Flood Plain Mapping
Taumarunui Township

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Project	Project No
Flood Plain Mapping Taumarunui Township	NZ50211
Authors Graham Macky Antoinette Taylor	Date August 2010 Approved by Colin Roberts

Revision	Description	By	Checked	Approved	Date
01	Final report (vol1) and Flooding Maps (vol 2)	GHM	CJR	CJR	25/08/10

Key words	Classification
Floodplain, flood mapping, MIKE Flood, flexible grid, Taumarunui, Whanganui River, Ongarue River	☐ Open ☐ Internal ☒ Proprietary

Distribution		No of copies Vol 1	Vol 2
DHI	Library	1	--
Horizons Regional Council	Jeff Watson	3	1

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1 INTRODUCTION

1.1 Background

Horizons Regional Council (HRC) have engaged DHI Water & Environment to undertake "*a Flood Plain Hazard Assessment for a range of Annual Exceedance Probabilities for the township of Taumarunui and the surrounding rural and semi-rural area*". The brief from HRC anticipates the development and operation of a numerical model to determine the risk of flooding in and near Taumarunui, both from the two major rivers that meet there and from local rainfall events that flood the township and rural land directly. The brief from HRC requires that "*the model ... will include both the urban drainage areas, the rural areas draining into the urban areas and those areas directly affected by the Whanganui River*", and goes on to say that "*the urban area of Taumauranui is of prime importance to this modelling project*".

The brief also requires mapping of model output, namely the extent, depth, level, velocity and hazard of floodwaters during flood events of various return periods.

There are two principal reasons for needing to know the extent and progression of flooding during extreme events: land use planning and emergency response. For both of these applications, the most important parameter (but not the only one of interest) is the maximum extent of flooding during design extreme events such as the 1% Annual Exceedance Probability (AEP) event.

The area to be modelled is shown in Figure 1-1 and includes the urban drainage areas, the rural areas draining into those urban areas and those areas directly affected by the Whanganui River.

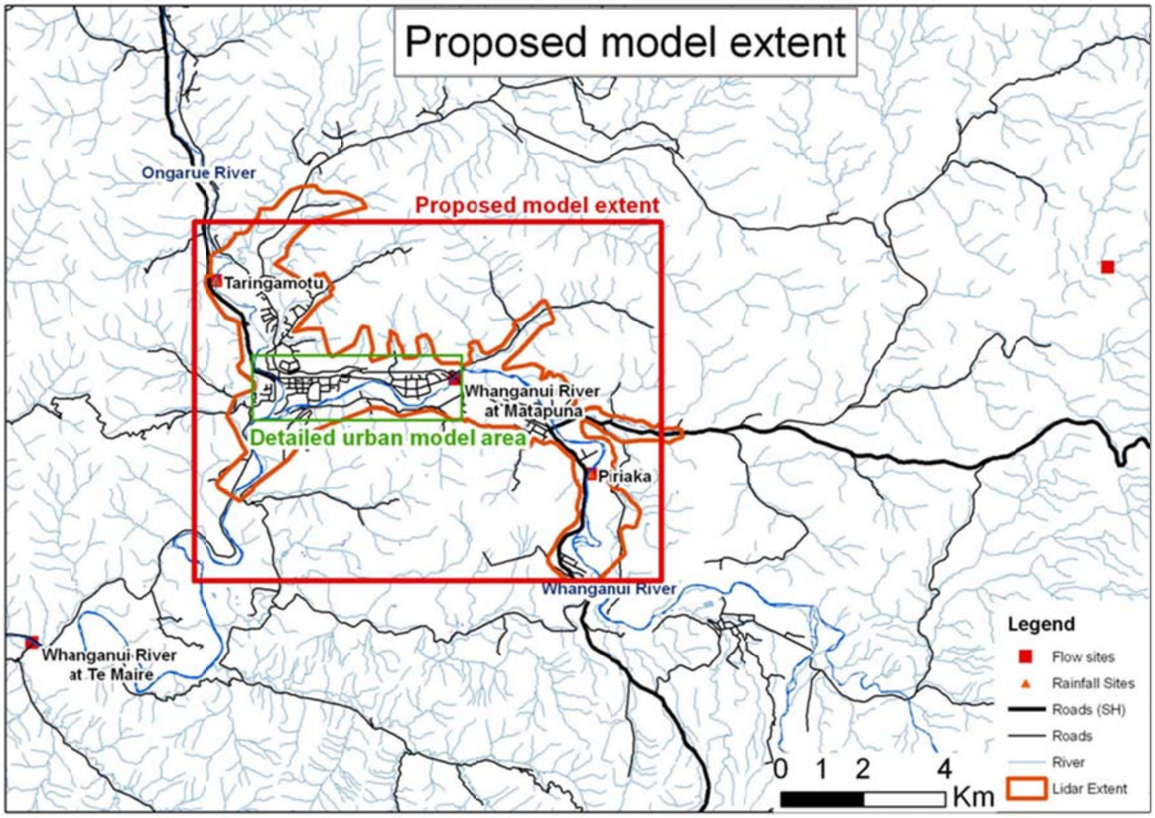

Figure 1-1 Study area, also showing river level /flow gauging stations

1.2 Client

Horizons Regional Council (HRC) has a number of responsibilities relating to the flooding risk and to land based activities undertaken within the region:

- Advising the community and the District Council on the setting of minimum floor levels and the development of infrastructure within floodable areas.
- Providing information on the flooding risk to private and public infrastructure owners for future planning and design and for the upgrade of existing facilities.
- Defining floodable areas.
- Emergency management.
- Providing river engineering works to minimise or eliminate flood hazards.

To assist with the development of hydraulic models Horizons Regional Council has undertaken a number of data-gathering projects. The data from these initiatives used in the present study are detailed in Section 2 below.

1.3 Study Area

Taumarunui township lies on the true right flood plain of the Whanganui River, at and upstream of the confluence with the Ongarue River. The flood plains immediately upstream of Taumarunui are largely rural land but also include the settlements of Manunui and Piriaka on the true left bank.

The Ongarue and Whanganui Rivers at Taumarunui have quite different catchments. The Ongarue drains approximately 1075km^2 of rolling to steep farmland to the north of Taumarunui, in sedimentary geology. The Whanganui catchment also includes similar land to the south of Taumarunui, but originates in the streams running off the volcanoes Ruapehu and Ngaruahoe. A significant portion of its catchment is volcanic, with highly permeable soils and steep headwaters. The catchment area upstream of the Piriaka gauge is 835km^2.

The study area (Figure 1-1) extends from upstream of the Piriaka power station to the Tunakotekote Stream, about 2 km downstream of the river confluence, and includes 3 km of the lower reaches of the Ongarue River, where it joins the Whanganui on the western edge of the town centre.

The centre of Taumarunui is protected from flooding from the Whanganui River by a stopbank which extends from the SH4 bridge to downstream of the Morero Tce bridge. Upstream of SH4 toward the power station the town of Manunui is essentially unprotected but is located on relatively high ground. Within the Ongarue catchment in Taumarunui most development is also located on elevated land.

1.4 Design Hydrological Events

The brief for this investigation from Horizons requires as the main deliverable maps of flooded areas with AEPs of 2%, 1% and 0.5%. This specification is what is needed for land use planning and zoning, but in locations like Taumarunui it can be quite difficult to firmly identify the 1% AEP flood zone. This is because it is not clear what the causative event is which may in fact vary between different locations.

The floodplains and township are at risk of flooding from two types of event:

First, extreme flood events in the Whanganui and Ongarue catchments may result in river levels high enough to inundate the floodplain. The major flood risk from these rivers is believed to exist along the unprotected reaches of the Whanganui River (i.e. upstream of the stopbank) with a possibility for flooding along the Ongarue left bank just upstream of the confluence.

The flow records show that peak flood flows upstream of their confluence are significantly higher in the Whanganui River than in the Ongarue River, so that annual peak flows at Te Maire, downstream of the confluence, usually occur with annual peak flows in the Whanganui upstream.

Because of this and because of the backwater effect, these events are the critical events for flooding in the lowest few hundred metres of the Ongarue River, rather than the Ongarue annual maximum floods. This conclusion has been confirmed by a frequency analysis of the Ongarue annual maxima (not presented in this report). Further upstream in the Ongarue, adjacent to and upstream of the SH4 bridge, Ongarue flows determine the river water levels, but the adjacent land is high enough to avoid inundation. Ongarue extreme flow events have therefore not been applied to the model in this study.

Second, there is also a risk of flooding within the stopbanked-protected area of Taumarunui caused by local rainfall runoff. Runoff from small hill catchments and from the township itself drains through a system of open drains and pipes, discharging through the stopbanks into the river via a series of flapgates. Should these events coincide with high river levels, the risk will be exacerbated because no drainage through the flapgates will be possible.

Hence a comprehensive flood mapping study must take into account both the local drainage system as well as river flows, and must consider the probability of joint flood-producing events in each system.

Identifying the 1% AEP event is therefore complicated because of the several types of flooding event. From consideration of the catchments, and from inspection of the flow and rainfall records, the possibilities for modelling have been reduced to two types of extreme event:

- An extreme combined river flow in the Whanganui River (at both Piriaka and Te Maire) (accompanied by local rainfall that might typically accompany this design flow event); and

- Extreme local rainfall (accompanied by river flows that might typically accompany this design rainfall event).

These two types of flood events between them appear most likely to be responsible for extreme flood levels within the region being modelled. However, only with operation of the numerical model can it be determined whether there are further events that are critical for particular locations. A recorded event in 1990 has been identified for modelling to help answer this question.

1.4.1 Choice of Design Events

HRC's brief has specified events of 2%, 1% and 0.5% AEP. These events provide a range of extreme occurrences that might be considered for different land use planning decisions where different levels of risk are appropriate. The inclusion of the 0.5 AEP event is a recent development in New Zealand that reflects a recognition (largely unofficial at this stage) that consideration of very extreme events can be justified by the consequences of flooding of some areas and assets, despite the lesser accuracy with which these extreme events can be identified. The 0.5 AEP event is also a useful indicator of the future 1% AEP design event that might be applicable after climate change.

In land use planning, and to some degeree in emergency response to flooding, accurate flooding information is highly desirable. If estimates are too low, risks are not fully appreciated and property are placed at risk. If estimates are too high, use and development of good land may be unnecessarily stifled and effective responses to the flooding emergency may be unnecssarily rejected.

In this study, a ”best estimate” of these extreme events has therefore been attempted, on the understanding that the choice of AEPs has already provided a suitable range of flood severities up to the present-day 0.5 AEP event. This approach is reflected in Section 4 below in the choice of the 45 years since 1965 for flood frequency analysis, and in the choice of rainfall hyetographs and river flow hydrographs to accompany the design extreme AEP Whanganui flows in one case and Taumarunui rainfalls in the other.

1.5 Delivery

The deliverables for this study comprise:

- The MIKE FLOOD numerical model (calibrated and operational so that HRC can carry our further modelling for other flood events);
- Tabulated flow, water level, flood depth, velocity, and flood hazard (as per the NSW Floodplain Development Manual) for representative locations on the rivers, drains and floodplains, for the 2%, 1% and 0.5% AEP events;
- GIS maps of the extent and depth of flooding, peak water levels, velocities and flood hazard, for the 2%, 1% and 0.5% AEP events; and
- Selected animations of the 2%, 1% and 0.5% AEP events.

2 DATA OVERVIEW

2.1 Projections and Datum

Data has been supplied in NZMG map projection (NZGD 1949 New Zealand Map Grid) and levels refer to the Moturiki datum. For simplicity the model will use the same projection and datum.

2.2 Ground Data

2.2.1 Land Level Data

HRC obtained LiDAR surveys of the study area in early 2009, to a vertical accuracy specification of 0.15m, which we understand has been largely achieved.

The LiDAR data includes "bare earth, unthinned" topography (i.e. with vegetation, buildings and structures removed). This data has sufficient horizontal resolution for fine-scale modelling and are not a limitation to eventual model grid size. The data have been processed to generate a 1m GIS grid which will be used as the basis for the model development.

In addition, LiDAR points representing buildings have been supplied in ESRI Shape file format.

2.2.2 Other GIS / Spatial Data and Aerial Photographs

HRC have supplied GIS layers of land use, soil type and geology. This data will be helpful in generally guiding the development of the model topography and roughness maps, as well as helping verification of the model results. The extent of the supplied data is acceptable as it covers the majority of the area to be used for the hydrological modelling.

Horizons have supplied the 1:5000 ortho-photography from 2004-2005, as well as unrectified aerial photographs taken with the 2009 LiDAR survey.

2.3 Hydrological Data

Relevant water level and flow data are available from several sites operated by NIWA, and provide between them a good record of flows in both rivers:

- Whanganui @ Piriaka (1970 – present) (NZMG S18:134531) and Whanganui @ Matapuna both within the study area and upstream of the Ongarue confluence. The Matapuna record extends from 1964-1973 only, but has previously been used to synthesize extend back to 1964 the Piriaka peak flows.
- Whanganui @ Te Maire (since 1962). This site provides a reasonable measure of the combined Ongarue and Whanganui flow at their confluence, with only minor additional inflows. Data exists from 1962.

- Ongarue @ Taringamotu (since 1963) (NZMG S18:043578). This site, on the outskirts of Taumarunui, provides a reasonable measure of the Ongarue flow at the confluence.

Data is also available for assessing flooding from local runoff:

- Two rainfall gauges operated by NIWA at the river gauging sites at Piriaka (site no. 859304, since 1999) and Taringamotu (site no. 858209, since 1993);
- A flow recording site on Punga Punga Stream operated for a year (1992-93) by Horizons RC.

The flow gauging sites are shown in Figure 1-1:

NIWA rain gauge site C85821 has been accessed for this investigation. This site is located within Taumarunui township and has provided continuously recorded rainfall data since 1986.

2.4 River Data

2.4.1 Cross Sections

A good coverage of approximately 54 cross sections is available on the Whanganui River within the proposed model area. In addition, there are eight cross sections available for the Ongarue between the confluence with the Whanganui River and 1300m upstream.

2.4.2 Hydraulic Structures and Bridges

The locations of the hydraulic structures were initially identified from aerial photos and the asset data provided. Size and length information is available for the major culverts. Invert levels are not available. However it should be sufficient to source these from the LiDAR. Some cross section information is available for the bridges crossing the Ongarue and Whanganui Rivers, but no other information. Pier sizes and other dimensions were therefore estimated during the site inspection.

2.4.3 Stopbanks and flapgates

HRC have provided details of the alignment of the Whanganui stopbanks. The levels can be ascertained from the cross section surveys. HRC also have provided data regarding the locations, geometries and invert levels of 11 flapgates that allow local drainage through the stopbank. As there are some inconsistencies in the shapefile locations of the flapgates, LiDAR and the RDC asset data were used to help confirm the locations. In addition the locations were checked in the site reconnaissance.

2.5 Urban Asset Data

HRC have provided urban stormwater asset GIS data comprising pipes, manholes, catchpits and open drains. Invert and ground levels of surveyed manholes were also provided as a separate dataset. Merging the datasets, results in a reasonably comprehensive stormwater network. Where ground levels were missing, levels were taken from the LiDAR data. Where invert levels were missing, intermediate levels were interpolated linearly or estimated with reference to the ground level, pipe diameter, and surrounding network.

2.5.1 Pipes

Pipe diameters and lengths are specified along with pipe material. No invert level data is available for the pipes, but this can be taken from the manhole invert levels.

2.5.2 Manholes and Catchpits

Manhole and catchpit locations and diameters are included in the GIS data set. In addition the invert and lid levels of a number of manholes was supplied. The available data along with the LiDAR data can be used to fill in any gaps.

2.5.3 Open Drains and Culverts

The significant drains have previously been captured in GIS in planform but without level data or cross-section information. The site inspection for the present project included some checks of the drains' connectivity with the urban pipe system.

Channel cross-sections were taken from LiDAR data for this investigation. This should provide sufficient accuracy for the present purpose.

The asset data provided by HRC includes the diameters, lengths and invert levels of all significant culverts. Most culverts, and in particular most of the flapgated pipes through the stopbank, were inspected to confirm their function.

3 MODEL SCHEMITISATION

3.1 Approach

The area to be investigated has a wide variety of hydraulic features, for which it is appropriate to use a numerical model combining three separate parts:

1. There are two major rivers and some drains and small streams, all of which are best modelled with a traditional one-dimensional model, which can readily capture the important flow characteristics of these channels.

2. HRC have supplied DHI with a high-accuracy land level data set acquired by an airborne laser survey (ALS, or LiDAR) of the floodplain area. The availability of such data lends itself to a two-dimensional modelling approach, in which the land level data are used as the topographic input for the overland and floodplain flow model.

3. HRC have also supplied network data for stormwater reticulation within Taumarunui township. A network model has been used to represent this network.

These three models – network model, 2D floodplain model and 1D channel model - are dynamically linked to form a combined model describing in detail the complete physical flow characteristics in the study area.

3.2 Model inflows

Water enters the model through three pathways:

1. Flow inputs in the two rivers, Ongarue and Whanganui, at the upstream boundaries of the MIKE 11 model.

2. Catchment runoff linked directly to the upstream ends of open drains also represented in the MIKE 11 model. Links with the MIKE 21 floodplain model represent the overflow of water from these drains.

3. Within reticulated urban areas, catchment runoff linked directly to nodes within the MIKE Urban model.

Rainfall directly onto areas that slope directly down to the Ongarue and Whanganui Rivers was ignored, for two reasons. This runoff has a negligible effect on flooding levels, because it is able to drain away freely before any flooding occurs from overflow from streams and drains or due to high river levels.

Derivation of the hydrographs is discussed in detail in Section 4 below.

3.3 Hydrodynamic modelling

The Taumarunui stormwater system can be grouped into four different categories: Rivers and streams, stormwater pipe network, open drains and overland flow. Three separate computational models are used to model the stormwater system. The urban pipe network is modelled using the MIKE Urban software. The rivers and the open drains use the MIKE 11 model, but the overland flow is modelled using the MIKE 21 flexible mesh module. Using the best-suited computational model for each of the flow categories allows for more accurate model results. The MIKE Flood software (Ref./6/) is used to dynamically couple the three models allowing the models to communicate and synchronise water levels and flows in real simulation time. Figure 3-1 illustrates the model area, showing the components of the model: the rivers and open drains modelled in MIKE 11, the urban pipe network and the area covered by the 2D overland model. Note that 2D flood maps will be available only for this area.

