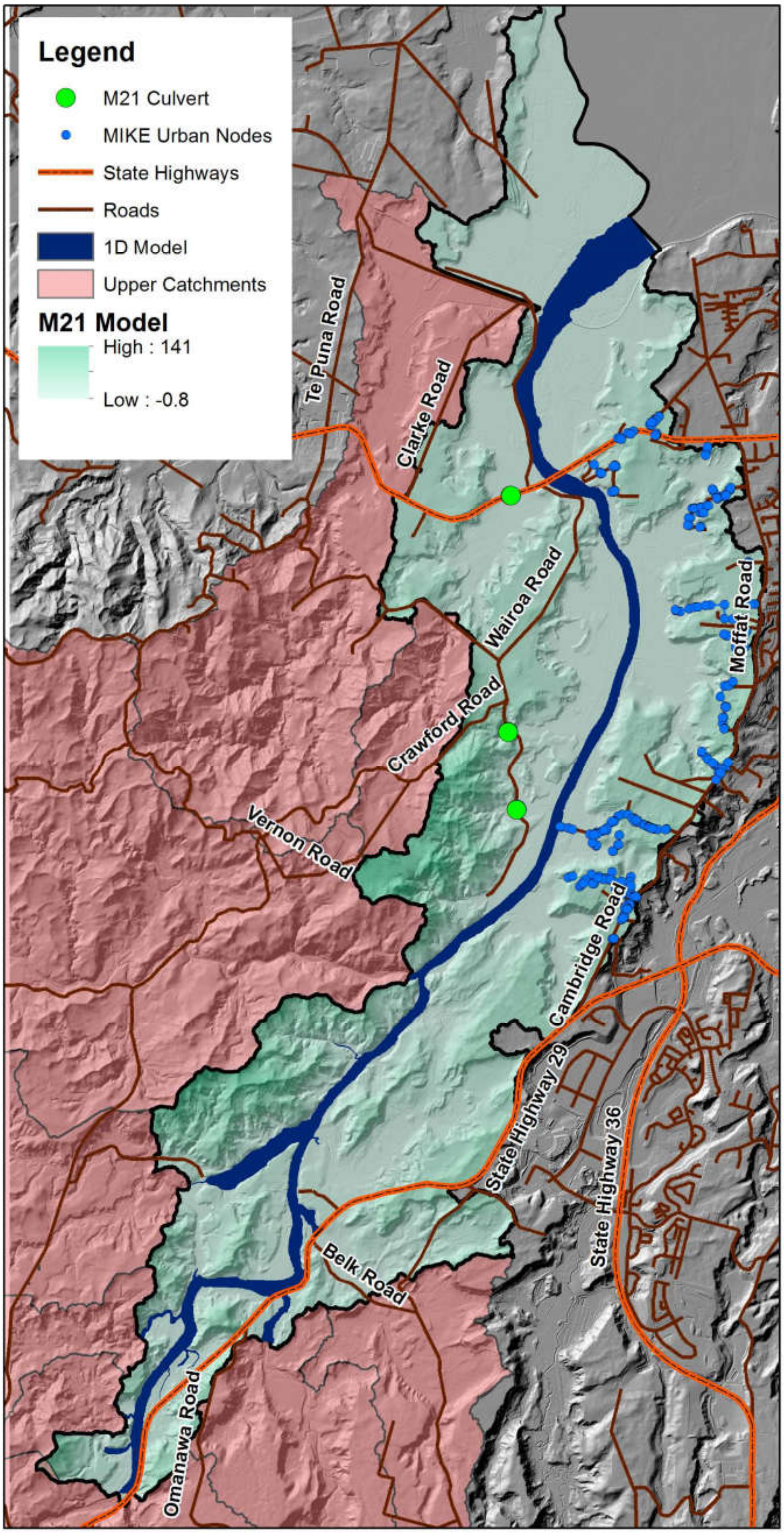

Figure 4-3 Model Layout

Table 4-5 MIKE 21 Structure Parameters

Name	Geometry	Size (m)	Length (m)
SH2 Culvert	Irregular Arch	1.75 x 1.4	8
Wairoa Rd 1	Circular	0.45	10
Wairoa Rd 1	Circular	0.45	10
Weir Wairoa Rd 1	Weir	18m RL, 16m wide	

Rainfall is applied directly to the mesh within the 2-D model area (Figure 4-2). In addition, two of the external sub-catchments were connected to the MIKE 21 FM grid as MIKE 21 source points. These catchments are Vernon Road and East Omanawa. The location of the sources is also shown in .

The 2-D roughness was derived from the LCDB version 4.1 and the TCC inland land use layer (based on aerial photography analysis), since the TCC layer only covered the TCC section of the catchment. Where data was not available on the road extent a new extent was created by applying a 5 m buffer to the road centre lines. The road centreline was first checked and adjusted so that the roads aligned with the aerial photography. Where data was not available on the buildings these were drawn manually based on the Aerial photography. The roughness was applied as a dfs2 file at 4 x 4 m resolution. The Manning's values used for each land use type are outlined in Table 4-6.

Table 4-6 MIKE 21 roughness

Landuse type	Manning's M
Forest	12
Crops and bushes	8
Grass	20
Water	80
Roads + other impervious	70
Buildings	5
Built up area	12

Infiltration was applied to the model using the constant infiltration with capacity method. To define the infiltration rates and initial losses the land use was split into four categories:

- Impervious areas, i.e. road, pavement and water;
- Buildings above an elevation of 3m;
- Buildings below an elevation of 3m; and
- Pervious areas, i.e. grass, vegetation, gravel.

The following parameters were used for these different land use types, Table 4-7.

Table 4-7 MIKE 21 infiltration values

Landuse Type	Infiltration mm/hr	Initial loss depth (mm)
Impervious	.0083333	0.05
Buildings > 3m	13.1	16.1
Buildings < 3m	8.5	20.8
Pervious areas	2.54	5

The MIKE 21 FM simulation uses a maximum time step of 0.25 s and a minimum time step of 0.1 s. The drying, flooding and wetting values are 0.002, 0.005 and 0.01 m respectively.

The initial water level is set to that of the starting level of the tidal boundary condition.

4.4 MIKE FLOOD

The model is dynamically linked using the MIKE FLOOD software. The model contains a total of two standard links, 72 lateral links, 168 MU inlet links, one MU outlet to river link and one side structure link.

5 Model validation

Model validation has been a two-stage process: firstly, validation of the hydrological model (HEC-HMS) of the upper catchment, and secondly validation of the hydraulic model (MIKE FLOOD).

The storm event of 28th – 29th January 2011 was used as the validation event, because of the size of the event and the availability of calibration data. The estimated peak Wairoa River flow at Ruahihi was 1,193 m^3/s, the largest since records began in 1990. Debris marks indicating peak water levels were subsequently surveyed along the river between Ruahihi and the harbour.

This storm was short and intense, as can be seen in the flow record at Ruahihi (Figure 5-1). It also followed a lesser, but still significant, storm on 23rd – 24th January 2011. As a result, the catchment would have been primed and runoff was more rapid in the second storm. (Note also that the recorder failed at or just before the peak, and hence there is no recorded recession. Peak flow estimates have provided by a slope-area calculation carried out by BOPRC.)

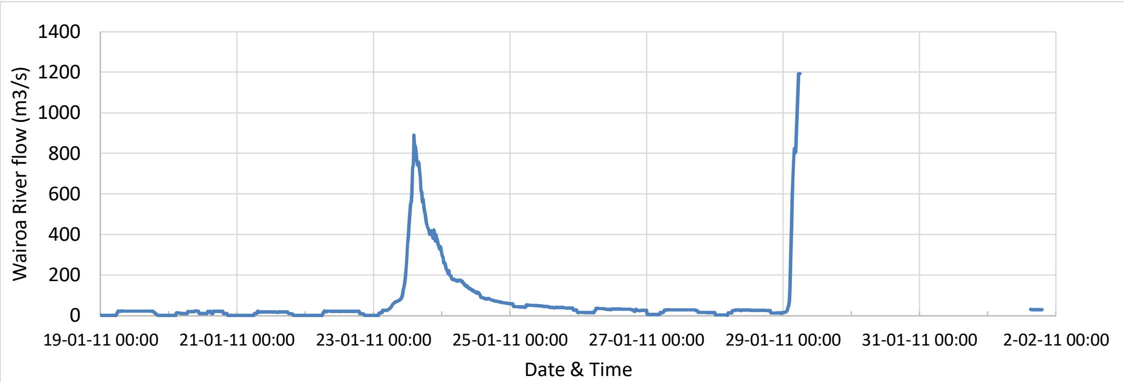

Figure 5-1 Wairoa River flow, immediately downstream of Ruahihi Power Station

5.1 Hydrological model validation

The existing HEC-HMS model of the Kaimai HEPS (Riley Consultants, 2005) has been updated as part of an exercise being carried out by Opus for Trustpower. The update has included a recalibration, taking into consideration five storm events: July 1998, May 1999, February 2004 and both January 2011 events. The rainfalls Opus used for the recalibration were based on a single rain gauge: the Lloyd Mandeno recorder, but scaled up in upper parts of the catchment (as explained in the original Riley Consultants report).

The recalibrated HEC-HMS model parameters (losses, times of concentration, etc.) are given in Appendix B. Comparison of the recorded and model predictions for the 29th January 2011 event are given in Figure 5-2.

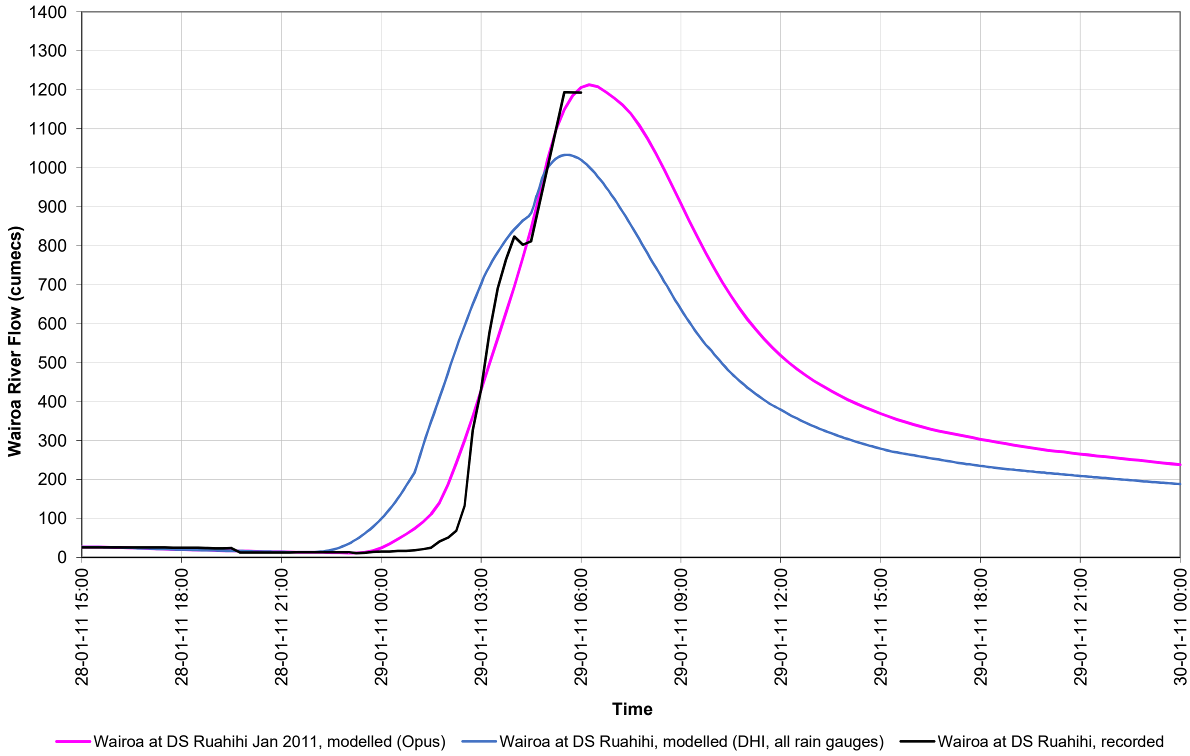

Figure 5-2 Recorded and modelled flows, Wairoa River downstream of Ruahihi, 29 January 2011

DHI has also used the same HEC-HMS model but with records from several rain gauges in and around the Wairoa catchment, as a further check on the calibration (Appendix B). Results for that check are also shown in Figure 5-2.

The simpler approach of Opus produces a good fit to the available data. Using all the available rain gauge information does not produce as good a fit. A possible reason is that the Mangakarengorengo Diversion rain gauge, which is applied over a significant part of the mid-catchment in the more detailed method, may have underestimated rainfall. The total recorded at that gauge is somewhat less than for all other gauges (Figure B-3). The nearby Lower Wairoa rain gauge has not been used in the analysis, as only daily rain totals were recorded at that time.

In addition to the rain gauge information, rain radar records were available. Peter West of Blue Duck Design has compared the rain totals suggested by the rain radar with the weighted sub-catchment rainfall depths calculated for the DHI approach. Figure B-4 S presents his comparison. An interpretation of that figure is:

- the rain radar seems to confirm that the Mangakarengorengo recorder underestimated the rain totals;
- the rain radar has underestimated the rain in the south of the catchment (a blind spot in that area is more clearly evident in the hourly rain radar records); and
- elsewhere there is no major discrepancy between the rain gauge and rain radar totals.

Given that the Opus work also considered a number of other storms for recalibration, the recalibrated HEC-RAS model is considered acceptable for design flow estimations.

5.2 Hydraulic model validation

5.2.1 Boundary conditions

The actual recorded river flow at Ruahihi has been used for the upstream boundary condition, at least on the rising limb of the hydrograph. For the falling limb, a rate of recession similar to that of the 23rd January 2011 event has been used (falling at a rate of 4% every 15 minutes), rather than using the more gradual model predictions of river flow. Regardless the rate of recession is not expected to have any significance on downstream peak water levels for this short storm.

For sub-catchments downstream of Ruahihi, the Opus HEC-RAS model has been extended to include these sub-catchments and so provide inflow hydrographs for them. Similar loss parameters used in the Opus recalibration work for this particular storm have been used for those sub-catchments, and times of concentration estimated with the aid of empirical formulae. Appendix B summarises the parameters used.

Over the MIKE 21 FM domain, a rain on grid approach has been used. The rain hyetograph assumed is that of the Landing Road rain gauge, being the gauge closest to that area of the model.

The downstream boundary condition is the harbour water level record from the "Tauranga Harbour @ Omokoroa" recorder, about 5 km to the northwest. This recorder is the closest to the river mouth of the three harbour level recorders maintained by BOPRC.

5.2.2 Calibration information

Peak water level data on the event was available in the form of a good number of debris marks which were picked up by BOPRC along the length of the Wairoa River between the Ruahihi flow gauge and the railway bridge. The confidence the surveyors had in the debris observations, based on how well-defined the debris marks were, was also recorded with the levels. (Figure 5-3 to Figure 5-5).

Several photographs of the flooding at various locations along the river are presented in Tonkin & Taylor (2013).

5.2.3 Simulations and results

Several simulations were run to attempt to match the model to the measured flood levels. In the process a few different variables were adjusted; these were: the 1-D channel roughness, the 2-D floodplain roughness, the infiltration from the 2-D mesh and the runoff from the tributary catchments. Except for the 1-D roughness, which had a larger effect on water levels, it was found that the model was not particularly sensitive to these parameters. The lack of sensitivity can be explained by comparing the relative catchment area upstream and downstream of the Ruahihi gauge. The catchment area upstream is approximately 380 km^2 while downstream the area is approximately 80 km^2, which is only 17% of the total catchment area. This means that any error in the flow measurement at the gauge will have a significant effect on the flood flows and levels downstream.

The findings from the validation iterations are detailed here:

1-D bed resistance: Lowering channel resistance along the length of the river by 10% from the initial assumption had the effect of lowering the water levels by between 380 mm and 90 mm. The final values adopted are shown in Figure 5-6. These values are considered to be at the lower end of the expected range of Manning's n values for such a river.

2-D bed resistance: The 2-D bed resistance for the vegetated open space area was adjusted from a Manning's M of 20, to 30. This lowered the Wairoa water levels up to 40 mm.

2-D infiltration: The pervious infiltration rates for the area of the MIKE 21 FM mesh were adjusted from 2.54 mm/hr to 5 mm/hr. This had the effect of lowering the water levels in the Wairoa by 10 mm.

Runoff from the tributary catchments: The runoff from the tributary catchments, except Omanawa, was removed completely to test the sensitivity of the runoff on the Wairoa water levels. This had the effect of lowering the water levels in the Wairoa by approx. 40-120 mm, indicating the relative insensitivity of the model results to the lower tributary inflows.

The final validation event combined a number of the model adjustments together. These adjustments were to decrease the tributary catchment runoff by approximately 10% by adjusting the infiltration parameters and adjusting the 1-D roughness to the values in Figure 5-6. In the process of the validation some additional adjustments were also made to the model such as improving the definition of vegetation areas in the 2-D roughness file, increasing the roughness of the roads, adjusting the flooding and drying values, and small adjustments to the flood links and MIKE 11 cross sections. However, for the most part these changes were made to improve model stability and the detail of the model rather than as an explicit calibration exercise.

The results are illustrated in Figure 5-3 to Figure 5-6. The average difference between the predicted and observed peak levels was 26 cm, with the average absolute difference being 31 cm.

In the upper reaches of the model, above the Omanawa River confluence, the model predictions are close to the measured data. However, in the middle reaches the predicted water levels are still showing as significantly higher than the measured levels, with differences up to 68 cm for measurements with medium confidence.

Possible reasons postulated for the discrepancies included:

- River bed changes between the flood event and the time of the cross-section survey (an interval of five years);
- Underestimation of peak debris levels on site (deciding upon appropriate debris levels is a subjective exercise, notwithstanding the level of confidence that was recorded in this case);
- Inaccuracies in the river flow rating; and
- Inaccuracies in the peak flow estimate (based on a slope area calculation).

Limited cross-section information from around the time of the flood event was subsequently obtained and assessed. That information does not explain the differences between model predictions and debris levels, as the 2016 cross-sections seem to have a lower invert than the earlier ones (Figure 4-1).

BOPRC also was comfortable with the rating curve at the Ruahihi site (Mark James, BOPRC, pers. comm.). Thus, it remains unclear why the model overpredicts the calibration event as much as it does.

Without further lowering the channel resistance, to unrealistic values, a closer fit to the observed data is unlikely.

Figure 5-7 and Figure 5-8 provide a comparison between flood photographs from the event provided in Tonkin & Taylor (2013) and model predictions. The photographs were presumably taken after the flood peak (that occurring before daybreak), but still will give an approximate indication of peak levels (as no debris lines above the water level are evident in the photographs). The modelled results may slightly over-predict the flood depths, particularly at the Hospice site, but nonetheless are considered in to show reasonable agreement with the photographs. (Other photographs are also available, but these do not add any information additional to the surveyed debris levels.)

Overall, even though the model has over-predicted levels in the middle reaches of the modelled river, realistic resistance values have been used, and the model validation is considered acceptable.

