HYDRAULIC MODELLING OF LOWER RUAMAHANGA RIVER

Report prepared for
Wairarapa Division
Greater Wellington Regional Council
by

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1 Introduction

The Wairarapa Division of the Wellington Regional Council manages the Lower Wairarapa Valley Development Scheme, a large flood protection scheme covering the Ruamahanga River downstream of the Waiohine River, the Tauherenikau River downstream of the railway, Lakes Wairarapa and Onoke and all the eastern and western tributary streams (Figure 1).

Construction of the Scheme started in 1963 and was completed in 1983. The Scheme has assets of over $70 million, including 190 km of stopbanks, 112 culverts and floodgates and several pump stations. The total land area of benefit is 31500 hectares.

The Council is currently reviewing the Scheme. To assist with the review, the Council required that a model of the lower Ruamahanga River, including its connections to Lakes Wairarapa and Onoke, be developed.

Past hydraulic modelling has included manual calculations at the time of the Scheme design, and computer models of smaller areas of the river and floodway system developed in the 1990s. With advances in GIS capabilities, computer hardware and modelling software and techniques, and several further years of hydrological observations and cross-section changes, a revised model is warranted.

The main objectives of producing such a model are

- to determine the standard of protection (i.e. flood capacity) offered by the flood protection scheme in its present state
- to identify any measures that would optimise or further improve the effectiveness of the scheme
- refine the accuracy of the Waihenga rating site
- to update and refine flood hazard maps of the lower valley
- to assist in flood warning

The hydraulic modelling that is the subject of this report aims to specifically meet the first three objectives. Particular outputs include

- an improved rating for the Waihenga site
- an assessment of the scheme performance in relation to original design standards, including the impact of bed aggradation and berm/floodway siltation
- identification of where stopbanks will overtop in over-design events
- proposals for optimising the current system, (including the possibility of raising the standard of protection).

Future modelling work, outside the scope of this current exercise, would meet the last two objectives. Future work could also develop strategies for planned system failure in over-design flood or assess environment impacts of Barrage Gate operating regime options.

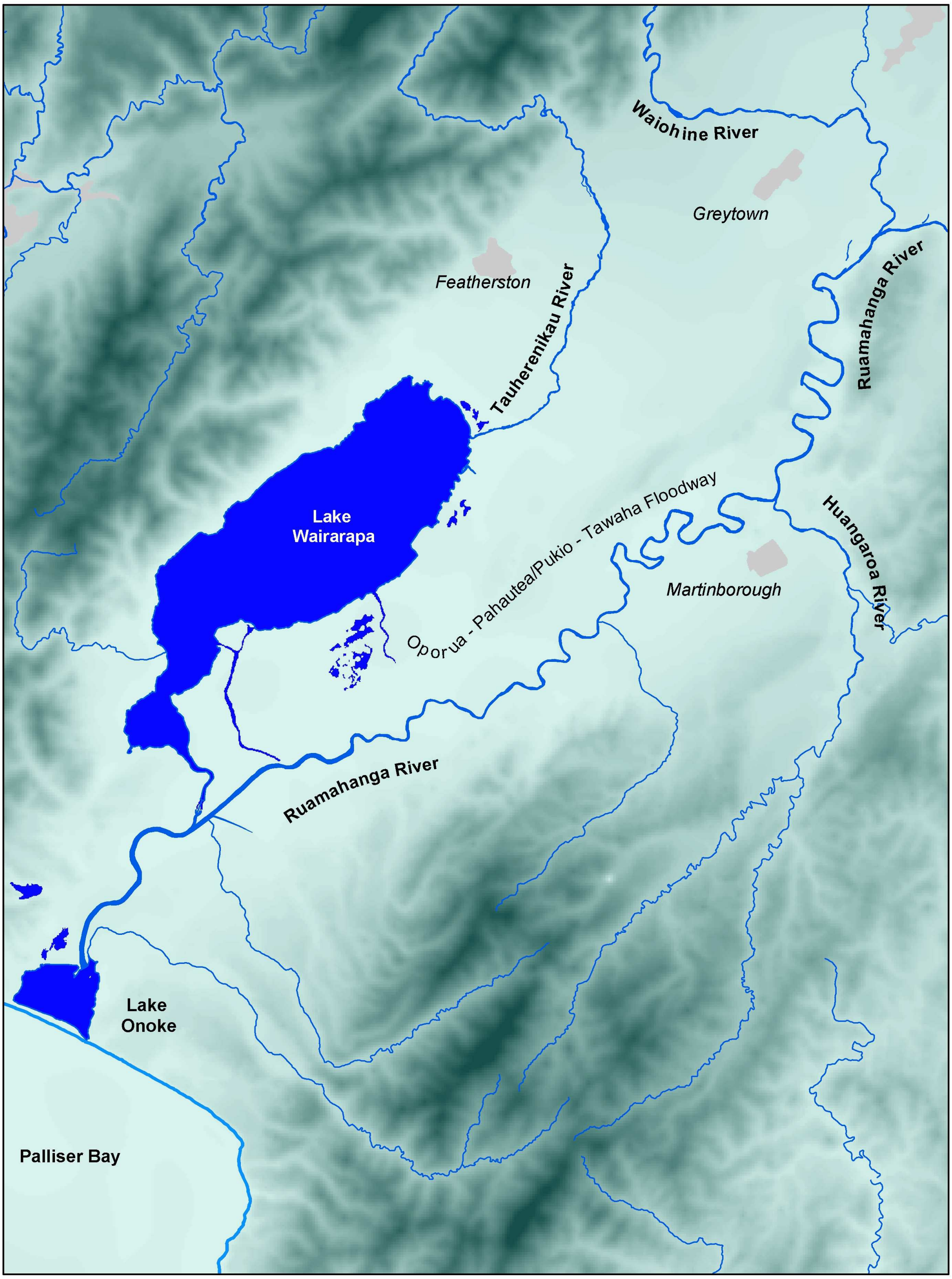

Figure 1 Model Study Area

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2 Extent of Model

The upstream extent of the model is the Waiohine River confluence with the Ruamahanga River. However, the prime area of interest is downstream of the Tawaha floodway (just upstream of Martinborough) and modelling effort has concentrated on that area.

The model includes the various floodways constructed as part of the Lower Wairarapa Valley Scheme, but does not include the floodplain beyond the stopbanks of the river and floodways.

The model also includes Lakes Wairarapa and Onoke and the barrage in the outlet of Lake Wairarapa. These are all important elements of the flood control scheme. The Tauherenikau River is represented by a point inflow into Lake Wairarapa.

Four versions of the model have been built. The first version has its downstream boundary at the Lake Onoke recorder, and is used for calibration and verification runs. This model therefore has known stage-time hydrographs as boundary conditions; no records of the sea level in Palliser Bay are available for calibration, and the Lake Onoke outlet geometry is not known with any precision for calibration events.

The other models extend downstream to Palliser Bay. The Lake Onoke mouth geometry varies depending on recent sea and river conditions, and three scenarios have been modelled: a short mouth, a long mouth and a blocked mouth (Figure 2).

Title: Figure 2 Lake Onoke Mouth Opening Scenarios

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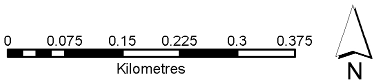


3 Model Software

MIKE 11, a software program developed by DHI, has been used as the principal modelling tool. It has had extensive use around the world, and has been widely used in New Zealand, including by the Wellington Regional Council, for 15 years.

MIKE 11 uses the Saint-Venant one-dimensional unsteady flow equations to model open channel flow. It allows branched and looped networks of channels to be modelled (and can also thus model flood plains flow in a quasi two-dimensional manner if required). An unsteady state model is considered essential for this exercise, due to the storage associated with the lakes and the floodways.

Weirs, culverts and other hydraulic structures can also be modelled with MIKE 11. Thus Kahutara Rd and Jenkins Dip culverts, the overflow sills to the floodways and the stopbanks can all be represented within the model. Likewise the bridges (Waihenga, Tuhitarata and Jenkins Dip) are included, using a culvert as a de facto bridge. Use of the MIKE 11 add-on “structures” module has allowed the barrage gates to be modelled with time-varying openings.

4 Model Layout

4.1 General

Prior to setting up the model, a site visit was made with Greater Wellington staff. Extensive use was then made of aerial photographs, flood photographs and videos, various cross-section and topographical plans in building the model. The council’s GIS proved invaluable in this process.

Figure 3 shows the full extent of the model, while Figures 4 to 7 show portions of the model in more detail. (Note that each of Figures 4 to 7 is at a different scale).