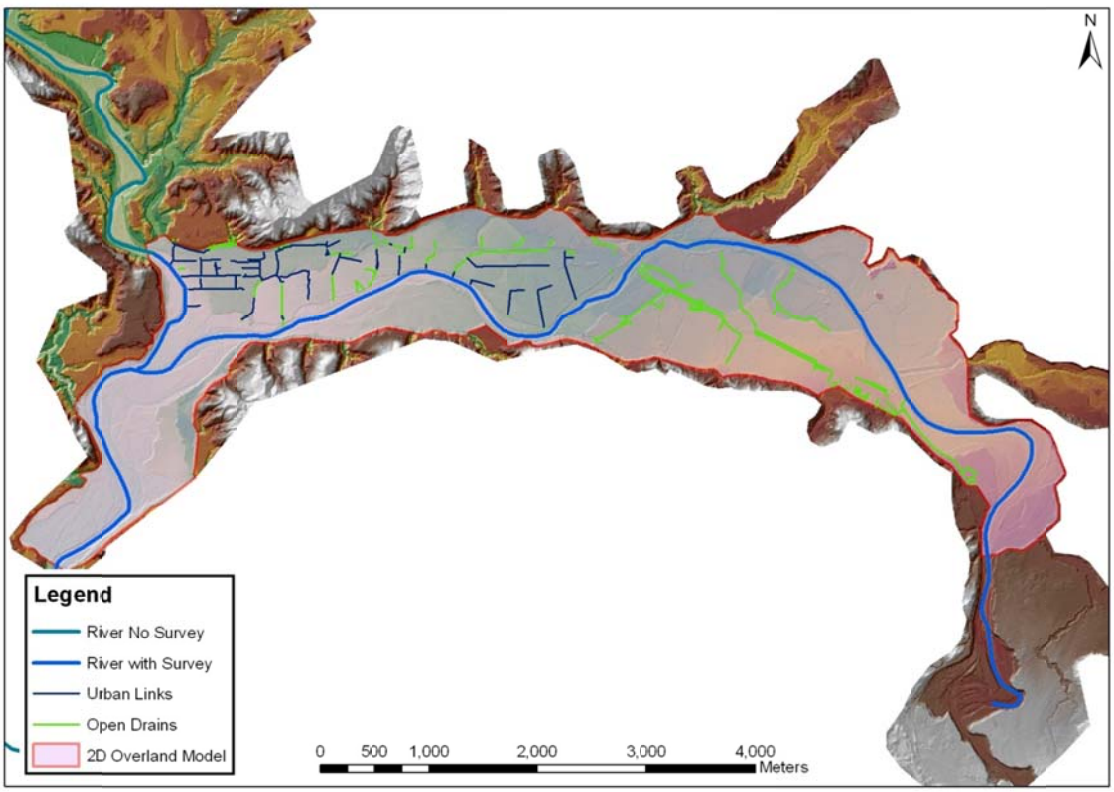

Figure 3-1: MIKE Flood Model Coverage

3.3.1 The Major Rivers

The Ongarue and the Whanganui rivers are modelled in the MIKE 11 river model. The river model extends upstream to the Taringamotu gauge on the Ongarue and upstream to Piriaka on the Whanganui River. The model's downstream end is the Te Maire gauge on the Whanganui River. Where stopbanks are located the river model includes the flow up to these banks.

On the Ongarue River, surveyed cross-sections were not available further than 1.3 km upstream of the Whanganui confluence. Upstream of the surveyed cross sections, as far as the Taringamotu flow gauge, further cross-sections were synthesized using floodplain data extracted from LiDAR. The main channel in these cross-sections was defined by linear interpolation between the gauge station section and the first surveyed cross section.

Cross sections downstream of the surveyed cross-sections on the Whanganui River were interpolated between the last of those cross-sections and the gauging site at Te Maire gauge (the model's downstream boundary).

3.3.2 Urban links

The urban model covers the Taumarunui floodplain and does not extend to the steeper areas to the north. The decision was made to only include the Taumarunui town network on the floodplain due to data availability (manhole invert levels are available only for this area) and because the floodplain area has a much higher potential of flooding than the surrounding hills.

The urban model has been simplified by removing catchpit links and other small peripheral links where no invert level data was available. Omitting catchpits is common practice in urban flood modelling, partly for simplicity but also because the performance of individual catchpits is quite variable due to debris and other obstructions.

The minor stormwater pipes in Manunui, Mahoe and other outlying locations were also not modelled.

3.3.3 Streams and Open Drains

Open drains have been identified from the LiDAR survey and from those drains included in the urban stormwater asset GIS data. The significant drains have been included in the MIKE 11 model. Where the drains run underneath roads via culverts or bridges the structure is also modelled in the 1D MIKE 11 model.

Minor drains are simply incorporated in the MIKE 21 model due to the LiDAR survey having recorded ground levels at or near the drain invert level. These may be modelled imperfectly, but the general direction of drainage will be captured by the model.

The lower reaches of tributary streams, including the Punga Punga River as well as several smaller streams, have also been included in the MIKE 11 model, with cross-sections again derived from the LiDAR survey. In the case of the Punga Punga, the cross-sections had to be deepened to allow for the depth of flow when the LiDAR survey was flown; from the site inspection this depth was estimated as being up to 0.5 m.

3.3.4 Overland Flow Paths and Surface Ponding

The study area comprises (besides the river channels) reasonably flat floodplains (including the central parts of Taumarunui town) where runoff may pond and flow. Flow over these floodplains has been modelled using MIKE 21FM, which is a flexible-mesh two-dimensional hydraulic model. The flexible mesh allows a better fit to irregular areas like the Taumarunui floodplains, and therefore runs faster and should produce more accurate results than the comparable rectangular mesh. The two-dimensional flexible mesh also allows variable resolution to suit the lie of the land and the required accuracy of results: rural parts of the modelled area have been represented by coarse mesh elements compared to the urban areas.

4 HYDROLOGY

4.1 Flow and Rainfall data analysis

Input river flows have been derived from the gauged river flow records. All the flow sites have long records covering several decades, from which the design flow can be estimated reasonably accurately.

On the other hand, inflows directly to Taumarunui township have been estimated from design rainfall depths and estimates of hydrological parameters such as infiltration and time of concentration. These hydrological estimates have relied on the hydrological response derived for the nearby Punga Punga catchment.

Two types of event have been analysed:

- Flood events in the Whanganui and Ongarue Rivers; and
- Rainfall events in the Taumarunui region, with small hillside catchments contributing to local flooding in the township.

The likelihood of both types of event occurring at the same time has been assessed by examining significant events within the historical data. This is further discussed below.

4.1.1 The River Flow Event

As advised in HRC's brief (except for the start date at Piriaka), flow recording sites exist at:

- Whanganui River at Matapuna (1964-1973)
- Whanganui River at Piriaka (1970-present)
- Ongarue River at Taringamotu (1963-present)
- Whanganui River at Te Maire (1962 – present)

The peak flows at Piriaka have previously been correlated with those at Matapuna, so that peak flow values of 45 years are available for both major rivers entering the site as well as the combined flow in the Whanganui River leaving the site at Te Maire.

Early Flood Events

Prior to the establishment of these sites, there were some very significant floods in the Whanganui River, in 1958 and earlier years. Certainly peak water levels in the 1958 event were significantly higher than any levels observed since the recording sites were established. A note on the Te Maire site file states that three earlier events were comparable.

There are two difficulties with accepting these earlier events at face value. First, flows were not gauged during these floods. A flow rate is quoted for the 1958 event for both Piriaka and Te Maire, and indeed used in determining the ratings at each site, but these flow rates, like any earlier estimates that might be found in the archives of flow rates in large events, were based on slope-area calculations. This technique has been shown to be very imprecise, and elsewhere in New Zealand extreme flow rates calculated by slope-area have subsequently been revised downwards.

Secondly, the absence of comparable events since 1964 casts further doubts on the flow rates. Different channel conditions might be a contributing factor to the high water levels observed.

There is also a practical reason for confining attention to the flows recorded since 1964: only for this era are complete hydrographs available for all three sites.

For all these reasons, it has been decided to ignore the 1958 event and those before it, and to instead analyse annual flow maxima since 1964.

Abstractions for Hydro-power

In the present analysis, any abstraction of flows for the hydro-power scheme at Piriaka has been ignored, as have abstractions to the Tongariro Scheme.

In both cases, the rigorous approach, if practicable, would be to correct the flows to what they would have been without the diversions, carry out the extreme value analysis, and then reinstate the diversions if it seemed likely that they would have been left in place during an extreme event. However, it was considered difficult to be sure of past abstraction flow rates during floods, and quite impracticable to be certain of abstraction flows that might occur during future floods. As abstractions are in any case minor compared to peak flood flows (50 m^3/s versus 1000 m^3/s) they were therefore ignored altogether.

Extreme Value Analyses

The standard extreme value analyses using the Gumbel and GEV distributions were applied to the three flow sites (Ongarue @ Taringamotu and Whanganui @ Piriaka and @ Te Maire).

Extreme value analysis of the Te Maire and Piriaka annual flow peaks show that with the addition of the last decade of data the GEV distribution is now a better fit than the Gumbel. Most (but not all) of the Piriaka annual peaks occurred during the same event as the Te Maire peaks. A second extreme value analysis was therefore carried out replacing non-coinciding Piriaka peak flows with those that did coincide with the Te Maire peaks; there was no significant change to the calculated 2%, 1% and 0.5% AEP flows. Figures 4-1 and 4-2 graph the GEV analyses of the Te Maire and modified Piriaka annual peak flow series respectively, and the data from these analyses are included as Appendix A-1. The original Piriaka GEV analysis is in Appendix A-2 for comparison.

A GEV analysis of the Ongarue River data (not presented) gives a 1% AEP flow of 597 m^3/s and a 0.5% AEP flow of 623 m^3/s. The Ongarue annual flood peaks, however, have occurred during different events from the peak flows in the Whanganui River. As a consequence, and given that the Whanganui peak flows are considerably higher than the Ongarue peak, a different approach to Ongarue peak flows has been adopted in this study.

It has been assumed that Ongarue peak flows do not provide the design conditions for flooding within the study area. This is not quite true for the Ongarue River banks (except within 500 m of the Whanganui confluence, where backwater effects from the Whanganui prevail). To evaluate how much effect the Ongarue extreme events have on the flooding within the study area, the MIKE 11 model of the river system has been run for a modified event with an Ongarue River peak flow of 623 m^3/s. Peak water levels just downstream of the State Highway 4 bridge are about 0.2 m higher than those for a flow of 600 m^3/s (the flow modelled with the Whanganui 0.5% event). These levels are still too low to cause flooding within Taumarunui or to affect drainage from the town, so there is no need to specifically model the Ongarue events.

Therefore, to obtain the Ongarue peak flow likely during the extreme event in the Whanganui River, peak Ongarue flows during the Te Maire annual peak events were plotted against the Te Maire peak flows (Figure 4-3).

A similar approach has been taken for other parameters of the river flows applicable during these extreme events. Measured values of the following parameters were graphed against Te Maire peak flow as scatter plots (Figures 4-4 to 4-8 respectively):

- The time difference between the Te Maire peak and the Ongarue peak (which generally came later)
- The time difference between the Te Maire peak and the Piriaka peak (which occurs earlier)
- The 48-hour flow volume in the Ongarue
- The 48-hour flow volume at Piriaka
- The 48-hour flow volume at Te Maire

Where a trend of the particular parameter with Te Maire peak flow is evident, the parameter has been extrapolated by eye to the 2% - 0.5% AEP events. Some parameters show no such trend, and for these a representative but reasonably conservative value has been chosen.

The flow parameters resulting from this process are given in Table 4-1, for the 2%, 1% and 0.5% AEP design events. 0.1% AEP Whanganui flow rates are included for comparison, as are the original Piriaka flow rates. A negative lag time is quoted for the time of peak flow at Te Maire compared to that on the Ongarue, to indicate that the Te Maire peak comes first.

Table 4-1: River flow parameters for the design events

			Annual Exceedance Probability (AEP)			
			2%	**1%**	**0.50%**	**0.10%**
Te Maire	peak flow	m^3/s	1649	1779	1880	2156
Piriaka	*peak flow*	*m^3/s*	*1072*	*1151*	*1212*	*1377*
Piriaka	peak flow for Te Maire event	m^3/s	1073	1155	1217	1385
lag Te Maire from Piriaka		hours	3.0	3.0	3.0	
lag Te Maire from Ongarue		hours	**-2.0**	**-2.0**	**-2.0**	
Ongarue	peak flow for Te Maire event	m^3/s	560	580	*600*	
Piriaka	peak flow for Te Maire event	m^3/s	1020	1080	*1170*	
Piriaka	48-hour volume for Te Maire event	m^3	7.0E+07	7.7E+07	8.4E+07	
Ongarue	48-hour volume for Te Maire event	m^3	6.9E+07	7.4E+07	7.9E+07	
Te Maire	48-hour volume for Te Maire event	m^3	1.60E+08	1.70E+08	1.78E+08	

To complete the derivation of design hydrographs, the largest recorded flow hydrographs for both Piriaka and Ongarue have been averaged, and the resulting hydrographs then modified to assume the peak flow, lag and flow volume few, and modified to match all these parameters to get the design inflow events.

In similar manner, a complete hydrograph for each of the design AEPs can be obtained for Te Maire.

Rain falling in Taumarunui during the annual river flood events has been similarly analysed. First, 6-hour blocks of rainfall were plotted to determine the trend of rainfall depth with peak Te Maire flow and thus assign rainfall depths for the 2%, 1% and 0.5% AEP flow events. Then the hyetographs for seven of the largest of these events were averaged, and this average hyetographs scaled to the rainfall depths just determined.

This approach has omitted any detail of variations shorter than 2 hours. It is difficult to know in advance whether this omission is significant. In general, we are relying on the other design scenario (the 2% - 0.5% AEP Taumarunui rainfall) to identify flooding due to short-term high-intensity rainfall. However, the model output for the river flow events will be inspected to determine whether a revised hyetograph is needed, with short-term events nested within it at critical times.

4.1.2 The Local Rainfall Event

The 2%, 1% and 0.5% AEP rainfall depths from HIRDS (Ref./1/) for all durations up to 24 hours have been used to determine “design” local rainfall events at Taumarunui. (It is reasonable to assume that the HIRDS rainfall depths have been derived from the rain gauges available for this study, so there is little to be gained from repeating the extreme value analysis.)

There is not much difference between the HIRDS rainfall depths at different locations within the study area. These differences have therefore been ignored, and the depth at Taumarunui township applied throughout the local catchments.

The hyetographs from the three NIWA rain gauges (Piriaka, Taringamotu and C85821) for selected major events have been inspected and compared major events for all 3 rainfall sites. The temporal details of these hyetographs do not correlate that well, but in general the rainfall totals appear to be quite similar.

Use of the Chicago-type hyetograph is common, almost standard, in New Zealand rainfall-runoff analyses. This form of hyetograph nests the rainfall depths of various durations within one another, to produce a hyetograph that is near-symmetrical and with a single central peak. In an attempt to derive a more realistic hyetograph, this method was varied by choosing one of the largest rainfall events recorded by gauge C85821, recorded on 16 March 1994 and modifying it to contain the design rainfall depths for the various durations from 10 minutes to 24 hours. Each of these design rainfall depths was placed at the same time as the corresponding peak rainfall depth on 16 March 1994. The resulting hyetographs (Figure 4-9) are not symmetrical in time like the "standard" Chicago" hyetograph, but because of the nature of the 1994 event the hyetographs retain the single sharp peak typical ofd the method.

Several of the largest rainfall events at Taumarunui in the historical records were inspected to decide on river flows to go with these design rainfall events. Some of the largest rainfall events occurred with negligible change in river flows. However, moderate freshes in the rivers occurred with two of the rainfall events inspected. The flow hydrographs for one of these events (28 December 2000) were chosen for the design events (Figure 4-9).

It appears uncommon for a major rainfall event to be accompanied by significant river flow as on 28 December 2000. Furthermore, the sharp rise in river flows just 2.5 hours after the rainfall peak appears unusual, and most likely attributable to significant earlier rainfall. Both these features point to the rainfall and flows in Figure 4-9 being a conservative combination. Any scaling-up of the river flows from the observed hydrographs has therefore not been considered warranted in obtaining these design events.

4.1.3 Event of 1990

There is a risk with the above approach of considering just the two diverse types of event without checking whether at some locations the critical conditions for flooding are when neither local rainfall nor river flows is extreme, but when both are moderately high. This is particularly the case at locations where river flows can contribute to flooding just by being high enough to keep flapgates closed.

One event on 9 March 1990 (Figure 4-10) epitomises this type of event. The MIKE 11 model of the river system was therefore modelled with this event "as is", (i.e. without any scaling to approximate an extreme event of given AEP). The peak river levels for this event at the flap-gated culvert outlets were close to the culvert inverts, slightly higher than the inverts in two cases and at pipe mid-depth at Culvert 9.

It was decided that the obstruction to runoff from the urban area would have been very minor in the 1990 event, with correspondingly minor effects on the extent of ponded runoff in the town. Events like the 1990 one have therefore not been considered further in this study.

4.1.4 Climate Change

The hydrographs and hyetographs as described above are intended to represent events of 2%, 1% and 0.5% AEP under existing climatic conditions. They do not include any allowance for climate change, being derived from direct analyses of recorded data.

Should 2%, 1% and 0.5% AEP events be required for conditions following climate change, these could be represented at reasonable accuracy by applying the recommended scaling factors to the flow hydrographs for the Whanganui and Ongarue Rivers and to the rainfall hyetographs.

4.2 Rainfall Runoff parameters of Local Catchments:

The sub-catchments defined for this model (excluding the local catchments of inflows to the stormwater network modelled in MIKE Urban) are shown in Figure 4-11, and the MIKE Urban catchments are shown in Figure 4-12.

4.2.1 Larger local catchments: "Urban Model A"

Punga Punga Stream in particular, and several other tributary streams, are long enough to have a significant time of concentration, a feature that Urban Model B does not provide. These streams have therefore been modelled using "Urban Model A" (Ref./2/)

There are no measurements of runoff from local catchments, except that for less than a year, flows were measured in the upper catchment of Punga Punga Stream. That flow record was used for this study to calibrate runoff from the entire Punga Punga Stream and the other catchments to be modelled using Urban Model A:

- The time of concentration t_c at Punga Punga @ Old Goat Gorge was estimated by eye by comparing the Taumarunui rainfall record with measured hydrographs there. Only some events showed a strong relationship between the observed rainfall and the observed flow. Those that did indicated t_c of between 2 and 4.5 hours, with an average of 3.6 hours.

- Two different empirical formulas for t_c were accessed: the SCS formula (Ref./3/) and the formula adopted in the Auckland Regional Council's guideline, TP108 (Ref./4/). Both these formulas use the thalweg length and the (area-weighted) slope, and these parameters were used to scale the 3.6 hours at Old Goat Gorge to obtain estimated times of concentration for the entire Punga Punga and for the other streams.

- The McKerchar & Pearson (1989) (Ref./5/) regional method of peak flow estimation was used to estimate the peak 100-year flow rate for selected catchments.

- Urban Model A was then run for these catchments for the Taumarunui 100-year rainfall obtained from HIRDS, and the runoff factor C adjusted until these peaks were obtained. C=0.27 worked well except for the very smallest of these catchments.

4.2.2 Smaller local catchments: "Urban Model B"

The small catchments that discharge onto the floodplains within the study area have been modelled using "Urban Model B" (Ref./2/). We consider this catchment model ideal for these catchments because it is simple to operate, and provides a plausible recession limb.

Model B incorporates infiltration losses for which initial and final infiltration capacities must be specified. These losses are dependent on the soil properties, and for this information the soil maps provided by Landcare research on their website were accessed. These describe the hill soils north of the Whanganui River as "silt loams" whereas soils on the river flats and in the catchments south of the Whanganui River are described as "sandy loams".