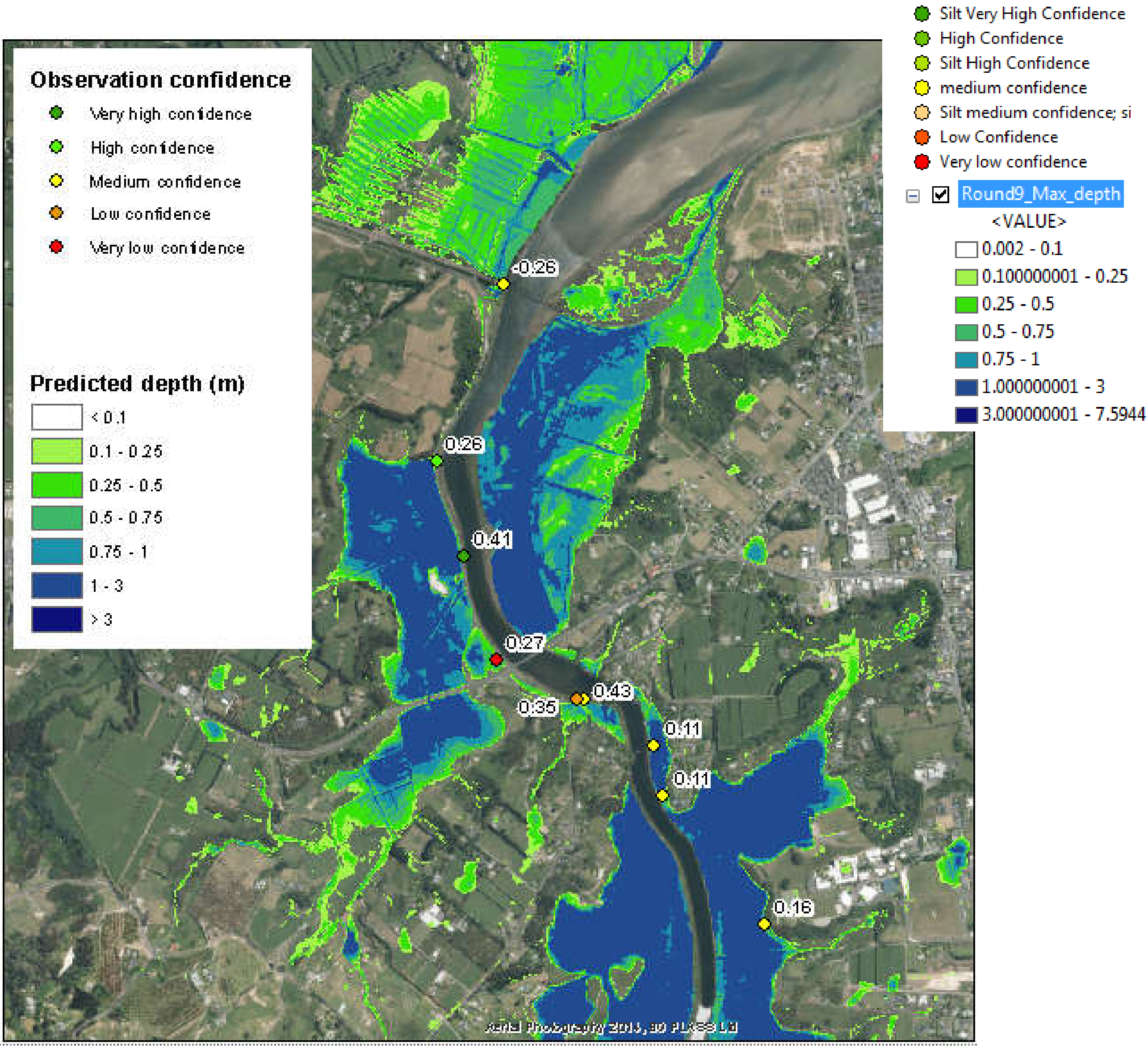

Figure 5-3 Difference between predicted and recorded peak flood levels, 29 January 2011 event, lower reaches.

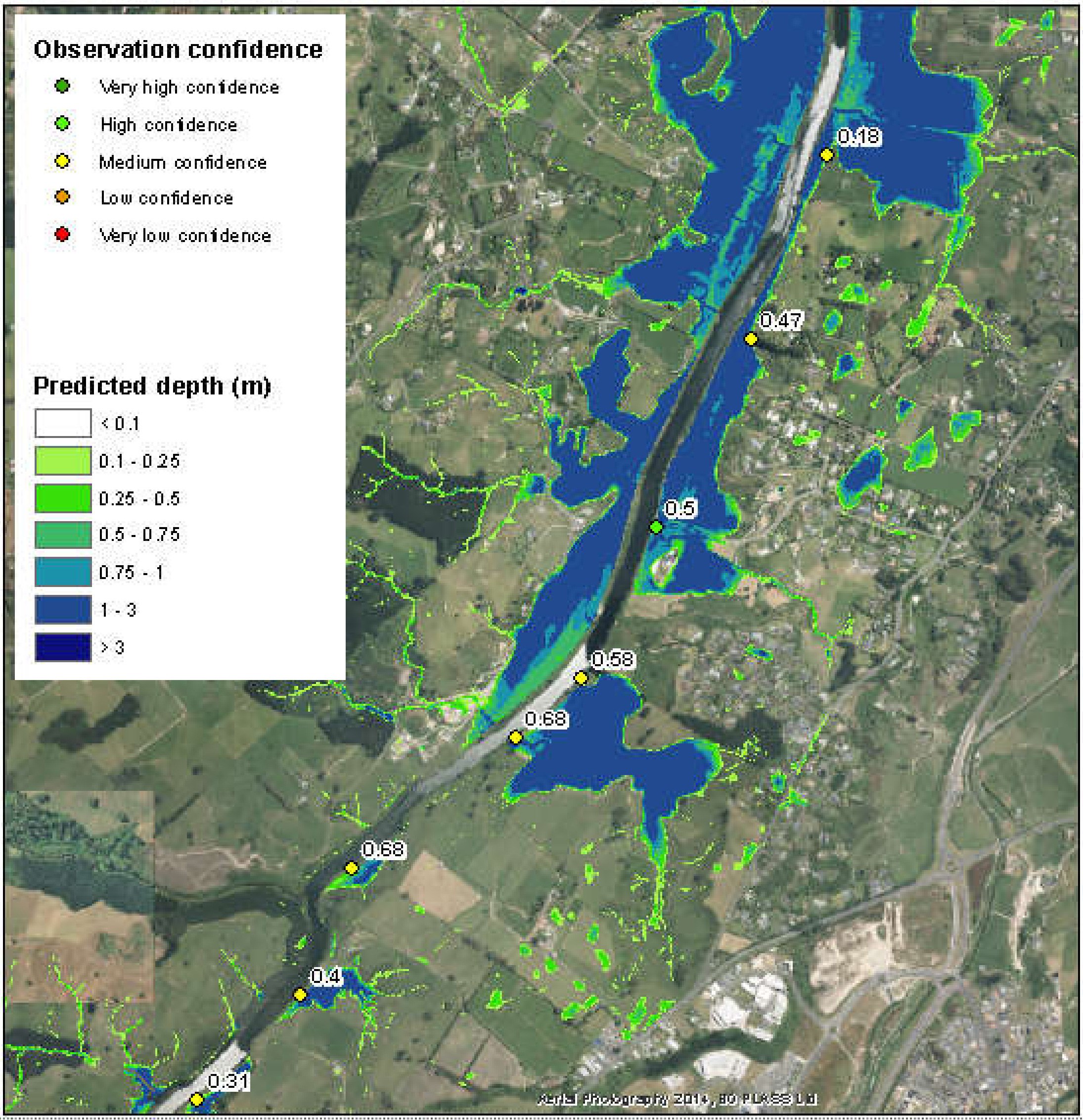

Figure 5-4 Difference between predicted and recorded peak flood levels, 29 January 2011 event, mid reaches

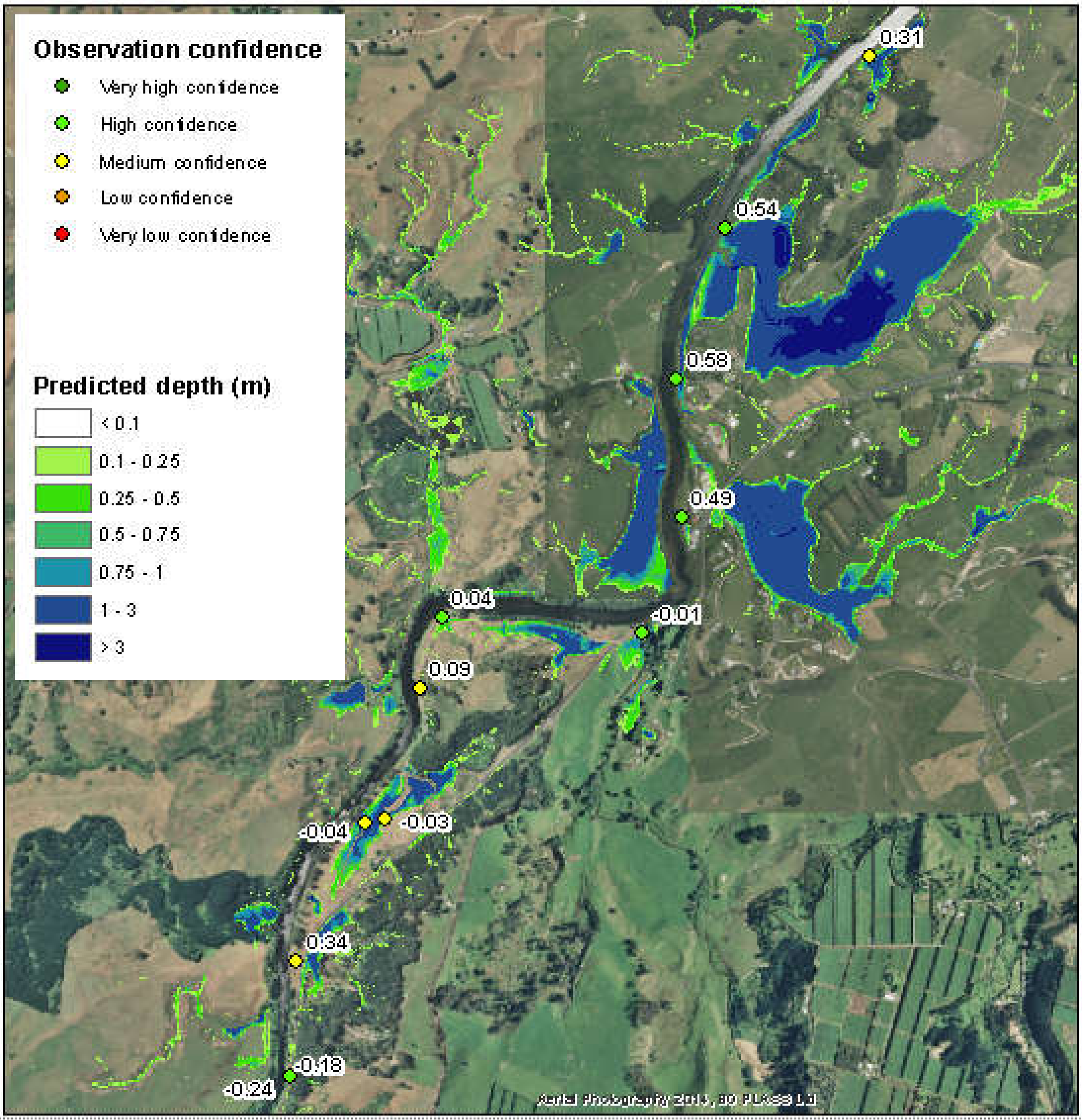

Figure 5-5 Difference between predicted and recorded peak flood levels, 29 January 2011 event, upper reaches

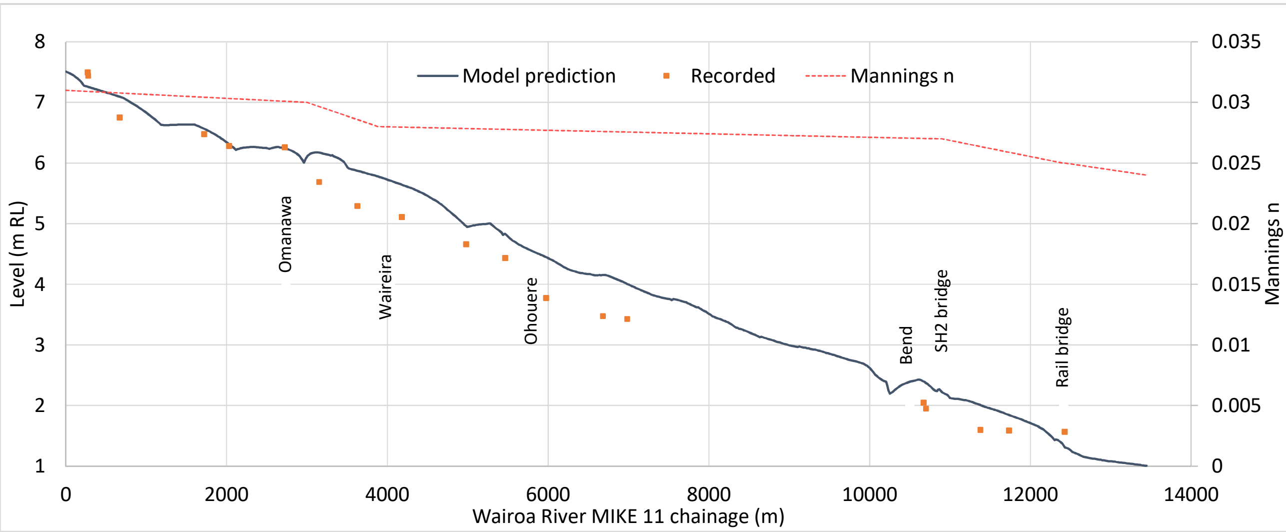

Figure 5-6 Final model validation runs, river channel peak levels

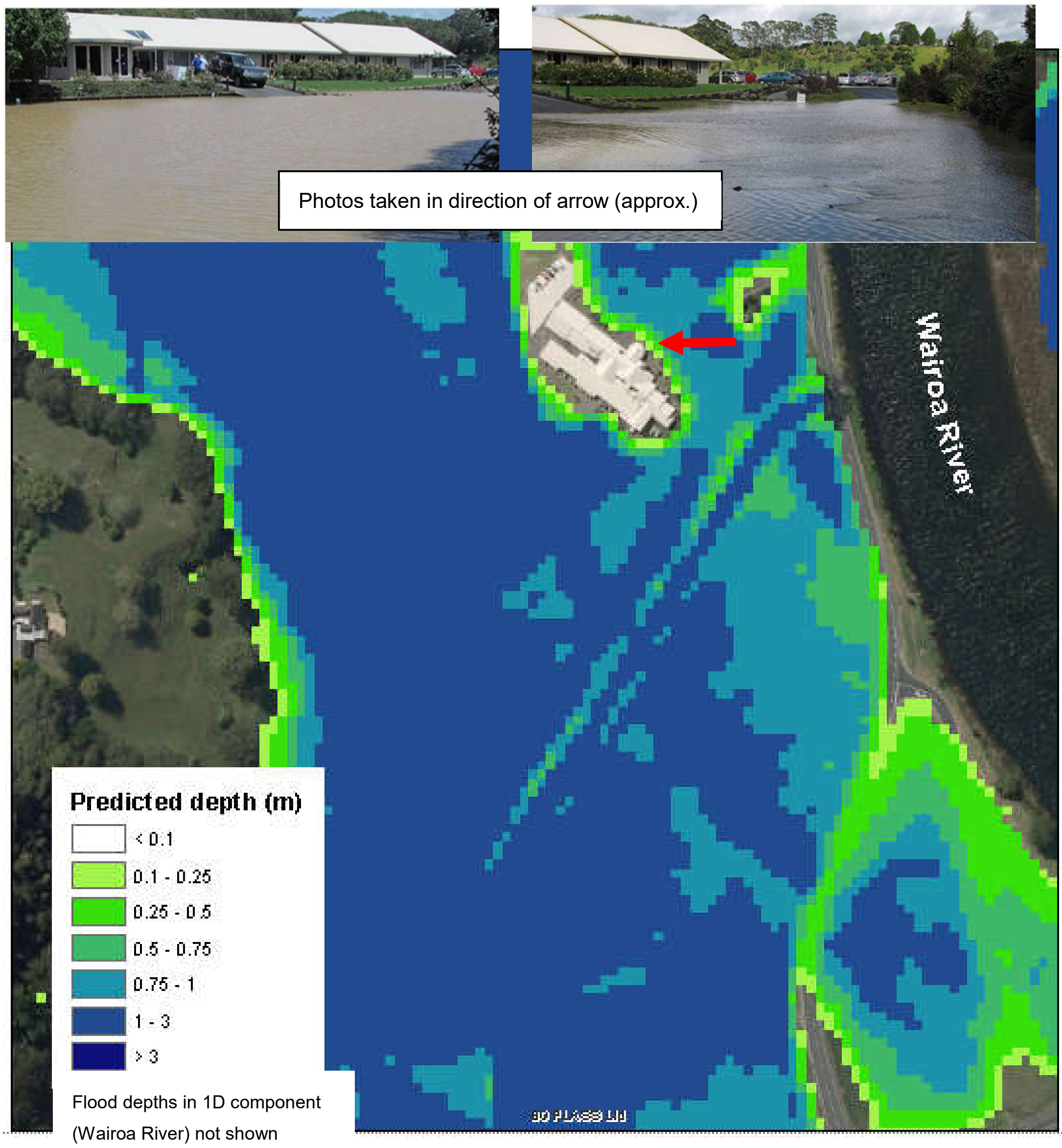

Figure 5-7 Flood depths at Hospice, 29 January 2011: modelled and photographs

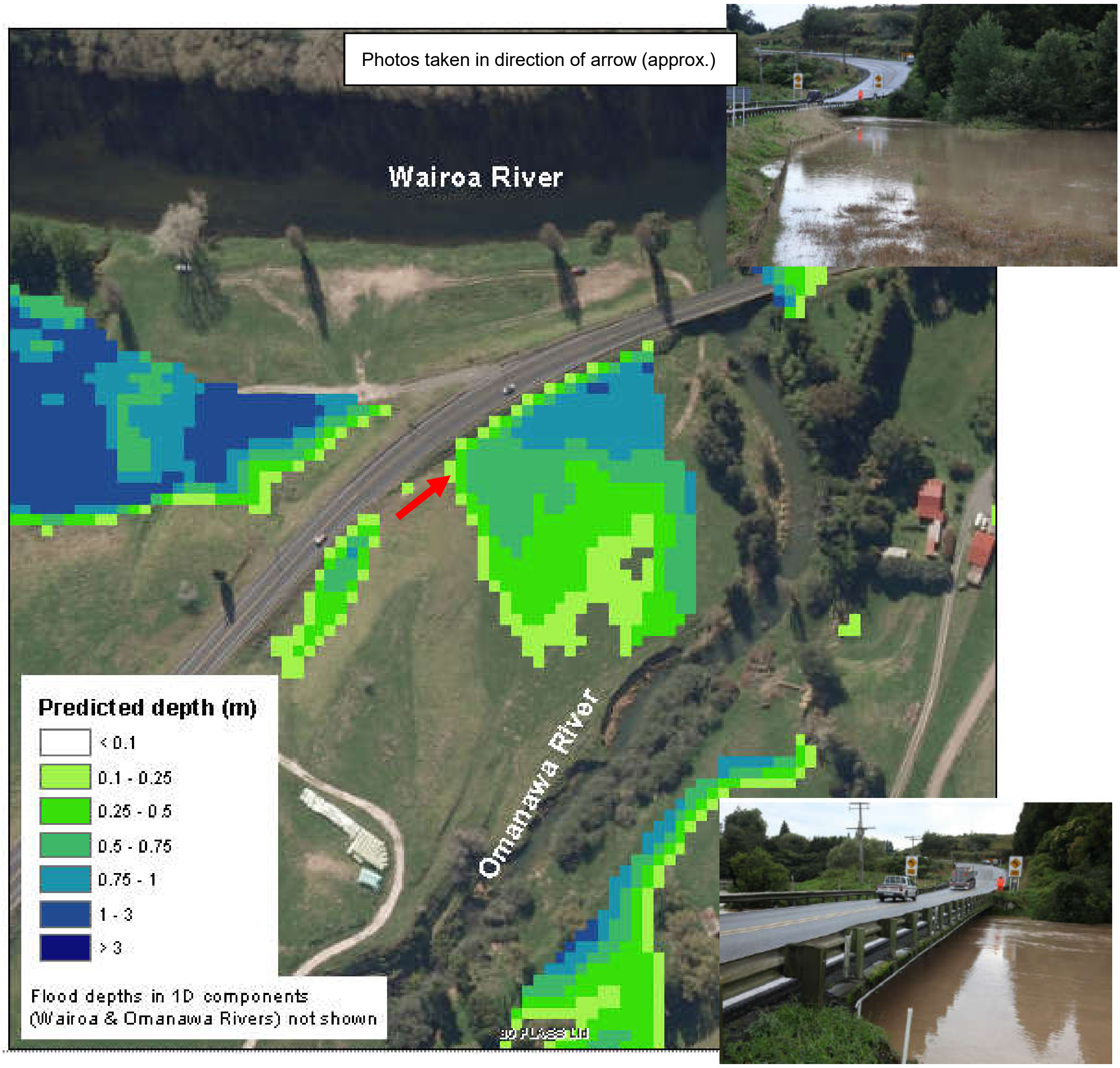

Figure 5-8 Flood depths at Omanawa confluence, 29 January 2011: modelled and photographs

6 Design events

6.1 Current climate

Two rainfall and tide combinations were modelled:

- 100-year river flow and rainstorm combined with a 10-year tide (from storm surge);
- 100-year tide combined with a 10-year river flow and rainstorm.