Figure 3 MIKE 11 Model Layout

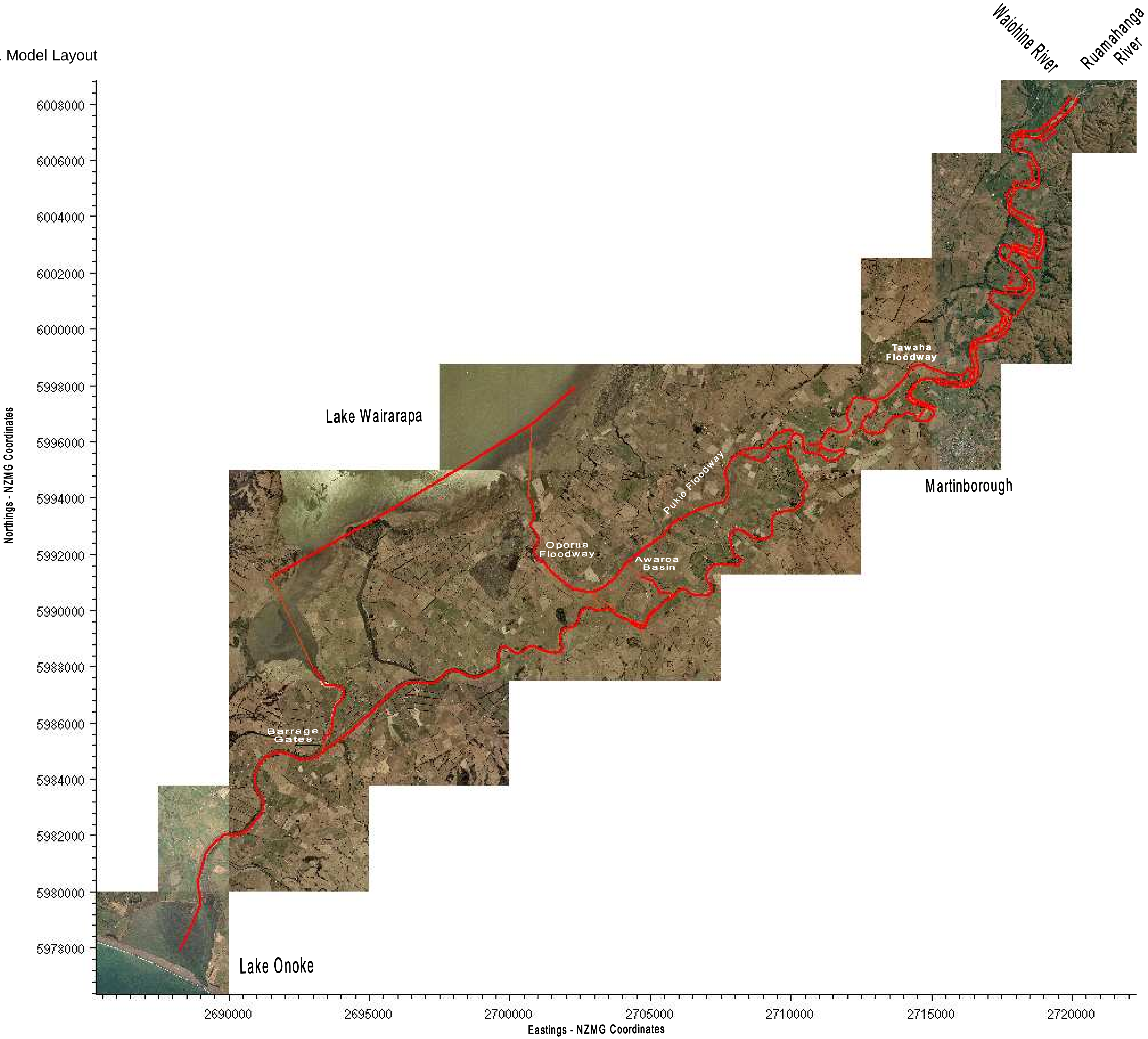


Figure 4 MIKE 11 Model Layout – Waiohine to Moiki

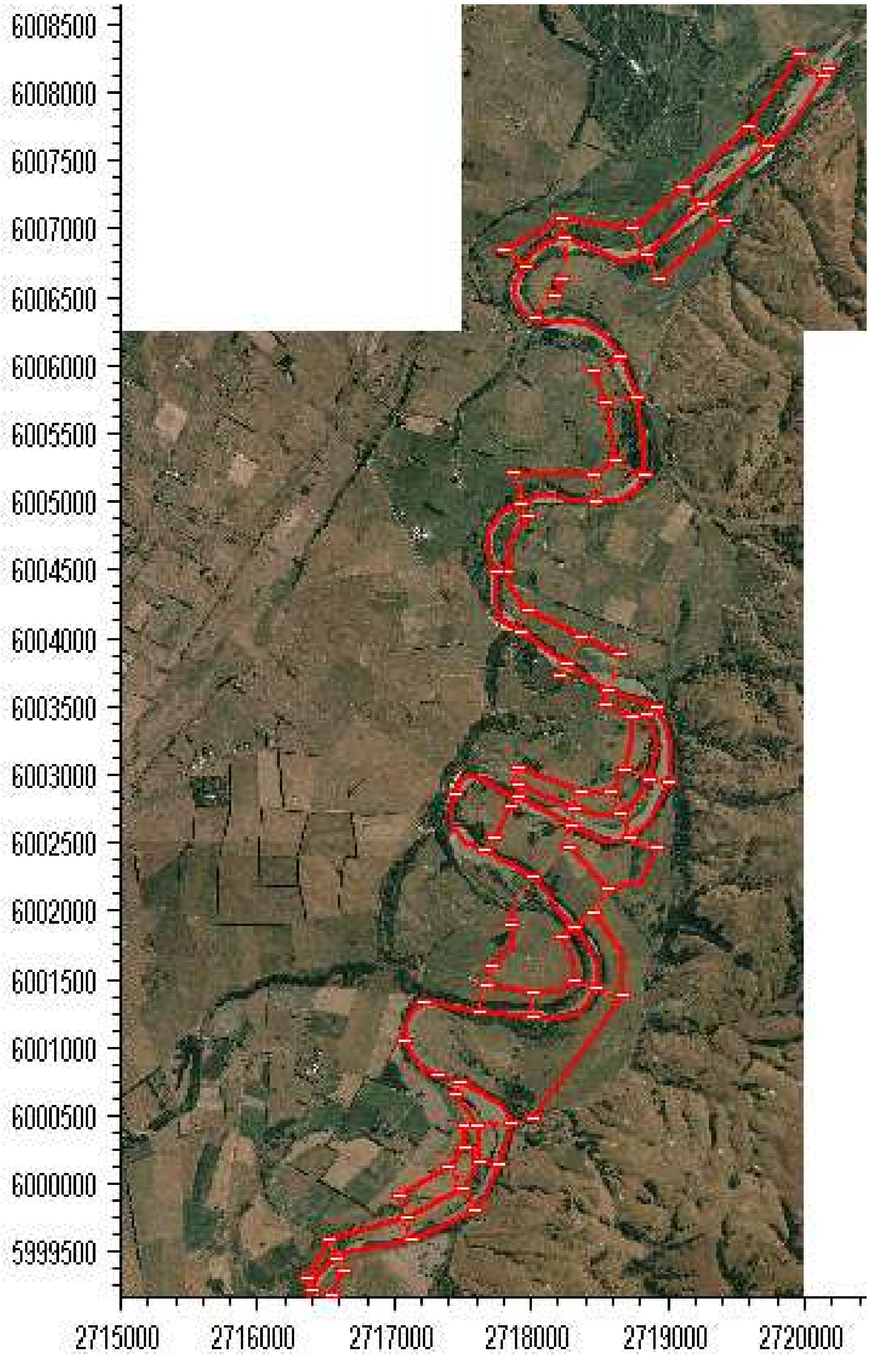


Figure 5   MIKE 11 Model Layout – Moiki to Hikinui

Figure 6   MIKE 11 Model Layout – Hikinui to Tuhitarata

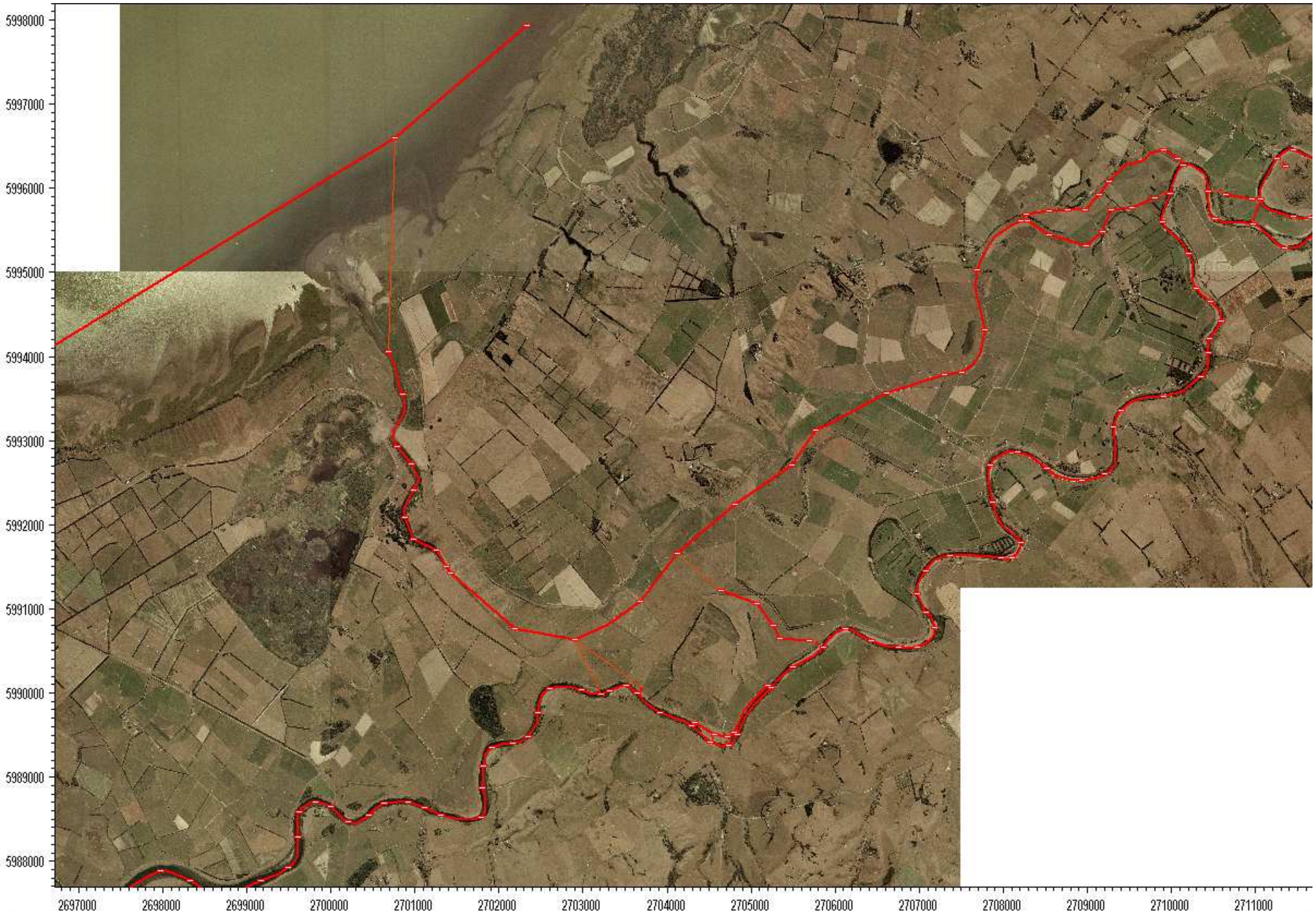
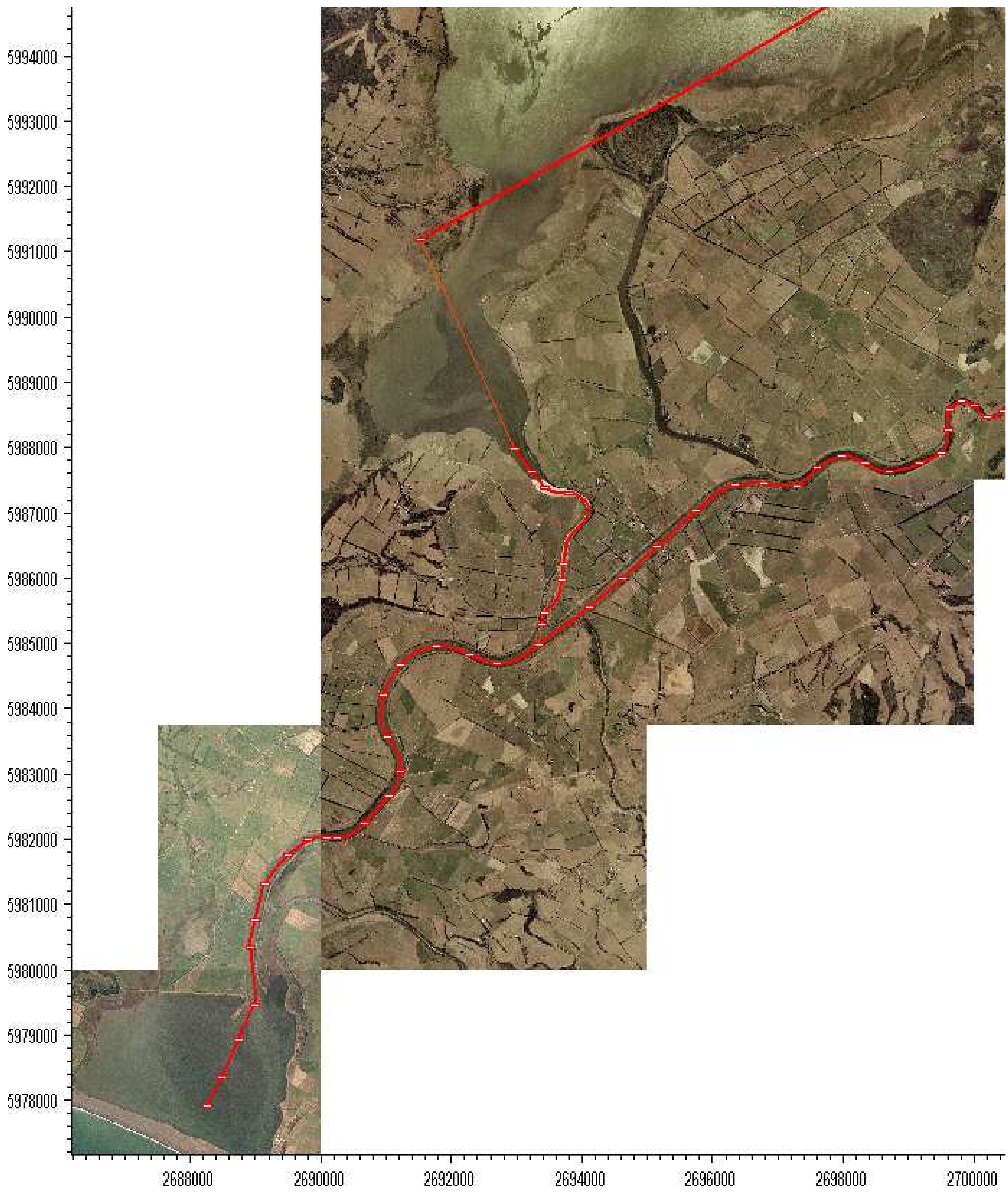


Figure 7 MIKE 11 Model Layout – Tuhitarata to Lake Onoke



4.2 Cross-section and Topographical Information

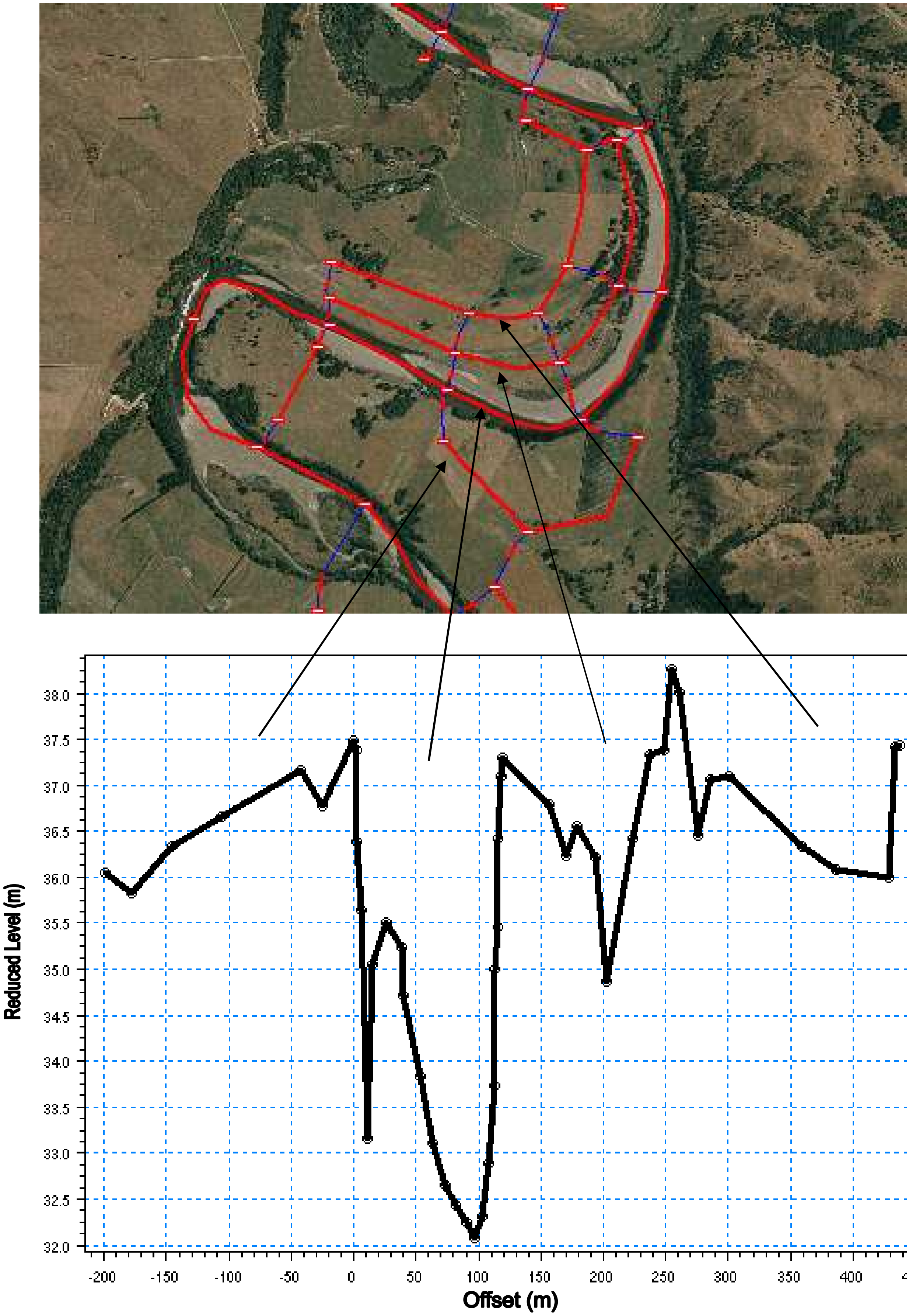
4.2.1 Ruamahanga River

Given the length of the Ruamahanga River being studied, it will never be practical to obtain a snapshot set of cross-sections, and so the cross-section data used will be from a range of dates. Appendix I shows the dates of cross-section surveys undertaken in the Ruamahanga River. The most recent survey was used for each section. In general, sections above the Moiki are from a 1999 survey, sections from Moiki to Tuhitarata are from a 1998 survey and those downstream are from a 1992 survey. As having such a spread of survey dates is less than ideal, results will need to be treated with some caution (and so an adequate freeboard for design levels will be required).

A range of miscellaneous survey information supplied by Greater Wellington staff, generally from over the last 20 years but from 1971 for some of the barrage channel sections, has been used for cross-sections in other parts of the model. Information held in the MIKE 11 cross-section file *wairarapa.xns11* identifies the information used.

Upon studying the river sections, it was apparent that a single channel formulation of the Ruamahanga would be inappropriate in many locations, particularly in the reach from the Waiohine to Waihenga. In some early trials it was found that separating the right berm from the main channel over the few hundred metres immediately downstream of the Waiohine confluence (sections 175-170) delayed the peak of a test flood by approximately 10 minutes a little further downstream. The cumulative effect of the berm flow further downstream was likely to be significant. While this reach was not the primary focus of the study, it was agreed that it should nonetheless be modelled reasonably realistically as doing so would have benefits for flood warning. Accordingly over much of the reach the river section was split into separate channel and berm branches. A similar split of river sections was made in a few locations between Waihenga and Awaroa. Figure 8 shows an example of this.

Figure 8 Example of Split of River Section into Main Channel and Berm Branches



4.2.2 Lake Wairarapa

Lake Wairarapa has been treated as a channel with a set of uniform cross-sections, with the width and shape specified so as to give the approximate volume of the lake. Early trials indicated that this was a better approach than attaching "additional storage" volumes (in the processed data tables of the MIKE 11 cross-sections) to a narrower channel. A 2-D model would be more realistic, but as in most flood situations the barrage gates will be shut, this MIKE 11 model is considered adequate.

4.2.3 Lake Onoke

Surveyed sections across Lake Onoke have been attached to the Ruamahanga branch of the model. Additional storage has been attached to section 171940 (see Appendix I for a list of cross-sections and their model chainages). In the calibration version of the model, this represents the remaining portion of the lake between that section (being the downstream end of the model) and the mouth. In the other model versions, this additional area has been halved, as the use of a further section downstream (173400) at the mouth will effectively provide approximately half of that area or volume by interpolation of these two cross-sections.

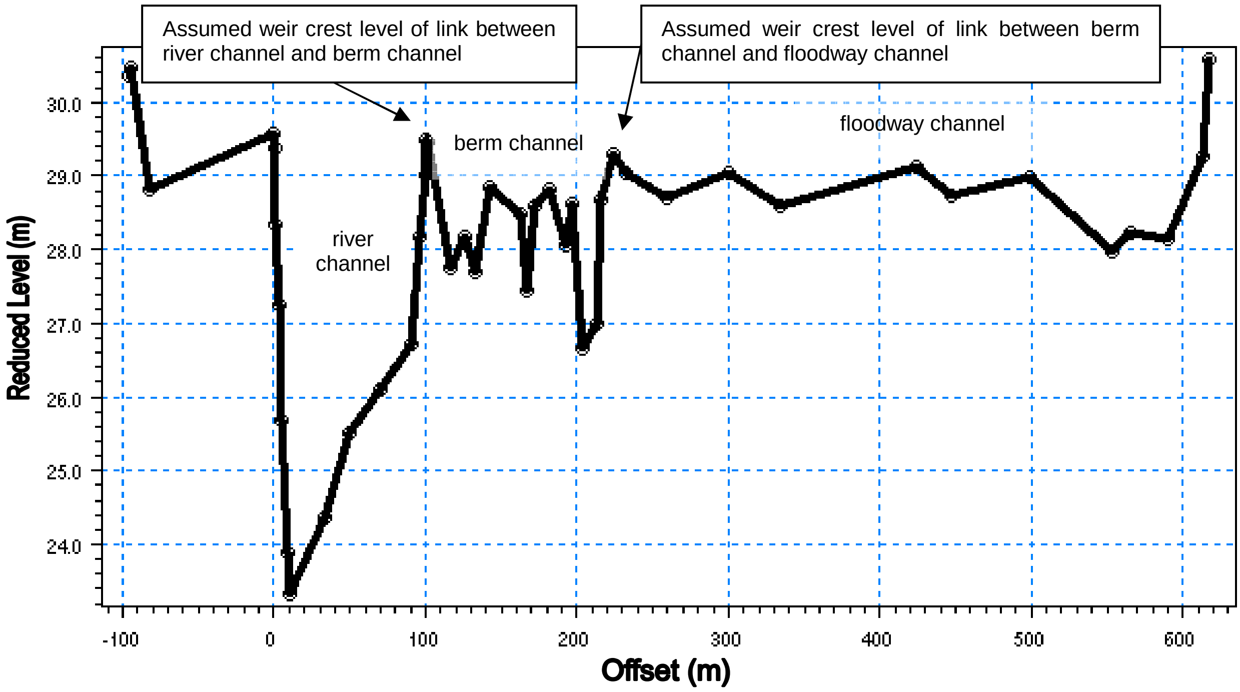
Subsequent trials for Lake Onoke were undertaken using a narrower channel (between 350m and 800m wide as suggested by aerial photographs and the cross-sections themselves) and additional storage volumes to each cross-section (extracted from a crude DTM based on cross-section data). This is perhaps a more realistic representation of the system. However, results were not significantly different to the original wide channel approach. Thus the original, simpler approach was adopted for the final runs.

The last three downstream sections of the capacity and design models are all artificial sections. The two through the mouth were produced after discussion with Greater Wellington staff. Mouth dimensions are quite variable, but a top width of 150m is considered realistic during a flood event. The expected depth would be 3-5m, the side slopes about the angle of repose for coarse sand and the bottom oval shaped. The downstream section of the model, in Palliser Bay, has been made 500m wide to represent an estimation of the width of the river plume just offshore. A shallower depth was also modelled in a low flow simulation (see Sections 6.6 and 9 of this report).

4.2.4 River – Floodway – Berm Connections

Connections between the river channel and berm channels or floodways have been treated as MIKE 11 "link channels". These treat the connection as a weir. With the exception of the Hikinui north and south sills, a simple geometry for all connections has been assumed. This geometry takes the level from the adjacent river cross-section that represents the boundary between the river and the berm or floodway channel and uses it as the minimum crest or low point of the "weir". (See Figure 9 for an example). The weir width has been measured from the council GIS, and generally goes to either midway to the next cross-section (where a further link is specified if required) or to what is judged to be the end of the length of overflow weir. These links are stored as an ArcView shape file ("sills&links-etc"), with the extent of the overflow weir shown.