Based on literature searches, DHI has previously decided on suitable infiltration rates for the common soil types; those for silt loam and sandy loam are set out in Table 4-2:

Table 4-2: Assumed Model B hydrological parameters for Taumarunui rural catchments

Parameter	**Silt Loam**: Catchments north of Whanganui River	**Sandy Loam:** Catchments south of Whanganui River, and river flats
Initial Infiltration Capacity	36 mm/hr	50-72 mm/hr
Final Infiltration Capacity	3.8 mm/hr	9-18 mm/hr
Decay exponent	$1.5 \times 10^{-3} s^{-1}$	$1.5 \times 10^{-3} s^{-1}$
Manning M (=1/n)	30	30

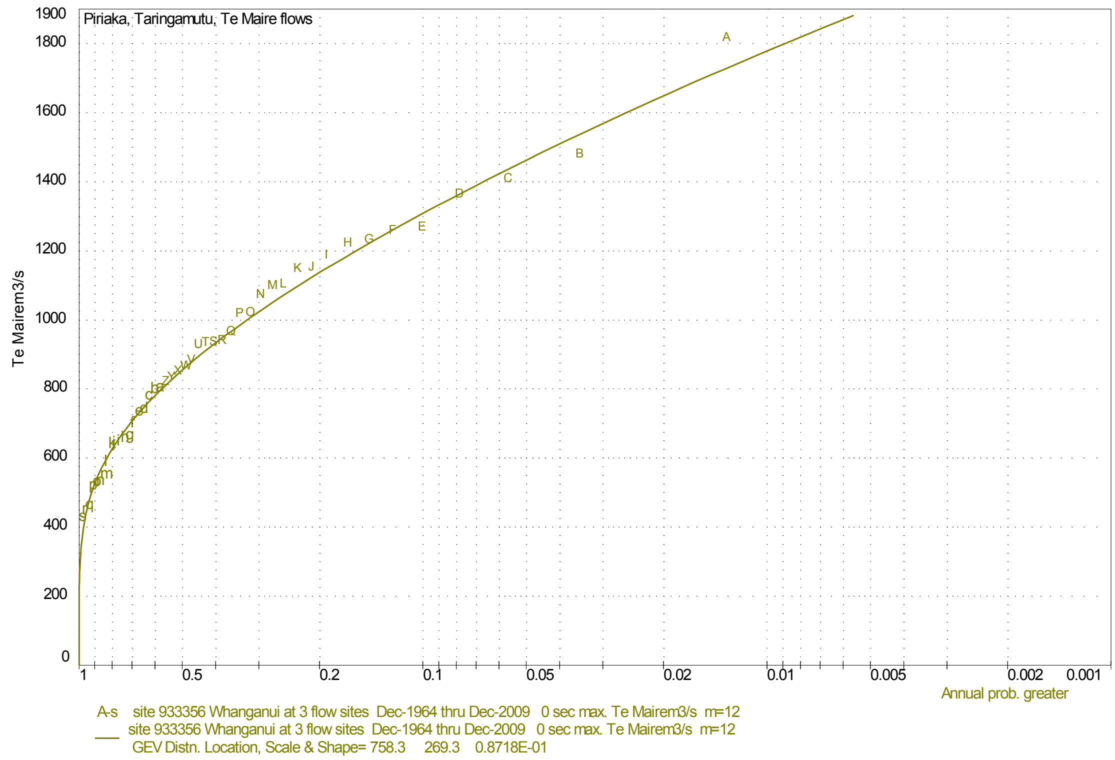

Figure 4-1: Whanganui @ Te Maire (site 33302): GEV analysis of annual peak flow rates since 1965.

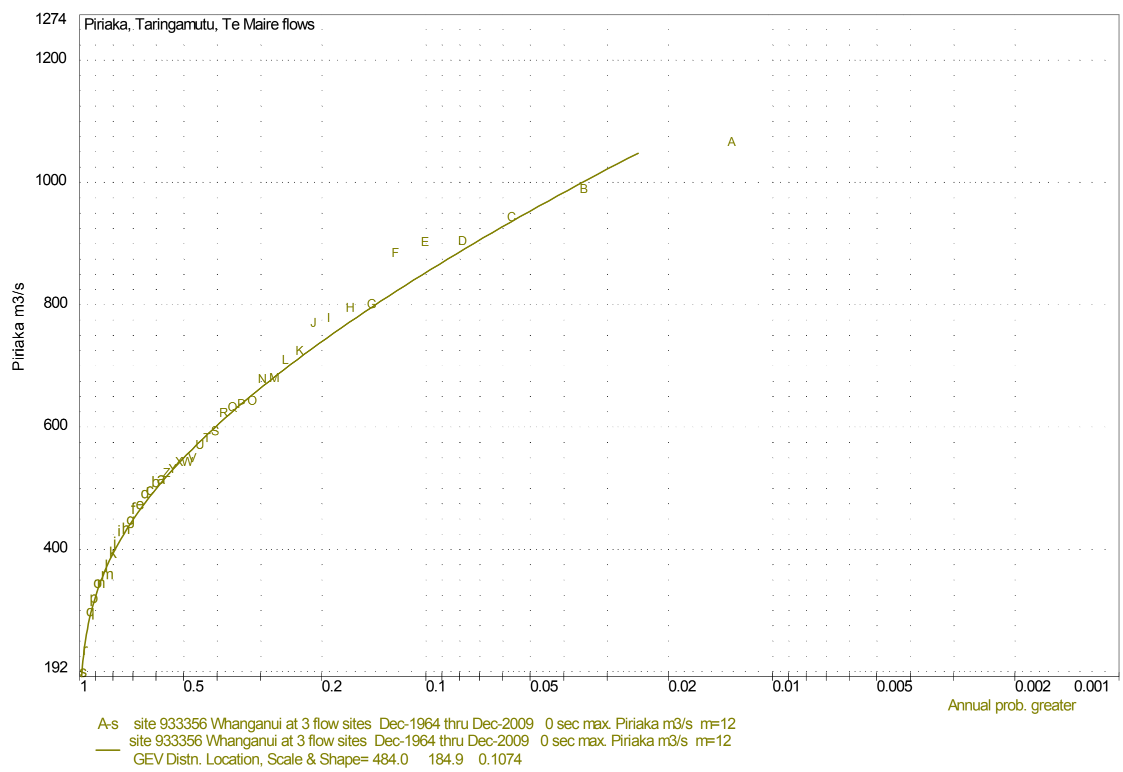

Figure 4-2: Whanganui @ Piriaka (site 33356): GEV analysis of annual peak flow rates for Te Maire annual peak events, since 1965.

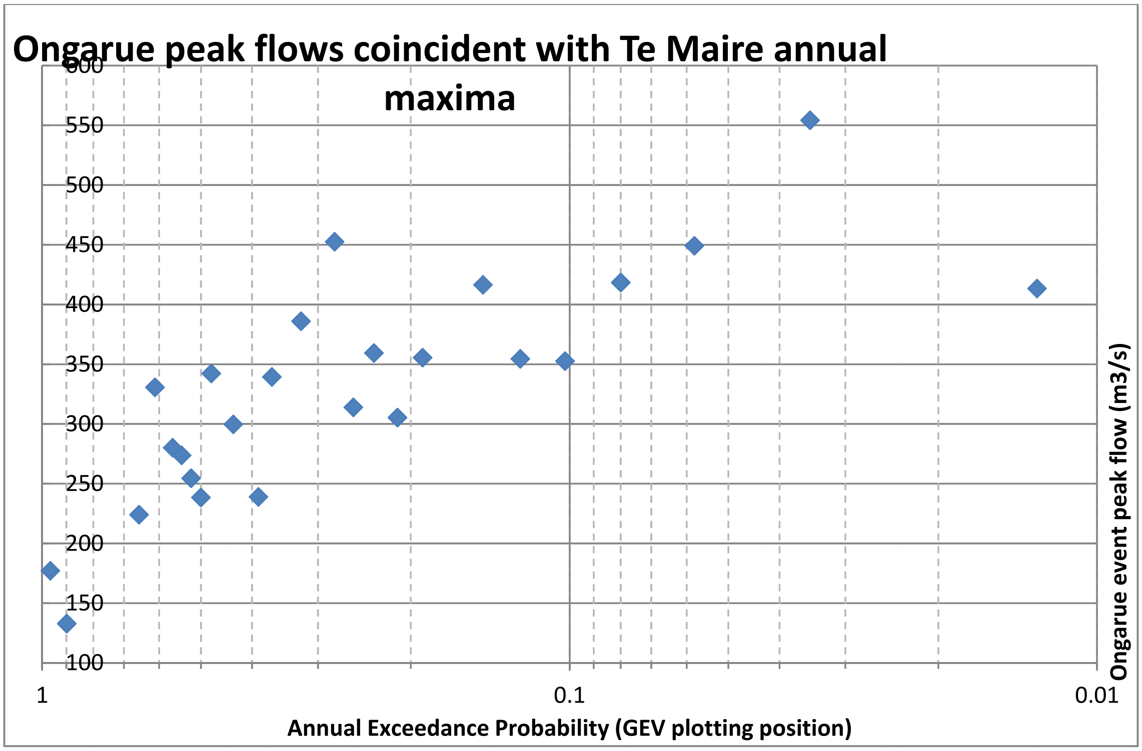

Figure 4-3: Scatter plot: AEP of Te Maire peak flow events vs. coincident Ongarue flows

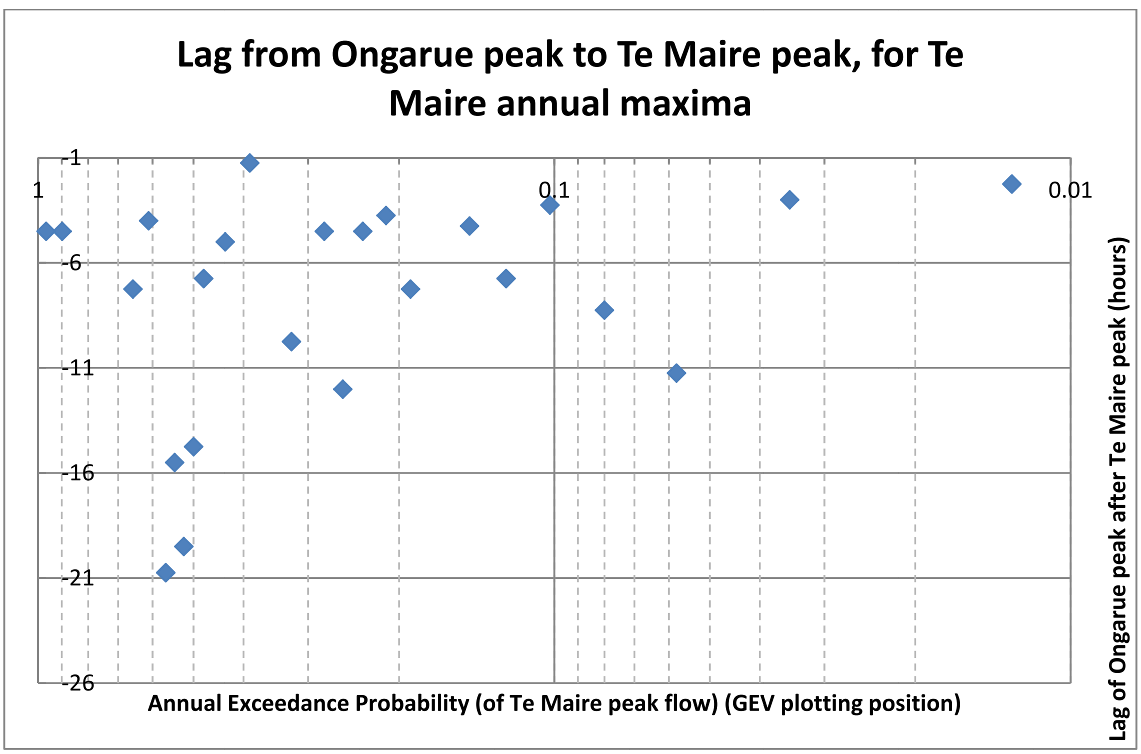

Figure 4-4: Scatter plot: AEP of Te Maire peak flow events vs. time difference between the Te Maire peak and the Ongarue peak (which generally came later)

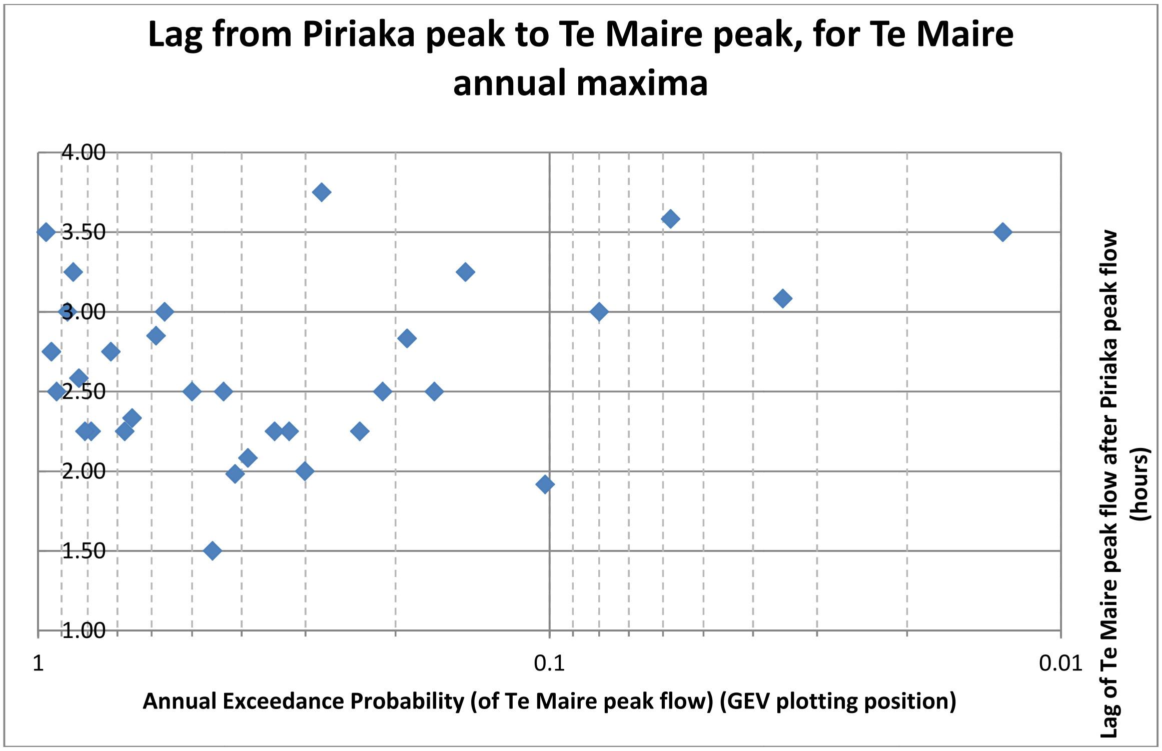

Figure 4-5: Scatter plot: AEP of Te Maire peak flow events vs. time difference between the Te Maire peak and the Piriaka peak (which occurs earlier)

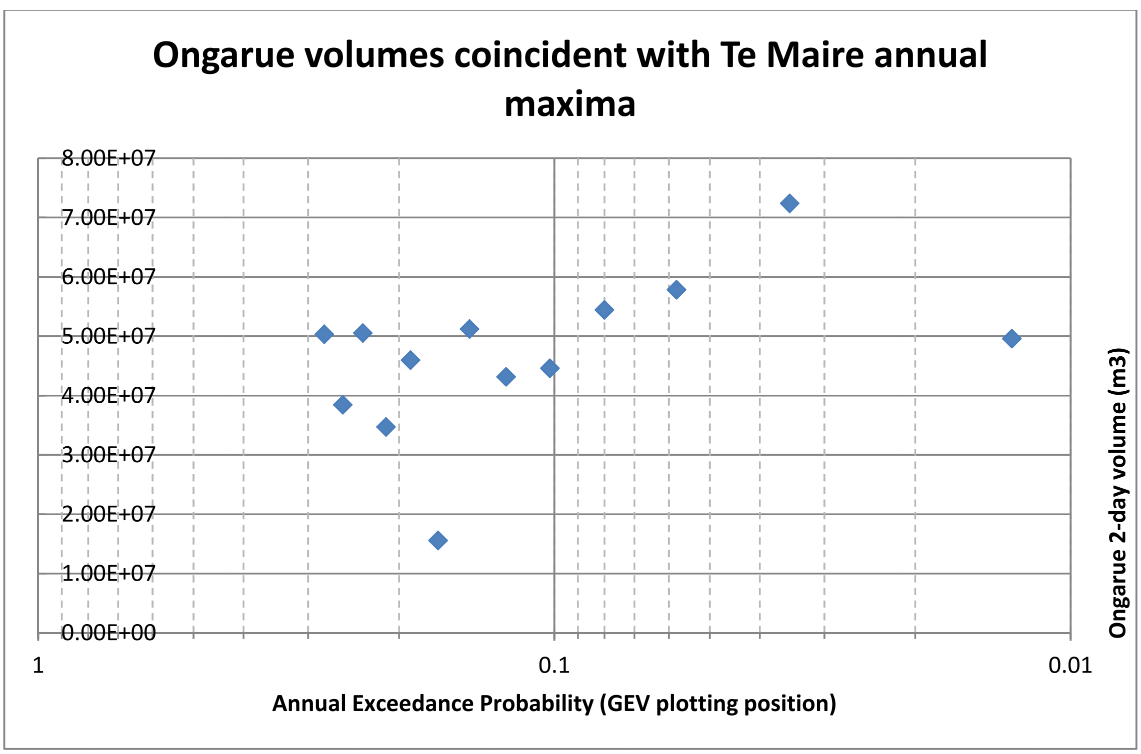

Figure 4-6: Scatter plot: AEP of Te Maire peak flow events vs. the 48-hour flow volume in the Ongarue River

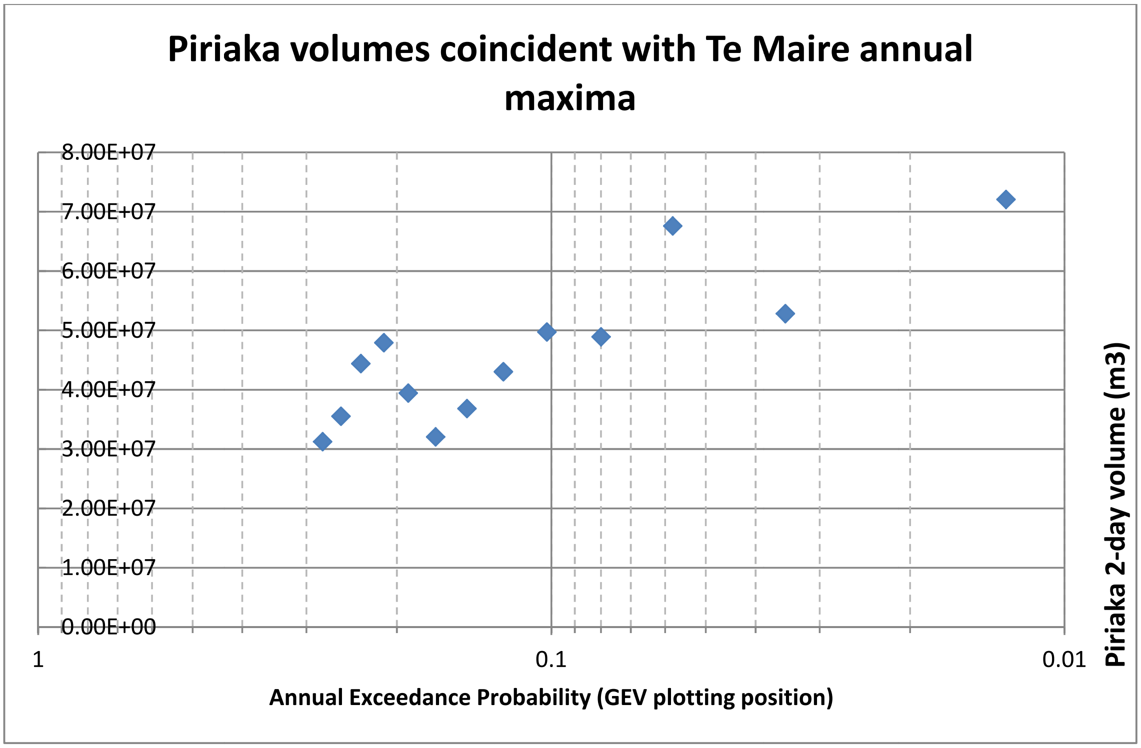

Figure 4-7: Scatter plot: AEP of Te Maire peak flow events vs. the 48-hour flow volume at Piriaka