6.1.1 Design flows

Initial model simulations were based on a Wairoa River 100-year design flow at the upstream end of the model (immediately downstream of the Ruahihi power station) of 2,200 m^3/s. This flow was obtained from the HEC-HMS model of the upper catchment (refer Appendix B), with rainfall inputs as described in Appendix C.

However, a flood frequency analysis using annual maxima of recorded flows gave a lower design flow: 1,138 m^3/s (albeit upstream of Ruahihi). After some discussion (Appendix F), TCC, BOPRC and DHI agreed to adopt a 100-year design flow of 1,500 m^3/s, upstream of Ruahihi. That corresponds to 1,582 m^3/s downstream of Ruahihi, at the upstream end of the MIKE 11 model.

The Wairoa River 10-year design flow adopted (downstream of Ruahihi) was 755 m^3/s.

Appendix D also documents the design flows for other sub-catchments.

6.1.2 Design rainstorm

All scenarios modelled assume 48-hour nested rainstorms.

Over the lower floodplain, the TCC design hyetographs have been applied as rain-on-grid to the MIKE 21 FM model. This assumption gives consistency with models of other stormwater catchments in the Tauranga district.

In the upper catchment (i.e. for the hydrological model), design rainstorms were derived from HIRDS v3, as described in Appendix C. These hyetographs have less intense peaks than the TCC hyetographs. As noted above and in Appendix D however, the resulting flows provided by the HEC-HMS hydrological model were subsequently scaled down.

6.1.3 Design tide levels

Downstream harbour levels were as used in other stormwater catchment studies commissioned by TCC, with peak 10-year and 100-year levels of 1.36 m and 1.52 m respectively. The tides were timed so that the peak high tide level coincided (approximately) with the arrival of the river flood.

6.1.4 Results

The maximum results of the two simulations were combined to create the final “100-year ARI” results.

Figure 6-1 is a summary of the flood depth map. (Note that no allowance for freeboard has been made in any of the maps presented in this report. Furthermore, depths within the 1-D component of the model are not shown.) More detailed maps have been provided to TCC separately.

A long-section of peak flood levels down the Wairoa River is presented in Figure 6-2.

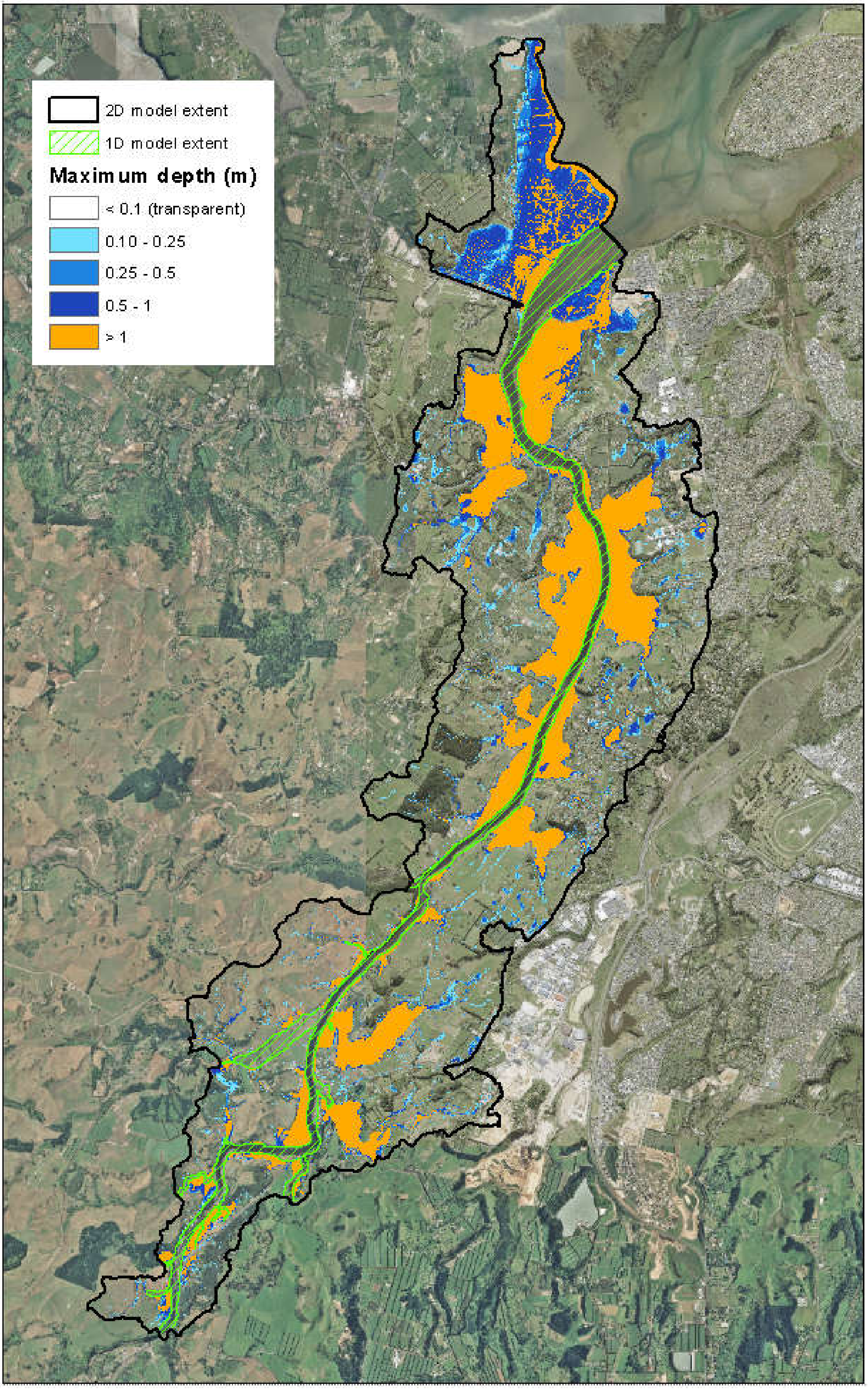

Figure 6-1 100-year ARI flood map, peak depths (current climate, "2005")

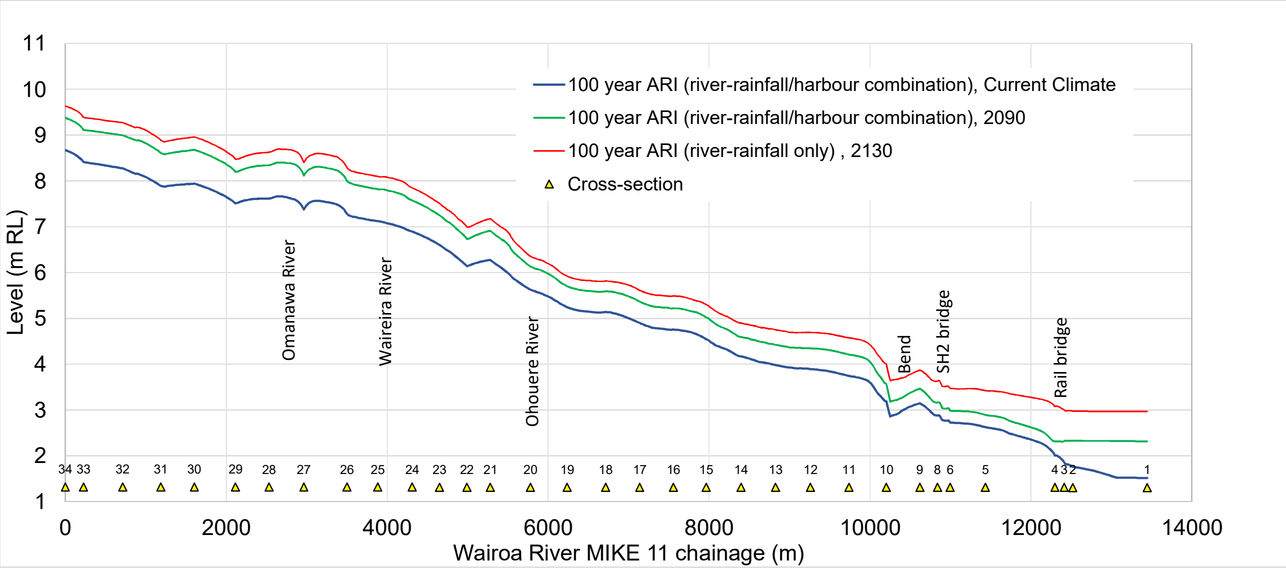

Figure 6-2 100-year ARI peak flood levels, Wairoa River

6.2 2090 climate

6.2.1 Assumptions

As noted in section 2.1 above, TCC requested 2090 climate change simulations to aid in the assessment of the TNL proposal.

To allow for climate change, an 800 mm sea level rise and a 2.1°C warming were both assumed.

Initial simulations with and without the TNL were undertaken with the earlier design flows; these were based on an upstream river flow of 2,600 m^3/s (downstream of Ruahihi) for the 100-year 2090 scenario. Results were provided to TCC in March 2017[8].

However, as noted in section 6.1.1 and in Appendix G, design flows were subsequently reduced. In the case of the 100-year (2090 climate change) scenario, the assumed upstream river flow is now 1,870 m^3/s. This an 18.2% increase on the current climate river flow, arrived at by applying the ratio of the HEC-HMS outputs for the 2090 and current climate rainstorm inputs (i.e. 2,600/2,200) to the revised current climate river flow estimate. The same scaling has been applied to other sub-catchment inflows.

As the TNL design has now progressed since that time and as TCC is now considering climate change to 2130, the updated flows have not been rerun with TNL proposal.

Nonetheless, the current situation has been modelled with the 2090 climate change assumption, with the updated flows. Again, two scenarios have been modelled:

- 100-year river flow and rainstorm combined with a 10-year tide (from storm surge);
- 100-year tide combined with a 10-year river flow and rainstorm.

The TCC (2090) rainfall has been applied as “rain-on-grid” to the MIKE 21 FM domain.

[8] Email from Philip Wallace (DHI) to Graeme Jelley (TCC), 3 March 2017

6.2.2 Results

The maximum results of the two simulations were combined to create the final “100-year” 2090 results. Figure 6-3 is a summary of the flood depth map. More detailed maps have been provided to TCC separately. Figure 6-2 presents a long-section of peak levels in the Wairoa River.

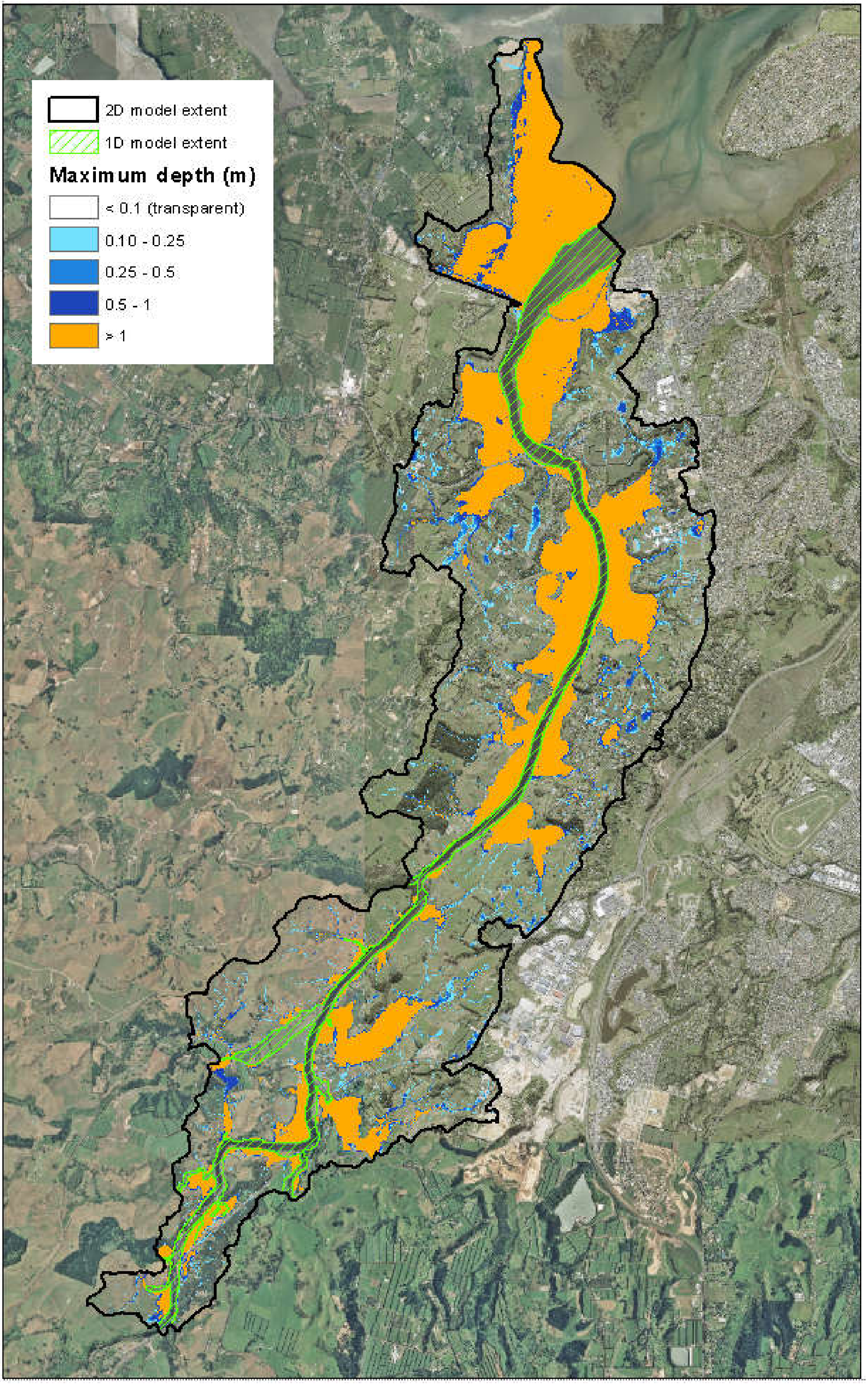

Figure 6-3 100-year ARI flood map (2090), peak depths

6.3 2130 climate

6.3.1 Assumptions

As noted in section 2.1 above, TCC requested 2130 climate change simulations to aid in the preliminary assessment of infilling to support urban development of the Tauriko West area, as well as further assessment of the TNL proposal.

The 100-year (2130 climate change) scenario assumes 3°C temperature increase and a corresponding 24% increase in rainfall depths (Appendix D). Given the uncertainties inherent in climate change forecasts (as well in the current flow statistics), a simple scaling up of river sub-catchment inflows by 25% from the current day values is considered appropriate. Thus, for example, the assumed inflow at the upstream end of the model is 1,978 m^3/s.

In the case of the existing situation (i.e. without the TNL or the Tauriko West development), TCC requested that a sea level rise of 1.25 m be assumed. The 100-year river flow/rain event was to be modelled in conjunction with a 20-year tide (storm surge) condition. Results of that scenario are presented in this report.

With regards to the Tauriko West and TNL proposals, TCC requested 100-year and 500-year river flow/rainfall scenarios with 1.25 m and 1.9 m sea level rises to be modelled. Results from those scenarios were presented to TCC in August and September 2017[9]. Since that time, the Tauriko West proposals have since been revised and the assessment is the subject of a new project. Hence those results have not been reproduced in this report.

6.3.2 Results

Peak flood depths for 100-year (2130) rainfall/river flow scenario are shown in Figure 6-4. Figure 6-2 presents a long-section of peak levels in the Wairoa River. Note that, unlike the current climate and 2090 results presented above, the 2130 results do not include the 10-year river flow plus 100-year harbour tide condition.

[9] Emails from Philip Wallace (DHI) to Graeme Jelley (TCC), 25 & 31 August 2017 and 12, 13 & 15 September 2017

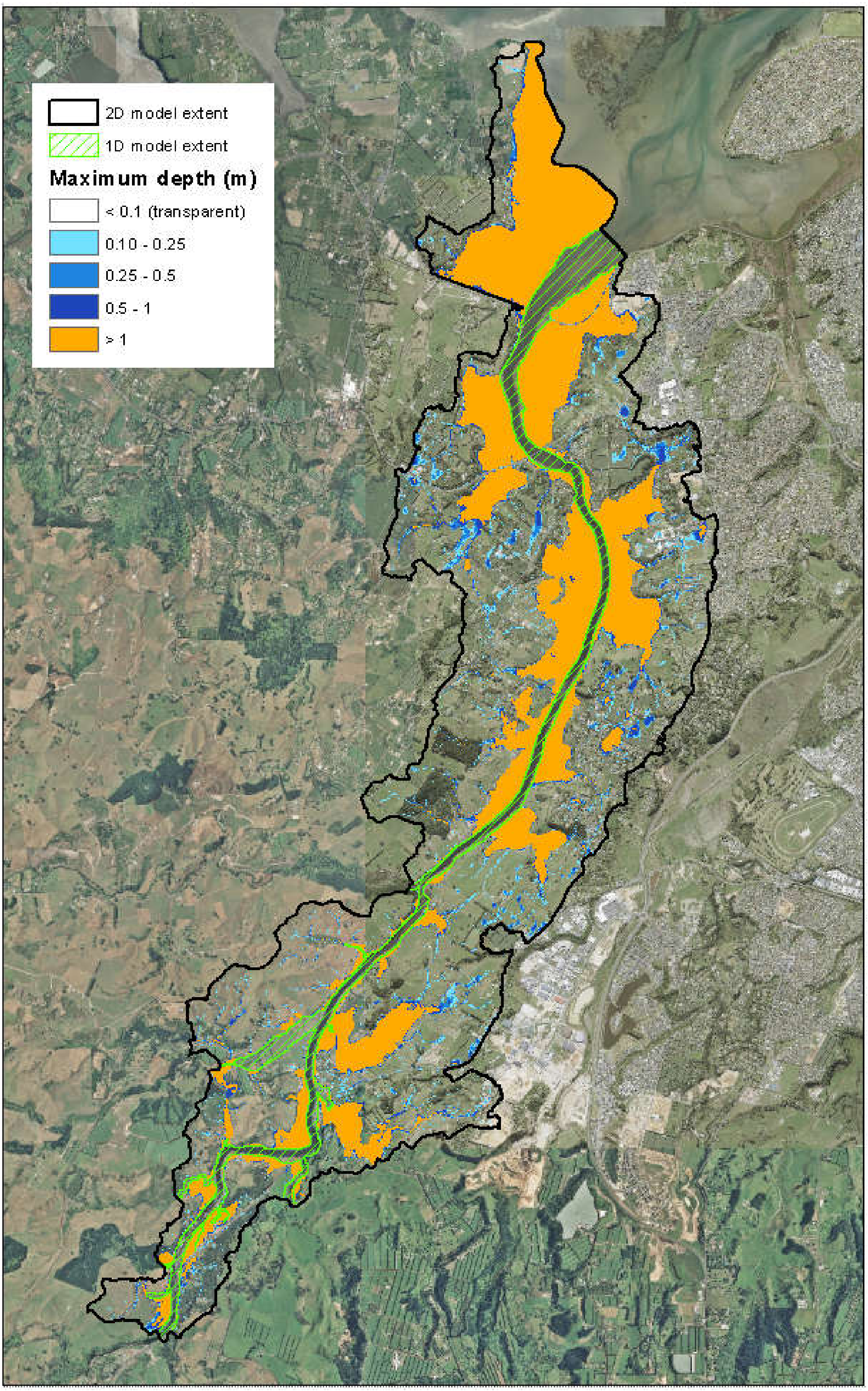

Figure 6-4 100-year ARI flood map (2130), peak depths

7 Results presentation and reporting

The following deliverables were provided to Tauranga City Council in digital format:

- All cross section information that has been surveyed for the purposes of this study;
- All model files required to run the verification events, the design events and the sensitivity scenarios;
- All result and summary files for all components of the MIKE FLOOD model for the verification events, the design events and the sensitivity scenarios;
- GIS polygon of the outline of the modelled catchment extent;
- ArcGIS geodatabase containing the post-processed rasters of:
    - maximum flood depth,
    - maximum flood surface elevation,
    - maximum velocity, and
    - maps of maximum flood depth, surface elevation and velocity, for the verification events and the design events.
- This technical report in electronic format.