Figure 9 Example of links between parallel channels (Ruamahanga Section 134)



In effect, this assumes that the level of the crest is constant over its length – i.e. it is at the level indicated by the cross-section. (However the weir crest has been given some small arbitrary "dish" shape so as to improve the computational stability of the model.)

The Hikinui sill geometry has been taken from a recent survey. Surveys of other sills that were available were considered too out of date to be representative of the actual geometry. With each flood, the sill levels have risen as silt is deposited on them. Comparison of river cross-sections with these older sill surveys confirmed that sills had risen.

Superelevation

As superelevation of the water surfaces occurs around bends, an adjustment of the crest level has been made at bends. A simple estimation of superelevation is

$$\Delta H = v^2 b/gr$$

where ΔH = the superelevation of the water surface, v = velocity, b = channel width, g = gravitational constant and r = radius of curvature of bend (Chow, 1959).

This formula gave some large superelevation figures (up to 600mm) and, as used by Webby et al (2002) in a recent study of the Waipaoa River, it was decided to use half the value from the formula.

Peak main channel velocities from earlier trials with the October 1992 flood simulation were used. This does mean that the effect of superelevation will be overestimated at less than peak flows (although there is not likely to be overtopping then anyway.) The crest level was lowered by the superelevation on the outside of bends (where water surface will be greater than the nominal output) and raised on the inside.

Note that the superelevation has only been taken into account in the predicted overflows to floodways and berms. Design water surface profiles and capacity results have not otherwise allowed for superelevation.

4.2.5 Ponding Areas

In a few locations, areas were identified where water was more likely to pond rather than flow. These areas were treated as sections or links with ”additional storage”. Topographical data for the areas was supplemented from GPS surveys undertaken over the past summer by council staff. The extent of these areas has been stored in the Arc shape file “storage_areas”.

4.3 Barrage Gate Structure Operation

The barrage is controlled by Council staff. For calibration simulations, the actual gate operation has been input into the model. Records of the actual gate operation were available for the 1992, 1998, 2000 and 2003 calibration events, while those for the 1994 event were inferred from records of water level either side of the gates (and assuming that all six gates were either open or shut at any one time in that flood). Nonetheless, the gate levels for that flood cannot be guaranteed and it is also understood that even for the other events the records may not be entirely reliable.

The gates take 15 minutes to shut or open, and are otherwise entirely shut or fully open.
For design simulations (Section 8 of this report), the gate operation has been set by defining general guidelines – e.g. to be a function of river and lake levels. After discussion with Council staff, a set of rules has been incorporated as best as practicable into the model for design runs. Figure 10 shows the rules for gates 1 and 6 (the outer gates) while Figure 11 shows the rules for gates 2 to 5.

The "priority" numbers shown in Figures 10 and 11 indicate each stage of the decision tree as described within the MIKE 11 network file.

During the rising limb of a flood the gates are closed, as shown in the first decision of the decision trees. In the model, a “flood” has been defined as the stage at Waihenga being above 26.5m RL[1], i.e 2.74m gauge height. (Note that in practice this level is lower than what would be considered as a flood but that other information, for example rainfall forecasts or upriver water levels, would be used to assess whether a flood was imminent and/or rising). The flood is defined as being on the rising limb if either it is rising at Waihenga (defined in the model as $Q_{120515} > Q_{120110}$, where the subscript refers to the MIKE 11 chainage) or if it is rising near Awaroa ($Q_{148475} > Q_{148785}$) (MIKE 11 chainages increase in a downstream direction). Defining a rising limb as occurring at Waihenga only leads to the gates being opened too early, while defining it at Awaroa leads to the gates being closed too late.

The level of Lake Onoke and the state of its mouth are important considerations in the gate operation. The gates may be opened, particularly when the mouth is blocked, if the level of Lake Onoke rises above approximately 11.3m (the exact level will depend on the wind). This allows backflow into Lake Wairarapa and reduces the flooding risk to the land surrounding Lake Onoke and the lower end of the river. For simplicity the model assumes that the gates will be opened if Lake Onoke rises above 11.3m provided that it is not during the rising limb of the flood, regardless of the state of the mouth.

[1] RL = Reduced Level. Unless otherwise noted, all levels in this report are in terms of the datum used by Greater Wellington for this Scheme, which defines mean high water spring tide as 10m.

Otherwise the gates are operated so as to keep the level of Lake Wairarapa as specified in the barrage gate resource consent requirements, as far as practical. The target levels vary according to the time of year, as indicated in Figures 10 and 11.

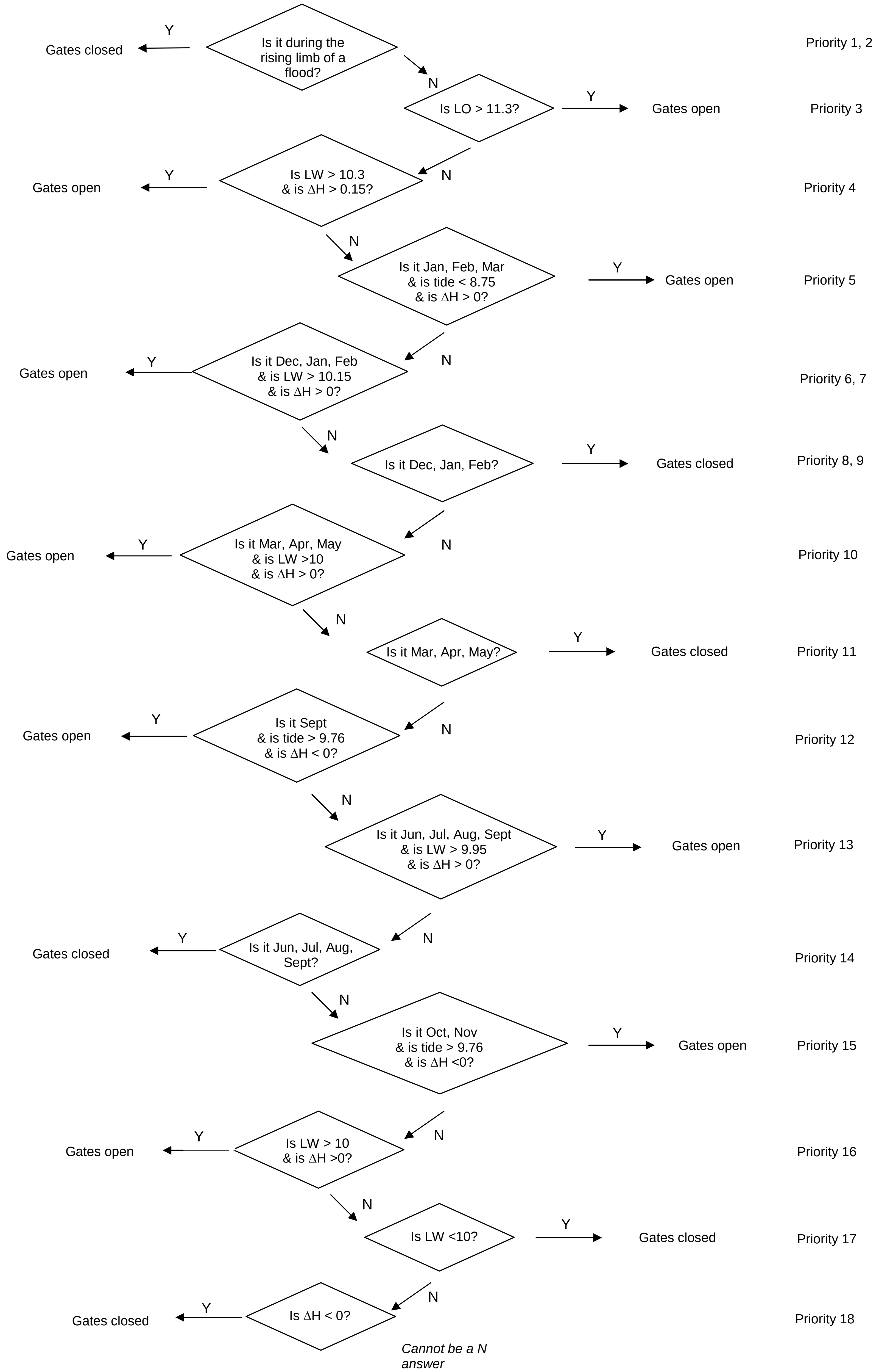
The resource consent requires Gates 1 and 6 to be opened so as to allow fish passage at defined times of year relating to the spawning cycle. These gates are to be opened one hour either side of high tide from September to November. As it is not possible to have a general rule in the model for this, it has been approximated by setting the gate to open if the downstream sea level rises above 0.46m tide datum, i.e 9.76m RL[2]. Inspection of the tide cycles for Palliser Bay over a sample few months, as predicted by NIWA, reveals that such a rule will allow the gates to open over almost all tide cycles for approximately the right length of time. Likewise, the requirement to have the gates open 1 hour either side of low tide from January to March can be approximated by setting the gates to open when the tide drops below -0.55m tide datum i.e 8.75m RL.

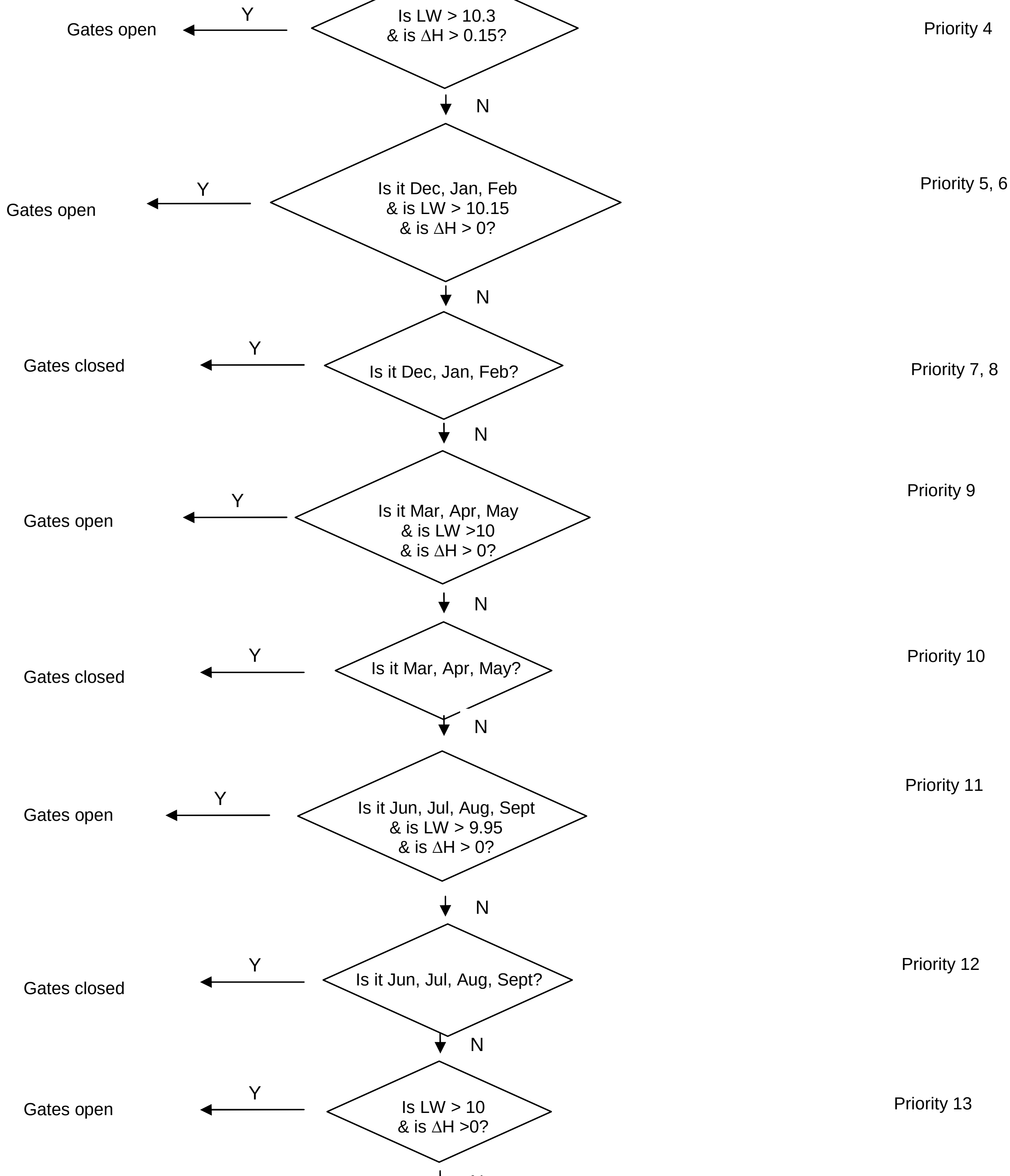
The following notes apply to Figures 10 and 11:

LW = level of Lake Wairarapa
LO = level of Lake Onoke
ΔH = water level northern side of barrage minus the water level on the southern side
Gate open/closed – i.e. gate is entirely open/closed

[2] Data from the NIWA tidal model (http://www.niwa.co.nz/services/tides) for the Palliser Bay area indicates that a typical spring tide level is approximately 0.7m above mean sea level (tide datum). The datum used by Greater Wellington for this Scheme defines mean high water spring tide as 10m. Thus mean sea level has been assumed to be 9.3m RL.

Figure 10 Barrage Gate Operation Rules – Gates 1 and 6





Is LW > 10.3
& is ΔH > 0.15?
Y
Gates open
N
Priority 4
Is it Dec, Jan, Feb
& is LW > 10.15
& is ΔH > 0?
Y
Gates open
N
Priority 5, 6
Is it Dec, Jan, Feb?
Y
Gates closed
N
Priority 7, 8
Is it Mar, Apr, May
& is LW >10
& is ΔH > 0?
Y
Gates open
N
Priority 9
Is it Mar, Apr, May?
Y
Gates closed
N
Priority 10
Is it Jun, Jul, Aug, Sept
& is LW > 9.95
& is ΔH > 0?
Y
Gates open
N
Priority 11
Is it Jun, Jul, Aug, Sept?
Y
Gates closed
N
Priority 12
Is LW > 10
& is ΔH >0?
Y
Gates open
Priority 13

5 Calibration and Clarification

5.1 Calibration Events and Data

Data from five periods of flooding, October 1992, November 1994, October 1998, October 2000 and June 2003, have been used to calibrate and (verify) the model. In each case, the periods in fact contained a series of floods, and simulations over approximately one month have been made.

The data consists of the following:

- October 1992
  - Peak flood levels along the Ruamahanga between Waihenga and Tuhitarara bridges
  - Peak flood levels in the Tawaha (Jenkins Dip) floodway.
  - Peak flood levels in the Pukio floodway.
  - Recorded water level hydrographs records at Waihenga Bridge, Lake Wairarapa (Burlings) and either side of the Barrage gates.
  - Recorded barrage gate openings

- November 1994
  - Recorded water level hydrographs records at Waihenga and Tuhitarata Bridges, Lake Wairarapa (Burlings), either side of the Barrage gates, Jenkins Dip, Oporua floodway and Awaroa sill.

- October 1998
  - Recorded water level hydrographs records at Waihenga Bridge, Lake Wairarapa (Burlings) and either side of the Barrage gates.
  - Recorded barrage gate openings

- October 2000
  - Peak flood levels along the Ruamahanga between Moiki and Waihenga.
  - Recorded water level hydrographs records at Waihenga Bridge, Lake Wairarapa (Burlings), either side of the Barrage gates, Jenkins Dip, Oporua floodway and Cross sill.
  - Recorded barrage gate openings

- June 2003
  - Peak flood levels along the Ruamahanga between sections 50 and 136.
  - Recorded water level hydrographs records at Waihenga Bridge, Lake Wairarapa (Burlings) and either side of the Barrage gates.
  - Recorded barrage gate openings

Use was also made of the flood photographs and, to a lesser extent, videos.

5.2 Calibration Assumptions

5.2.1 Inflows

Ruamahanga

A recorder is located at Waihenga Bridge, providing a valuable set of data for calibration. However, as the model needs to incorporate the Tawaha/Jenkins Dip floodway, a boundary condition is needed upstream of this. The Waihenga River gauge incorporates the Huangarua inflow but misses the flow bypassing via Jenkins Dip. The next upstream recorder is at Wardells, 45 km upstream - outside of the model area, beyond the available cross-sections and upstream of the major Waiohine inflow. Another upstream boundary condition must therefore be created.