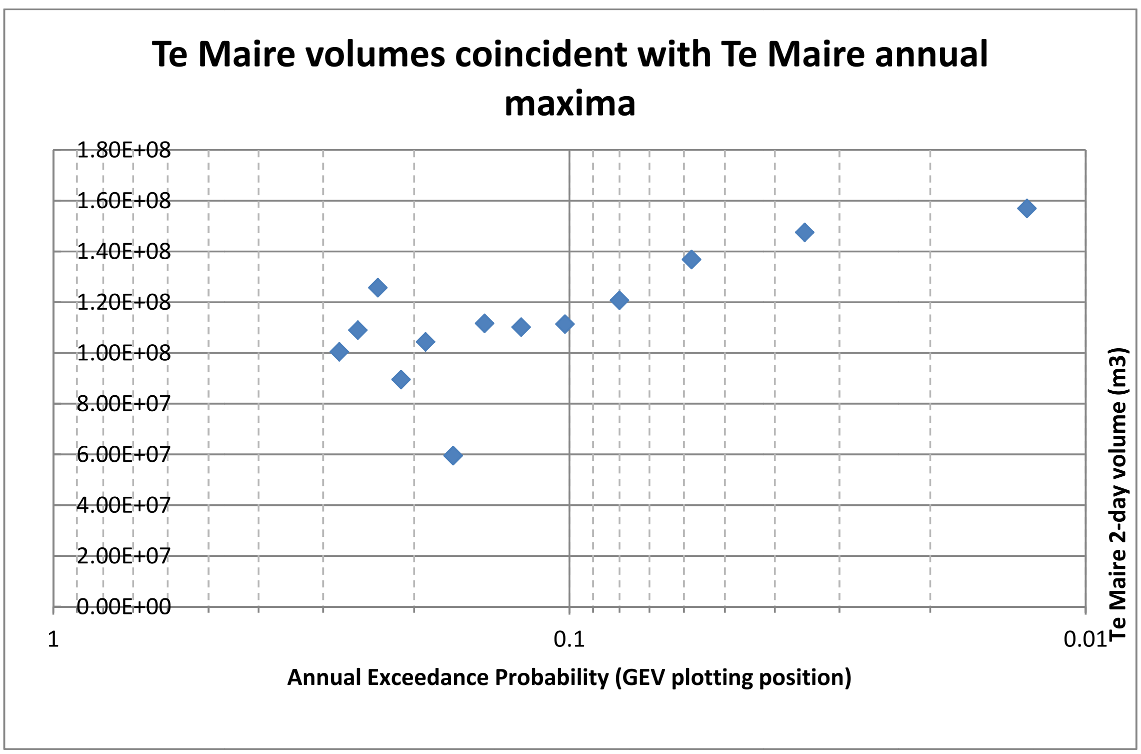

Figure 4-8: Scatter plot: AEP of Te Maire peak flow events vs. the 48-hour flow volume at Te Maire

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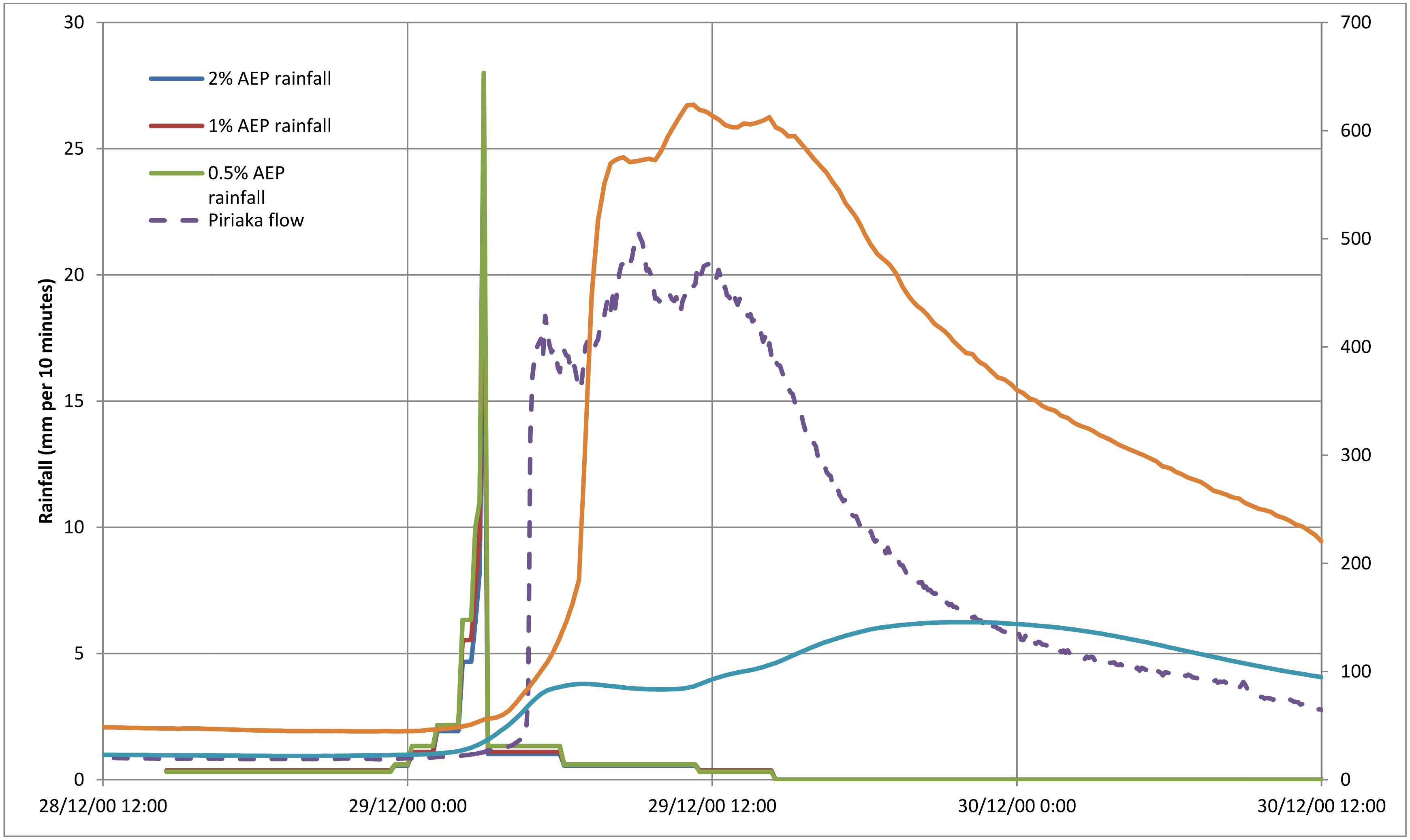

Figure 4-9: Hyetographs for Taumarunui derived from HIRDS Rainfall depth, with assumed coincident river flows

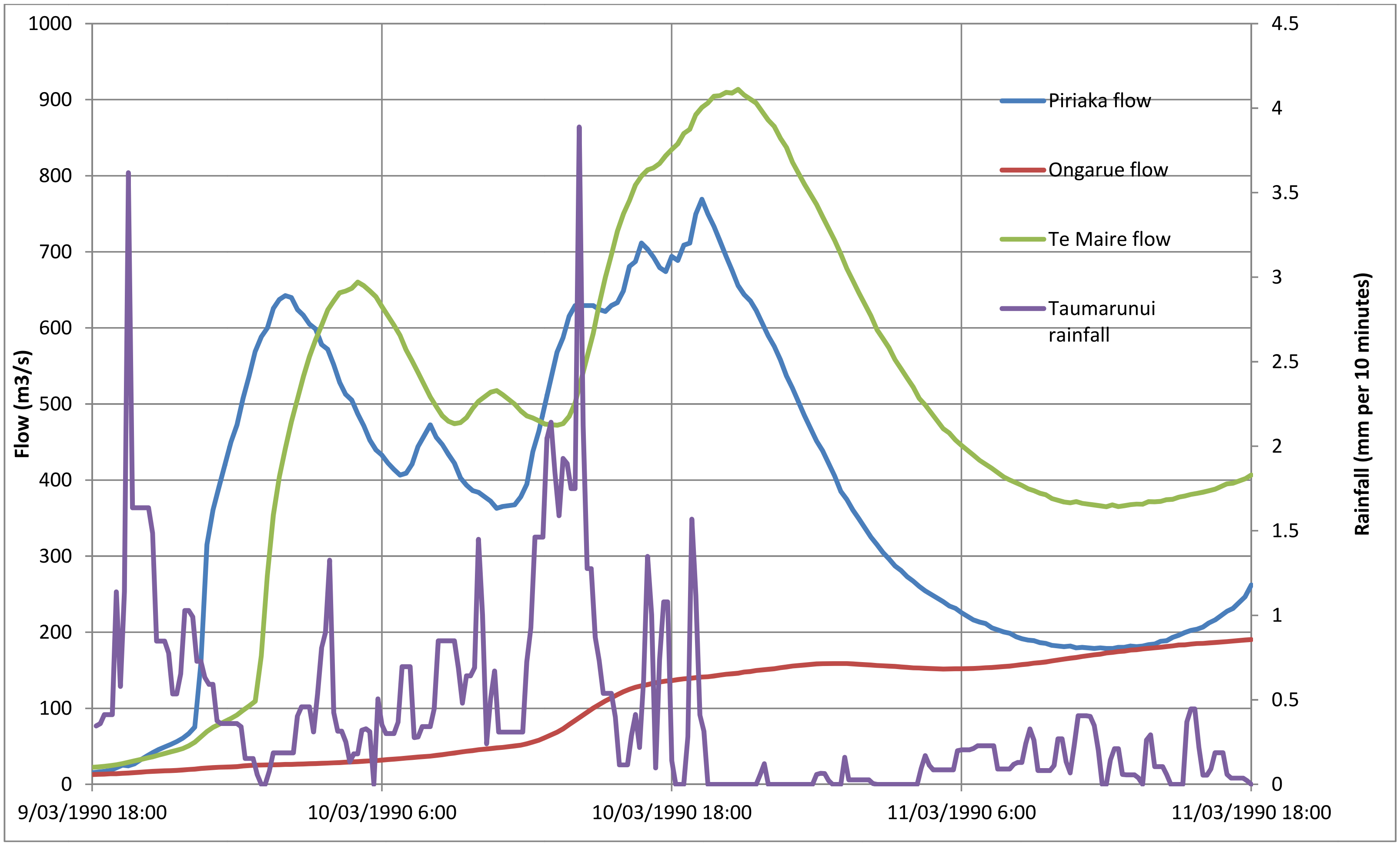

Figure 4-10: Measured flows and rainfall for event of 9-11 March 1990

Figure 4-11: Map showing Sub-catchments modelled in MIKE 11..

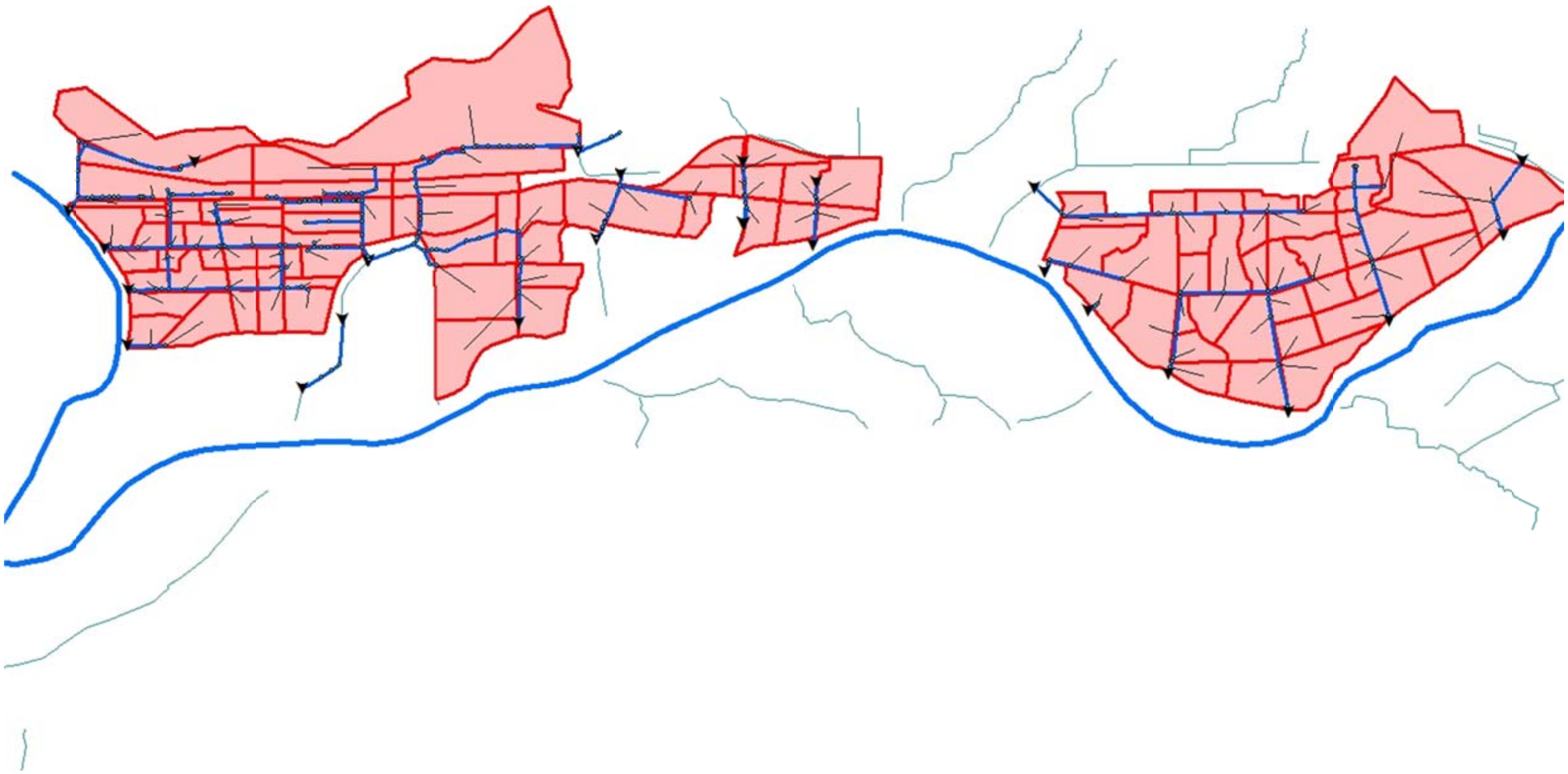

Figure 4-12: Map showing those urban Sub-catchments modelled in MIKE Urban.

5 HYDRODYNAMIC MODEL BUILD

This section outlines the detailed methodology and model parameters used in each of the model components; the rivers, the stormwater network and overland flow. The models are linked together using the MIKE Flood software by DHI to create a fully dynamic model of the hydrodynamics at and around the Taumarunui township.

5.1 River network

Within the reaches for which surveyed cross-sections are available, the survey data have been relied on for the MIKE 11 model. However, as a check, cross-section data at the same locations was obtained from the LiDAR survey.

At the Taringamotu gauge on the Ongarue River, and at the Te Maire gauge on the Whanganui River, data from the gauging themselves was used to derive at each site an equivalent cross-section with the same hydraulic properties as the real ones (which were not immediately available). On the Ongarue, this cross-section and the floodplain cross-sections derived from the LiDAR survey have been combined to produce model cross-sections in MIKE 11 of the channel with floodplain and upstream to Piriaka on the Whanganui River.

The Whanganui River reaches downstream of the surveyed cross-sections to Te Maire are outside the scope of this study, but it is sound modelling practice to use realistic downstream boundary conditions. The Te Maire rating has therefore been used directly as a boundary condition, with cross-sections interpolated between the synthetic one at Te Maire and the most downstream of the surveyed cross-sections. Tributary stream reaches within the floodplain have also been included in the model, connected to the main river network. With the possible exception of the Punga Punga River, their flow is too minor to significantly affect river levels, but overbank spills onto the floodplains may be important. The cross-sections have been taken from LiDAR, with a few minor adjustments, including addition of some channel depth in the Punga Punga River below the water levels evident in the LiDAR survey.

Calibration of the two main rivers is described in Section 6 below. No calibration data were available for the tributary stream channels. Their channel resistance was therefore estimated from experience. Some have been assigned the default Manning's n value of 0.032, but others with dense vegetation have been given higher values.

The network map for the MIKE 11 model is shown in Figures 5-1 and 5-2.