Maximum flood extents and surface elevation files were named according to the TCC requirements and provided in the format specified in *Guidelines for Stormwater Modelling using MIKE FLOOD* (DHI, 2017).

8 Quality assurance

As a part of the internal QA at DHI, integrity checks have been made throughout the modelling process to ensure quality in both the computational model and the outputs.

Detailed checks were performed during the model build, validation and production run phases to ensure the models were setup in the correct way and the outputs produced are sensible and without evidence of any significant numerical instabilities.

9 Conclusions and recommendations

The production of the flood maps for the lower Wairoa catchment has involved development and refinement of both a hydrological model and a hydraulic model. The hydrological model in particular was reasonably complex, given that much of the upper catchment is part of the Kaimai HEPS, with its multiple diversions, lake storages and power stations.

The HEC-HMS model of the Kaimai HEPS, developed by Opus, was a work-in-progress at the time of this report. Further refinement and development of that model is possible in the course of the investigations that Opus is carrying out for Trustpower. It is recommended that TCC keep abreast of any revisions of the HEC-HMS model made on behalf of Trustpower.

Future refinement of the hydrological model could consider making use of rain radar data, to complement data from the rain gauge network.

The three-way coupled MIKE FLOOD hydraulic model of the river, floodplain and stormwater network, all downstream of Ruahihi, has been built using recent data. Although a comprehensive set of peak flood levels along the river measured following the 29 January 2011 storm event was available for model validation, the model predictions in the mid to lower reaches were higher than the observations. Nonetheless, realistic model parameters (primarily channel resistance values) have been used, and the model validation is considered acceptable.

In the event of future flood events, it is recommended that efforts be made to carry out high flow gauging of the Wairoa River recorder immediately upstream of the Ruahihi power station outfall. Peak flood levels along the river and on the floodplain should again be recorded and the maximum flood extent should be captured (e.g. by aerial photography and ground truth surveys). Observations of flows and blockages at all major structures should also be made.

The MIKE FLOOD model and the flood maps will need regular review and refinement to ensure that they are kept up-to-date in light of the ongoing urban development and land use changes in the lower catchment.

10 References

/1/ Tauranga City Council (2015); *Request for Proposal TC39/15 for Mike Flood Model Build for Wairoa Stormwater Catchment*

/2/ DHI; *Guidelines for Stormwater Modelling using MIKE FLOOD*; Report produced for Tauranga City Council; July 2017.

/3/ DHI; *MIKE FLOOD model conversion guidelines - Classic grid to Flexible mesh;* Technical note produced for Tauranga City Council, November 2014.

/4/ BECA; *TCC Rainfall Profile Review*; Report produced for Tauranga City Council, November 2014.

/5/ AECOM; *Joint Probability Analysis of Sea Level and Rainfall*; Report produced for Tauranga City Council, June 2014.

/6/ USGS; *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains;* Paper; 2009.

/7/ DHI; *Soakage and Infiltration - spreadsheet describing soakage and infiltration parameters for different catchment types, land covers and elevations*; *September 2013.*

/8/ Tonkin & Taylor Ltd; *Kaimai Hydro Electric Power Scheme: PIC Assessment;* Report produced for Trustpower, 2013.

/9/ Riley Consultants Ltd; *Kaimai HEPS Flood Study;* Report produced for Trustpower, 2005.

/10/ Ryder Consulting Ltd; *Wairoa River at Tauriko – Floodplain Assessment*; Report produced for Boma No. 2 Trust, May 2010.

/11/ DHI; *Modelling Approach for Soak Holes*; *Email describing recommended approach for soak holes for different catchment types and elevation;* Sent out 3rd of April 2014.

APPENDICES

APPENDIX A – Roles

Project Team

A Project Team

Table A-1 Wairoa stormwater catchment model project team

Organisation	Name	Role
Tauranga City Council	Dayananda Kapugama	Project Manager (Client)
DHI	Dragan Tutulic	Project Manager and Internal QC
DHI	Philip Wallace	Technical Lead
DHI	Henrik Locke	Project Engineer
DHI	Antoinette Taylor	Senior Project Engineer
DHI	Nancy Zhang	Project Engineer
DHI	Aloïs Denervaud	Project Engineer

APPENDIX B–Hydrological Model

Sub-catchments and HEC-HMS model

B Sub-catchments and HEC-HMS model

B.1 Sub-catchments and parameters

The HEC-HMS model of the Kaimai HEPS recently updated by Opus, as part of an exercise being carried out by for Trustpower, includes sub-catchments upstream of Ruahihi as well as the Omanawa sub-catchment. These sub-catchments are shown in Figure B-1 and listed in Table B-1. More detail on the sub-catchment parameters adopted during the calibration Opus undertook is given in Table B-1 (email from Paul Mitchell to Philip Wallace, 27 May 2016).

As described in sections 5.1 and 4.2 of this report, that model has been validated against the 29 January 2011 storm event and then used to generate design hydrographs for the hydraulic modelling.

DHI has also prepared HEC-HMS models for the sub-catchments downstream of Ruahihi (those shown in Figure 4-2). Table B-1 gives the key model parameters adopted after consideration of those in Table B-1. These lower sub-catchment HEC-HMS models have then been used to generate sub-catchment runoff hydrographs for the validation and design events.

Table B-1 Sub-catchments in upper HEC-HMS model

Sub-catchment		Area (km^2)	Initial loss (mm) Jan11/Design	Continuing loss(mm/hr) Jan11/Design	Time of concentration (hours)
A	Upper Opuiaki	54.7	10/8	4/2	2.3
B	Tauwharawhara	1.3	10/8	4/2	0.5
C	Ngatuhoa	21.1	10/8	4/2	2
D	Awakotuku	1.23	10/8	4/2	0.5
E	Lake Mangaonui	4.85	10/8	8/2	1
F	Upper Mangapapa	42.8	10/8	8/2	2.8
G	Ruakaka Stream	3.06	10/8	8/2	1.8
H	Omanawa (upper)	52.1			
I	Opuiaki	25.1	10/8	4/2	2.5
J	Lower Mangapapa	11.4	10/8	8/2	1.5
K	Mangakarengorengo	108.3	10/8	8/2	2 (Jan11) 1.8 (Design)
L	Lake McLaren	4.7	10/8	8/2	1
M	Ruahihi tributaries	7.7	10/8	8/2	1.5
N		0.29			
O		2.51			
P		0.56			
Q	Wairoa local	14	10/8	8/2	1

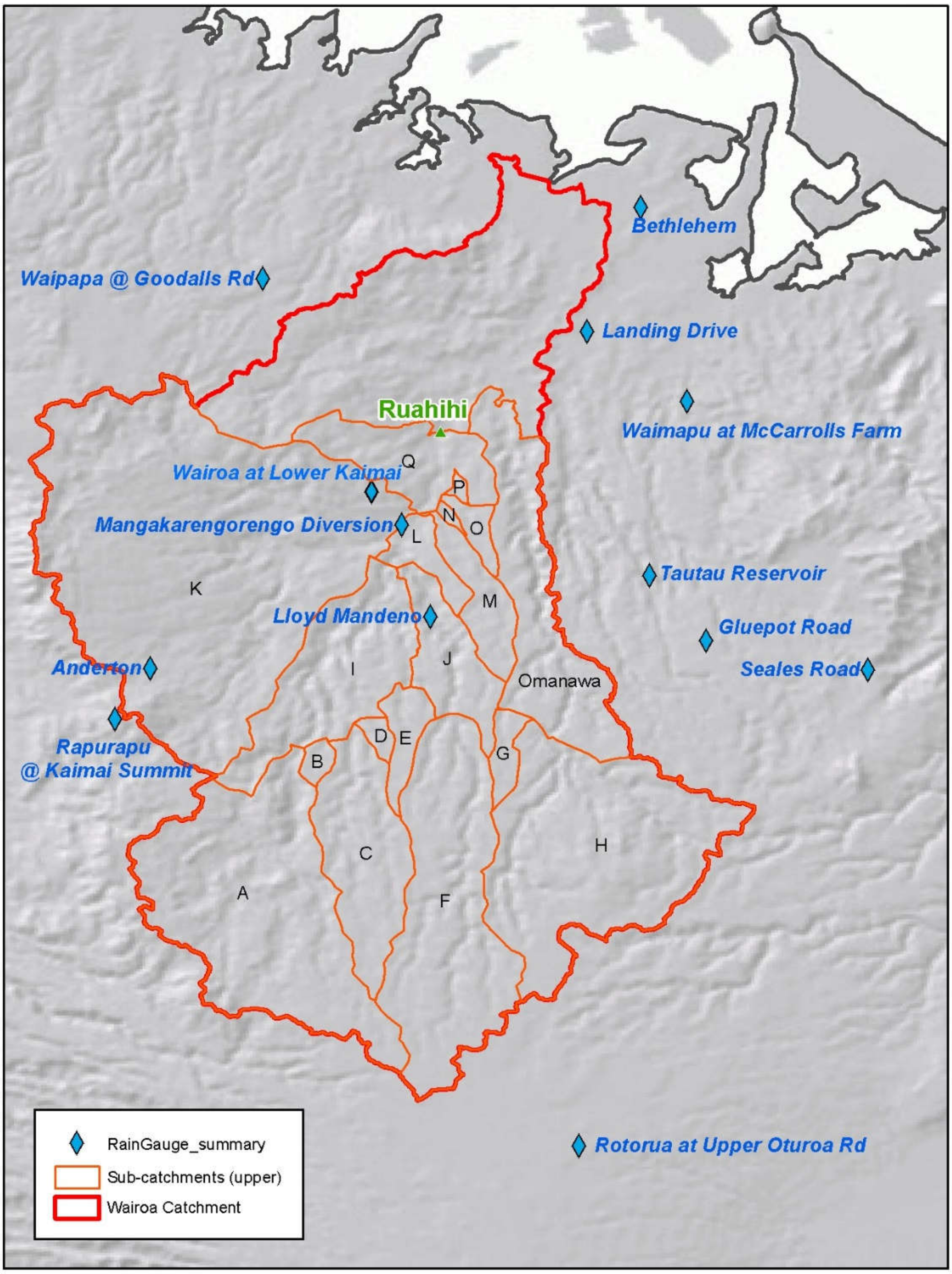

Figure B-1 Sub-catchments (upper) used in HEC-HMS model and rain gauge locations

Table B-1 Model parameters for upper catchment HEC-HMS model, Opus calibration

Components	Area km^2	Initial Loss (mm)	Constant Loss (mm/hr)	Tc (hrs)	Storage (hrs)	Baseflow	Initial Baseflow (cumecs)	Baseflow Recession Constant	Baseflow Ratio
Omanawa and Ruakaka c/ments									
July 1998		8.0	2.0	1.8	2.5	None	NA	NA	NA
May 1999		20.0	15.0	1.8	2.5	None	NA	NA	NA
Feb 2004		30.0	22.0	1.8	3.5	None	NA	NA	NA
Jan 2011a	55.2	30.0	8.0	1.8	2.0	None	NA	NA	NA
Jan 2011b		10.0	8.0	1.8	2.0	None	NA	NA	NA
Average		*19.6*	*11.0*	*1.8*	*2.5*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**1.8**	**2.0**	**None**	**NA**	**NA**	**NA**
Upper Mangapapa c/ment									
July 1998		8.0	2.0	2.8	3.5	None	NA	NA	NA
May 1999		20.0	15.0	2.8	3.5	None	NA	NA	NA
Feb 2004		30.0	22.0	2.8	4.5	None	NA	NA	NA
Jan 2011a	42.8	30.0	3.0	2.8	3.0	None	NA	NA	NA
Jan 2011b		10.0	8.0	2.8	3.0	None	NA	NA	NA
Average		*19.6*	*10.0*	*2.8*	*3.5*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**2.8**	**3.0**	**None**	**NA**	**NA**	**NA**
Lake Mangaonui									
July 1998		8.0	2.0	1.0	1.5	None	NA	NA	NA
May 1999		20.0	15.0	1.0	1.5	None	NA	NA	NA
Feb 2004		30.0	22.0	1.0	2.5	None	NA	NA	NA
Jan 2011a	4.86	30.0	7.0	1.0	1.5	None	NA	NA	NA
Jan 2011b		10.0	8.0	1.0	1.5	None	NA	NA	NA
Average		*19.6*	*10.8*	*1.0*	*1.7*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**1.0**	**1.5**	**None**	**NA**	**NA**	**NA**
Upper Opuiaki c/ment									
July 1998		8.0	2.0	2.3	3.0	None	NA	NA	NA
May 1999		20.0	12.0	2.3	3.0	None	NA	NA	NA
Feb 2004		20.0	15.0	2.3	4.0	None	NA	NA	NA
Jan 2011a	54.7	30.0	2.0	2.3	2.5	None	NA	NA	NA
Jan 2011b		10.0	4.0	2.3	2.5	None	NA	NA	NA
Average		*17.6*	*7.0*	*2.3*	*3.0*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**2.3**	**2.5**	**None**	**NA**	**NA**	**NA**
Tauwharawhara c/ment									
July 1998		8.0	2.0	0.5	0.5	None	NA	NA	NA
May 1999		20.0	15.0	0.5	0.5	None	NA	NA	NA
Feb 2004		20.0	15.0	0.5	1.0	None	NA	NA	NA
Jan 2011a	1.3	30.0	2.0	0.5	0.5	None	NA	NA	NA
Jan 2011b		10.0	4.0	0.5	0.5	None	NA	NA	NA
Average		*17.6*	*7.6*	*0.5*	*0.6*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**0.5**	**0.5**	**None**	**NA**	**NA**	**NA**
Ngatuhoa c/ment									
July 1998		8.0	2.0	2.0	3.0	None	NA	NA	NA
May 1999		20.0	15.0	2.0	3.0	None	NA	NA	NA
Feb 2004		20.0	15.0	2.0	3.5	None	NA	NA	NA
Jan 2011a	21.1	30.0	2.0	2.0	2.0	None	NA	NA	NA
Jan 2011b		10.0	4.0	2.0	2.0	None	NA	NA	NA
Average		*17.6*	*7.6*	*2.0*	*2.7*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**2.0**	**2.0**	**None**	**NA**	**NA**	**NA**
Awakotuku c/ment									
July 1998		8.0	2.0	0.5	1.0	None	NA	NA	NA
May 1999		20.0	15.0	0.5	1.0	None	NA	NA	NA
Feb 2004		20.0	15.0	0.5	1.5	None	NA	NA	NA
Jan 2011a	1.23	30.0	2.0	0.5	0.5	None	NA	NA	NA
Jan 2011b		10.0	4.0	0.5	0.5	None	NA	NA	NA
Average		*17.6*	*7.6*	*0.5*	*0.9*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**0.5**	**0.5**	**None**	**NA**	**NA**	**NA**

Components	Area km^2	Initial Loss (mm)	Constant Loss (mm/hr)	Tc (hrs)	Storage (hrs)	Baseflow	Initial Baseflow (cumecs)	Baseflow Recession Constant	Baseflow Ratio
Lower Mangapapa c/ment									
July 1998	11.4	8.0	2.0	1.5	2.0	None	NA	NA	NA
May 1999		20.0	15.0	1.5	2.0	None	NA	NA	NA
Feb 2004		20.0	22.0	1.5	3.0	None	NA	NA	NA
Jan 2011a		30.0	3.0	1.5	2.0	None	NA	NA	NA
Jan 2011b		10.0	8.0	1.5	2.0	None	NA	NA	NA
Average		*17.6*	*10.0*	*1.5*	*2.2*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**1.5**	**2.0**	**None**	**NA**	**NA**	**NA**
Lake McLaren c/ment									
July 1998	4.7	8.0	2.0	0.7	1.0	None	NA	NA	NA
May 1999		20.0	15.0	0.7	1.0	None	NA	NA	NA
Feb 2004		20.0	15.0	0.7	2.0	None	NA	NA	NA
Jan 2011a		30.0	8.0	0.7	1.0	None	NA	NA	NA
Jan 2011b		10.0	8.0	0.7	1.0	None	NA	NA	NA
Average		*17.6*	*9.6*	*0.7*	*1.2*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**0.7**	**1.0**	**None**	**NA**	**NA**	**NA**
Opuiaki R c/ment									
July 1998	25.1	8.0	2.0	1.8	2.5	None	NA	NA	NA
May 1999		20.0	15.0	1.8	2.5	None	NA	NA	NA
Feb 2004		20.0	15.0	1.8	3.5	None	NA	NA	NA
Jan 2011a		30.0	2.0	1.8	2.5	None	NA	NA	NA
Jan 2011b		10.0	4.0	1.8	2.5	None	NA	NA	NA
Average		*17.6*	*7.6*	*1.8*	*2.7*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**1.8**	**2.5**	**None**	**NA**	**NA**	**NA**
Ruahihi Tribs									
July 1998	11.1	8.0	2.0	1.5	2.0	None	NA	NA	NA
May 1999		20.0	15.0	1.5	2.0	None	NA	NA	NA
Feb 2004		20.0	15.0	2.0	2.5	None	NA	NA	NA
Jan 2011a		30.0	8.0	1.5	1.5	None	NA	NA	NA
Jan 2011b		10.0	8.0	1.5	1.5	None	NA	NA	NA
Average		*17.6*	*9.6*	*1.6*	*1.9*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**1.5**	**1.5**	**None**	**NA**	**NA**	**NA**
Mangakarengorengo catchment									
July 1998	108.3	8.0	2.0	2.5	2.7	Recession	30	0.37	0.70
May 1999		20.0	15.0	1.8	2.5	Recession	12	0.40	0.85
Feb 2004		20.0	12.0	2.5	4.0	Recession	25	0.46	0.99
Jan 2011a		30.0	8.0	2.0	2.2	Recession	15	0.40	0.85
Jan 2011b		10.0	4.0	2.0	2.2	Recession	15	0.40	0.88
Average		*17.6*	*8.2*	*2.2*	*2.7*	Recession	*19*	*0.41*	*0.85*
For Design		**8.0**	**2.0**	**1.8**	**2.2**	**Recession**	**30**	**0.50**	**0.90**
Local c/ment ds McLaren									
July 1998	14.0	8.0	2.0	1.0	1.5	None	NA	NA	NA
May 1999		20.0	15.0	1.0	1.5	None	NA	NA	NA
Feb 2004		20.0	15.0	1.5	2.0	None	NA	NA	NA
Jan 2011a		30.0	8.0	1.0	1.0	None	NA	NA	NA
Jan 2011b		10.0	8.0	1.0	1.0	None	NA	NA	NA
Average		*17.6*	*9.6*	*1.1*	*1.4*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**1.0**	**1.0**	**None**	**NA**	**NA**	**NA**
Omanawa local c/ment to Wairoa									
July 1998	31.8	8.0	2.0	2.0	2.0	None	NA	NA	NA
May 1999		20.0	15.0	2.0	2.0	None	NA	NA	NA
Feb 2004		10.0	15.0	2.0	2.0	None	NA	NA	NA
Jan 2011a		30.0	8.0	2.0	2.0	None	NA	NA	NA
Jan 2011b		10.0	8.0	2.0	2.0	None	NA	NA	NA
Average		*15.6*	*9.6*	*2.0*	*2.0*	None	NA	NA	NA
For Design		**8.0**	**2.0**	**2.0**	**2.0**	**None**	**NA**	**NA**	**NA**

Table B-2 Sub-catchments in lower HEC-HMS models

Sub-catchment	Area (km^2)	Initial loss (mm) Jan11/Design	Continuing loss(mm/hr) Jan11/Design	Time of concentration (hours)
Wairoa6	0.63	10/8	8/2	0.5
Wairoa5	1.85	10/8	8/2	0.83
East Omanawa	2.21	10/8	8/2	0.75
Waireia (Wairoa3)	8.78	10/8	8/2	1.2
Ohourere (Wairoa4)	29.78	10/8	8/2	2.5
Vernon Rd (Wairoa1)	1.32	10/8	8/2	0.7

B.2 Analysis of 29 January 2011 storm

As an alternative check of the Opus HEC-HMS model calibration, Thiessen polygons were derived around the rain gauges for the 29 January 2011 storm and used to derive weighted rainfall hyetographs for the sub-catchments in the HEC-HMS model. The location of the gauges and the Thiessen polygons are shown in Figure B-2 The total rainfall recorded at each gauge is shown in Figure B-3. As noted in section 5.1, the Wairoa at Lower Kaimai recorder has not been used, despite having a recorded total more in line with other rain gauges than the Mangakerengorengo recorder, as only daily rainfall totals were recorded.