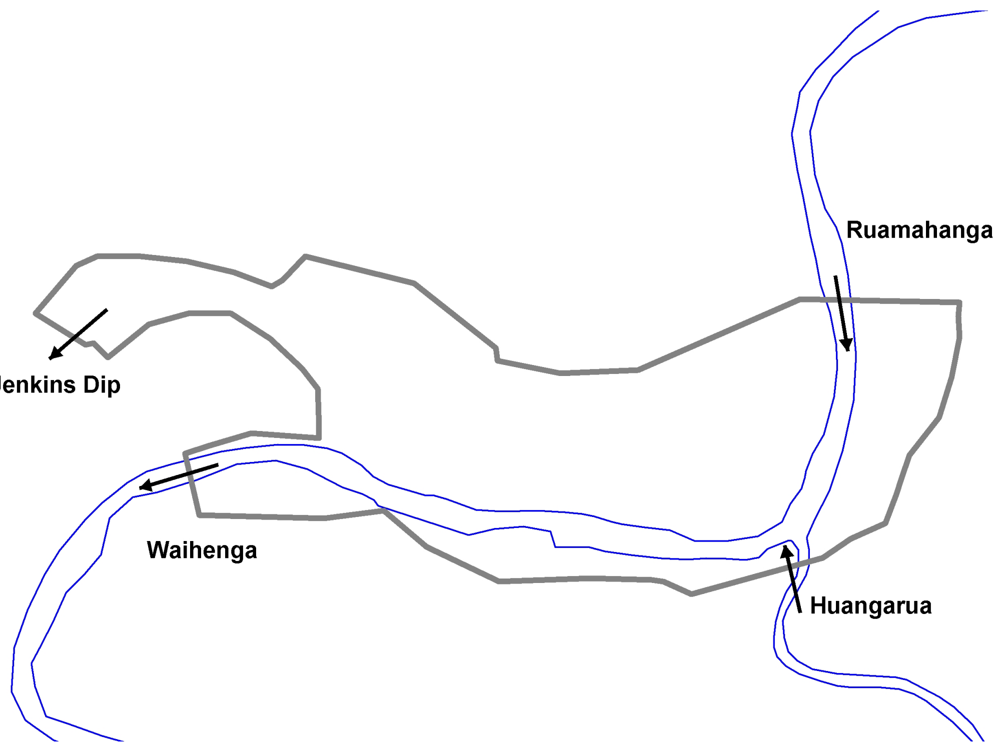
Consider Figure 12. From conservation of mass, and as the length of channel between the Huangarua inflow and the Waihenga bridge is small and no off-channel storage is present:

Ruamahanga flow + Huangarua flow = Waihenga flow + Jenkins Dip flow

or

Ruamahanga flow = Waihenga flow + Jenkins Dip flow - Huangarua flow.

Figure 12 Balance of Flows in Vicinity of Waihenga



The hydrograph “Waihenga flow + Jenkins Dip flow” has been estimated by Greater Wellington for each flood event, and although based on limited gaugings at Jenkins Dip it has been used in the calibration process here. An assumed Huangarua inflow hydrograph has been created (see below), and thus an estimate of the Ruamahanga inflow hydrograph upstream Jenkins Dip has been derived.

However, the upstream end of the model has been set at the Waiohine confluence, and this hydrograph has been further modified before being applied. It has been assumed that the extra inflow between Waiohine and Waihenga (excluding the Huangarua) is insignificant compared to the total flow in the Ruamahanga (the extra catchment area between Waiohine and Huangarua is approximately 25km^2, compared to a catchment of approximately 1970km^2 at the Waiohine). A earlier flood warning chart prepared by Greater Wellington indicates that the flood peak takes 2.5 hours to travel from Waiohine to Waihenga, so the hydrograph has been shifted back by 2.5 hours and applied at the upstream end of the model.

Huangarua and Waitawatautau

The Huangarua has a recorder at Hautotara, leaving approximately half of the catchment ungauged. Furthermore, no records are available from the 1992 flood event.

For the 1992 flood, the Huangarua (plus Waitawatautau) inflow at the Ruamahanga was derived from the Waihenga hydrograph and the assumption that $Q \alpha A^{3/4}$, where Q = flow and A = catchment area. Thus

$$Q_H = Q_W(A_H/A_W)^{0.75} = 0.237Q_W$$

Q_H = inflow from Huangarua and Waitawatautau at Ruamahanga
Q_W = total flow at Waihenga Bridge and Jenkins Dip
A_H = area of Huangarua and Waitawatautau catchment
A_W = area of Ruamahanga catchment at Waihenga

As the Waihenga plus Jenkins Dip flows were correspondingly reduced by this amount, in this case model results will not be particularly sensitive to the assumptions regarding the Huangarua inflow. (Note that this flood was due to a north-west storm, whereas the Huangarua flows tend to be higher in southerly storms).

In the case of the other calibration and verification floods, inflow from the Huangarua/Waitawatautau catchment has been derived from the Hautotara recorder, so that

$$Q_H = Q_{Hautotara}(A_H/A_{Hautotara})^{0.75}$$

These estimates should be more reliable than the 1992 estimate, being based on data from within the catchment rather than from a separate catchment.

Tauherenikau

Flow records are available at a site as the river enters the Wairarapa Plains, covering 112km^2 of the 154km^2 catchment. In addition, there are a number of streams on the western side of Lake Wairarapa, so that the total contributing area of these streams and the Tauherenikau is approximately 360km^2.

Using the relationship $Q \alpha A^{3/4}$ as before would imply that the Tauherenikau flows could be scaled by a factor of 2.4. However, this appeared to raise Lake Wairarapa levels in the model too much. Furthermore the recorder would catch the most intense rain in the Tararua Ranges, whereas the remainder of the catchment is in the foothills or plains where rain would be less intense. After discussions with Greater Wellington staff, it was decided to scale the Tauherenikau recorded flows by a factor of 1.5.

Evaporation on the lake surface was not incorporated into the model, as it is not significant in such a flood model.

Turanganui, Tauanui and Whangaehu/Paharakeke Rivers

Initially, estimates of inflows from these rivers were made from the relationship Q α $A^{3/4}$ and from the Waihenga flows (for the 1992 event) or the Hautotara flows. However, some early difficulties in calibrating the lower end of the Ruamahanga for the 1992 flood were experienced – levels tended were higher than observed – and it was decided not to include these flows to help lower river levels. In any case, the calculated inflows were not high. For consistency, the same was done for the other calibration floods.

5.2.2 Downstream Boundary Conditions

Since the Onoke outlet geometry in flood events cannot be known in detail and as there are no sea level records in Palliser Bay, sea level could not be used as the downstream boundary for the model. Instead, Lake Onoke level records have been used as the calibration event boundary conditions. The Lake Onoke levels are not however independent of river inflows, and in the calibration runs the water level predictions immediately upstream of the lake may be affected.

No allowance has been made for waves spilling over the bar and so adding to the volume of water in Lake Onoke.

5.2.3 Mannings n

Within each channel, i.e. in the main river (Ruamahanga branch) or in a berm/floodway channel, a uniform Mannings n across a section has been assumed. This simplification has been made because:

- With the large number of sections and observation points, the number of permutations of resistance profiles across a section and down the river becomes too large.
- Water levels tend to be less sensitive to variations across a section than to the overall section Mannings n.
- The way Mike 11 treats relative resistance across a section is not always as expected if the "hydraulic radius" formulation of MIKE 11 is used. Increasing the relative resistance of part of a section may not necessarily result in a decreased section conveyance as calculated by MIKE 11.

However, variation across a section could be justified for future detailed investigations over short reaches. At the scale of this current exercise, it is not justified.

5.3 Results and Discussion

5.3.1 October 1992

(Refer Figures 13-20 and Oct92-calibration.xls)

This flood was approximately a 20 year return event and was the result of a northwest storm. It has been regarded as the primary calibration event, as it has the most extensive set of recorded data of all the flood events simulated.

As with other calibration simulations, water levels at Waihenga are overpredicted at high stages. As the bed in the vicinity of the bridge has been aggrading, and as the section used in the model are from a later date (1998), one would expect the model to overpredict in this event.

Peak water levels are reproduced to within an average of 140mm, an acceptable result for such modelling (Figure 13). Specific areas where there is a greater discrepancy are

- around Waihenga bridge (see above),
- the reach between the Tawaha floodway exit and the Hikinui sills where the flow patterns are complex and a 1-D model could be expected to struggle and where ground topography is limited, and
- immediately upstream of the Dry River inflow. No inflow was modelled at this point, but it is possible that the actual inflow would have raised water levels.

Some attempt was made to determine whether these discrepancies could be due to superelevation; it was concluded that this was not the case, as the inconsistencies tended to be over reaches with both inner and outer bends (Figure 14). Nonetheless, it would be possible to refine the plot of observed and model peak water levels.

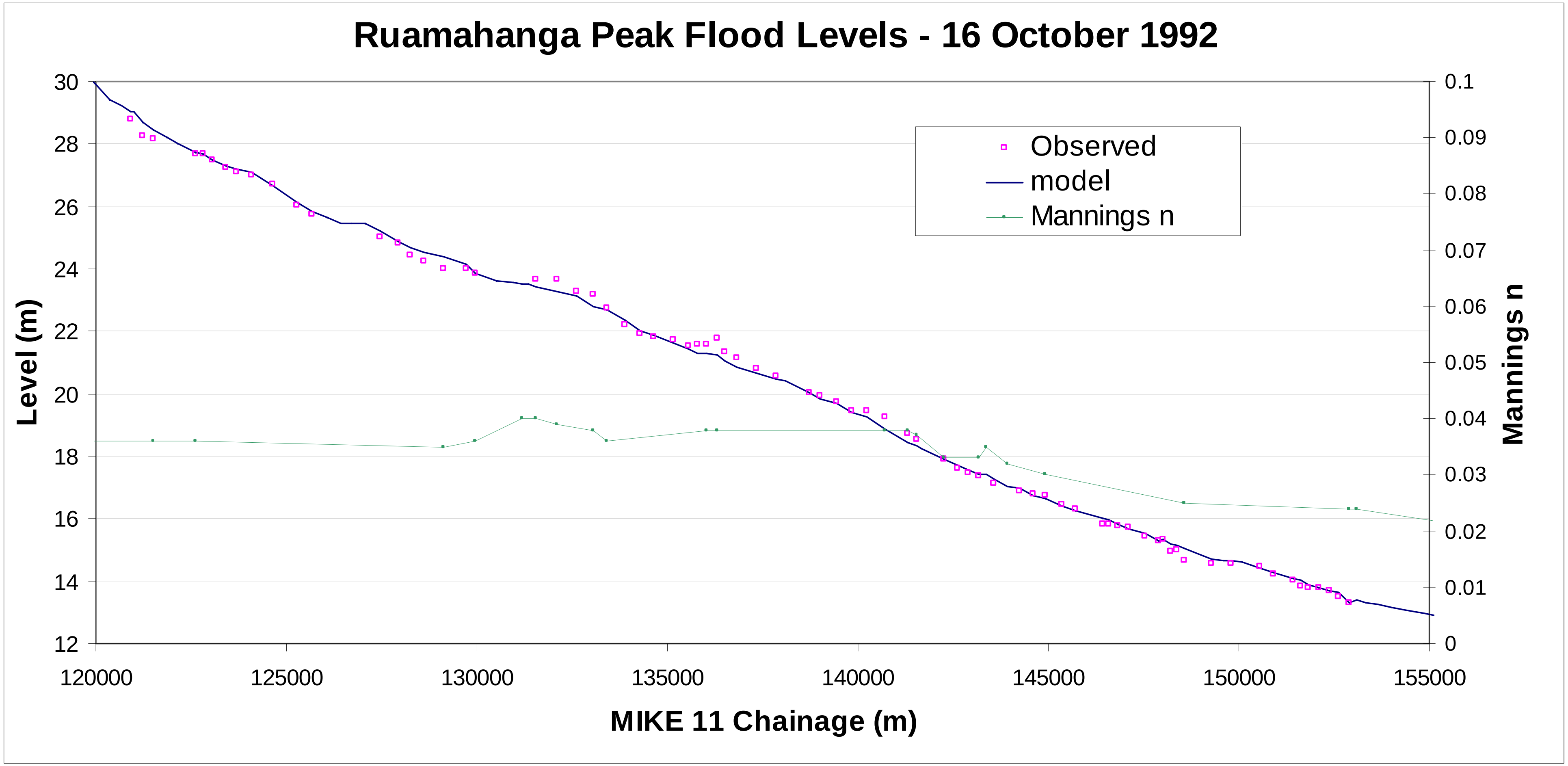
Note also, as discussed further in section 5.3.4, that it is possible that the location of the observed data may better correspond with model results in berm channels rather than the river channel. As no more detailed information on the location of the observations was available, this has not been investigated.

Figure 13 also plots the main channel Mannings n profile. Further adjustment of Mannings n could reduce the discrepancies between the predicted and observed levels, although as noted above, overflows tend to reduce the sensitivity of water level to Mannings n. Such adjustment however would require greater variation in Mannings n down the channel. There is no apparent obvious physical reason for doing so, and so the Mannings n profile has been left reasonably smooth with a general trend for n to decrease downstream (as bed particle size and river sinuosity decrease).

Further peak level observations in the lower reaches of the river would have been useful, particularly in light of the fact that predictions either side of the Barrage gates do not match observations while the gates are shut.

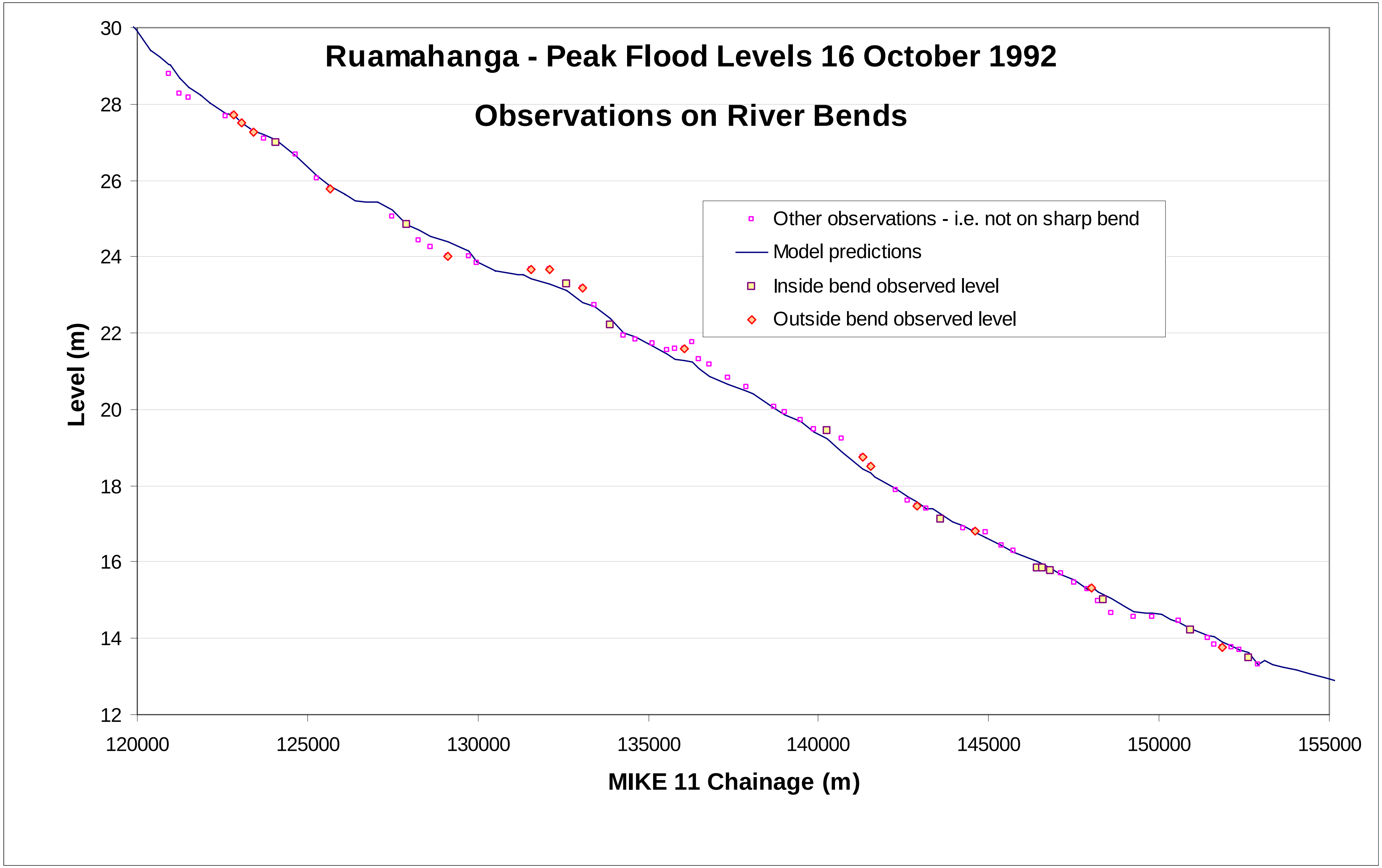
Lake Wairarapa levels are underpredicted during the peak of the event (Figure 15). It is unclear whether this is due to model inadequacies, the assumptions made regarding tributary inflows or the effect of wind waves on recorded levels.