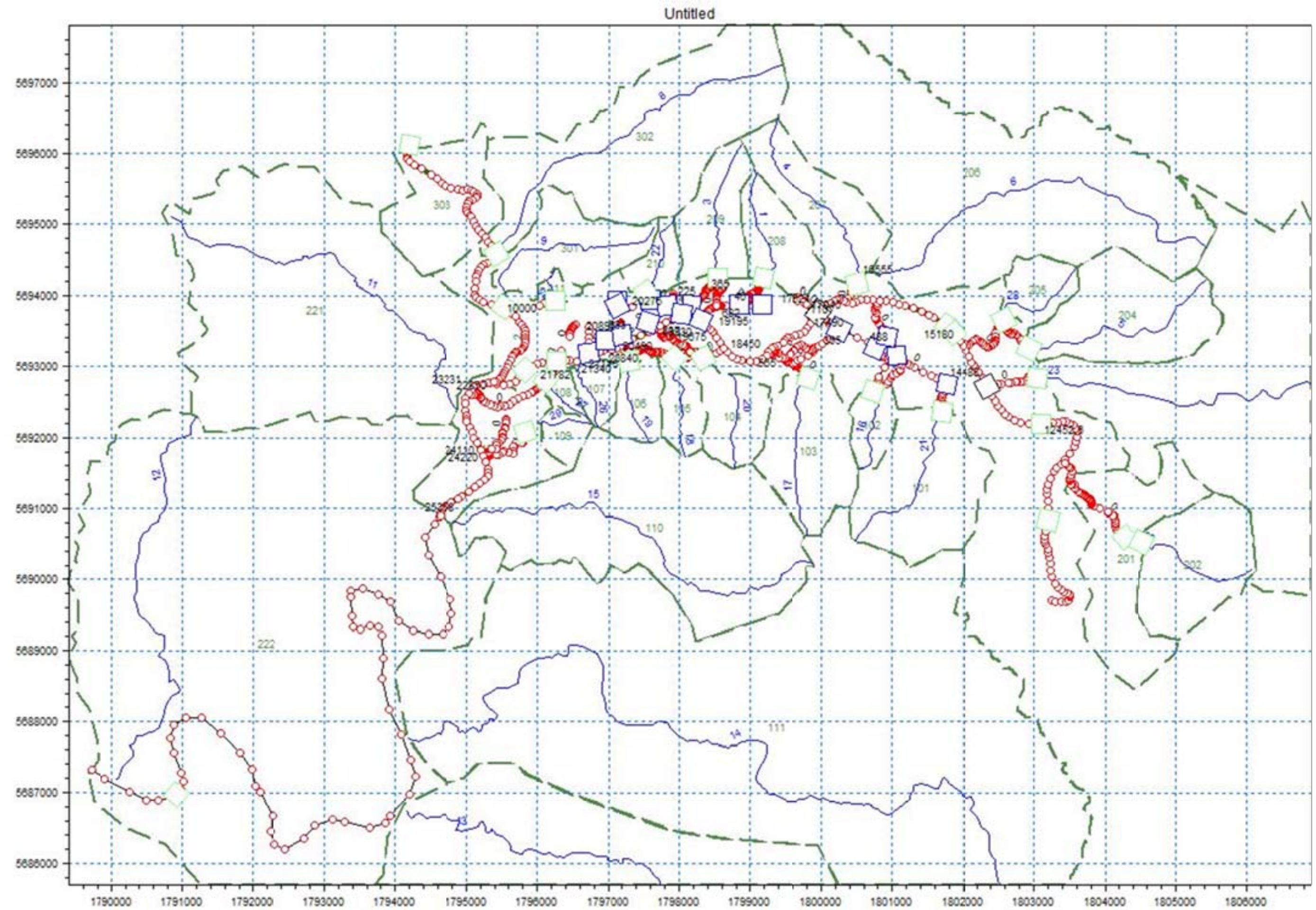

Figure 5-1: MIKE 11 model layout

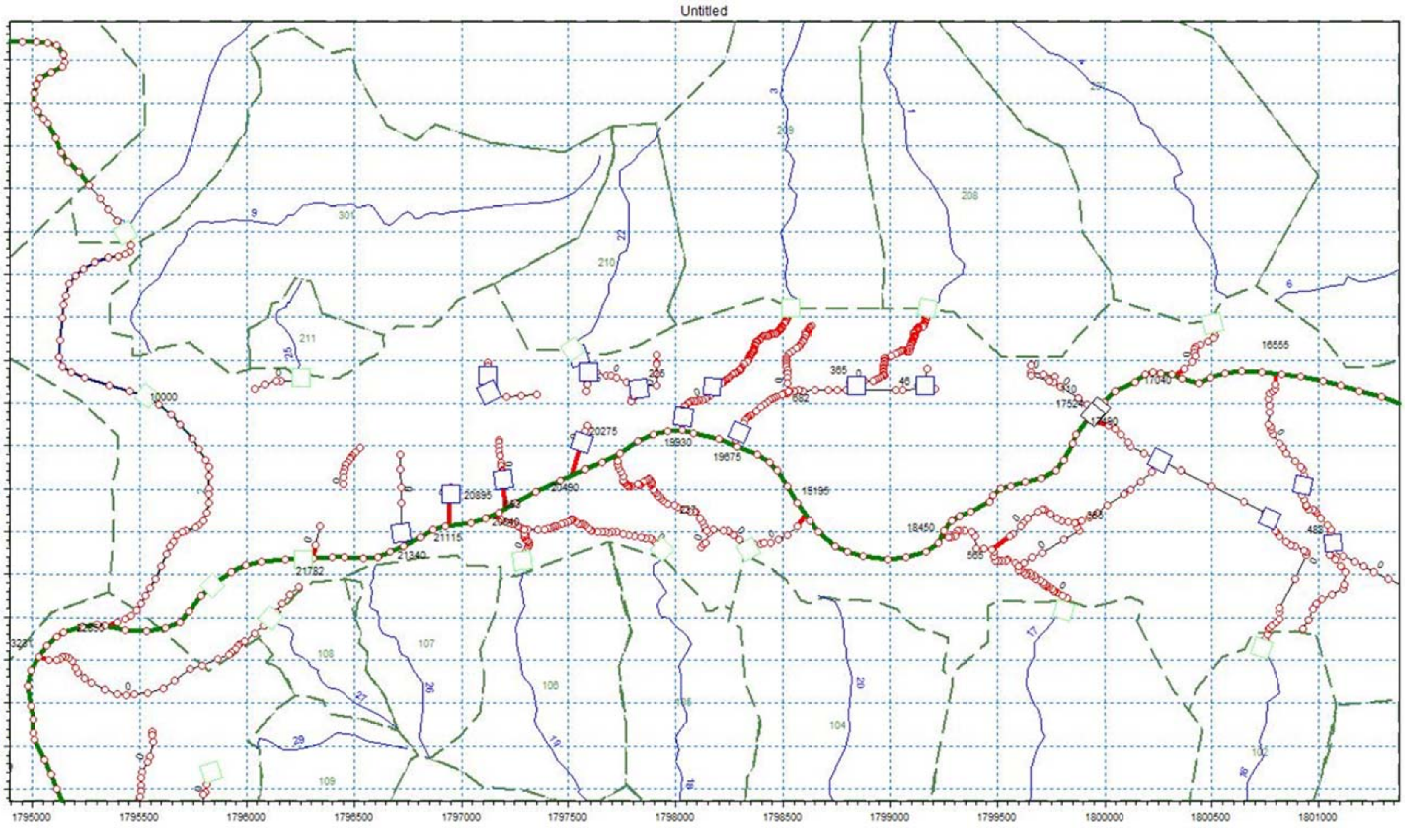

Figure 5-2: MIKE 11 model layout (detail: At and upstream of Whanganui – Ongarue confluence)

5.2 Stormwater network

The culverts conveying flow through the stopbank from Taumarunui into the Whanganui River are fitted with flapgates to prevent reverse flow when the river is high. These have been modelled as culverts with standard head losses and positive flow only. Any head loss specifically attributable to the flapgates is believed to be minor and has been ignored.

Surveyed invert levels were not available for many other culverts, particularly those on the smaller ditches. For these, the pipe diameter was measured on site if not already known, and the upstream and downstream inverts taken to align with the adjacent channel inverts derived from LiDAR.

Pipe network data was provided from RDC's database, covering the main town area of Taumarunui on the true right bank of the Whanganui River. Data was not available for the hill areas of urban Taumarunui, nor for the outlying settlements of Piriaka, Manunui, and Taumaruiti. These areas were therefore either not modelled (except for including their runoff as hydrological input) or included within the MIKE 21 overland flow model (see below).

A stormwater network model was built in MIKE Urban from the pipe network data, incorporating a few modifications that were considered necessary:

- At some manholes where invert levels have not been measured, these were interpolated.

- Other invert levels were varied slightly to provide a consistent pipe grade where the raw data indicate widely varying grades or even reverse grades. These changes do not in principle change the hydraulic performance in flood conditions, but are made to ensure model stability.

- At one location (Taupo Road at Porou Street) a pipe has been included that was noticed during the field inspection but is missing from the database.

- At the eastern end of Taupo Road, a 600mm pipe running away from the rail line has been presumed to connect with another 600mm pipe that ultimately discharges into the Whanganui River.

With no calibration data to hand, standard practice was followed in assigning head losses to the pipes and manholes.

5.3 Overland Flow

The overland flow is modelled using the MIKE 21 Flexible Mesh. The mesh is made up of triangular and quadrangular elements of varying sizes. Higher resolution was chosen for the more detailed areas, where large differences in elevation occur and around rivers, roads and other obstructions. Where rivers and open channels are modelled in the 1D model the grid is set as true land to avoid double-counting of both flood storage and flow conveyance (Figure 5-3). The size of the elements ranges from $7m^2$ up to $130m^2$, with a total of 144,932 elements in the mesh.

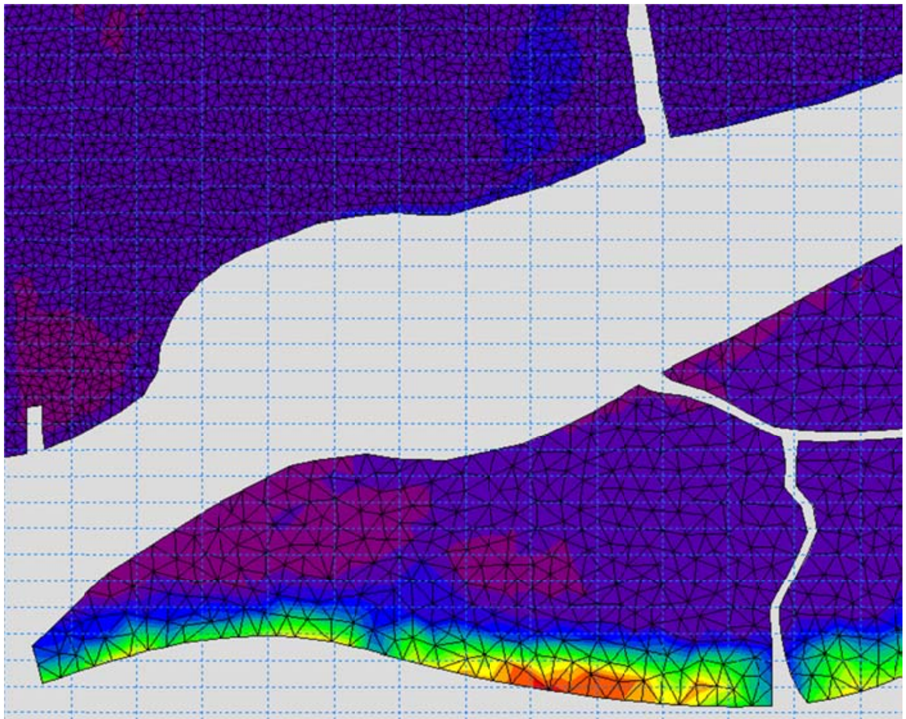

Figure 5-3: An example of flexible mesh grid showing finer and coarser mesh and blocked-out open channels

The 2D model covers the low-lying area in and around the Taumarunui town (Figure 1-1). The outer edges of the model grid lie where the terrain begins to steepen rapidly where flooding is unlikely to occur.

A constant eddy viscosity of 0.002 m^2/s was used, a nominal low value. This arbitrary choice is sufficient because bed friction dominates in determining the velocity pattern in overland flow on floodplains such as these.

The roughness distribution is based on the aerial photos and the land use shapefiles from LINZ. Table 5-1 presents the roughness value used for each land use type. The majority of the floodplain is grassland, with an assigned Manning's n roughness value of 0.05, and urban development using a Manning's n roughness value of 0.1.

Table 5-1: Roughness values used for different landuse types

Description (LCDB2)	Code	N	M
High Producing Exotic Grassland	40	0.050	20
Pine Forest - Closed Canopy	66	0.125	8
Pine Forest - Open Canopy	65	0.125	8
Indigenous Forest	69	0.125	8
Built-up Area	1	0.100	10
Other Exotic Forest	67	0.125	8
Urban Parkland / Open Space	2	0.033	30
River	21	0.020	50
River and Lakeshore Gravel and Rock	11	0.020	50
Deciduous Hardwoods	68	0.125	8
Lake and Pond	20	0.020	50

The initial condition for the flood plain is initially dry, with any water contained to the river and pipe network.

Most flow into and out of the MIKE 21 model is handled through the MIKE Flood links to the urban and river models (see 5.4 below). When water spills from the rivers and manholes it will enter the 2D model and flow overland. At one location, at the freezing works site on the Whanganui River south bank, catchment runoff computed in MIKE 11 has been applied directly to the MIKE 21 overland flow model.

Buildings can be included in the model mesh at a later point, as a way of refining the model.

5.4 MIKE Flood

MIKE FLOOD (Ref./6/) is the software that dynamically couples the three models to synchronise water levels and flows in real simulation time. Several types of coupling have been applied in the present model.

5.4.1 MIKE 11 – MIKE 21 coupling

Lateral links have been used to model spill flow between the stream reaches (and open drains) within the MIKE 11 model and the surrounding floodplain modelled in MIKE 21. These lateral links typically connect several MIKE 21 cells to each MIKE 11 h node, and (following standard practice) the maximum distance between MIKE 11 cross-sections has been limited to 20 m to avoid significant inaccuracies in the overland flow water levels.

The links are coded as simple weirs, which are intended to reflect the free over-bank weir flow that will generally occur. The coupling is bi-directional, allowing for both in-flow to the stream channel and outflow from it. The default values for weir friction (0.05) and coefficient (1.838) are applied for the links.

The spill level is chosen as the higher of the MIKE 11 cross section bank marker and the level in the linked 2D element. To prevent the double counting of volume the rivers and open channels were “blocked out” in the 2D mesh. Figure 5-4 shows an open channel that can spill laterally from the left bank onto the playing field in a flood event.

Figure 5-4: Open drain with no stopbanks: Taumarunui Domain, adjacent to Morere Terrace

5.4.2 MIKE 11 – MIKE Urban coupling

There are some locations within Taumarunui itself where lengths of open drain and pipe network alternate, requiring coupling between the MIKE 11 and MIKE Urban models. MIKE Urban includes a facility to describe outlets from the pipe network to an open channel, and this facility has been applied for all these couplings. Figure 5-5 shows an example of the urban network spilling into the Ongarue River.

Figure 5-5: Stormwater pipe discharging into the Ongarue River

This arrangement has proved difficult for the inlets to the pipe network from an open drain, but careful choice of the inlet parameters has avoided any model instabilities.

5.4.3 MIKE 21 – MIKE Urban coupling

Individual catchpits and other inflow points have not been included in the MIKE Urban network model, so these inflow points have been lumped to their downstream manhole with these manholes directly linked to the 2D grid. The capacity of the links is based on the number of catchpits feeding into the manhole and the size of these catchpits. The location of catchpits was taken from the RDC stormwater shapefiles supplied for the project.

Flows into the network – and out of it – are represented by weir flows at manholes. An upper limit has been applied to the flow rate to or from each modelled manhole, representing the maximum flow that can pass through the catchpits and other inlets. When this inflow capacity is exceeded the additional water will stay in the 2D model. Where catchments are linked to the urban model when the inlet capacity is exceeded the additional water will spill into the 2D model. All manholes with either catchpit or catchment connections are linked to the 2D model.

Detailed information on the catchpits and other inlets was not to hand, so these inflow /outflow limits are estimates rather than precise values. A detailed hydraulic study aimed at improving specific aspects of the network performance would warrant a closer assessment of inlet conditions.

Figure 5-6 shows a typical Taumarunui grated catchpit in Short Street. This type of catchpit has been assumed for the model; for each catchpit the inflow area was taken to be $0.1818m^2$ with a maximum inflow capacity of $0.03m^3/s$.

Figure 5-6: Typical catchpit found in Taumaranui stormwater network

6 HYDRAULIC MODEL CALIBRATION

Where possible, hydraulic models should be calibrated so that past events are accurately modelled. This generally involves modelling one or more past events for which the inflow hydrographs are known, and adjusting hydraulic properties (particularly the flow resistance provided by the channel) so that modelled water levels match observed levels.

At Taumarunui, despite some anecdotal observations of floods, the only firm data available for calibration is at the flow gauging sites at Piriaka on the Whanganui River and Taringamotu on the Ongarue River. The flow hydrographs obtained from these sites have been used as upstream boundary conditions for the model, but the stage records can be use as well to calibrate the adjacent river reach.

The MIKE 11 model was therefore run a few times for the 1998 flood, using different values of Manning's n for different runs. A single Manning's n value was applied to the entire length of the Whanganui River for which cross-sections were available, and a separate value was applied to the entire modelled length of the Ongarue River. This simplification is broadly consistent with our observations of the channels on site.

Modelled water levels were compared with the recorded data at Piriaka and Taringamotu. The Flow and depth hydrographs plotted in Figures 6-1 and 6-2 respectively. In Figures 6-3 (Whanganui @ Piriaka) and 6-4 (Ongarue @ Taringamotu) the same data are presented as flow plotted against stage. The measured data thus recreate the ratings for the gauged sites.

Manning's n was then set to the value needed to match gauging and modelled values for the peak flows in the 1998 event. Both the resulting Manning's n values appear consistent with the observed state of the channel:

- 0.39 in the Whanganui River from Piriaka to about 2km downstream of the confluence; and
- 0.51 in the Ongarue River.

The rather high value for the Ongarue can be ascribed to the effect of willows and other riverbank vegetation, a recognised problem there.

Subsequent to this limited calibration of the model, Manning's n values were further varied in the Ongarue River (from 200 m upstream of the State Highway 4 bridge to the Whanganui confluence). This was to attempt to replicate a quoted observation during the October 2000 flood of a water level just downstream of the bridge within 0.4 m of flooding the town. Increasing Manning's n for this lower reach raised the peak flood level for that event by about 0.4 m, but the modelled peak level is nevertheless a further 1 m lower.

This difficulty illustrates the uncertainties in both numerical modelling and field observations. In particular, higher bed levels than the more recent survey, and/or debris trapped in the bank vegetation, could have offered more flow resistance than has been modelled. Exactly what was observed during the flood is also uncertain. The flow-gauging site at Taringamotu is likely to be one of New Zealand's more reliable sites, but some uncertainty in gauged flows is always present.

From Figure 6-1 it is clear that the volume measured at Te Maire exceeds that modelled using the measured inflows at Piriaka and Taringamotu. This is also evident with other events, though not quite to the same degree. Some of the discrepancy could be due to under-estimating the inflows from the Punga Punga and minor catchments, but not all of the discrepancy.

It therefore appears that the gauging sites are inconsistent with one another at high flows. Whilst it is not clear which site or sites might be in error, it is quite likely that the Te Maire high flows are over-estimated, being based on slope-area gauging prior to the gauging site being established.

In this study, therefore, the Piriaka and Ongarue flows have been taken as correct. For the end purpose of fixing design flood levels, this is the more conservative assumption.