As concluded in 5.1, the simpler Opus approach using only the rain gauge data from the Lloyd Mandeno recorder gives a better fit to the recorded flow at Ruahihi than the approach of using all the rain gauges and Thiessen polygons.

Figure B-4 gives a comparison of the rain gauge data with rain radar data for the storm (ratio of rain gauge total to rain radar total).

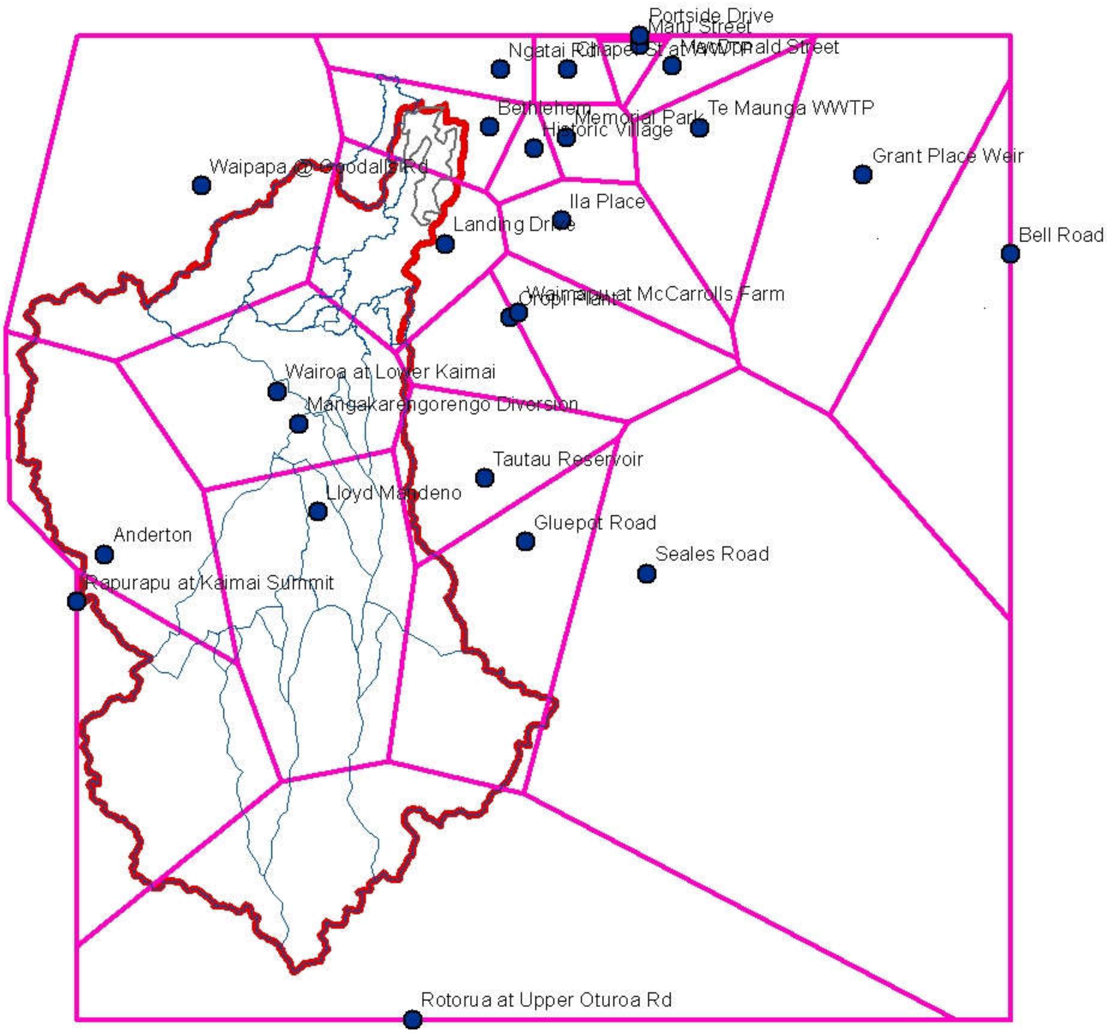

Figure B-2 Wairoa sub-catchments, rain gauges and Thiessen polygons

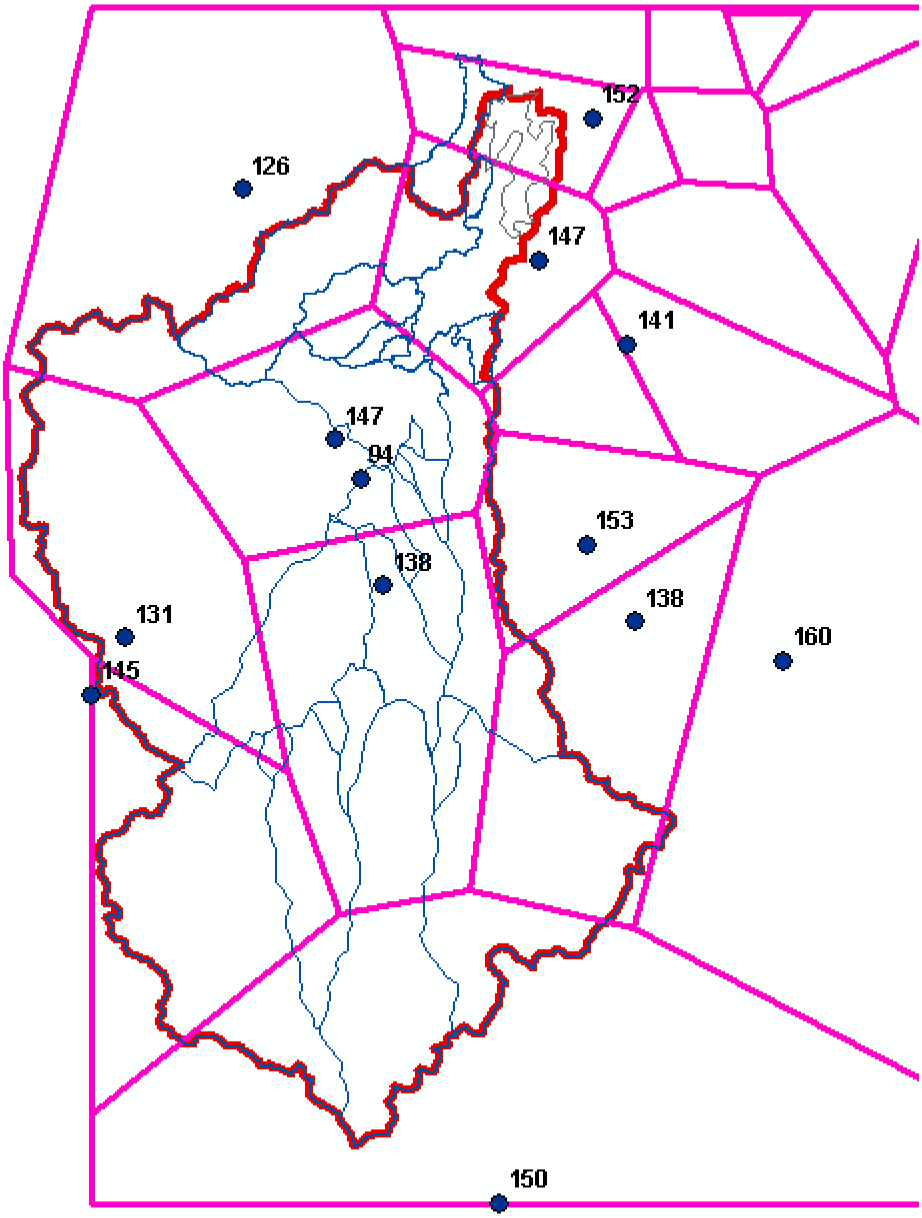

Figure B-3 Rainfall totals (mm), 28th – 29th January 2011 event

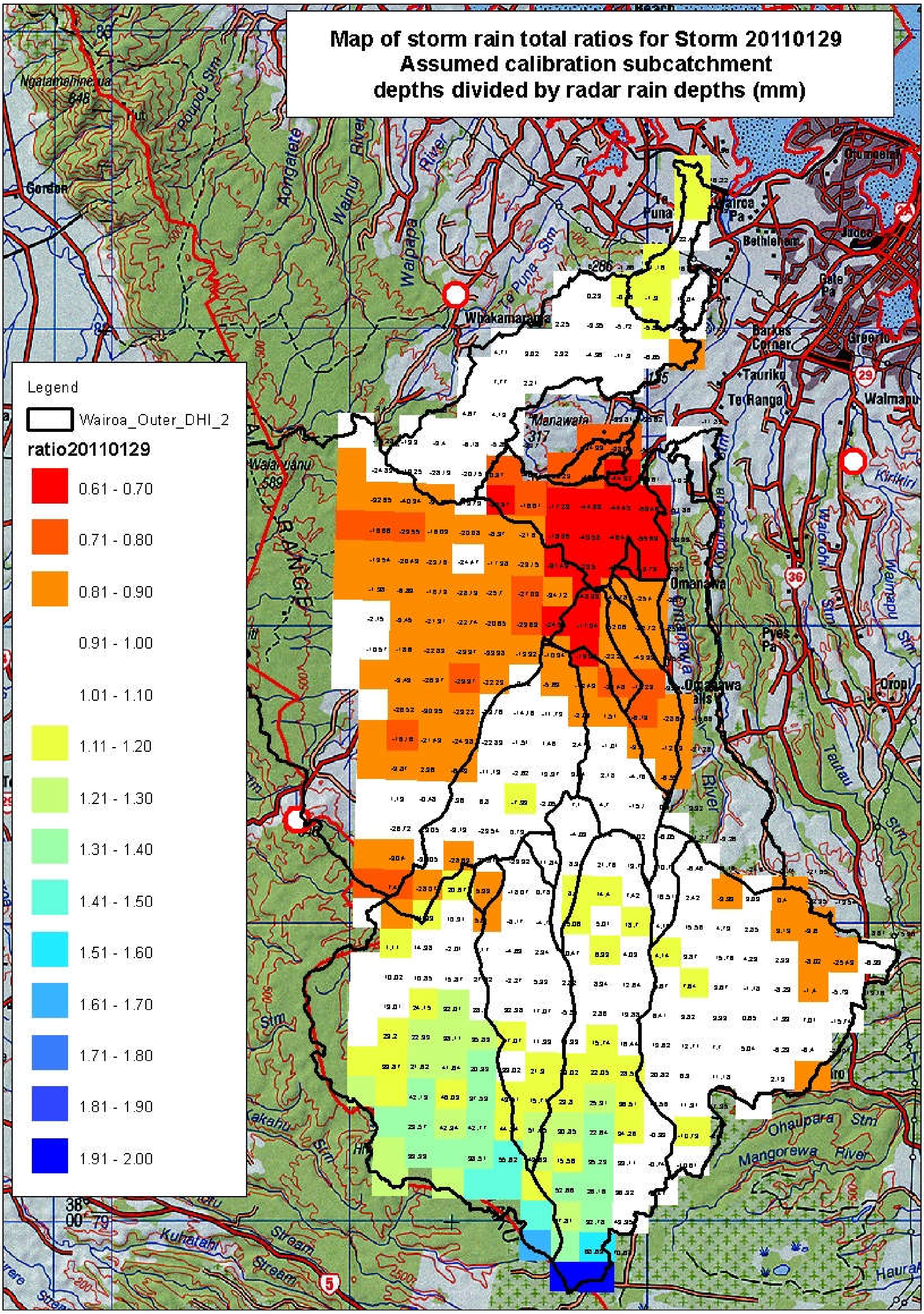

Figure B-4 Storm rain total ratios for the 29th January 2011 event

APPENDIX C–Design Rainstorm

Design Storm Model Inputs

C Design Storm Model Inputs

Design rainstorm hyetographs for the hydrological model were supplied by Blue Duck Design Ltd (on behalf of BOPRC). The approach taken is described in a letter from Peter West, reproduced below.

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Whakatane 3191
Peter.West@BlueDuckDesign.co.nz
Ph. 022 049 9601

Blue Duck Design Ltd
Consulting River Engineers

14 June 2016

Philip Wallace
Principal Engineer
DHI
Level 6, EMC2 House
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Wellington

Wairoa River Hydrologic Modelling – Design Rainstorm Model Input Time-series Files

Phil,

As requested I've prepared the Wairoa River catchment design synthetic rainstorm data that we discussed (delivered by email). I believe that the data is suitable for direct input to the HEC-HMS hydrological model that you described.

The input files describe each of two storms, one using depth duration frequency relationships from NIWA's HIRDS v3; the other uses rainstorm depths from Tauranga City Council's (TCC) Infrastructure Development Code. Both storms are supplied as a single, mobile band of rain moving north (bearing 360 degrees) at 2 metres per second. This direction is based on the general direction of flow when considering the catchment as a whole; the rate of movement has been selected following inspection of radar rainfall data for several large storm events in the Bay of Plenty near Tauranga. Both storms have the same area reduction factors applied based on distance from the gross-catchment centroid, and corresponding to the sequence within the storm's temporal pattern.

The following notes describe the approach taken to generate the data. I've also attached a series of map images that illustrate the method being applied over the Rangitaiki River catchment.

In conjunction with Bay of Plenty Regional Council, we have been developing methods for applying a specifiable synthetic rainstorm to spatially distributed hydrological models for use in the design of river waterways and flood protection systems. We have produced a conceptual storm generator that fulfils the following criteria:

1. A single band of rain of varying intensity tracks across the catchment at specified bearing and speed.
2. The storm intensifies over a specified location (this is usually the area-centroid of the catchment or of a particular sub-catchment of interest).
3. Within this band of rain, at each location within the study area, rain is applied such that it delivers a fully-nested storm consistent with the depth-duration-frequency relations specific to that location, and at the specified probability.

   The default storm uses NIWA's HIRDS.v3 to prescribe these depths – although I've also sent you a storm that uses Tauranga City Council's preferred design storm depths[10]. The total duration of the NIWA storm that I've sent you is 72 hours; the TCC storm is 48 hours long.
4. The NIWA nested storm is applied as a semi-continuous storm, calculated at every time-step – rather than interpolated between table values.
5. The storm that I've sent through is temporally symmetrical, however we do specify the proportion of rain that falls before/after the arrival time of the band of most intense rain. We've found that some flood protection systems are sensitive to this aspect – flood-gated side catchments for example, are vulnerable to late-arriving peak intensities.
6. Each rainfall increment (temporally and spatially) is also factored according to the corresponding duration, and to its particular storm-area using area-reduction factors from Tomlinson (1980)[11]. We've defined the storm-areas by each location's radial distance from the location of most intense rain (see 2 above). A circular storm plan-form is assumed.
7. Although the storm you requested is for the present-day climate change scenario, we also generate storms factored for increased design rainfall intensities in accordance with MfE's 2008 guidance notes[12].

We have found it convenient to define the wider synthetic storm as a collection of point-located time-series files of cumulative rainfall depth. The point locations are taken to be at the area-centroid of each model sub-catchment. This is the approach taken for the Wairoa input files provided, based on the geospatial sub-catchment polygon file supplied by DHI.

---

[10] OPUS International Consultants, *Tauranga City High Intensity Rainfall Update,* October 2005;
Also BECA Ltd, *Tauranga City Rainfall Profile Update (using the Chicago Profile),* 2014.

[11] Tomlinson, A.I., TP19, *The Frequency of High Intensity Rainfalls in New Zealand*, Water and Soil Publications, Ministry of Works and Development, 1980

[12] MfE publication number 891, *Preparing for Climate Change, A guide for local government in New Zealand*, Ministry for the Environment, July 2008.

The method is intended to run multi-scenario analyses (to determine the critical storm location, direction etc). Therefore, now that it is set up for Wairoa River catchment, we can generate additional storm scenarios (probabilities, climate, locations, directions, storm speed) with minimal effort. Let me know if you want any other scenarios produced.

Please let me know if I can help further.

Peter West
B.E. (hons), CPENG, MIPENZ, IntPE

APPENDIX D–Design Flows

Design Flows, Wairoa River

D Design Flows, Wairoa River

D.1 Wairoa River design flow discussion document

A discussion document, on the discrepancies between design flow estimates derived from a flood frequency analysis and those derived from the HEC-HMS model, was prepared in June 2017. This was distributed to TCC and BOPRC staff and relevant consultants, in advance of an on-line meeting held on 5 July 2017. The discussion document is reproduced below.

The expert in WATER ENVIRONMENTS

MEMO

To:	Graeme Jelley (TCC), Peter Blackwood (BOPRC)
Cc:	Daya Kapugama (TCC), Peter West (Blue Duck Design), Paul Mitchell (Mitch Hydro)
From:	Philip Wallace (DHI)
Date:	29 June 2017
Subject:	Wairoa River – 100 year design flow

1 Introduction

In recent Wairoa River modelling, DHI has taken the 100 year design flow at Ruahihi (immediately downstream of the Ruahihi power station) to be 2200 m^3/s. This flow was obtained from a HEC-HMS model of the upper catchment (including the Kaimai HEPS), with rainfall inputs provided by Peter West for each of the various subcatchments in the model.

However, new information suggests that this design flow is possibly too high.

2 HEC-HMS model of upper catchment

The HEC-HMS model of the upper catchment was originally prepared for Trustpower by Paul Mitchell when he was at Riley Consultants. At that time, the model predicted a peak 100 year flow of 1555 m3/s, using a 36 hour rainstorm and using rainfall data from the Kaimai School recorder site.

The DHI obtained a copy of that model, with the intention that it be checked against the January 2011 storms and then be used for design predictions.

However, DHI was aware that the HEC-HMS model was being worked on again by Paul, by that time working for Opus, for Trustpower. DHI's work for TCC was due to be completed in advance of Paul's work for Trustpower, and thus we were aware that the HEC-HMS model might subsequently be revised. Nonetheless, Paul did some preparatory work and supplied an updated (but draft) HEC-HMS model in mid-2016. It was that model that DHI has been using.

At that stage, Paul's interim model refinements (including using the January 2011 events for calibration) and refined design rainfall estimates produced a 100 year flow of 2234 m^3/s.

Paul subsequently revised the model further, as well as his design rainfall assumptions. His final results now suggest a 100 year flow of 1559 m^3/s (i.e. very close to his original estimate).

2.1.1 Model loss parameters

The HEC-HMS model uses an initial loss-continuing loss approach (IL/CL).

Paul had calibrated the HEC-HMS for a number of recorded flood events. For each storm, and for model subcatchment (basin), IL and CL values were adopted.