Figure 13



Ruamahanga Peak Flood Levels - 16 October 1992
Observed
model
Mannings n
Level (m)
Mannings n
MIKE 11 Chainage (m)
30
28
26
24
22
20
18
16
14
12
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
120000
125000
130000
135000
140000
145000
150000
155000

Figure 14



Ruamahanga - Peak Flood Levels 16 October 1992
Observations on River Bends
Other observations - i.e. not on sharp bend
Model predictions
Inside bend observed level
Outside bend observed level
Level (m)
30
28
26
24
22
20
18
16
14
12
120000
125000
130000
135000
140000
145000
150000
155000
MIKE 11 Chainage (m)

Figure 15

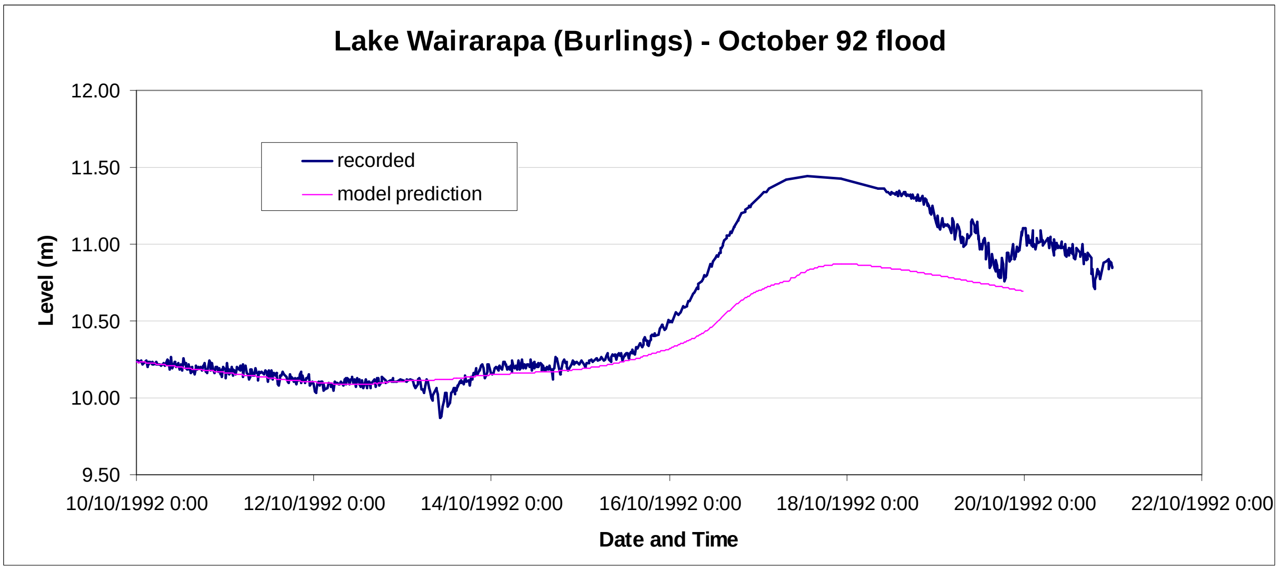


Figure 16

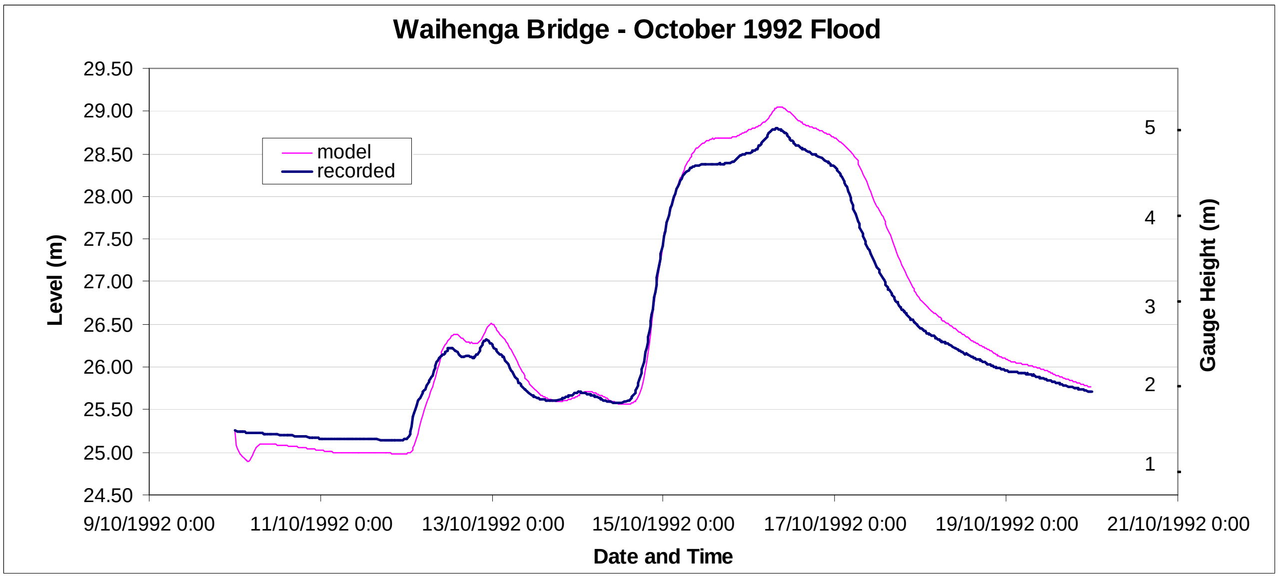


Figure 17

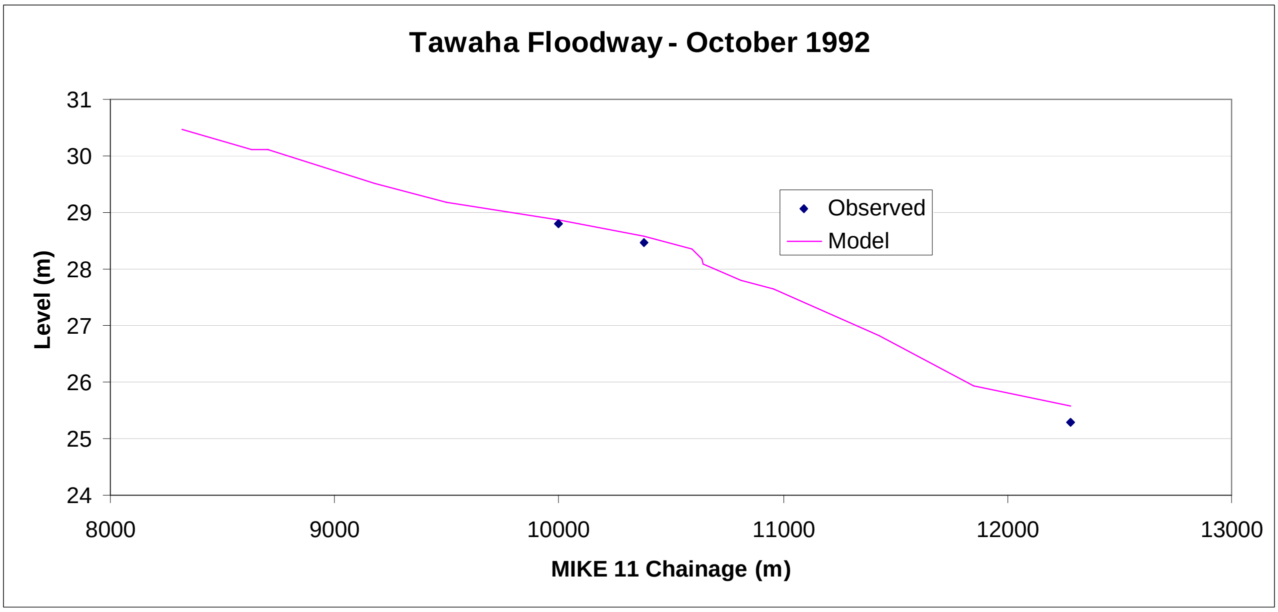


Figure 18

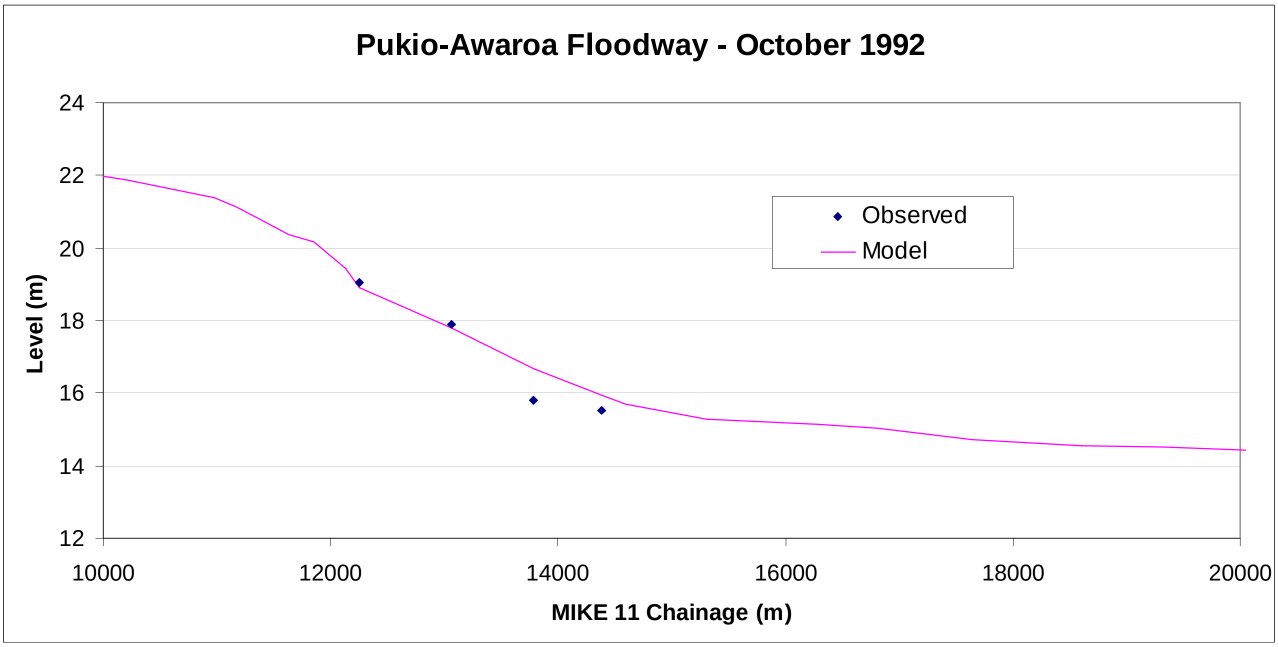


Figure 19

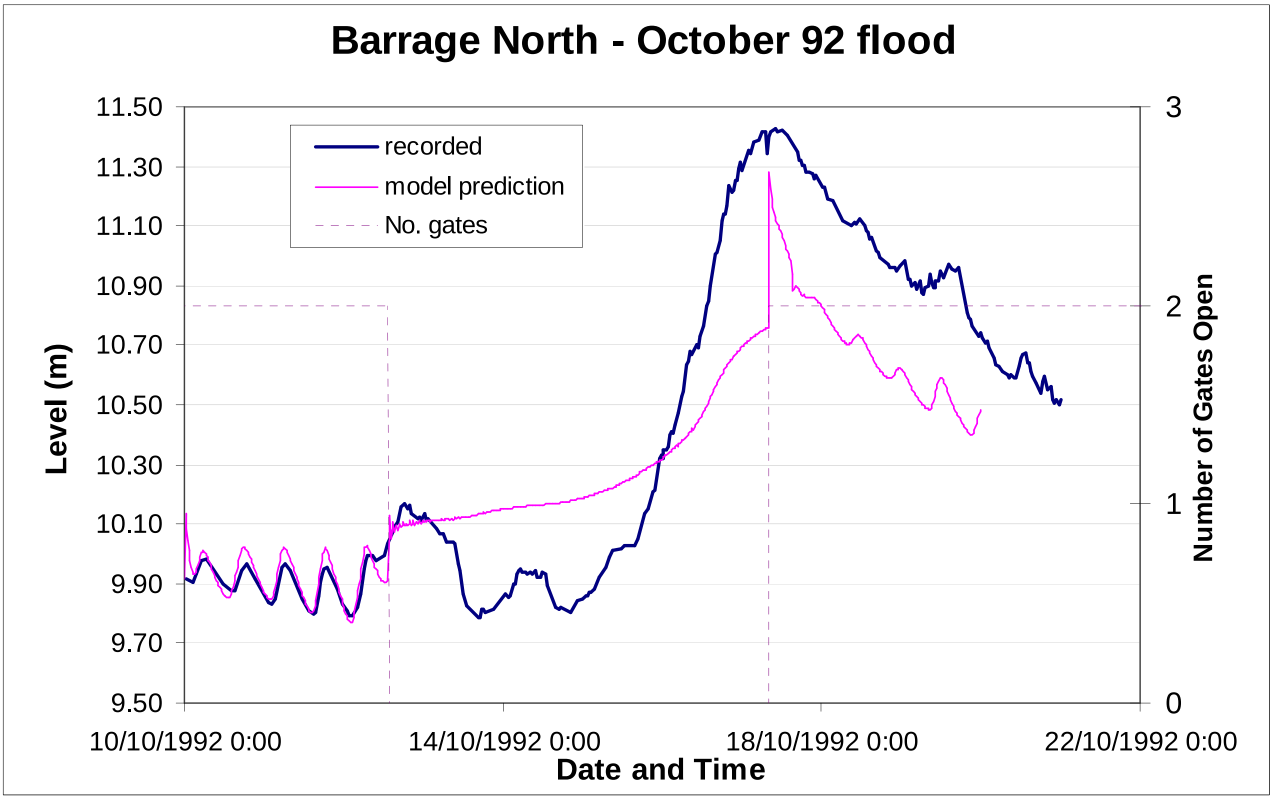
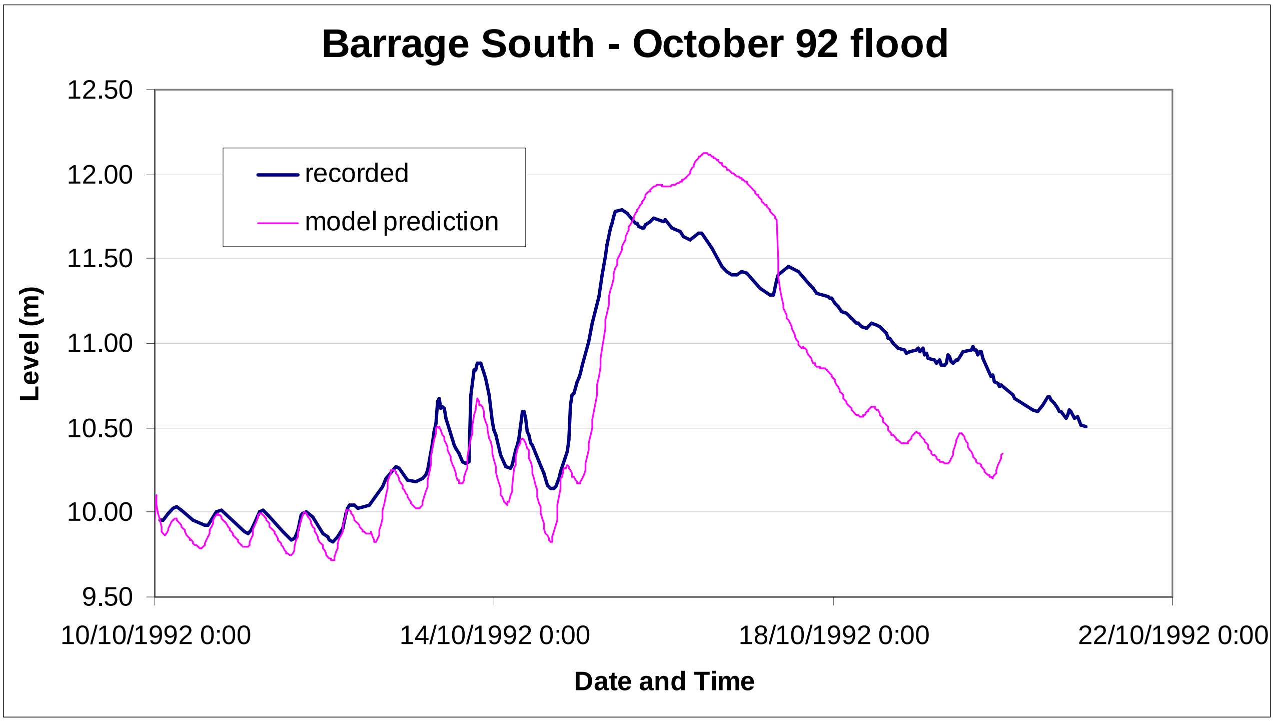


Figure 20



5.3.2 November 1994

(Refer to Figures 21-28 and Nov94-calibration.xls)

This flood was the largest of the calibration floods, being an approximately “50 year” event.

Results at Tuhitarata Bridge, Awaroa Sill and Oporua floodway are acceptable. Water level predictions at Jenkins Dip are less than recorded, while those at Waihenga Bridge are slightly more than the recorded levels on the recession of each flood peak. This could indicate that more water should be drawn off at the Tawaha Sill and into the Jenkins Dip floodway in this event, that the Huangaroa inflow was less than assumed or that there is more attenuation of floods between Waiohine and Waihenga than predicted.

The simulation shows some instabilities in the lower reaches of the Ruamahanga and in Lake Onoke. These are not present in the other flood simulations. Closer inspection of results shows that these instabilities are caused by peculiarities of the downstream boundary condition i.e. the Lake Onoke recorded water level time series. For instance, the recorder suggests that the lake level rose by nearly 500mm in 20 minutes on 22 November 1994 – far more than would be due to river or tide inflow.