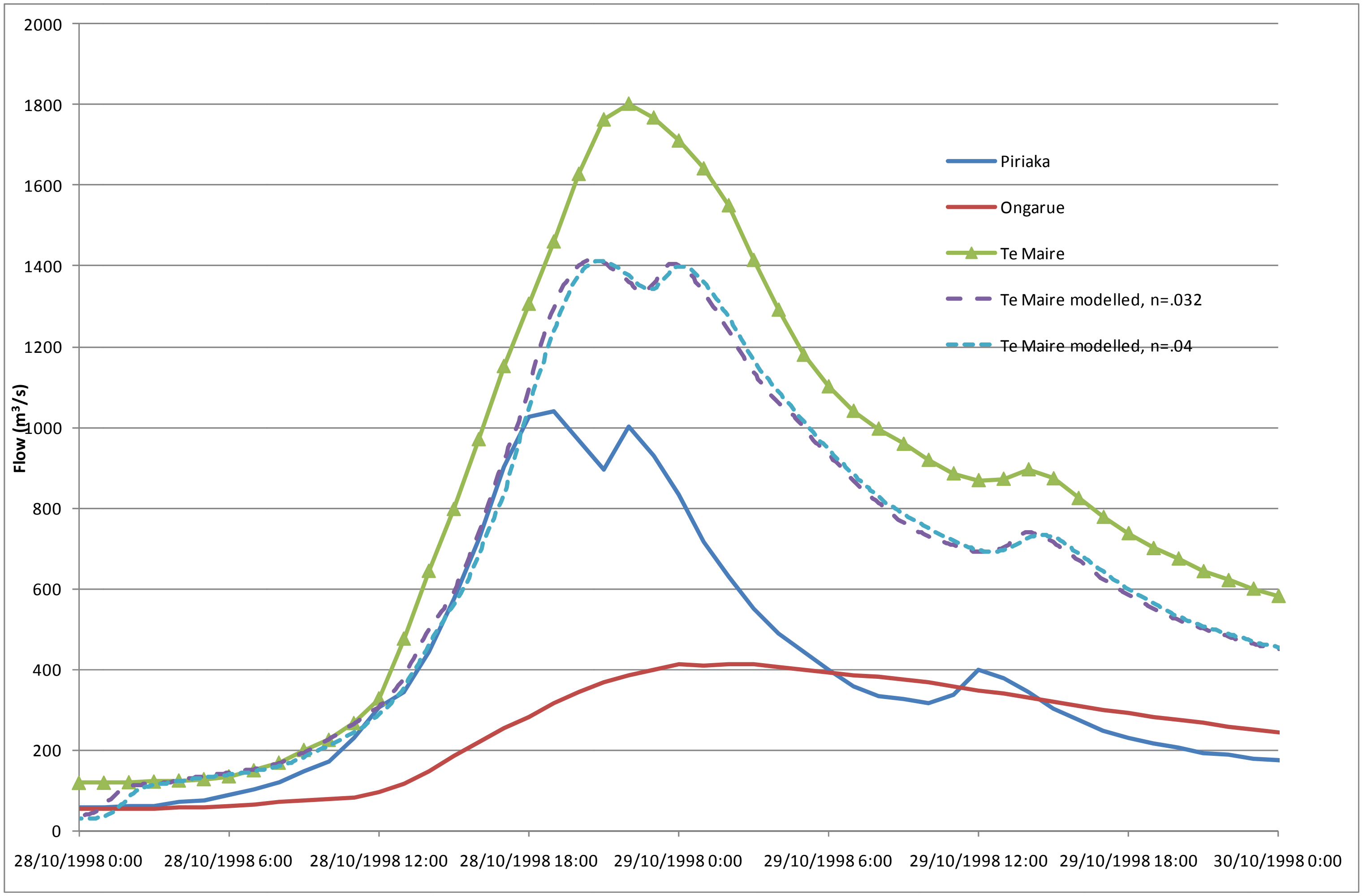

Figure 6-1: Modelled and measured flow hydrographs for event of 28 October 1998

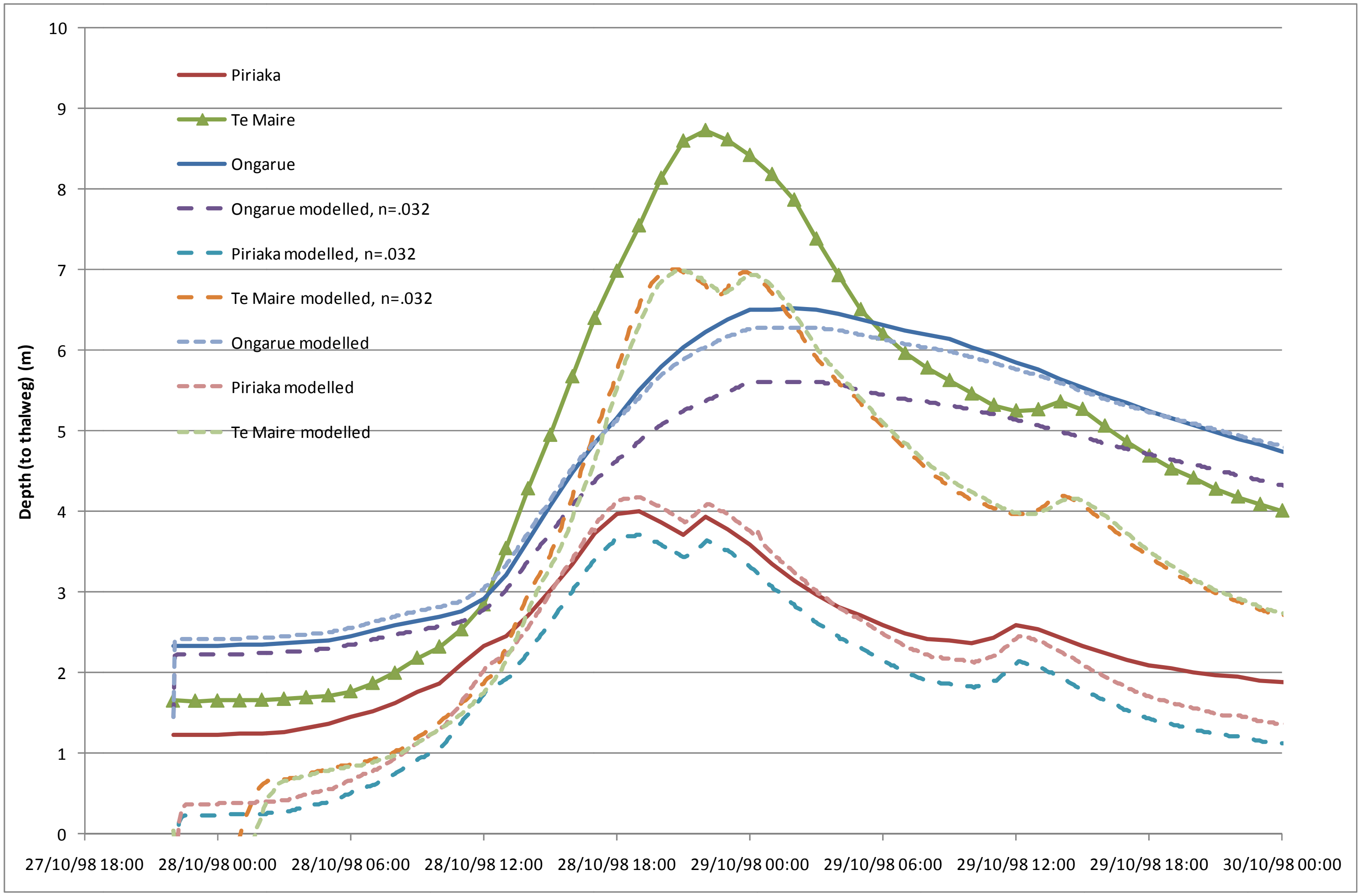

Figure 6-2: Modelled and measured flow depths (at gauging sites) for event of 28 October 1998

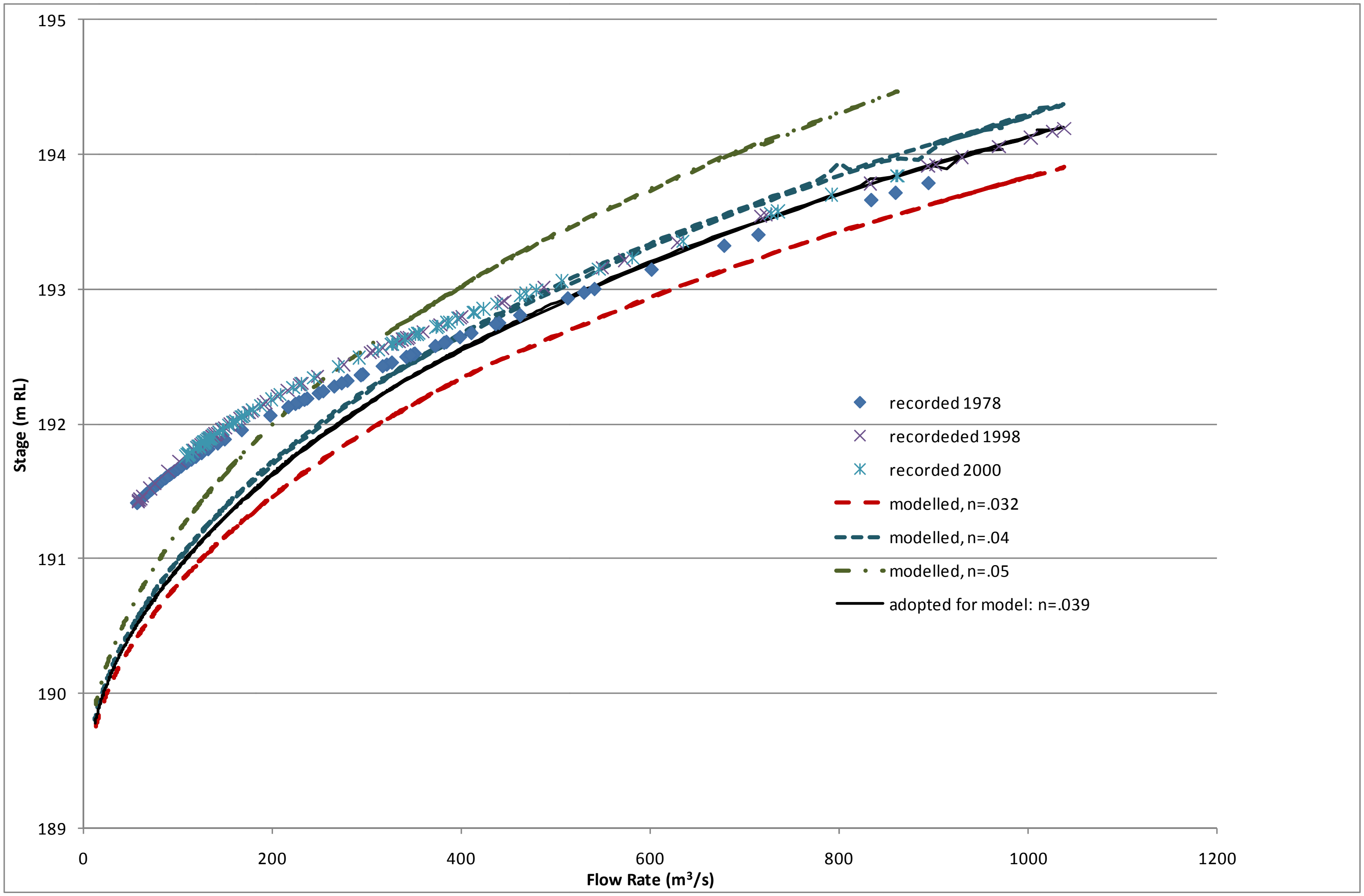

Figure 6-3: Whanganui @ Piriaka: Modelled flow and stage compared with ratings determined from gauging

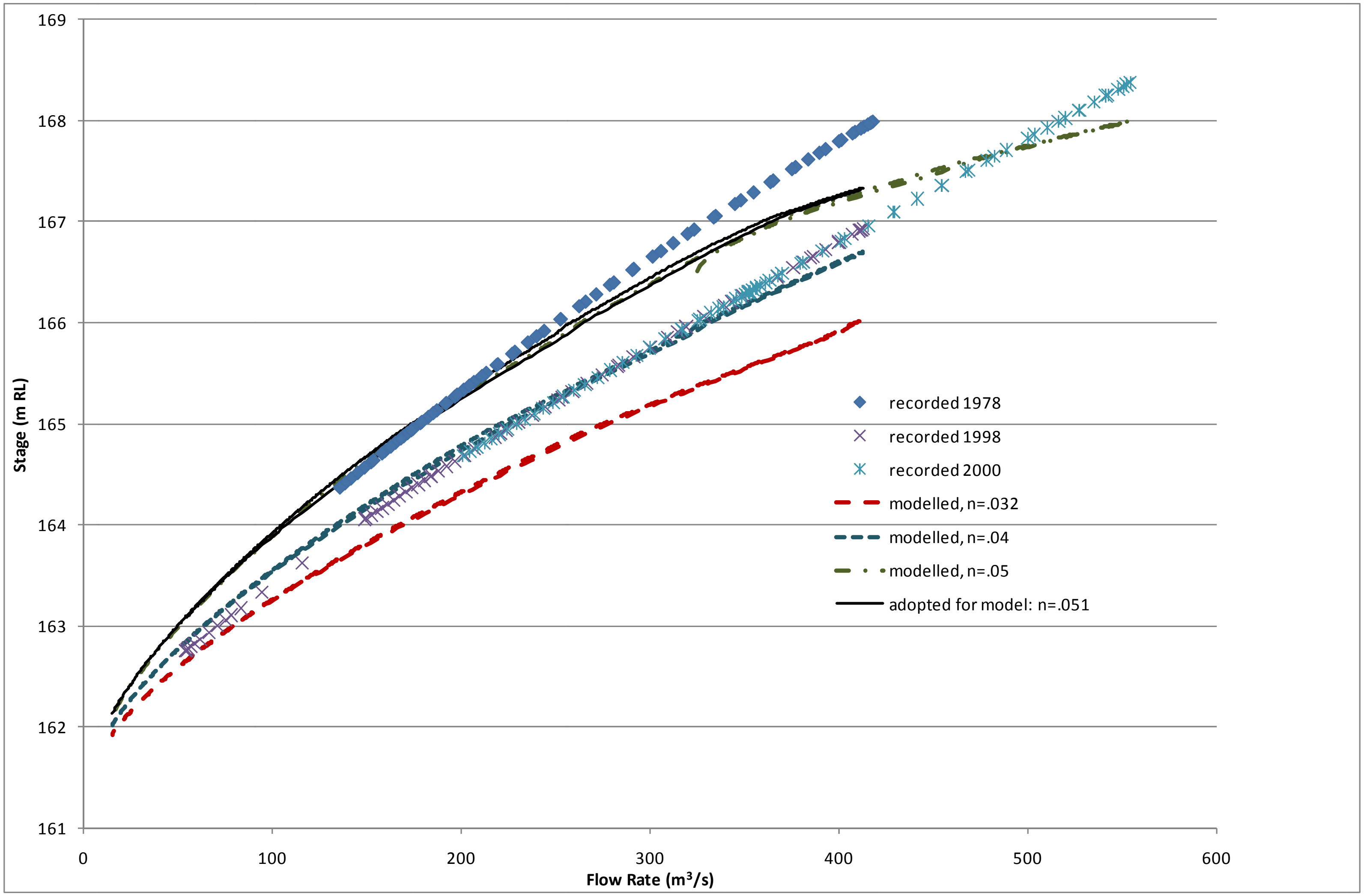

Figure 6-4: Ongarue @ Taringamotu: Modelled flow and stage compared with ratings determined from gauging

7 FLOOD RISK ASSESSMENTS

7.1 Design Events

The modelling described in this report has included two distinct types of event: a flood in the Whanganui River and a local rainfall event. For specifying the flood level and other flood parameters, for land use planning and other applications, the more severe of these two events at any particular location is needed.

7.1.1 The Flood Maps

Volume 2 of this report comprises flood maps that for each ARI combine the two event types. These maps should be taken as showing the best estimate of the true Flood map sets have been produced for ARIs of 50, 100 and 200 years, each comprising sets of 16 maps:

- Four different map coverages, two (at 1:5000 when printed at A3 size) covering much of Taumarunui itself, and two at 1:15,000 covering locations on the Whanganui River immediately upstream and downstream of Taumarunui.

- Event maximums for four flooding parameters:

  - Flooding depth (in which very shallow depths have been omitted);

  - Flow velocity;

  - Water surface elevation; and

  - Flood hazard.

Flood Hazard was computed using post-processing software developed by DHI following the definition set out in the NSW Floodplain Development Manual (see Fig. 7-1).

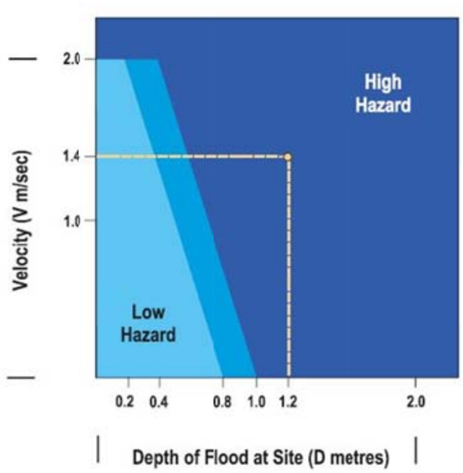

Figure 7-1: Hazard Calculation Criteria

The river channels have been excluded from mapping, on the presumption that these are excluded from land use planning. However, spot river level elevations are shown.

7.1.2 Selected peak flood levels

Appendix B includes a graph of modelled peak water levels in the Whanganui River adjacent to Taumarunui township. The 50-year, 100-year and 200-year levels are shown, as well as peak levels from two historical events in 1990 and 1998. For comparison, invert and soffit levels are plotted for the culverts that convey water through the stopbank.

This graph shows the extreme river flow events impeding drainage through some of the culverts, but not all, as their invert levels are exceeded. The culverts are provided with flapgates to prevent reverse flow from the river, but drainage from the township is then impossible (presuming that floodwaters in town have not ponded to above river level).

Also in Appendix B is a graph of modelled peak water levels from the upstream end of the MIKE 11 model (corresponding to the uppermost surveyed cross-section) to the edge of Taumarunui. The Piriaka gauging station is 2.24 km downstream from the model boundary.

Some representative peak flood levels within Taumarunui township are set out in Table 7-1. These show that flooding from local runoff, in response to rainfall events determined from HIRDS, is typically more severe within the township than flooding associated with Whanganui River floods.

Table 7-1: Some representative peak flood levels within Taumarunui township

Location	100-year local rainfall	200-year local rainfall	100-year Whanganui River flood	200-year Whanganui River flood
Taumarunui Domain near Morere Tce (Fig. 5-4).	165.57	165.97	164.23	164.30
Near stopbank, drain east of saleyards (south of 69 Taupo Rd).	168.35	168.40	166.66	166.82
Drain beside railway line adjacent 6 Totara Crescent (at culvert).	171.89	171.96	171.66	171.68
Drain beside railway line adjacent 9 Taupo Road.	169.11	169.12	169.07	169.07

7.1.3 Features of the flooding as modelled

The following comments follow from inspection of the model output, including animations of the flooding depth through the design event:

Within Taumarunui itself, the more extensive flooding occurs with the local rainfall. This is in part because flooding is exacerbated by the small streams that discharge into the town. These streams typically flow to the Whanganui River in open drains, although some smaller streams run into the pipe network. In both cases, the 100-year and 200-year events in particular generally overwhelm the drainage network. This is expected, as stormwater pipes and drains are typically sized for the 10-year ARI event or similar.

Parts of Manunui are also shown to be flooded in the 200-year ARI local rainfall event in particular. This flooding is shallow and not particularly extensive, but would be more significant but for the railway line and the drains that run parallel to it. These protect much of Manunui by diverting hill runoff towards the north-west, albeit at the expense of properties on Matai Street.

For most of Taumarunui and the other settlements, the model results do not show the river flood events to be a major direct threat. The 100-year and 200-year river levels are high enough to impede drainage from parts of Taumarunui (including Matapuna) by reaching the level of some of the culverts through the stopbank, where the effect is to close the flapgates that have been installed to prevent reverse flow. However, flood levels are considerably lower than the top of the stopbanks.