The design model then used the most conservative (i.e. lowest) loss parameters across all of the storms for each basin. In most cases, the adopted values were 8mm (IL) and 2mm/hr (CL).

As Paul has noted, his work was focussed on extreme flood events, e.g. 1000 year, PMF etc. Hence it is not unreasonable that conservative runoff parameters, for instance, would be used.

3 Design rainfall

After some discussion with TCC and BOPRC, DHI has used different design rainfall assumptions, as supplied by Peter West. These are based on HIRDS and a moving storm over the catchment. Hyetographs were supplied for each of the HEC-HMS model subcatchments (basins).

The resulting 100 year flow was 2200 m^3/s (Wairoa @ downstream of Ruahihi).

4 Flood Frequency Analysis

A flood frequency analysis using annual maxima for the 26 years of full record (1991-2016) gives a 100 year flow of 1138 m^3/s (upstream of Ruahihi), for a GEV distribution (Figure 4-1). The same analysis suggests that the 29 Jan 2011 event was around a 115-120 year return period.

The assumed 100 year flow of 2200 m^3/s (HEC-HMS & HIRDS) sits at ~ 2000 year ARI according to that analysis. (Although the 2200 m^3/s figure does include the contribution from the Ruahihi Power Station; the HEC-HMS model predicts around 100 m^3/s from the power station. A 2100 m^3/s flow would be around 1600 year return period.)

Note that the second biggest event on record, 890 m^3/s recorded on 23 January 2011, is not accounted for in that flood frequency analysis. If it was assumed that this event occurred in 2010, and was to replace that year's annual maximum, the 100 year estimate would rise to 1337 m^3/s. However, that event should not be considered independent of the event on 29 January 2011; it would have contributed to wet antecedent conditions.

Also of interest - I've compared rain gauge totals vs HIRDS for the 29 January 2011 event. The highest ARIs are for the 6hr totals and are around 30yr.

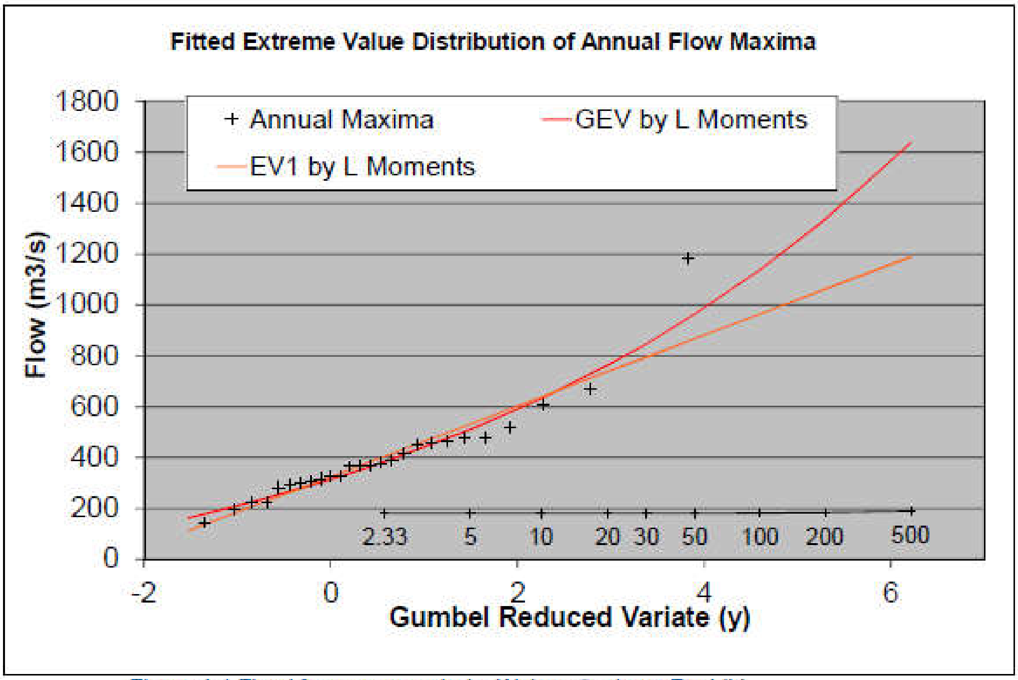

Figure 4-1 Flood frequency analysis, Wairoa @ above Ruahihi

5 Reanalysis with higher loss parameters

As noted above, the design HEC-HMS model assumed the most conservative of the loss parameters of all storms. If the loss parameters that Paul used to calibrate the model to the 29 January 2011 event (typically 10mm IL and 4-8mm CL) were used, again with Peter West's hyetographs, the 100 year design flow would drop to 1722 m³/s.

6 Summary

Peak flow estimates from the various analyses are as in Table 6-1 and Figure 6-1.

Table 6-1 Design flow estimates

Method	Source	Peak flow (m3/s)
Flood frequency analysis (1991-2009) (above Ruahihi)	Ryder Consulting[1]	600
Flood frequency analysis (1991-2016) (above Ruahihi)	DHI	1138

[1] Ryder Consulting Ltd; Wairoa River at Tauriko – Floodplain Assessment; Report produced for Boma No. 2 Trust, May 2010.

HEC-HMS model (2005) (below Ruahihi)	Riley Consultants [2]	1555
HEC-HMS model (2016 draft, Opus) (below Ruahihi)	Opus (Paul Mitchell)	2234
HEC-HMS model (2016 final, Opus) (below Ruahihi)	Opus (Paul Mitchell)	1559
HEC-HMS model (below Ruahihi)	DHI	2200
HEC-HMS model (below Ruahihi) (losses as per 29 Jan 2011)	DHI	1722

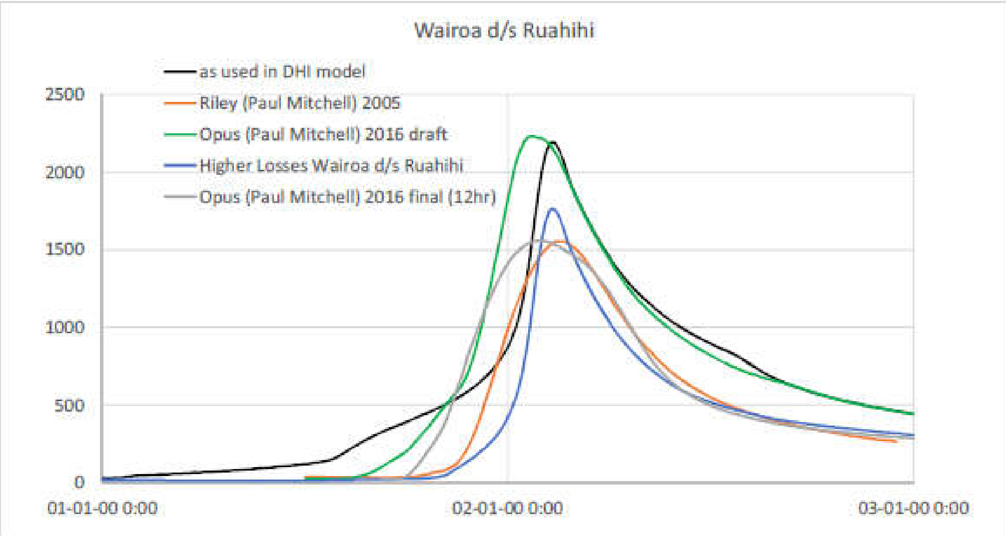

Figure 6-1 100 year design hydrographs from various HEC-HMS models

The HEC-HMS model with the design loss parameters and the HIRDS rainfalls from Peter West has also been adapted to provide subcatchment flows for lower Wairoa catchments. Any changes to the design flow method and hydrological model should also therefore be carried out for those subcatchments.

Note also that the Omanawa subcatchment hydrological model thus adopted was used to prepare a test HEC-HMS model for the adjacent Kopurererua catchment (the subject of a separate study). Results gave significantly higher flows that other analyses for the Kopurererua model.

[2] Riley Consultants Ltd; Kaimai HEPS Flood Study; Report produced for Trustpower, 2005.

D.2 Wairoa River design flow recommendations

Following the meeting of 5 July 2017, a memorandum with recommendations for design flows was prepared. The memorandum, dated 24 July 2017, also contained recommendations for climate change allowances to 2090 and 2130. It is reproduced below.

MEMO

To:	Graeme Jelley (TCC), Peter Blackwood (BOPRC)
Cc:	Daya Kapugama (TCC), Peter West (Blue Duck Design)
From:	Philip Wallace (DHI)
Date:	24 July 2017
Subject:	Wairoa River – 100 year design flow recommendation

1 Introduction

Following on from my memo of 29 June and our subsequent meeting on 5 July, I have prepared some recommendations for the design 1% AEP flow in the Wairoa River. This will need acceptance from both TCC and BOPRC before we rerun the flood model with the revised design flows.

2 Discussion of 5 July meeting

I presented flood frequency analysis of the annual maxima at "Wairoa @ upstream of Ruahihi" (Figure 4-1) as well as design flood hydrographs from various HEC-HMS models (Figure 6-1).

We provisionally agreed that the current 1% AEP flow should be 1700 m3/s, down from the original value of 2200 m^3/s. This was the approximate value obtained from rerunning the HEC-HMS model with losses used in calibrating the model to the 29 January 2011 flood.

However this provisional value was subject to checking results with the final HEC-HMS model Opus prepared, and subject to agreement by Peter Blackwood (not present at the meeting).

In addition, Peter West undertook to provide some error bands for the flood frequency analysis. There was interest in whether or not the 1700 m^3/s figure would fall within the standard error band of the flood frequency plot.

3 Final Opus HEC-HMS model

I spoke with Paul Mitchell (ex-Opus) again on 13 July. He advised that he no longer had access to the final HEC-HMS model he developed. As noted in our meeting and in my previous memo, the model we had used for our work was a provisional one and he finalised the HEC-HMS model later in 2016.

I requested a copy of the final HEC-HMS model and the associated report from Trustpower[1]. I received these on 19 July.

There are a few changes from the previous model we had obtained. These include the following:

- Losses used to calibrate to 29 January 2011 were lower for several of the subcatchments.
- Design model altered to reflect raised spillway from Lake McLaren

3.1 Reruns with the latest HEC-HMS model

I reran the latest HEC-HMS model with a few different variations as below. I also needed to split subcatchment Q ("Local catchment downstream of McLaren") into two, upstream and downstream of the recorder sites, in order to properly compare model results with the flood frequency results. The part of the subcatchment that is roughly bounded by Ruahihi, Jensen and Omanawa Roads enters the river downstream of both recorders. It does however need to be included in the upstream boundary condition flows for the hydraulic model.

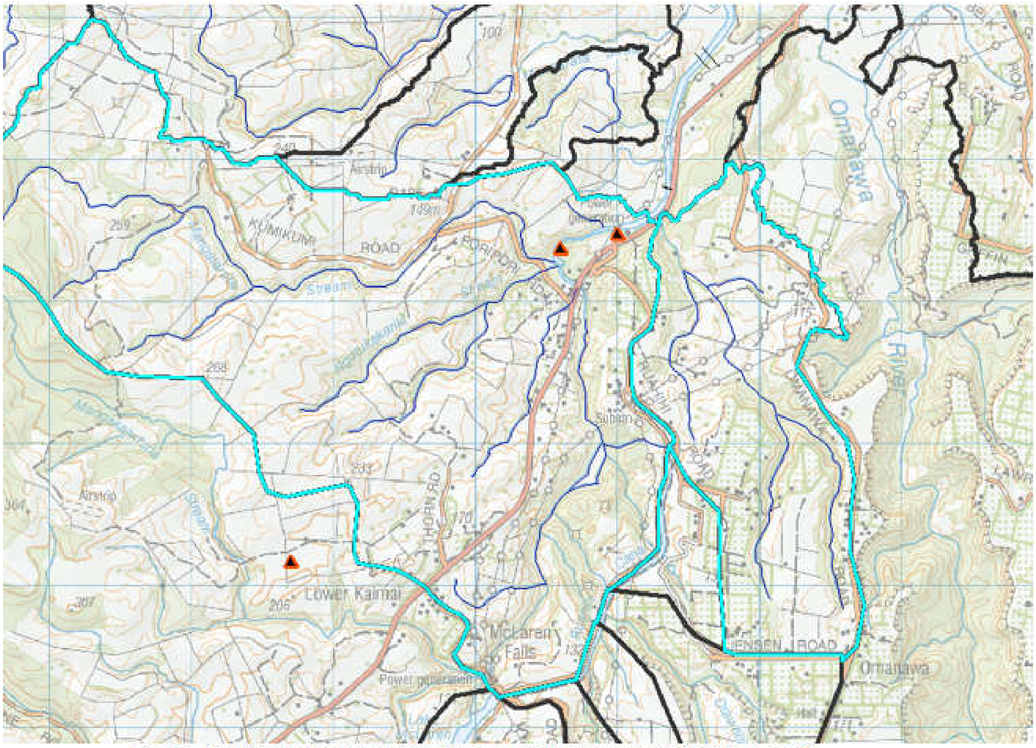

Figure 3-1 Split of subcatchment Q ("Local catchment d/s McLaren")

The variations run through the hydrological model are as follows:

- Latest model, with Opus design losses and subcatchment areas, run with Opus 12 hour design storm
- Latest model, with Opus 29Jan11storm losses and Opus subcatchment areas, run with Peter West's HIRDS-based rainfall hyetographs (72 hour nested moving storm)

[1] Opus (2016): *Kaimai Hydro Electric Power Scheme (HEPS): Flood Hydrology Review.* 5 December 2016.

- Latest model, with Opus design losses and DHI estimate of subcatchment areas, run with Peter West's HIRDS-based rainfall hyetographs (48 hour nested moving storm)
- Latest model, with Opus 29Jan11storm losses and Opus subcatchment areas, run with Opus 12 hour design storm

The results for each are as in Table 3-1.

Table 3-1 HEC-HMS model 1% AEP flow predictions

Simulation assumptions		"Wairoa @ u/s Ruahihi"	Ruahihi PS flows	"Wairoa @ d/s Ruahihi"	Design flow at upstream end of hydraulic model	Omanawa inflow
Losses Subcatchment areas Rainfall assumption	Opus Design Opus estimates Opus Design (12hr)	1485	62	1543	1559	146
Losses Subcatchment areas Rainfall assumption	Opus 29Jan11 calibration Opus estimates HIRDS nested moving storm, 72hrs	1904	85	1985	2013	179
Losses Subcatchment areas Rainfall assumption	Opus 29Jan11 calibration DHI estimates HIRDS nested moving storm, 72hrs	1904	81	1980	2008	173
Losses Subcatchment areas Rainfall assumption	Opus 29Jan11 calibration Opus estimates Opus Design (12hr)	1209	43	1248	1259	99

4 Design rainfall

As per my last memo, we have been using 72 hour nested storm rainfall assumptions, as supplied by Peter West. These are based on HIRDS and a moving storm over the catchment. Hyetographs were supplied for each of the HEC-HMS model subcatchments (basins).

Opus used a 12 hour design storm, based on rainfall records from the Lloyd Mandeno gauge (scaled up for upper catchments). Opus also modelled a 36 hour storm, but this gave lower flows.

The design rainfall totals derived by Opus for this site are significantly less than the equivalent duration totals from Peter West's HIRDS-based values for a similar location. For instance, the 36 hour totals are 290 mm (Opus, for Lloyd Mandeno) and 352 mm (Peter West - Subcatchment I). For a 12 hour duration, the totals are 205 mm (Opus) and 225 mm (Peter West).

The temporal pattern used by Opus is that of the North Island PMP storm of Tomlinson and Thompson. This has lesser peak intensities, as can be seen in Figure 4-1, for sample subcatchment I.

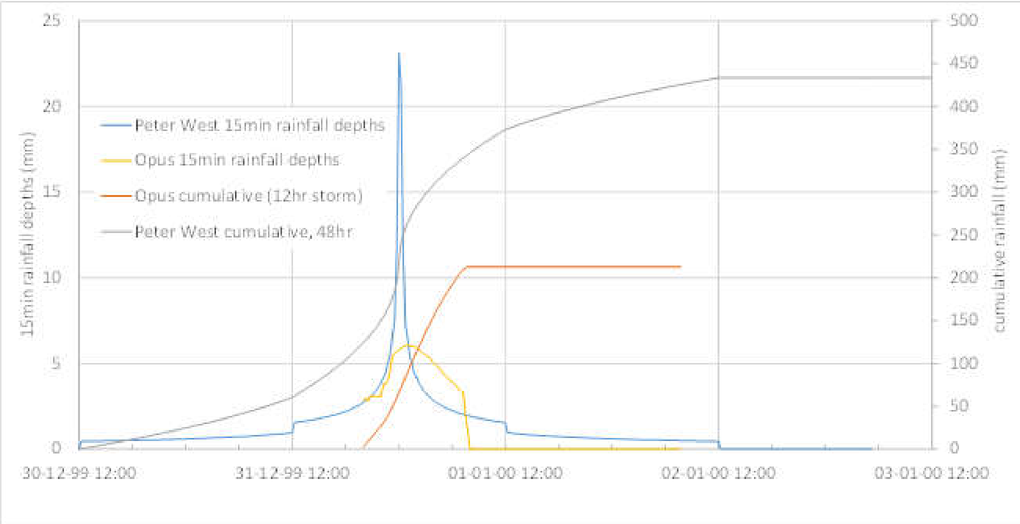

Figure 4-1 Design rainfall assumptions (subcatchment I)

5 Flood Frequency Analysis

A flood frequency analysis using annual maxima for the 26 years of full record (1991-2016) gives a 100 year flow of 1133 m^3/s (upstream of Ruahihi), for a GEV distribution. (There is some minor variation in results: Peter Blackwood gets 1131m^3/s, Peter West uses 1138 m^3/s.)

Peter Blackwood has also performed a partial duration series analysis, incorporating the 890 m^3/s flow recorded on 23 January 2011 (his email of 12 July). That gives a 1% AEP flow of 1400 m^3/s. I have some reservations about whether the 23rd January event should be regarded as a separate event to that of the 29th. While the two rainstorm events themselves could be regarded as independent, the antecedent conditions for the latter flood would have been an important reason why it was so large.

Peter West has produced error bands the flood frequency analysis. Results are given in Table 5-1 and Figure 5-1. The upper bound for the 1% AEP flow, to one standard deviation, is 1475 m^3/s. (Note that the results are based on using a value for the 29 January 2011 flow of 1185 m^3/s. This should be 1177 m^3/s.)