No records of the barrage gate openings were available for this flood event, and openings had to be inferred from records of the water level either side of the gates. It was also assumed that all six gates were either open or shut at any one time.

Accordingly, results in the lower end of this simulation may not be very reliable.

Figure 21

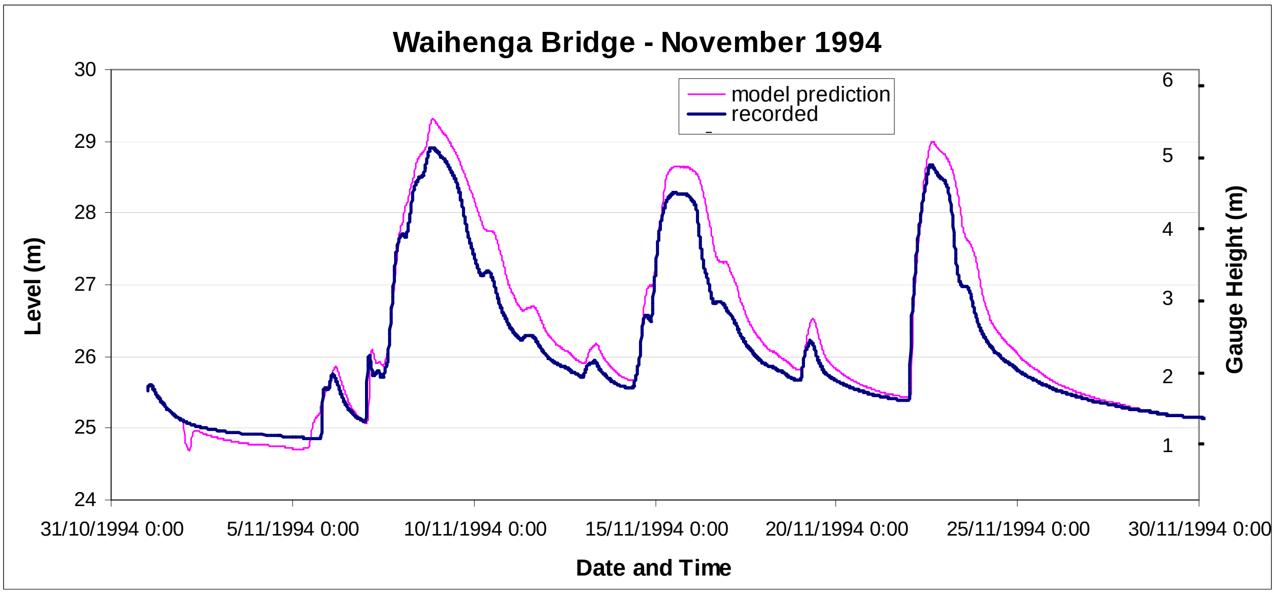


Figure 22

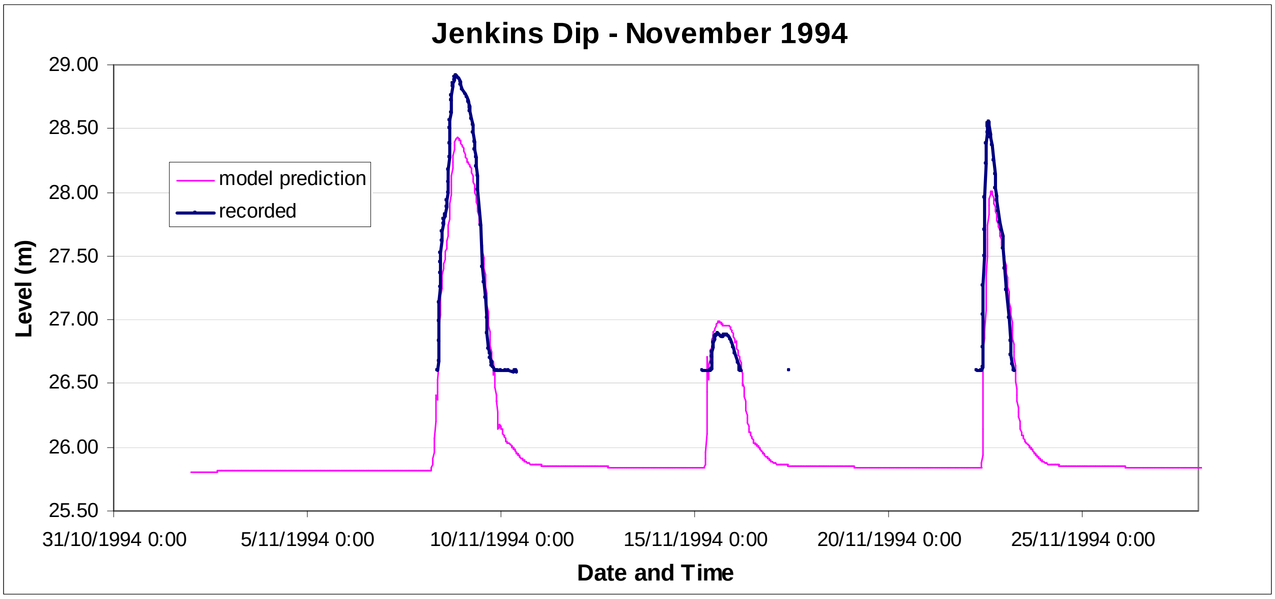


Figure 23

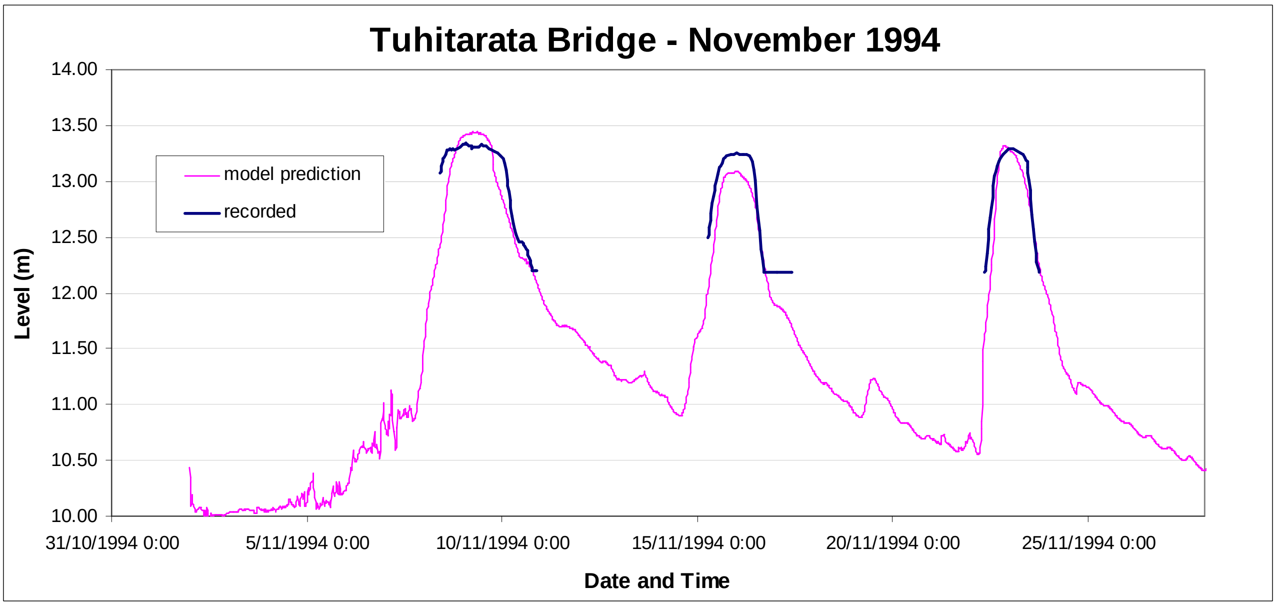


Figure 24

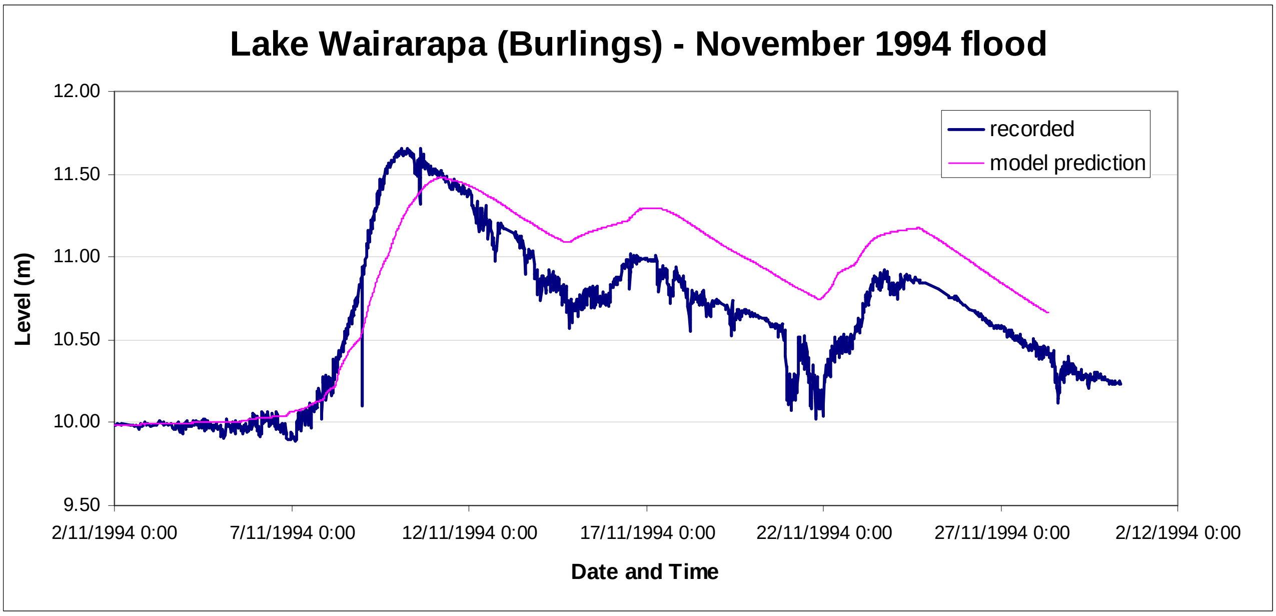


Figure 25

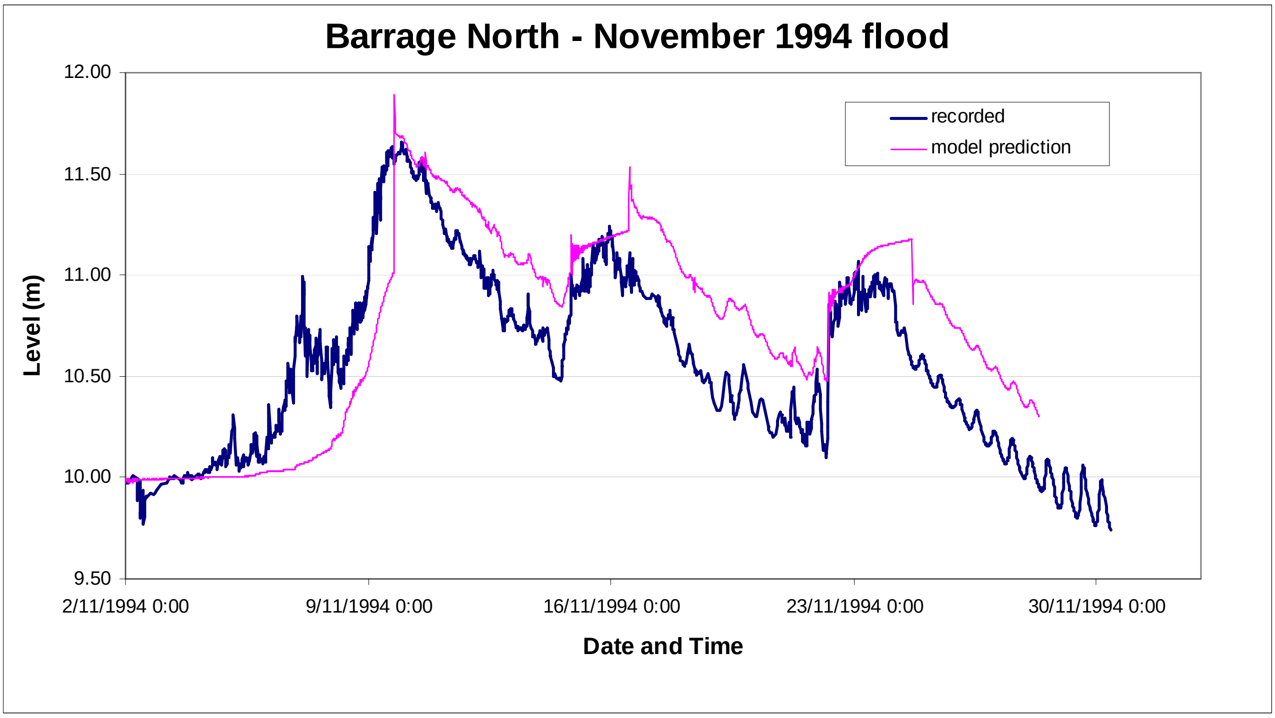


Figure 26

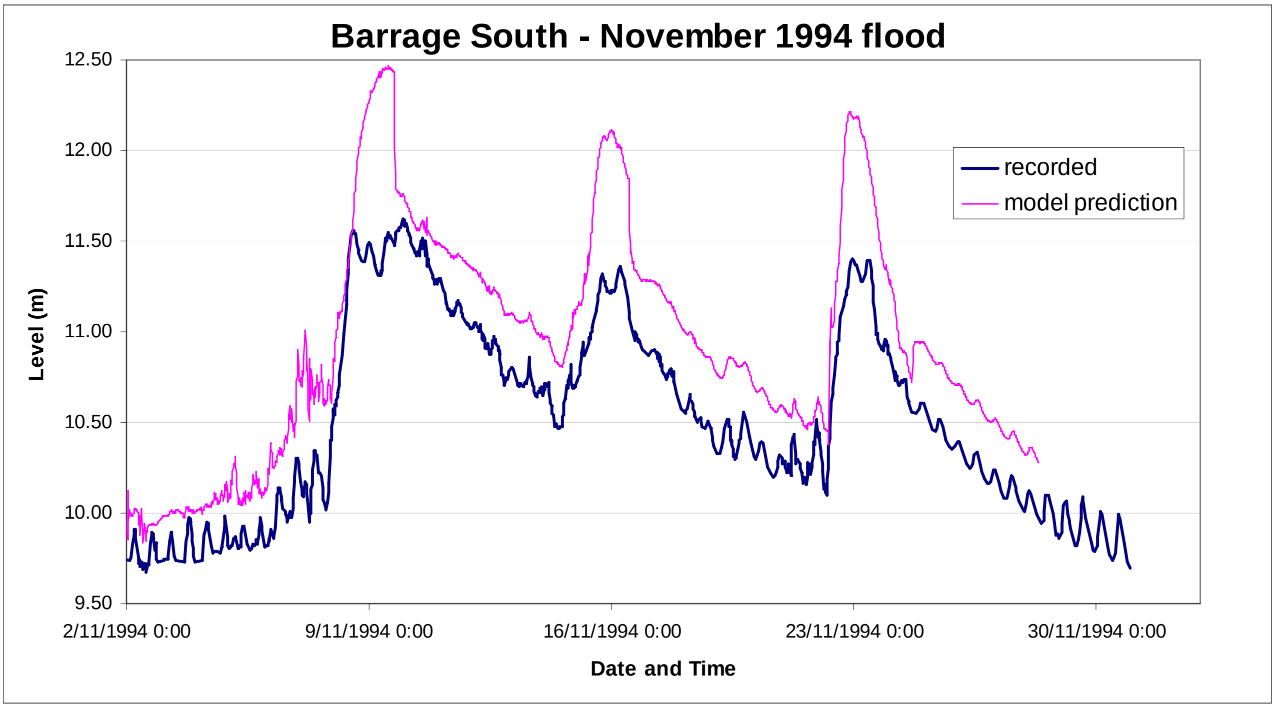


Figure 27

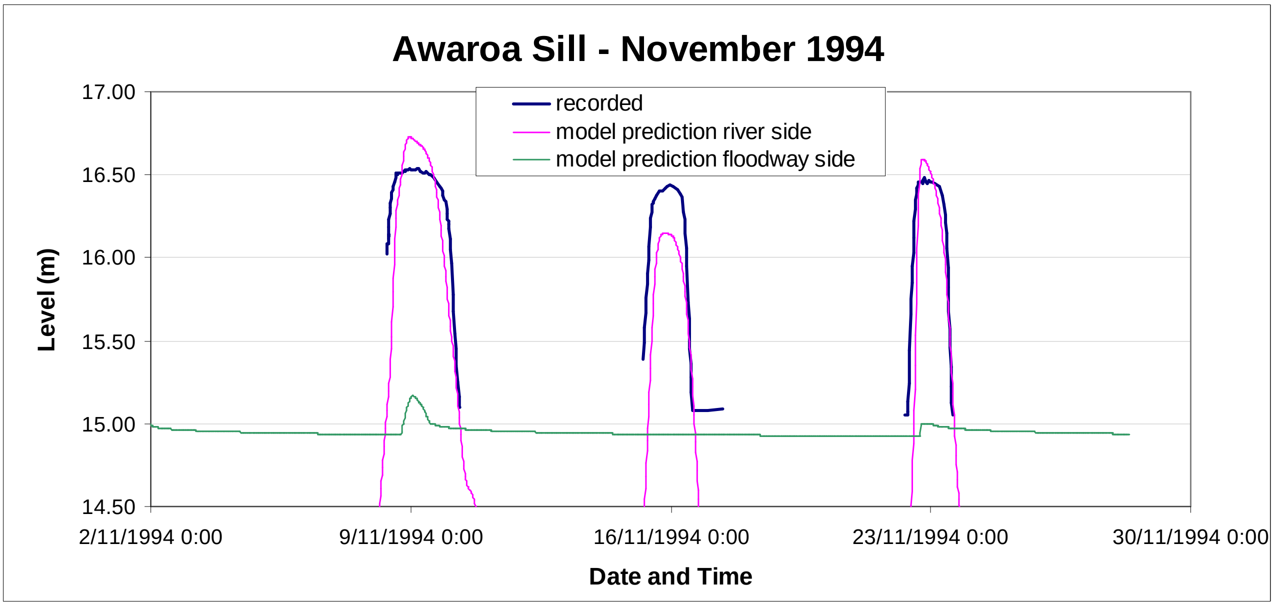
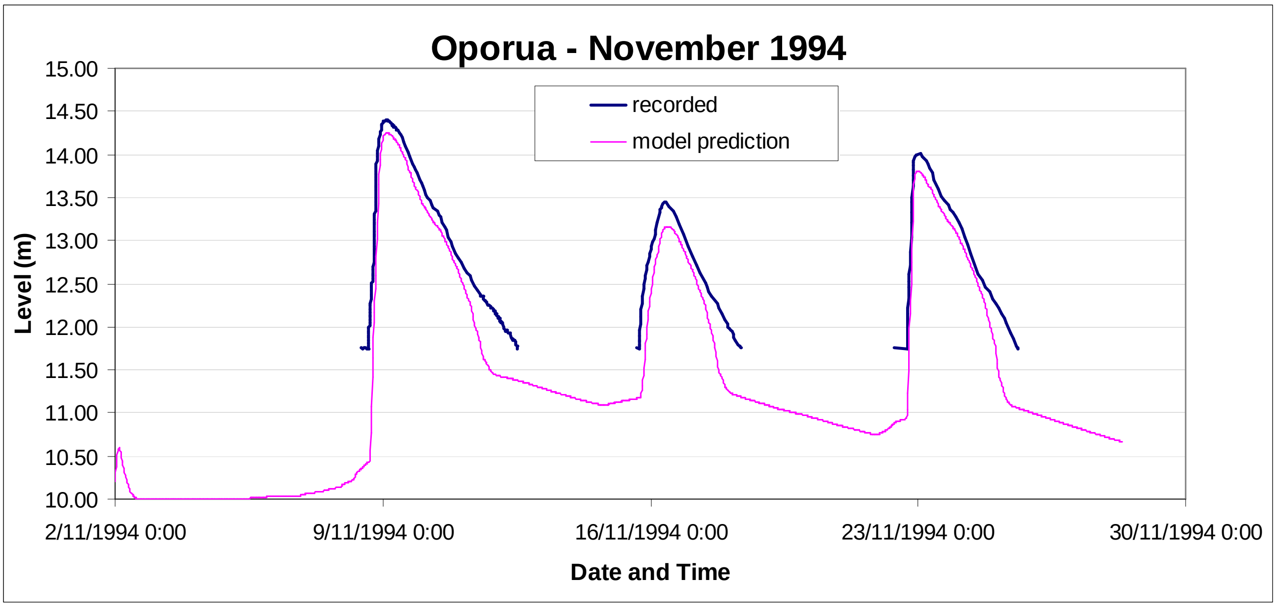


Figure 28



5.3.3 October 1998

(Refer to Figures 29-32 and calib-98.xls)

Although there were only four recorded water level hydrographs available for this flood event, model results compare reasonably well with them.

Figure 29

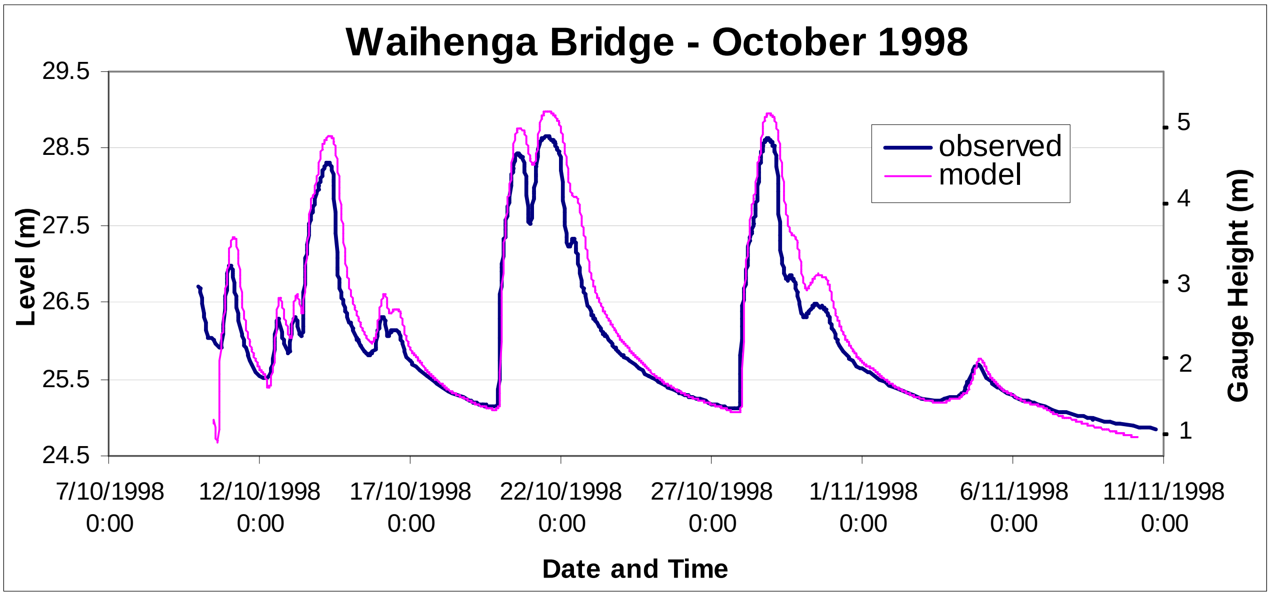


Figure 30

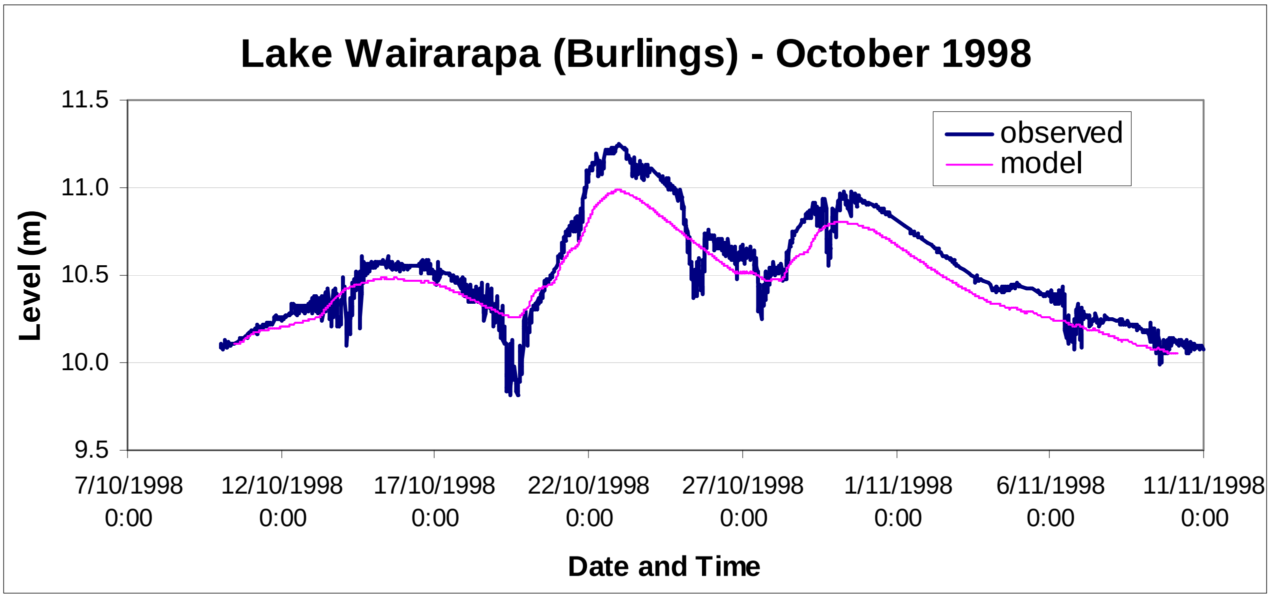


Figure 31

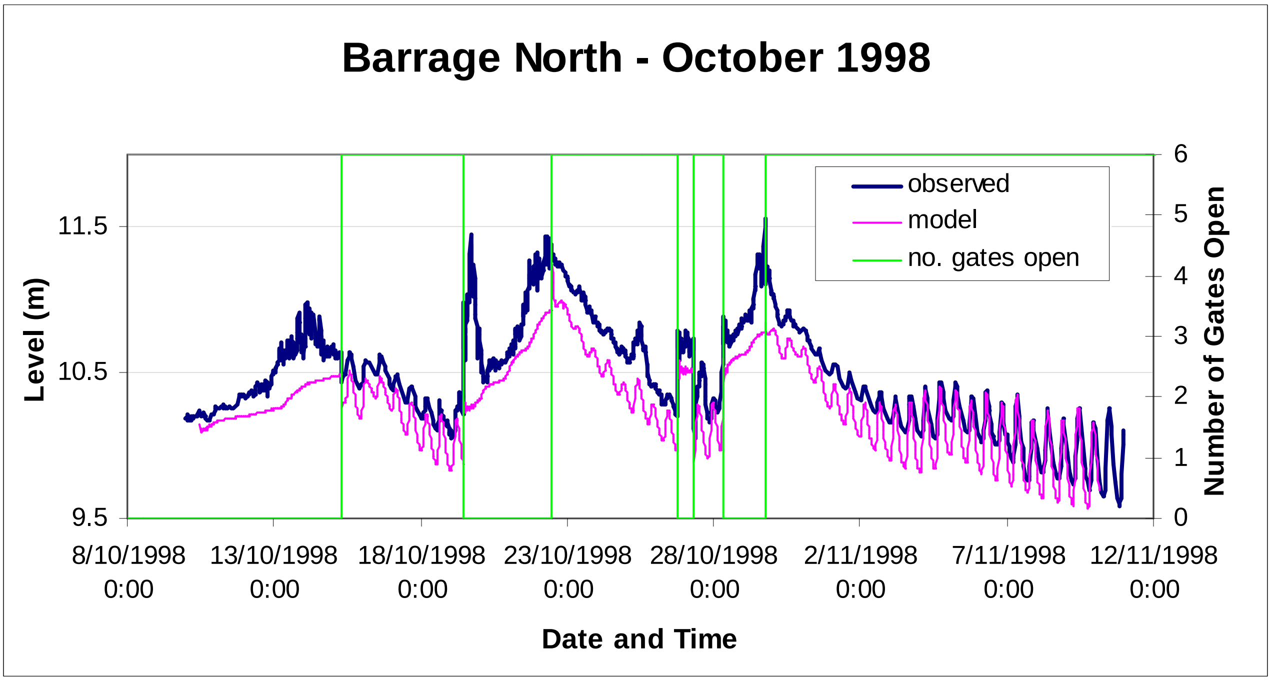
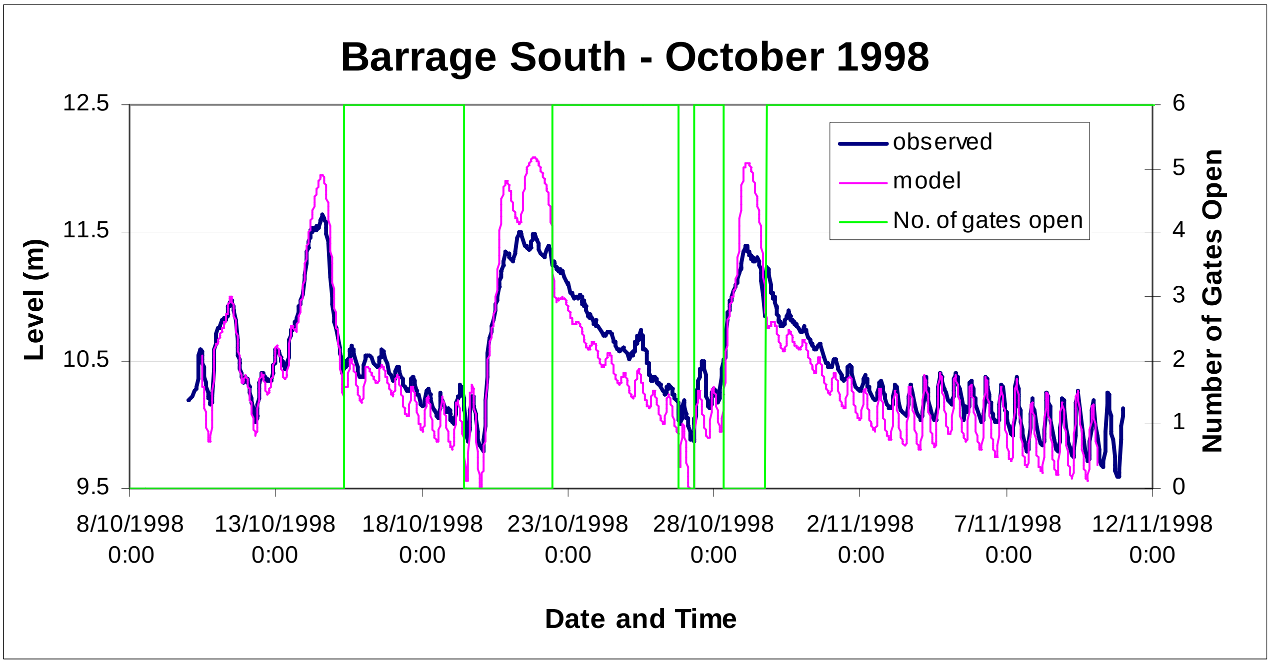


Figure 32



5.3.4 October 2000 (NW Storms)

(Refer to Figures 33-44 and Oct2000calibration.xls)

It is understood that the series of flood events during this month arose from north-west storms. A good set of observed data was collected from the event.

Results are again reasonable, with good results for Lake Wairarapa and the north side of the barrage gates in particular. As with the November 1994 event, the predictions at the peak and on the recession of the flood event are higher than was recorded. The model underpredicts slightly the peak levels in Jenkins Dip, although it has overpredicted the levels and gauged flow on the first flood peak (1 and 2 October).

Note that the actual locations of the flood peak levels recorded upstream of Waihenga are in many cases in the model berm or floodway channels rather than the main river channel. In Figure 33 the model results are plotted at the adjacent Ruamahanga chainage. This accounts for the uneven profile of the model results as plotted; i.e. the results jump from river to berm to floodway. In some cases the location is on the boundary of the channels, so it is not possible to get an exact level from the model.