The model results indicate that the settlements of Manunui and Mahoe are clear of the Whanganui River flood hazard zone, as are areas of flat land further downstream. From a geomorphic viewpoint, these results suggest that the Whanganui River may have become entrenched by down-cutting since the valley flats were formed.

The uncertainties in specifying the channel resistance (in the form of Manning's n) have been noted elsewhere in this report. However, all reasonable assumptions for this resistance result in the above conclusion that the stopbanks provide a generous freeboard for the extreme events considered here.

A reassuring feature is the "high flooding hazard" areas identified by the model runs. Although these areas in the 200-year ARI event do cover some residential sections, they are in general confined to undeveloped areas including sports fields, a sign that the community is already aware of the most severe flood risks.

7.2 Conclusions and Recommendations

7.2.1 "Design" Flood Levels

We are confident that the modelled flooding levels presented in the maps accompanying this report for 0.5%, 1% and 2% AEP events are the best estimate of the present flood risk that can be made with the information now available. The two scenarios tested, of a major flood in the Whanganui River and an extreme local rainfall event, represent the likely range of events contributing to this flood risk. The flooding maps attached to this report are therefore suitable for land use planning and other statutory uses, and for civil defence planning.

The model results indicate that most river flats and terraces outside Taumarunui town in fact lie above the 1% AEP (100-year ARI) flood level. However, there are some locations within and near the urban area that are prone to flooding. For the most part, this seems to have been recognised, as most of this low-lying land is used for sports fields and parks rather than housing or industrial uses.

The flood event of March 1990, with a peak Whanganui River flow of 800 m^3/s, represents the approximate threshold above which water levels in the Whanganui River are high enough to close the flapgates and prevent drainage of Taumarunui. The computed river level is midway between invert and soffit at the Culvert 9 outlet and above invert level for two more culvert outlets. In more extreme events, with higher river levels, much of Taumarunui's urban drainage system is compromised by river levels, with the flapgates then being essential to prevent worse flooding.

The flow of 800 m^3/s has a computed AEP of 14% (i.e. an ARI of 7 years). It is likely that the stormwater drainage system has been designed for a comparable AEP such as 20% or 10%.

7.2.2 Effect of channel roughness on flood levels

As part of the calibration process, and to test the model's robustness, some model runs (MIKE 11 only) were carried out with different assumed channel roughness values for the Whanganui and Ongarue Rivers. Variation of Manning's n of $\pm$20% resulted in water level variations typically of $\pm$ 0.4m.

At locations where it appears that improving the channel hydraulics would significantly reduce flood risk, this conclusion could be tested and refined, by modelling a reduction to channel resistance in specific locations that could be achieved by willow clearance and other channel improvements.

Channel improvements could similarly be tested for the many small streams and ditches within the model. However, results for the ditches in particular would be indicative rather than quantitative, given that their present channel roughness is unmeasured and somewhat difficult to assess.

7.2.3 River gauging sites

At peak flows, there appears to be an inconsistency between the three river gauging sites at Piriaka, Taringamotu and Te Maire, with that at Te Maire indicating higher flows than can reasonably be explained from gauged and estimated contributing flows. This appears worthy of further investigation, given the long length of all three river flow records.

7.2.4 Future work to refine the model

The MIKE FLOOD model described in this report is believed to include a reasonable representation of the hydraulic features of the rivers and floodplains, and can therefore be expected to provide reasonably accurate flood predictions. Furthermore, the flooding data that it has provided appear very suitable both for land use zoning and for civil defence planning.

However, more certainty in the output data could be provided, and also a greater level of detail, could be attained with further site data and with a more detailed description of the hydraulic features. This more detailed modelling might include more blocking out of large buildings to provide improved description of the flow in their vicinity.

River flood levels

Any water level observations will help provide a more fully calibrated model, especially observations when flow in the Whanganui at Piriaka exceeds 400m^3/s. These can be either peak water levels (which can often be identified after the event from debris lines) or ad hoc water levels at a particular location and time. All these data can be compared with a model hindcast of the particular event, and the model then calibrated primarily by adjusting the assumed channel roughness values.

Local flooding
In principle, a better understanding of flooding of local origin requires improved knowledge of rainfall, infiltration to ground and other losses, the local stormwater sewer network, and the hydraulics of the flapgate outlets. As long as the three presently operating rain gauges continue, rainfall information can be considered adequate. To better describe the conversion of this rainfall to runoff and the drainage of runoff in Taumarunui, the following data would help:

Network information

- Pipe connectivity, particularly details of any pipes missing from the network captured in GIS;

- Typical blockage and hydraulic conditions at pipe and culvert intakes (blockage that typically occurs during a major event may well not be predictable from a dry-weather inspection, but a pipe intake blocked most of the time can be expected to be well blocked during the major events); and

- The location and flow capacity of catchpits and other inlets to the pipe network.

Storm event observations

Both on-site measurements and ad hoc observations are valuable, and photographs particularly so, but the time of day is needed for quantitative interpretation:

- Water level observations for large ponded areas (ideally these should be accompanied by information on the amount of blockage, if any, of pipes and channels draining the site);

- Reports of flows out of manholes and catchpits, and overflows from streams and ditches;

- The runoff hydrograph from one of the hill catchments behind Taumarunui. (this would require at least three gaugings during a storm event, and the time of peak discharge would be useful additional information); and

- Observations of flow rates from the flapgates, particularly if ponding levels immediately upstream can also be obtained.

7.2.5 Future applications of the model:

It is expected that the present model results will be used to inform the District and Regional Plans, so that flood-prone land is not developed (particularly for housing) without flood mitigation works.

It is also expected that civil defence planning will rely on the present model output.

In addition, the model might also be used for:

- Improvements to the urban stormwater reticulation network: Targeted maintenance and new or upgraded pipes. Some further drainage details (smaller sub-catchments at catchpit details) may be needed;

- Developing a maintenance regime for Whanganui & Ongarue Rivers: Willow clearance and other vegetation control; and

- Design of any proposed stopbanking, or modification of existing stopbanks.

However, to address local drainage problems in rural areas and in outlying settlements such as Manunui, or very local drainage problems in Taumarunui at the catchpit level, the present model would need to be enhanced by incorporating closer detail of infrastructure and drainage patterns.

8 REFERENCES

/1/. National Institute of Water & Atmospheric Research (NIWA), High Intensity Rainfall Design System (HIRDS) Reference Manual, June 2002.

/2/. DHI Water & Environment: Chapter 4: Rainfall-Runoff Module Reference Manual, MIKE 11 Rainfall-Runoff Module Reference Manual, 2009.

/3/. U.S. Soil Conservation Service (SCS), 1972, National Engineering Handbook, Section 4, U.S. Department of Agriculture, Washington, D.C.

/4/. Auckland Regional Council, 1999, Guidelines for stormwater runoff modelling in the Auckland Region, Technical Publication No. 108.

/5/. McKerchar, A.I.; Pearson, C.R., 1989: Flood frequency in New Zealand. Pub. No. 20, Hydrology Centre, DSIR, Christchurch. 87 p.

/6/. DHI Water & Environment: MIKE FLOOD Reference Manual, 2009.

A P P E N D I C E S

A P P E N D I X   A

Extreme value analyses of annual peak flow events in the Whanganui River: GEV distribution

Appendix A-1: GEV analyses of Te Maire annual peak flow events: peak flows at Te Maire and at Piriaka

Whanganui @ Te Maire (site 33302): GEV analysis of annual peak flow rates since 1965.

~~~ NIWA Tideda ~~~ DHI New Zealand 19-MAR-2010 11:43
~~~ XFPLOT ~~~
Reading data from C:\Program Files\NIWA\Tideda\Working\EVENTS.DAT
(Source file is C:\PROGRAM FILES\NIWA\TIDEDA\WORKING\3FLOWSITES.MTD)
Site 933356 Whanganui at 3 flow sites
From 1-Jan-1965 00:00:00 to 31-Dec-2009 24:00:00
Data selected from months January to December inclusive

12 mth Partition starts yyyymm	Recorded at yyyymmdd:hhmmss	maximum Value measured Te Mai m3/s		(Alpha=0.400) ann. Prob. 1/y	ret. per y
199801	19981028:220000	1801	A	0.013	75
200001	20001002:234500	1466	B	0.035	28
200401	20040229:064500	1394	C	0.058	17
197801	19781113:001500	1350	D	0.080	13
200301	20031004:040000	1252	E	0.102	10
199601	19960421:114500	1244	F	0.124	8
198601	19860726:044500	1217	G	0.146	7
199701	19970409:083000	1208	H	0.168	6
200801	20081007:220000	1172	I	0.190	5
199301	19931121:174500	1138	J	0.212	5
199501	19950907:063000	1134	K	0.235	4
196501	19651107:235900	1089	L	0.257	4
198801	19880608:124500	1084	M	0.279	4
200101	20010525:171500	1058	N	0.301	3
197901	19791014:191500	1007	O	0.323	3
199101	19910807:080000	1003	P	0.345	3
199401	19941108:174500	951	Q	0.367	3
200601	20061130:180000	925	R	0.389	3
196601	19660426:235900	921	S	0.412	2
199901	19990517:004500	919	T	0.434	2
199001	19900310:204500	914	U	0.456	2
198901	19891015:180000	869	V	0.478	2
198001	19800123:011500	852	W	0.500	2
196701	19670202:213000	837	X	0.522	2
197301	19730916:240000	819	Y	0.544	2
197701	19770628:213000	805	Z	0.566	2
197101	19710601:123539	796	a	0.588	2
197501	19750829:120000	791	b	0.611	2
200201	20020930:004500	773	c	0.633	2
196801	19681207:230000	732	d	0.655	2
198101	19810925:031500	725	e	0.677	1
197201	19720311:054642	691	f	0.699	1
198401	19841216:233000	656	g	0.721	1
199201	19920810:204500	649	h	0.743	1
197001	19700917:173000	639	i	0.765	1
200901	20090213:002000	635	j	0.788	1
197601	19760430:151500	633	k	0.810	1
200701	20070318:180000	579	l	0.832	1
198201	19820225:034500	544	m	0.854	1

197401 19740703:114500 525 n 0.876 1
198301 19831108:171500 522 o 0.898 1
198501 19851202:140000 511 p 0.920 1
196901 19690424:003000 456 q 0.942 1
198701 19870122:181500 443 r 0.965 1
200501 20050919:020000 420 s 0.987 1
Mean = 892

End of process
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Reading data from C:\Program Files\NIWA\Tideda\Working\EVENTS.DAT
(Source file is C:\PROGRAM FILES\NIWA\TIDEDA\WORKING\3FLOWSITES.MTD)
Site 933356 Whanganui at 3 flow sites
From 1-Jan-1965 00:00:00 to 31-Dec-2009 24:00:00
GEV exponent 0.08718 has magnitude greater than 0.07541 and so GEV fits better than Gumbel

Moments L1= 892 L2= 173 T3= 0.115 T4= 0.100
L-moments estimates of distributions's parameters:
Location = 758 Scale = 269 Shape = 0 100yr/2.33yr = 1.955
Data selected from months January to December inclusive

12 mth Recorded maximum -- GEV Distribution --
partition value measured 1.96 ann. ret.
starts at Te Mai Te Mai std. prob. per.
yyyymm yyyymmdd:hhmmss m3/s m3/s dev. 1/y y
2156 0.001 1000
199801 19981028:220000 1801 A 1801 0.009 113
1779 0.010 100
1649 0.020 50
200001 20001002:234500 1466 B 1466 0.049 20
1463 0.050 20
200401 20040229:064500 1394 C 1394 0.069 15
197801 19781113:001500 1350 D 1350 0.084 12
1309 0.100 10
200301 20031004:040000 1252 E 1252 0.127 8
199601 19960421:114500 1244 F 1244 0.131 8
198601 19860726:044500 1217 G 1217 0.146 7
199701 19970409:083000 1208 H 1208 0.152 7
200801 20081007:220000 1172 I 1172 0.175 6
199301 19931121:174500 1138 J 1138 0.199 5
1137 0.200 5
199501 19950907:063000 1134 K 1134 0.202 5
196501 19651107:235900 1089 L 1089 0.239 4
198801 19880608:124500 1084 M 1084 0.243 4
200101 20010525:171500 1058 N 1058 0.267 4
197901 19791014:191500 1007 O 1007 0.318 3
199101 19910807:080000 1003 P 1003 0.322 3
199401 19941108:174500 951 Q 951 0.380 3
200601 20061130:180000 925 R 925 0.411 2
196601 19660426:235900 921 S 921 0.416 2
199901 19990517:004500 919 T 919 0.418 2
199001 19900310:204500 914 U 914 0.425 2
910 0.430 2.33
198901 19891015:180000 869 V 869 0.482 2
198001 19800123:011500 852 W 852 0.505 2
196701 19670202:213000 837 X 837 0.524 2

197301	19730916:240000	819	Y	819	0.548	2
197701	19770628:213000	805	Z	805	0.568	2
197101	19710601:123539	796	a	796	0.581	2
197501	19750829:120000	791	b	791	0.588	2
200201	20020930:004500	773	c	773	0.612	2
196801	19681207:230000	732	d	732	0.668	1
198101	19810925:031500	725	e	725	0.677	1
197201	19720311:054642	691	f	691	0.722	1
198401	19841216:233000	656	g	656	0.766	1
199201	19920810:204500	649	h	649	0.775	1
197001	19700917:173000	639	i	639	0.786	1
200901	20090213:002000	635	j	635	0.792	1
197601	19760430:151500	633	k	633	0.794	1
200701	20070318:180000	579	l	579	0.852	1
198201	19820225:034500	544	m	544	0.885	1
197401	19740703:114500	525	n	525	0.900	1
198301	19831108:171500	522	o	522	0.903	1
198501	19851202:140000	511	p	511	0.911	1
196901	19690424:003000	456	q	456	0.946	1
198701	19870122:181500	443	r	443	0.953	1
200501	20050919:020000	420	s	420	0.963	1
	Mean =	892				

End of process

Whanganui @ Piriaka (site 33356): GEV analysis of peak flow rates for Te Maire annual peak events, since 1965.

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~~~ XFPLOT ~~~
Reading data from C:\Program Files\NIWA\Tideda\Working\EVENTS.DAT
(Source file is C:\PROGRAM FILES\NIWA\TIDEDA\WORKING\3FLOWSITES.MTD)
Site 933356 Whanganui at 3 flow sites
From 1-Jan-1965 00:00:00 to 31-Dec-2009 24:00:00
Data selected from months January to December inclusive

12 mth Partition starts yyyymm	Recorded Value at yyyymmdd:hhmmss	maximum measured Piriak m3/s		(Alpha=0.400) ann. Prob. 1/y	ret. per y
199801	19981028:183000	1056	A	0.013	75
200401	20040229:031000	980	B	0.035	28
199701	19970409:060000	934	C	0.058	17
197801	19781112:211500	894	D	0.080	13
199301	19931121:151500	893	E	0.102	10
200001	20001002:204000	874	F	0.124	8
200301	20031004:020500	791	G	0.146	7
199601	19960421:090000	786	H	0.168	6
199001	19900310:191500	769	I	0.190	5
200801	20081007:191000	761	J	0.212	5
200101	20010525:151500	715	K	0.235	4
196501	19651107:220000	700	L	0.257	4
199101	19910807:054500	670	M	0.279	4
198601	19860726:013000	669	N	0.301	3
200601	20061130:155500	633	O	0.323	3
197101	19710601:094405	627	P	0.345	3
199501	19950907:041500	623	Q	0.367	3
196701	19670202:234500	614	R	0.389	3
198001	19800122:224500	583	S	0.412	2
197901	19791014:170000	572	T	0.434	2
200901	20090212:220500	562	U	0.456	2
200701	20070318:152500	540	V	0.478	2
196601	19660426:220000	534	W	0.500	2
198801	19880608:090000	533	X	0.522	2
198101	19810925:010000	522	Y	0.544	2
200201	20020929:215000	515	Z	0.566	2
199901	19990516:221500	509	a	0.588	2
198401	19841216:204500	503	b	0.611	2
197701	19770628:183000	491	c	0.633	2
197601	19760430:130000	485	d	0.655	2
199401	19941116:143000	467	e	0.677	1
197301	19730916:214500	460	f	0.699	1
198901	19891015:153000	442	g	0.721	1
199201	19920810:183000	427	h	0.743	1
196801	19681207:061500	423	i	0.765	1
198201	19820225:003000	404	j	0.788	1
198501	19851202:114500	387	k	0.810	1
197501	19750829:090000	366	l	0.832	1
197401	19740703:090000	353	m	0.854	1
197201	19720517:122725	339	n	0.876	1
197001	19700917:143000	339	o	0.898	1
198301	19831108:143000	314	p	0.920	1
198701	19870122:144500	291	q	0.942	1

196901	19690423:214500	229	r	0.965	1
200501	20050928:151500	193	s	0.987	1
	Mean =	573			

End of process
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Reading data from C:\Program Files\NIWA\Tideda\Working\EVENTS.DAT
(Source file is C:\PROGRAM FILES\NIWA\TIDEDA\WORKING\3FLOWSITES.MTD)
Site 933356 Whanganui at 3 flow sites
From 1-Jan-1965 00:00:00 to 31-Dec-2009 24:00:00
GEV exponent 0.10741 has magnitude greater than 0.07541 and so GEV fits better than Gumbel

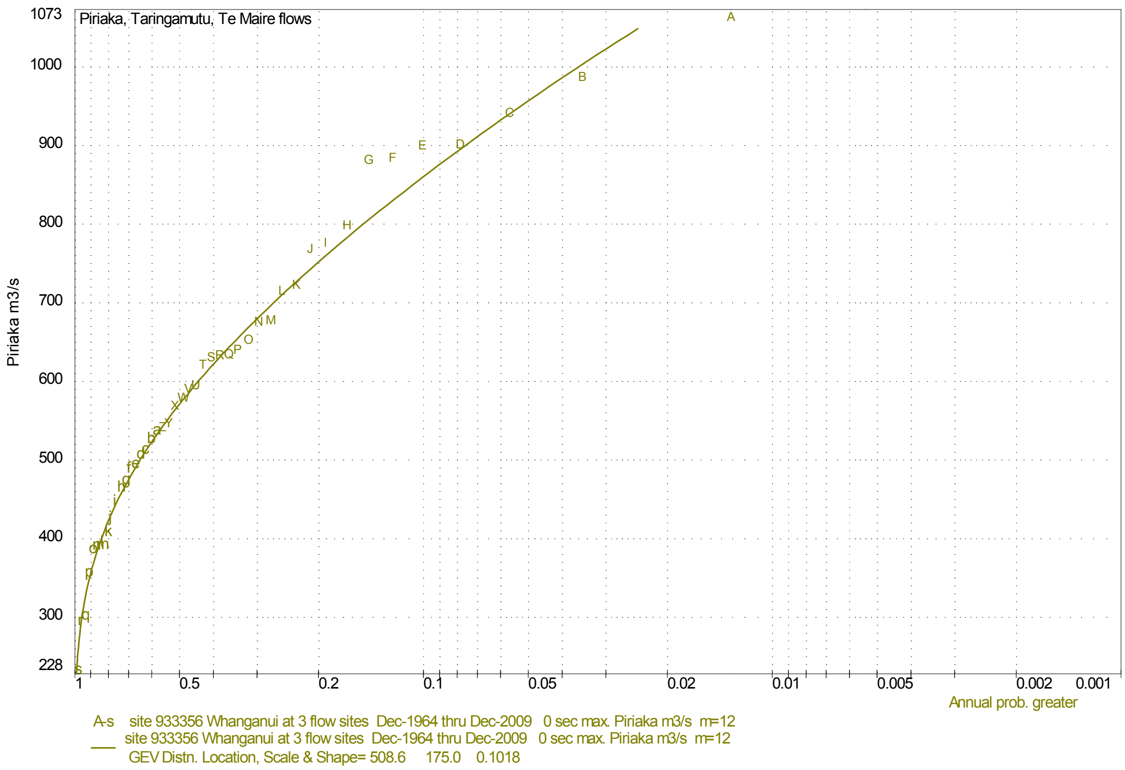
Moments L1= 573 L2= 117 T3= 0.103 T4= 0.117
L-moments estimates of distributions's parameters:
Location = 484 Scale = 185 Shape = 0 100yr/2.33yr = 1.966
Data selected from months January to December inclusive