Table 5-1 EV1 and GEV flows and standard errors, Wairoa River

Return Period		EV1	GEV	GEV +1SD
T	YT	QT1	QT2	
2.33	0.5786	404	383	420
5	1.4999	533	510	563
10	2.2504	637	631	705
15	2.6738	696	707	804
20	2.9702	737	763	881
25	3.1985	769	809	946
30	3.3843	795	848	1005
35	3.5409	817	882	1056
40	3.6762	836	912	1106
45	3.7954	852	939	1150
50	3.9019	867	963	1187
60	4.0860	893	1007	1254
70	4.2413	914	1045	1320
80	4.3757	933	1079	1380
90	4.4942	949	1110	1433
100	**4.6001**	**964**	**1138**	**1475**
125	4.8243	995	1199	1576
150	5.0073	1021	1250	1673
175	5.1619	1042	1295	1756
200	5.2958	1061	1335	1833
250	5.5195	1092	1404	1962
300	5.7021	1118	1463	2068
500	6.2136	1189	1638	2437

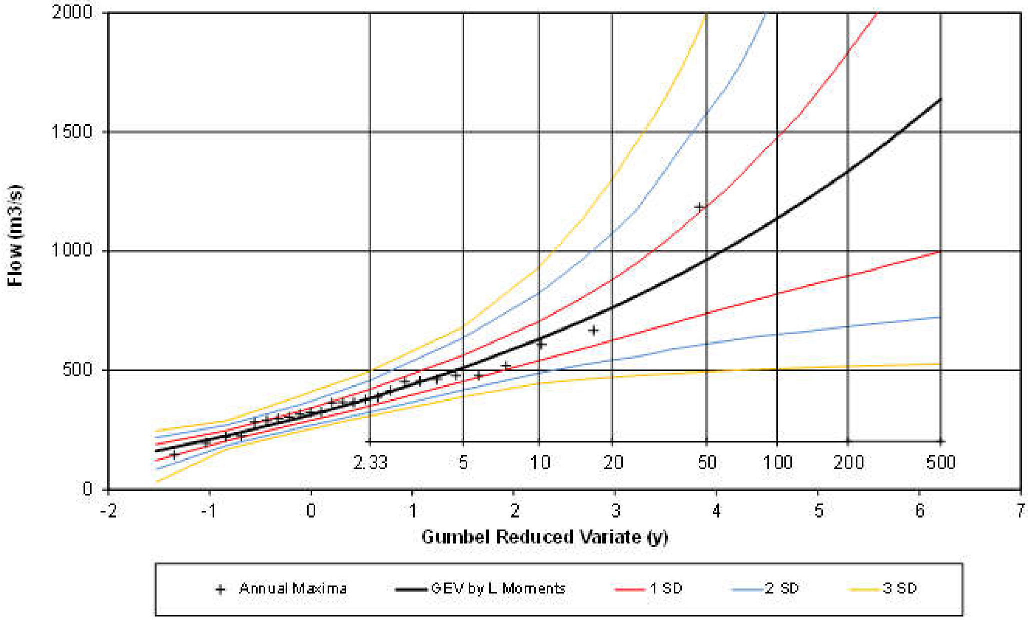

Figure 5-1 Standard error bands, GEV distribution, flow Wairoa River

6 Recommendations, 1% AEP flow, current climate

I propose that we adopt a 1% AEP design flow for "Wairoa @ upstream of Ruahihi" of 1500 m³/s, as

- this is around the value of the annual maxima GEV estimate plus one standard deviation (1475 m³/s)
- leaving aside the issue of whether it is valid to account for the 23 January 2011 event, the partial duration series result (GEV) is of the same order (1400 m³/s), and
- the Opus design results are similar (1485 m³/s).

I also propose that model inflow at the upstream end be scaled to

1500/1904 x 2008 = 1582 m³/s (refer Table 3-1),

and using the hydrograph shape previously obtained using the HEC-HMS model and the HIRDS rainfalls. Other inflows (e.g. Omanawa and other downstream contributing catchments) can be scaled by a similar approach.

7 Other return periods

We will also need to provide estimates of the 10 year, 50 year and 500 year flows (i.e. 10% AEP, 2% AEP and 0.2% AEP, respectively), for the current modelling exercises (flood maps for Wairoa, plus Tauriko West proposals).

Without doing a similar set of HEC-HMS model tests, I suggest that we simply round up the GEV + 1σ values that Peter West has provided in Table 5-1, as follows:

- 10 year ARI flow (Wairoa @ upstream of Ruahihi): 710 m³/s
- 50 year ARI flow: 1200 m³/s
- 500 year ARI flow: 2500 m³/s

8 Climate change

In the case of climate change to 2090, to date we have relied on the 2008 MfE guidelines, from which we have used 2.1°C warming and 16.8% increase in rainfall intensities (for the 1% AEP storm). From the work to date, using subcatchment hyetographs for 2090 supplied by Peter West, this resulted in an increase in flow at the upstream end of the model of 18.2%.

I propose that the upstream model flows be scaled by the same increase, giving a 2090 flow of 1582 x 1.182 = 1870 m³/s.

There are no definitive guidelines as yet for making allowances for climate change beyond 2090. However, in June 2016 MfE issued a report prepared for it by NIWA, on climate change projections[2]. Taking Figure 7 from that report, a 2.1°C increase by 2090 sits roughly midway between the RCP 6 and PCP 8.5 projections. Extrapolating those projections (linearly), and taking the mid-point, gives a temperature increase to 2130 of around 3°C (Figure 8-1). This is what we will assume for modelling involving climate change to 2130.

[2] MfE (2016); *Climate Change Projections for New Zealand: Atmospheric Projections based on Simulations from the 5th IPCC.*

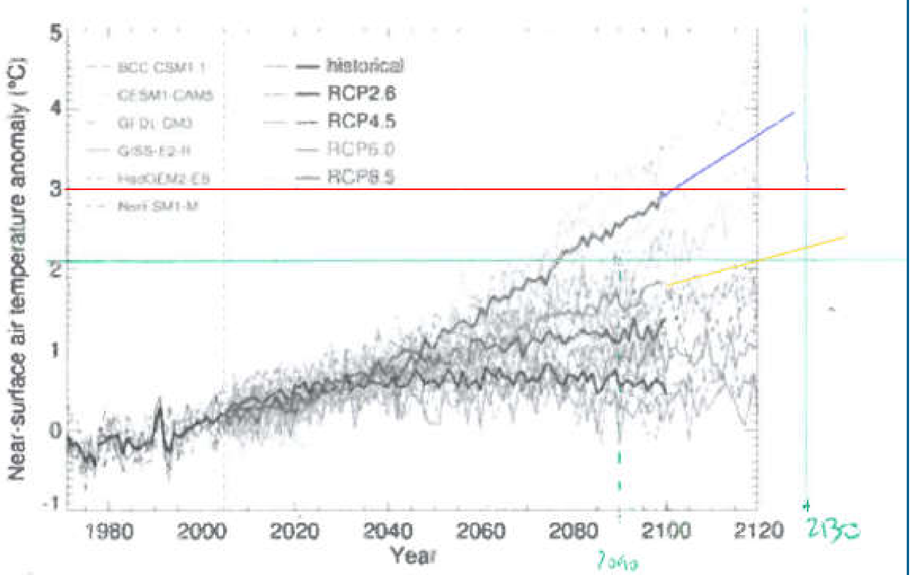

Figure 8-1 Temperature increase projections, from Figure 7 of MfE (2016)

Peter West has produced 1% AEP rainfall hyetographs for a 3°C, for the Wairoa subcatchments. I intend to run these through the HEC-HMS model to produce a flow estimate for the upstream end of the hydraulic model. I will compare that to the current climate predictions, using the same method (i.e. 2013 m³/s, Table 3-1), to provide a scaling factor for flow increase due to climate change. I will then apply that to the 1582 m³/s current climate flow, and scale the design hydrographs accordingly.

However, if we use the 8% increase in rainfall for every 1°C increase in temperature, as per the 2008 MfE guidelines, this would result in a 24% increase in rainfall. One could expect the increase in runoff to be at least 25% in that case, giving a flow of 1980 m³/s at the upstream end of the hydraulic model.

D.3 Peak design flows

Table D-1 presents the adopted peak design flows for the various sub-catchments (see section 4.2.4).

Table D-1 Hydrological inputs to hydraulic model – sub-catchment peak flows (m^3/s)

Scenario	Sub-catchment								
	Omanawa	East Omanawa	Wairoa2	Wairoa5	Wairoa6	Waireia	Ohourere	Vernon Rd	Upstream end of M11 model
10-year Current climate	71	8	15	6	3	27	66	4	755
100-year Current climate	136	15	29	12	5	51	126	8	1582
10-year 2090	84	9	18	8	3	32	78	5	893
100-year 2090	161	18	35	15	6	60	148	10	1870
10-year 2130	89	10	19	8	3	33	82	6	944
100-year 2130	170	19	37	15	6	63	157	11	1978
500-year 2130	283	32	61	26	10	106	262	18	3296

APPENDIX E–River Survey

Cross-section and bridge crossing survey

E Cross-section and bridge crossing survey

E.1 River cross-section survey

Aurecon carried out a river cross-section survey out between January and March 2016. Figure E-1 shows the cross-section locations.

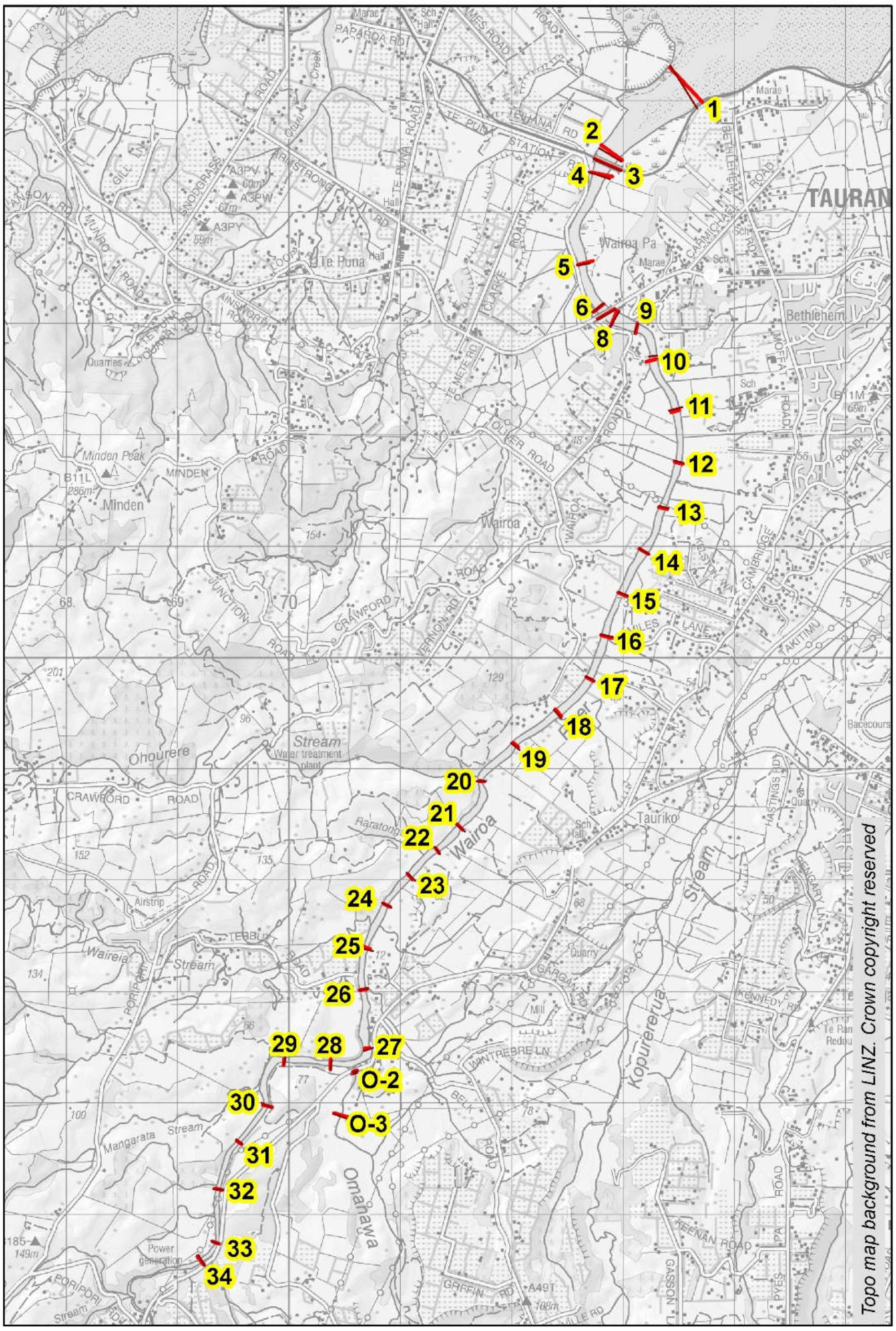

Figure E-1 River cross-section locations

The initial survey showed a deep hole near the bend upstream of the State Highway 2 bridge (Figure 4-1). To investigate this further, additional bed survey was subsequently undertaken around the bend (Figure E-2). At the same time, an extra cross-section was surveyed at a constriction upstream of that bend.

Figure E-3 shows the bed surface, confirming the existence of a hole.

Figure E-2 Follow-up survey

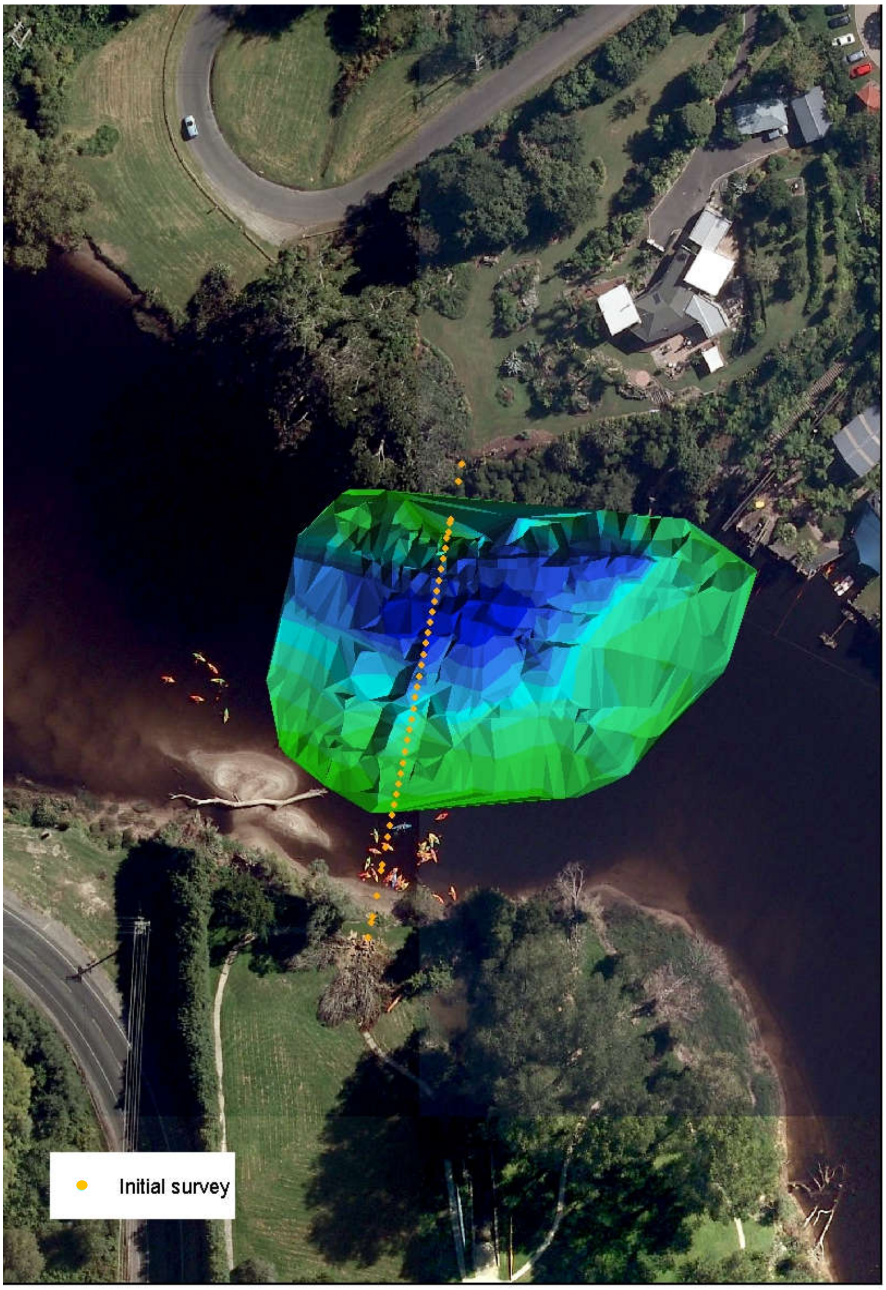

Figure E-3 Hole in river bed at bend upstream of SH2 bridge

E .2 Bridge survey

Aurecon also surveyed key dimensions of the three bridges over the Wairoa and Omanawa Rivers. Figures E-4 to E-6 show the surveyed features.