12 mth partition starts yyyymm	Recorded value at yyyymmdd:hhmmss	maximum measured Piriak m3/s		-- GEV Piriak m3/s	1.96 std. dev.	Distribution -- ann. prob. 1/y	ret. per. y
				1385		0.001	1000
				1155		0.010	100
				1073		0.020	50
199801	19981028:183000	1056	A	1056		0.023	44
200401	20040229:031000	980	B	980		0.041	24
				954		0.050	20
199701	19970409:060000	934	C	934		0.058	17
197801	19781112:211500	894	D	894		0.076	13
199301	19931121:151500	893	E	893		0.077	13
200001	20001002:204000	874	F	874		0.087	11
				854		0.100	10
200301	20031004:020500	791	G	791		0.148	7
199601	19960421:090000	786	H	786		0.153	7
199001	19900310:191500	769	I	769		0.169	6
200801	20081007:191000	761	J	761		0.177	6
				740		0.200	5
200101	20010525:151500	715	K	715		0.230	4
196501	19651107:220000	700	L	700		0.249	4
199101	19910807:054500	670	M	670		0.291	3
198601	19860726:013000	669	N	669		0.293	3
200601	20061130:155500	633	O	633		0.349	3
197101	19710601:094405	627	P	627		0.359	3
199501	19950907:041500	623	Q	623		0.366	3
196701	19670202:234500	614	R	614		0.382	3
				587		0.430	2.33
198001	19800122:224500	583	S	583		0.437	2
197901	19791014:170000	572	T	572		0.459	2
200901	20090212:220500	562	U	562		0.478	2
200701	20070318:152500	540	V	540		0.521	2
196601	19660426:220000	534	W	534		0.532	2
198801	19880608:090000	533	X	533		0.534	2
198101	19810925:010000	522	Y	522		0.556	2
200201	20020929:215000	515	Z	515		0.569	2
199901	19990516:221500	509	a	509		0.581	2
198401	19841216:204500	503	b	503		0.595	2

197701	19770628:183000	491	c	491	0.619	2
197601	19760430:130000	485	d	485	0.631	2
199401	19941116:143000	467	e	467	0.666	2
197301	19730916:214500	460	f	460	0.680	1
198901	19891015:153000	442	g	442	0.714	1
199201	19920810:183000	427	h	427	0.742	1
196801	19681207:061500	423	i	423	0.749	1
198201	19820225:003000	404	j	404	0.782	1
198501	19851202:114500	387	k	387	0.810	1
197501	19750829:090000	366	l	366	0.843	1
197401	19740703:090000	353	m	353	0.862	1
197201	19720517:122725	339	n	339	0.880	1
197001	19700917:143000	339	o	339	0.881	1
198301	19831108:143000	314	p	314	0.909	1
198701	19870122:144500	291	q	291	0.932	1
196901	19690423:214500	229	r	229	0.973	1
200501	20050928:151500	193	s	193	0.986	1
	Mean =	573				

End of process

Appendix A-2: Whanganui @ Piriaka (site 33356): GEV analysis of annual peak flow rates since 1965.

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~~~ NIWA Tideda ~~~ DHI New Zealand                    19-MAR-2010 17:06
~~~ XFPLOT ~~~
Reading data from C:\Program Files\NIWA\Tideda\Working\EVENTS.DAT
(Source file is C:\PROGRAM FILES\NIWA\TIDEDA\WORKING\3FLOWSITES.MTD)
Site 933356 Whanganui at 3 flow sites
From  1-Jan-1965 00:00:00 to 31-Dec-2009 24:00:00
Data selected from months January   to December  inclusive

12 mth     Recorded     maximum   (Alpha=0.400)
Partition    Value     measured    ann.   ret.
starts        at        Piriak   Prob.  per
yyyymm yyyymmdd:hhmmss      m3/s      1/y     y
199801 19981028:183000    1056   A 0.013   75
200401 20040229:031000     980   B 0.035   28
199701 19970409:060000     934   C 0.058   17
197801 19781112:211500     894   D 0.080   13
199301 19931121:151500     893   E 0.102   10
199601 19961201:110000     877   F 0.124    8
200001 20001002:204000     874   G 0.146    7
200301 20031004:020500     791   H 0.168    6
199001 19900310:191500     769   I 0.190    5
200801 20081007:191000     761   J 0.212    5
200101 20010525:151500     715   K 0.235    4
196501 19650212:180000     708   L 0.257    4
199101 19910807:054500     670   M 0.279    4
198601 19860726:013000     669   N 0.301    3
199401 19941116:143000     646   O 0.323    3
200601 20061130:155500     633   P 0.345    3
```

```
197101 19710601:094405   627  Q 0.367   3
197201 19720305:084448   626  R 0.389   3
199501 19950907:041500   623  S 0.412   2
196701 19670202:234500   614  T 0.434   2
200201 20021226:075000   588  U 0.456   2
198001 19800122:224500   583  V 0.478   2
197901 19791014:170000   572  W 0.500   2
200901 20090212:220500   562  X 0.522   2
200701 20070318:152500   540  Y 0.544   2
196601 19660426:220000   534  Z 0.566   2
198801 19880608:090000   533  a 0.588   2
198101 19810925:010000   522  b 0.611   2
199901 19990516:221500   509  c 0.633   2
198401 19841216:204500   503  d 0.655   2
197701 19770628:183000   491  e 0.677   1
197601 19760430:130000   485  f 0.699   1
197301 19730111:180320   470  g 0.721   1
199201 19920118:004500   461  h 0.743   1
198901 19890122:104500   443  i 0.765   1
196801 19681207:061500   423  j 0.788   1
198201 19820225:003000   404  k 0.810   1
197501 19750607:071500   391  l 0.832   1
197001 19701214:214321   388  m 0.854   1
198501 19851202:114500   387  n 0.876   1
198301 19830403:131500   383  o 0.898   1
197401 19740703:090000   353  p 0.920   1
200501 20050106:074500   298  q 0.942   1
198701 19870122:144500   291  r 0.965   1
196901 19690423:214500   229  s 0.987   1
             Mean =   593
```

End of process

~~~ NIWA Tideda ~~~ DHI New Zealand 19-MAR-2010 17:07

~~~ FRED ~~~

Reading data from C:\Program Files\NIWA\Tideda\Working\EVENTS.DAT

(Source file is C:\PROGRAM FILES\NIWA\TIDEDA\WORKING\3FLOWSITES.MTD)

Site 933356 Whanganui at 3 flow sites

From 1-Jan-1965 00:00:00 to 31-Dec-2009 24:00:00

GEV exponent 0.10184 has magnitude greater than 0.07541 and so GEV fits better than Gumbel

Moments L1= 593 L2= 111 T3= 0.106 T4= 0.117

L-moments estimates of distributions'ss parameters:

Location = 509 Scale = 175 Shape = 0 100yr/2.33yr = 1.898

Data selected from months January to December inclusive

```
12 mth    Recorded      maximum      -- GEV   Distribution --
partition  value      measured          1.96   ann.  ret.
starts     at         Piriak     Piriak  std.   prob.  per.
yyyymm yyyymmdd:hhmmss    m3/s       m3/s    dev.   1/y    y
                                     1377          0.001 1000
                                     1151          0.010  100
                                     1072          0.020   50
199801 19981028:183000   1056  A     1056          0.023   44
200401 20040229:031000    980  B      980          0.042   24
                                      957          0.050   20
199701 19970409:060000    934  C      934          0.059   17
197801 19781112:211500    894  D      894          0.079   13
```

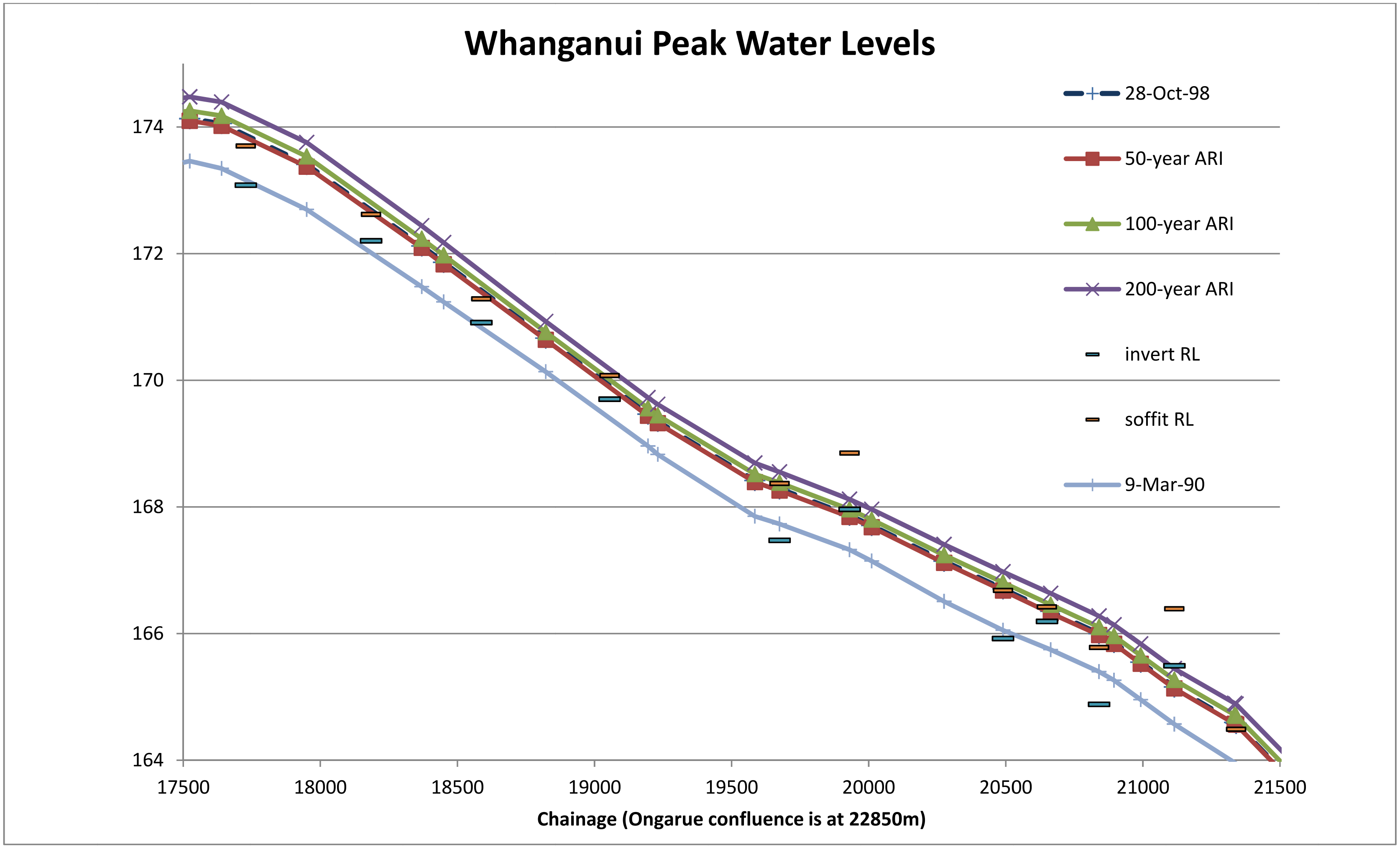
199301	19931121:151500	893	E	893	0.080	13
199601	19961201:110000	877	F	877	0.089	11
200001	20001002:204000	874	G	874	0.091	11
				861	0.100	10
200301	20031004:020500	791	H	791	0.157	6
199001	19900310:191500	769	I	769	0.181	6
200801	20081007:191000	761	J	761	0.189	5
				752	0.200	5
200101	20010525:151500	715	K	715	0.248	4
196501	19650212:180000	708	L	708	0.257	4
199101	19910807:054500	670	M	670	0.316	3
198601	19860726:013000	669	N	669	0.318	3
199401	19941116:143000	646	O	646	0.357	3
200601	20061130:155500	633	P	633	0.379	3
197101	19710601:094405	627	Q	627	0.391	3
197201	19720305:084448	626	R	626	0.394	3
199501	19950907:041500	623	S	623	0.398	3
196701	19670202:234500	614	T	614	0.416	2
				607	0.430	2.33
200201	20021226:075000	588	U	588	0.467	2
198001	19800122:224500	583	V	583	0.476	2
197901	19791014:170000	572	W	572	0.499	2
200901	20090212:220500	562	X	562	0.520	2
200701	20070318:152500	540	Y	540	0.566	2
196601	19660426:220000	534	Z	534	0.578	2
198801	19880608:090000	533	a	533	0.580	2
198101	19810925:010000	522	b	522	0.603	2
199901	19990516:221500	509	c	509	0.630	2
198401	19841216:204500	503	d	503	0.644	2
197701	19770628:183000	491	e	491	0.670	1
197601	19760430:130000	485	f	485	0.682	1
197301	19730111:180320	470	g	470	0.711	1
199201	19920118:004500	461	h	461	0.730	1
198901	19890122:104500	443	i	443	0.764	1
196801	19681207:061500	423	j	423	0.800	1
198201	19820225:003000	404	k	404	0.832	1
197501	19750607:071500	391	l	391	0.853	1
197001	19701214:214321	388	m	388	0.857	1
198501	19851202:114500	387	n	387	0.858	1
198301	19830403:131500	383	o	383	0.865	1
197401	19740703:090000	353	p	353	0.904	1
200501	20050106:074500	298	q	298	0.955	1
198701	19870122:144500	291	r	291	0.960	1
196901	19690423:214500	229	s	229	0.988	1
	Mean =	593				

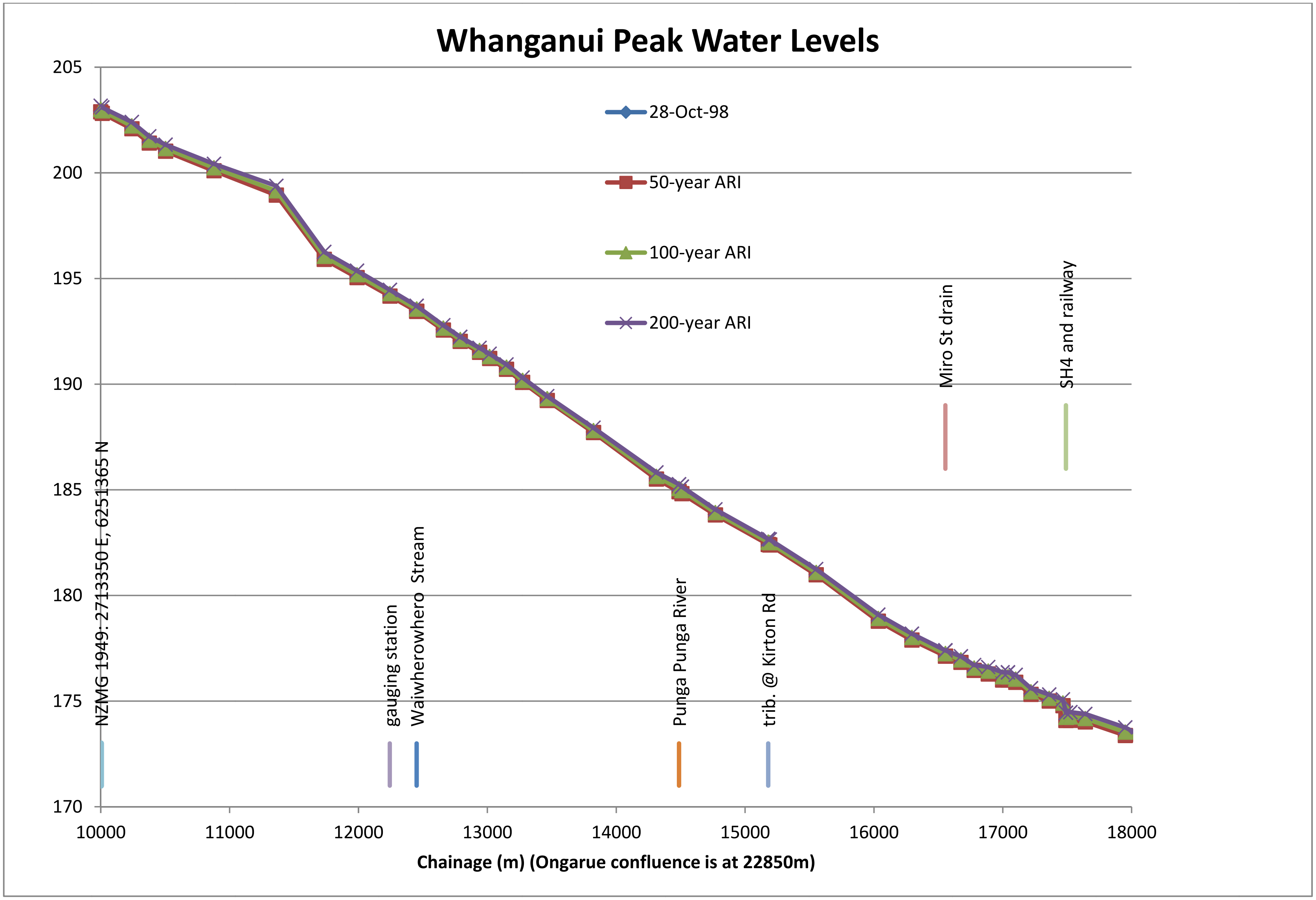
End of process

A P P E N D I X   B

Graphs of modelled peak water levels in the Whanganui River:

1. ***Adjacent to Taumarunui township***
2. ***From near Piriaka to Taumarunui***


Whanganui Peak Water Levels
28-Oct-98
50-year ARI
100-year ARI
200-year ARI
invert RL
soffit RL
9-Mar-90
174
172
170
168
166
164
17500
18000
18500
19000
19500
20000
20500
21000
21500
Chainage (Ongarue confluence is at 22850m)



Whanganui Peak Water Levels
205
200
195
190
185
180
175
170
10000
11000
12000
13000
14000
15000
16000
17000
18000
Chainage (m) (Ongarue confluence is at 22850m)
28-Oct-98
50-year ARI
100-year ARI
200-year ARI
NZMG 1949: 2713350 E, 6251365 N
gauging station
Waiwherowhero Stream
Punga Punga River
trib. @ Kirton Rd
Miro St drain
SH4 and railway