Figure E-4 SH29 bridge over Omanawa River

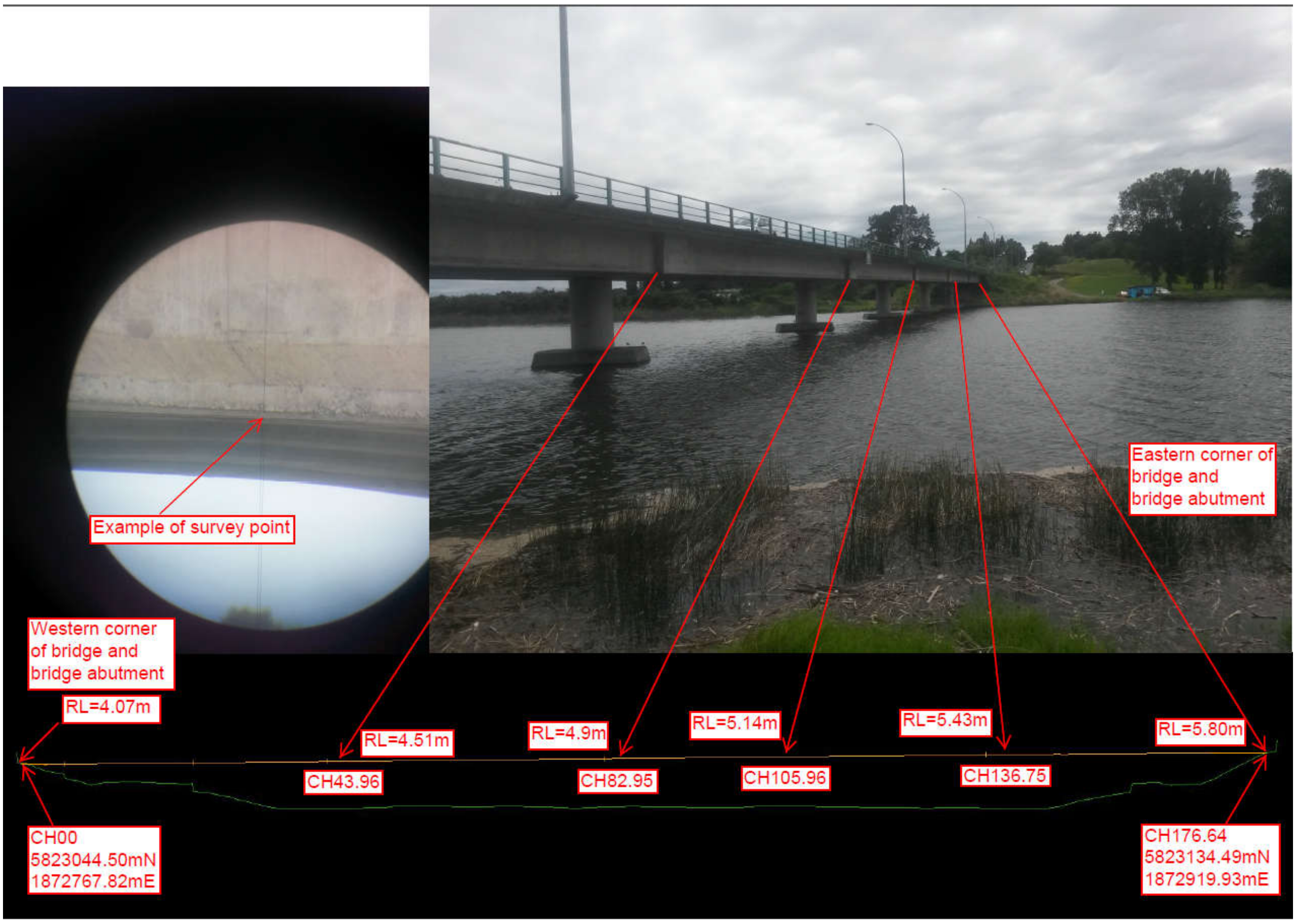

Figure E-5 SH2 bridge over Wairoa River

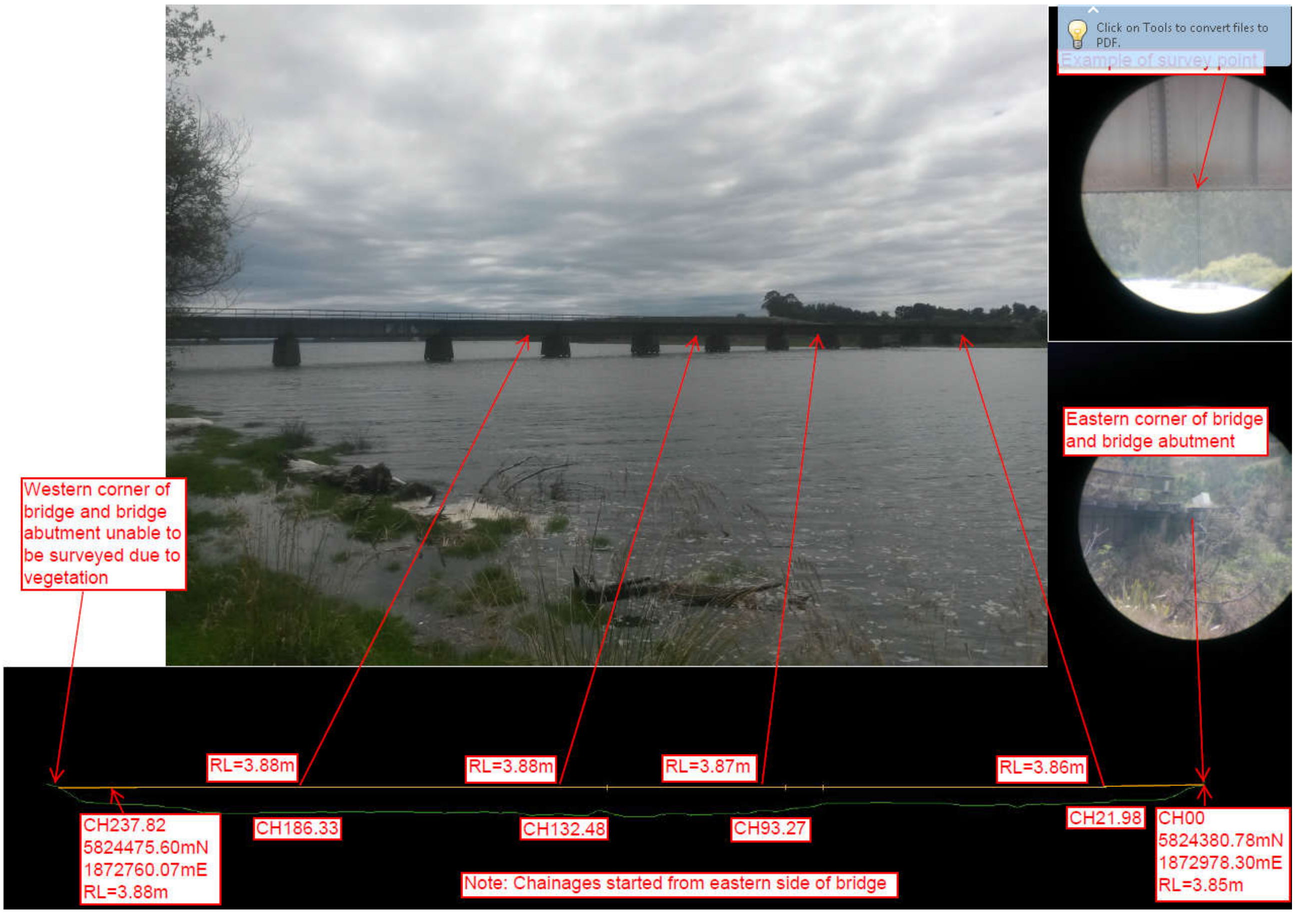

Figure E-6 Railway bridge over Wairoa River

E.3 BBO riverbed survey 2017

After the development of the Wairoa model, BBO commissioned a riverbed survey as part of its design work for the TNL, on behalf of NZTA. The extent of the survey is shown in Figure C-7.

DHI has created a surface from the BBO data, to visualise the river bed bathymetry (Figure C-8). A quick perusal suggests that the river bed varies between cross-sections 8 and 10b.

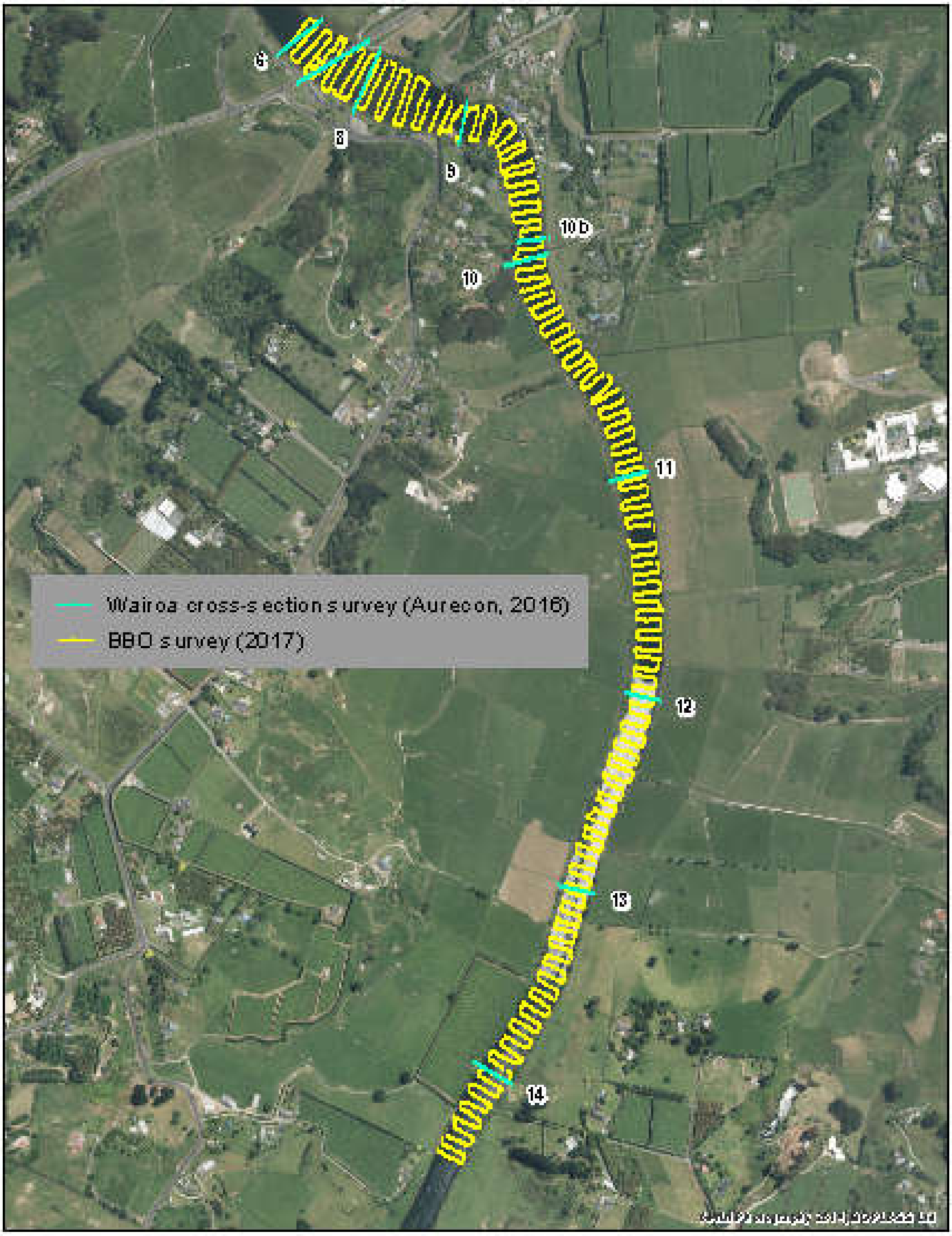

Figure E-7 BBO riverbed survey

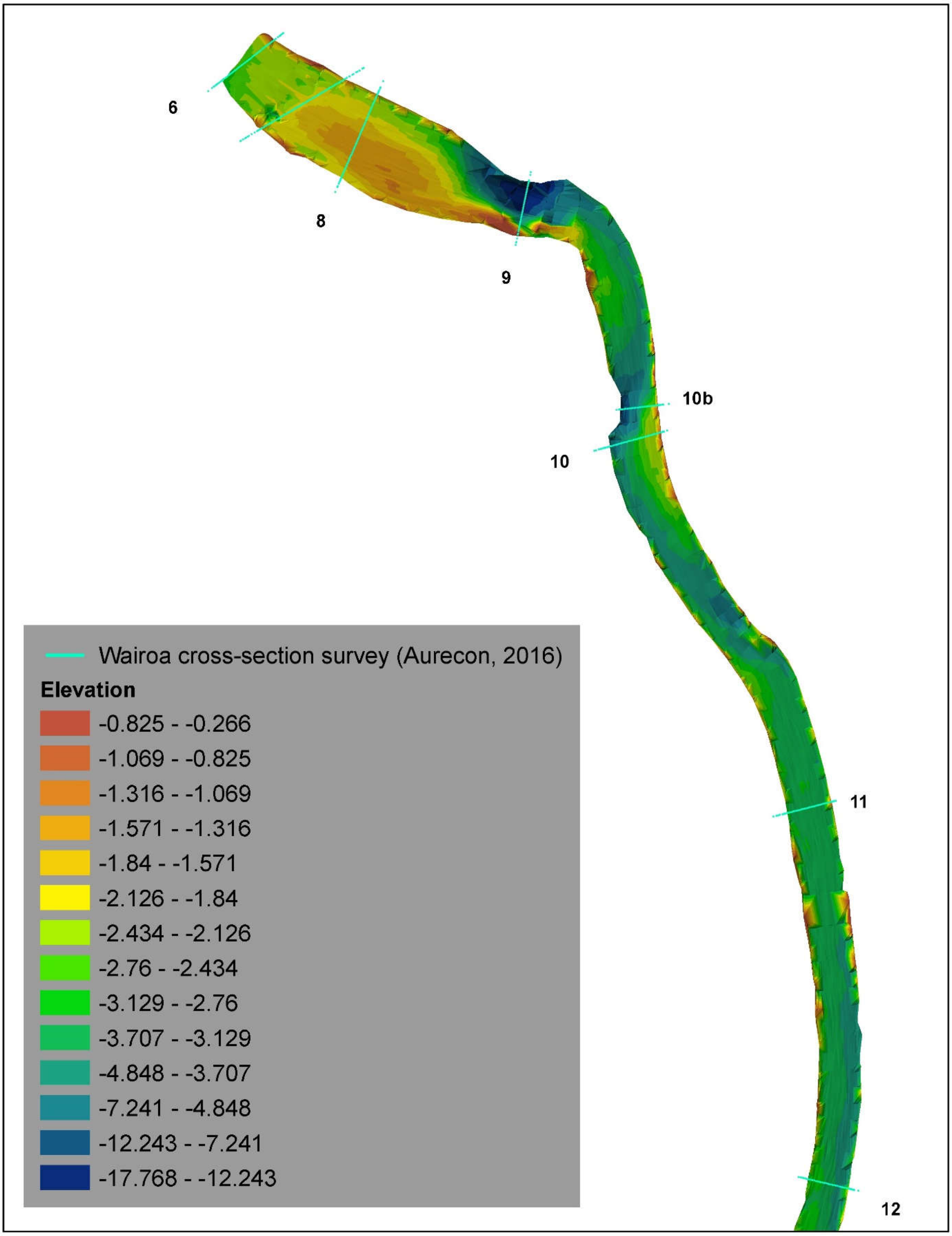

Figure E-8 Bed elevation model created from BBO riverbed survey

APPENDIX F–Model Files

Couple Files

F Couple Files

The input and output files for the final model simulations can be tracked via the .couple files noted in Table F-1. The location of the files in the TCC Model Warehouse is also provided.

MIKE 2016 SP3 was used for all final simulations.

Table F-1 MIKE FLOOD .couple files

Scenario	.couple file	TCC Model Warehouse location
29 January 2011 flood (calibration)	Wairoa_Calibration_29Jan2011	Wairoa\Model_Master\Model\MF
100-year rainfall, 10-year storm surge, current climate	Wairoa_Design_ED2005_Q100T10	Wairoa\Model_Master\Model\MF
10-year rainfall, 100-year storm surge, current climate	Wairoa_Design_Q10T100	Wairoa\Model_Master\Model\MF
100-year rainfall, 10-year storm surge, 2090 CC	Wairoa_Design_ED2090_Q100T10	Wairoa\Model_Master\Model\MF
10-year rainfall, 100-year storm surge, 2090 CC	Wairoa_Design_ED2090_Q10T100	Wairoa\Model_Master\Model\MF
100-year rainfall, 10-year storm surge, 2130 CC	Wairoa_Design_ED2130_Q100T20	Wairoa\Model_Master\Model\MF

APPENDIX G–Mass Balance

Validation and Design Simulations

G Mass Balance

The mass errors generated by the MIKE FLOOD simulations are insignificant, as shown in the examples for the January 2011 validation event and the 100-year ARI current climate, given in Tables G-1 to G-6 below.

G.1 January 2011 Validation Event

Table G-1 Mass Balance, MIKE 11 model – January 2011 Validation event

d:\DHI\Projects\Wairoa\TaurikoWest\Round 11 - Wairoa Calibration 29Jan2011\M11\Wairoa_Calibration_29Jan2011.sim11

Volume Balance Summary

A: Initial volume in model area					**2953302.12**	**m^3**
B: Final volume in model area					**3698520.79**	**m^3**
MIKE 21 lateral inflow	40175266.48	m^3				
MIKE 21 boundary inflow	536689.63	m^3				
MIKE URBAN CS lateral inflow	4871.22	m^3				
MIKE URBAN CS boundary inflow	0.00	m^3				
Lateral sources inflow	518574.00	m^3				
Lateral correction	0.00	m^3				
Open boundaries inflow	36332556.45	m^3				
C: Total inflow					**77567957.77**	**m^3**
MIKE 21 lateral outflow			39321831.50	m^3		
MIKE 21 boundary outflow			265262.75	m^3		
MIKE URBAN CS lateral outflow			2185.98	m^3		
MIKE URBAN CS boundary outflow			0.00	m^3		
Lateral sinks outflow			0.00	m^3		
Open boundaries outflow			37270842.06	m^3		
D: Total outflow					**76860122.28**	**m^3**
E: Continuity balance = B-A-C+D =					**37383.18**	**m^3**
Relative deficit E/max(A,B,C,D) =					**0.00**	**()**

Table G-2 Mass Balance, MIKE 21 model – January 2011 Validation event

d:\DHI\Projects\Wairoa\TaurikoWest\Round 11 - Wairoa Calibration 29Jan2011\M21\Wairoa_Calibration_29Jan2011.m21fm

Volume Balance Summary

A: Initial volume in model area					**347325.04**	**m^3**
Final volume in wet area					**1416641.95**	**m^3**
Final volume in dry area					**27621.77**	**m^3**
B: Final volume in model area					**1444263.72**	**m^3**
Source inflow	357815.78	m^3				
MIKE 11 inflow target	39587093.66	m^3				
MIKE 11 inflow correction	-0.00	m^3				
MIKE Urban inflow target	51220.72	m^3				
MIKE Urban inflow correction	0.00	m^3				
Source outflow			0.00	m^3		
MIKE 11 outflow target			40711955.50	m^3		
MIKE 11 outflow correction			-4.08	m^3		
MIKE Urban outflow target			54873.56	m^3		
MIKE Urban outflow correction			-0.49	m^3		
C: Total volume from source					**-949602.21**	**m^3**
D: Total volume from precipitation/evaporation					**2368240.98**	**m^3**
E: Total volume from boundaries					**-321710.12**	**m^3**
F: Continuity balance (B-A-C-D-E)					**10.02**	**m^3**

Table G-3 Mass Balance, MIKE URBAN model – January 2011 Validation event

Continuity Balance

1 :	**Start volume in Pipes, Manholes and Structures**				**17.1 m3**
2 :	**End volume in Pipes, Manholes and Structures**				**963.1 m3**
3 :	**Total inflow volume**				
	Non-specified inflows				
	Outlets (inflow) :	2193.5 m3			
	Inflow from 2D overland :	54873.6 m3			
		57067.1 m3		-->	**57067.1 m3**
4 :	**Total diverted volume**				
	Operational, non-specified outflows				
	Outlets :	4934.6 m3			
	Flow to 2D overland :	51220.7 m3			
		56155.3 m3		-->	**56155.3 m3**
5 :	**Water generated in empty parts of the system :**				**114.2 m3**
6 :	**Continuity Balance = (2-1) - (3-4+5) :**				**-79.9 m3**
	Continuity Balance max value :		1.1 m3		
	Continuity Balance min value :		-100.7 m3		

G.2 100-year ARI (current climate) scenario

Table G-4 Mass Balance, MIKE 11 model – 100 year ARI rainfall

e:\DHI_Large\44800847_Wairoa\Project\Modelling\2\ED2005_Q100T10\M11\Wairoa_Design_100yr_run0.sim11

Volume Balance Summary

A: Initial volume in model area					**2255673.25**	**m^3**
B: Final volume in model area					**5059436.31**	**m^3**
MIKE 21 lateral inflow	50855566.47	m^3				
MIKE 21 boundary inflow	1082652.66	m^3				
MIKE URBAN CS lateral inflow	3503.57	m^3				
MIKE URBAN CS boundary inflow	0.00	m^3				
Lateral sources inflow	826231.79	m^3				
Lateral correction	0.00	m^3				
Open boundaries inflow	74429324.38	m^3				
C: Total inflow					**127197278.87**	**m^3**
MIKE 21 lateral outflow			50743422.94	m^3		
MIKE 21 boundary outflow			340079.73	m^3		
MIKE URBAN CS lateral outflow			4646.14	m^3		
MIKE URBAN CS boundary outflow			0.00	m^3		
Lateral sinks outflow			0.00	m^3		
Open boundaries outflow			73412482.32	m^3		
D: Total outflow					**124500631.13**	**m^3**
E: Continuity balance = B-A-C+D =					**107115.33**	**m^3**
Relative deficit E/max(A,B,C,D) =					**0.00**	**()**

Table G-5 Mass Balance, MIKE 21 model – 100 year ARI rainfall

e:\DHI_Large\44800847_Wairoa\Project\Modelling\2\ED2005_Q100T10\M21\Wairoa_Design_100yr_run0.m21fm

Volume Balance Summary

A: Initial volume in model area					**69512.25**	**m^3**
Final volume in wet area					**2932820.98**	**m^3**
Final volume in dry area					**410.07**	**m^3**
B: Final volume in model area					**2933231.05**	**m^3**
Source inflow	935349.12	m^3				
MIKE 11 inflow target	51083500.01	m^3				
MIKE 11 inflow correction	-0.00	m^3				
MIKE Urban inflow target	96059.48	m^3				
MIKE Urban inflow correction	0.00	m^3				
Source outflow			0.00	m^3		
MIKE 11 outflow target			51938216.47	m^3		
MIKE 11 outflow correction			-2.51	m^3		
MIKE Urban outflow target			96239.72	m^3		
MIKE Urban outflow correction			-0.05	m^3		
C: Total volume from source					**-387219.61**	**m^3**
D: Total volume from precipitation/evaporation					**5231021.63**	**m^3**
E: Total volume from boundaries					**-1980084.92**	**m^3**
F: Continuity balance (B-A-C-D-E)					**1.70**	**m^3**

Table G-6 Mass Balance, MIKE URBAN model – 100 year ARI rainfall

Continuity Balance					
1 :	**Start volume in Pipes, Manholes and Structures**				**17.1 m3**
2 :	**End volume in Pipes, Manholes and Structures**				**4528.2 m3**
3 :	**Total inflow volume**				
	Non-specified inflows				
	Outlets (inflow) :	7855.5 m3			
	Inflow from 2D overland :	27838.3 m3			
		35693.7 m3		-->	**35693.7 m3**
4 :	**Total diverted volume**				
	Operational, non-specified outflows				
	Outlets :	3352.4 m3			
	Flow to 2D overland :	27917.9 m3			
		31270.4 m3		-->	**31270.4 m3**
5 :	**Water generated in empty parts of the system :**				**766.9 m3**
6 :	**Continuity Balance = (2-1) - (3-4+5) :**				**-679.1 m3**
	Continuity Balance max value :		0.0 m3		
	Continuity Balance min value :		-711.1 m3		