Figure 33

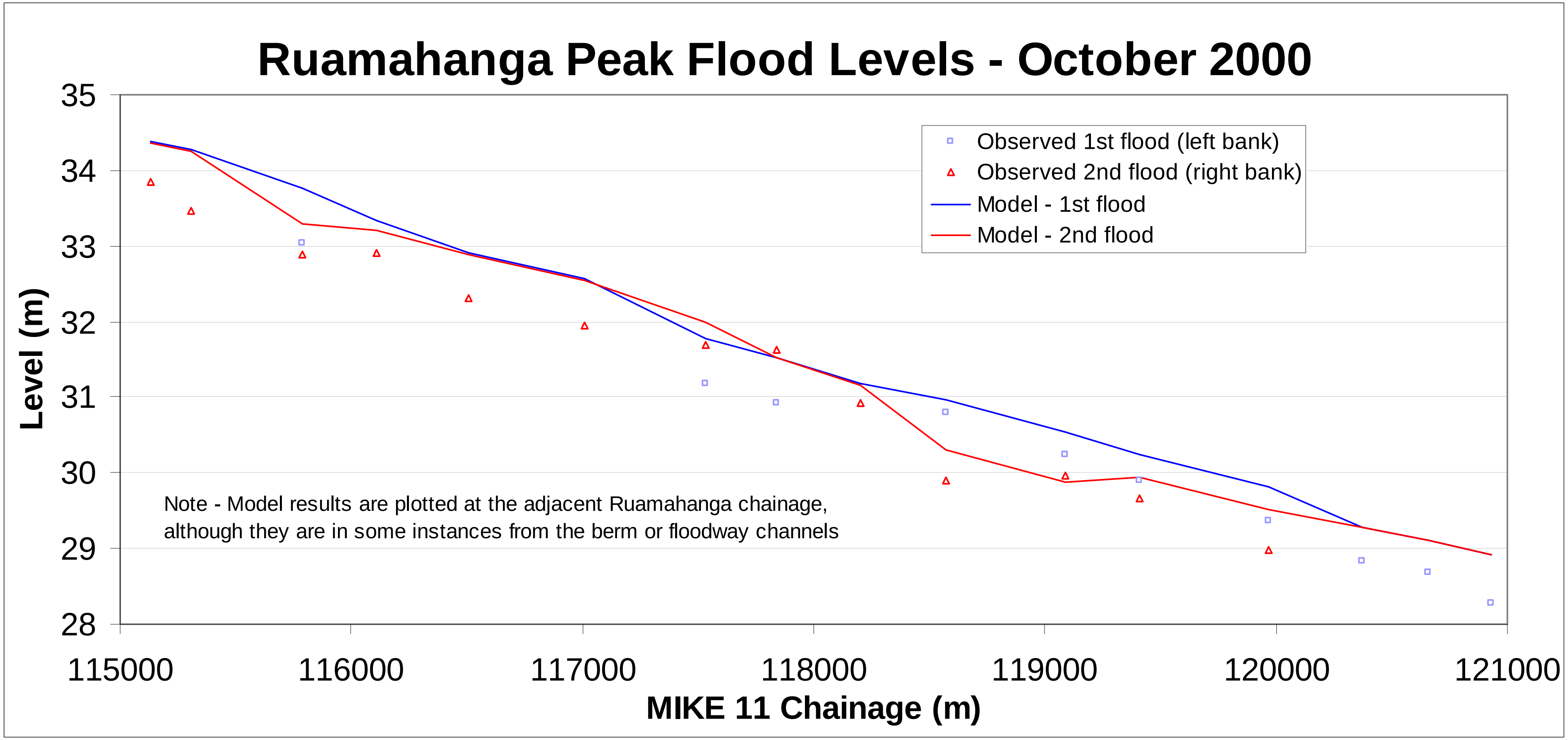

Ruamahanga Peak Flood Levels - October 2000
Observed 1st flood (left bank)
Observed 2nd flood (right bank)
Model - 1st flood
Model - 2nd flood
Note - Model results are plotted at the adjacent Ruamahanga chainage,
although they are in some instances from the berm or floodway channels
Level (m)
35
34
33
32
31
30
29
28
115000
116000
117000
118000
119000
120000
121000
MIKE 11 Chainage (m)

Figure 34

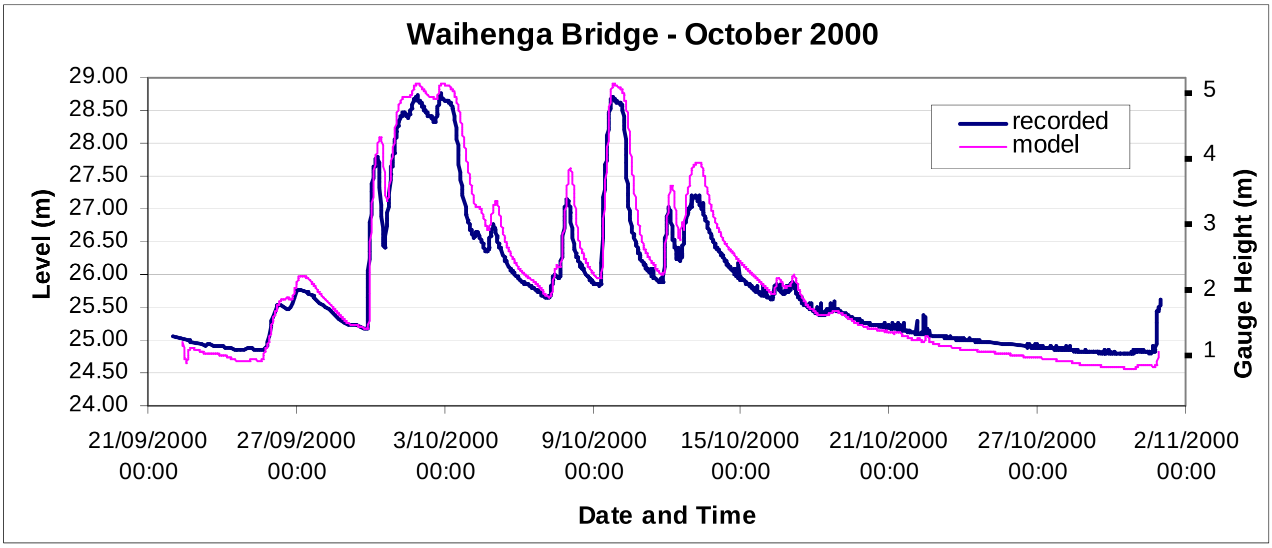


Figure 35

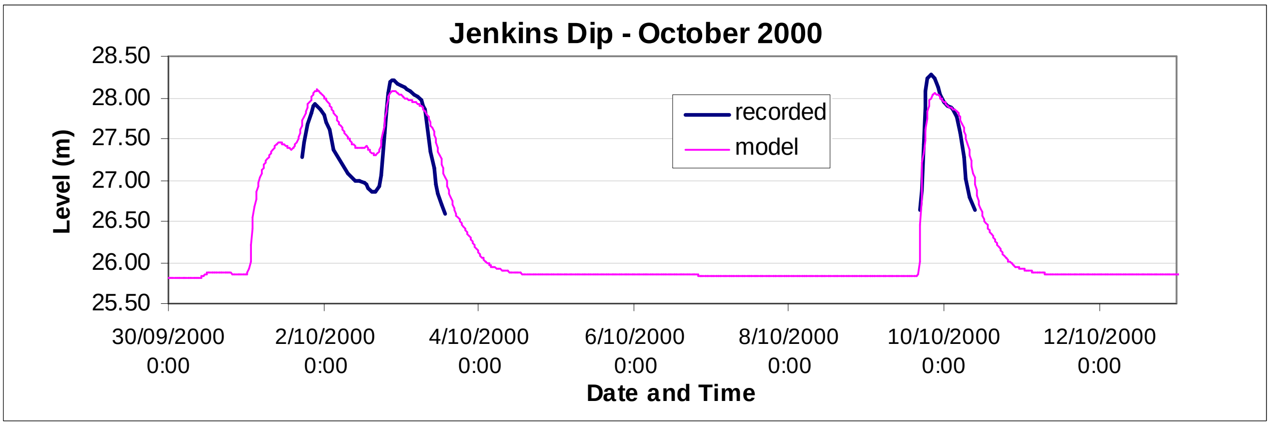


Figure 36

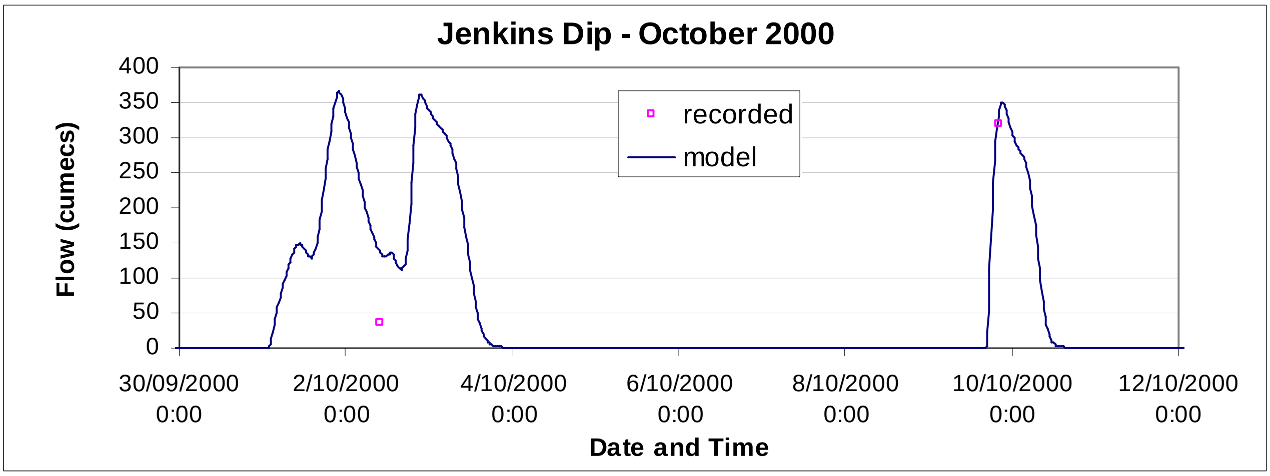


Figure 37

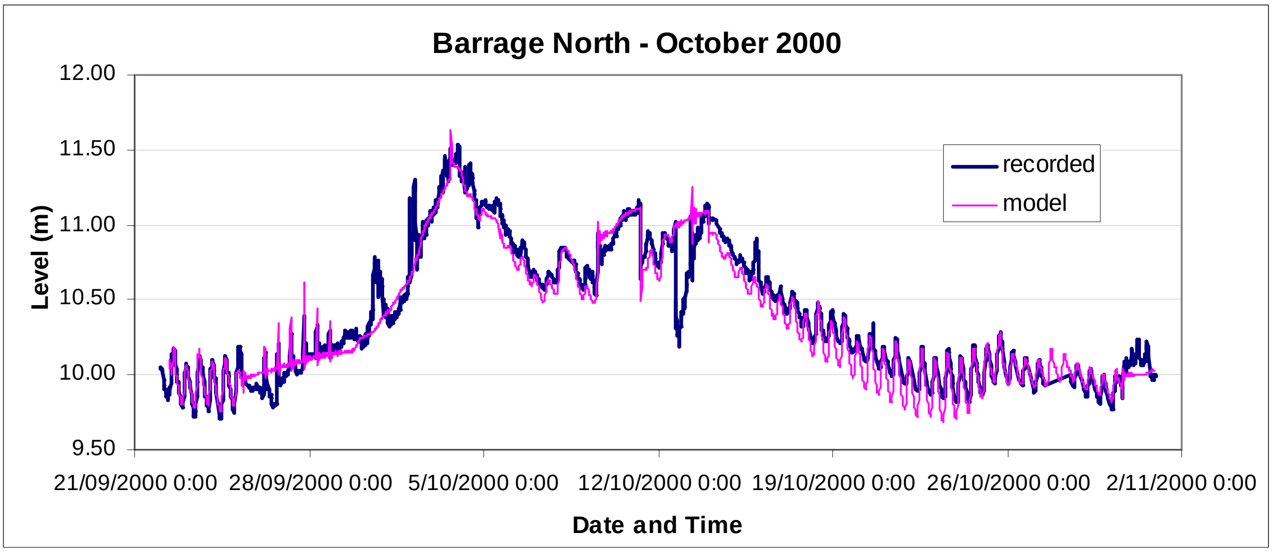


Figure 38

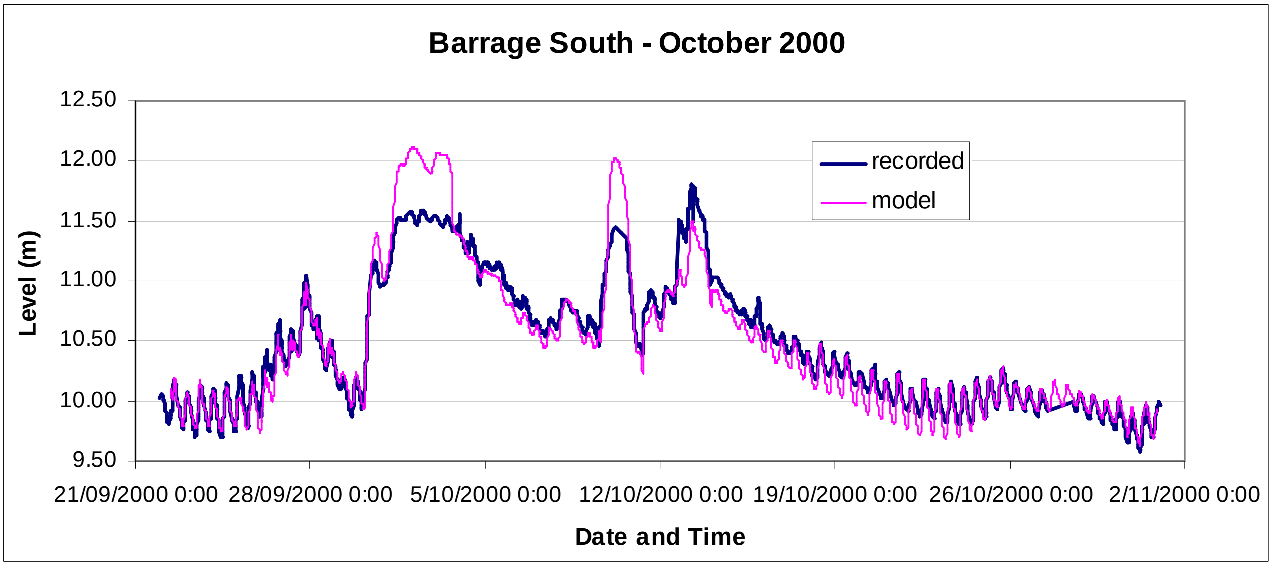


Figure 39

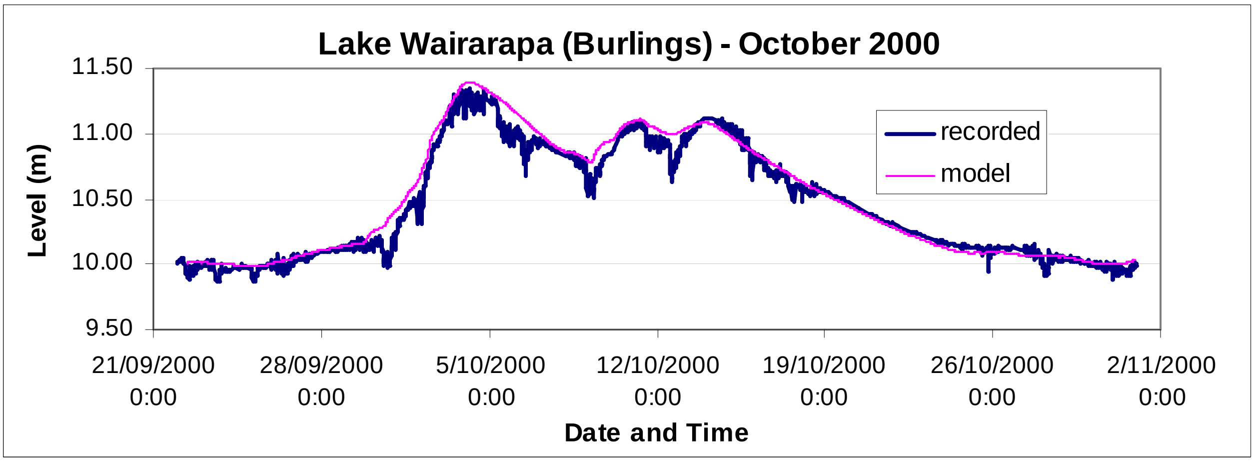


Figure 40

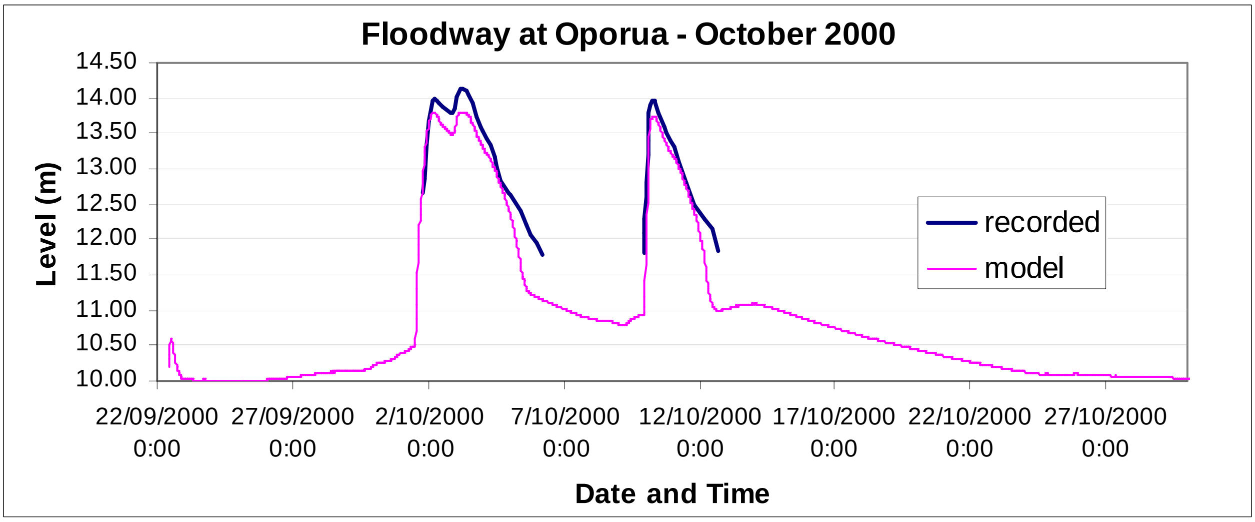


Figure 41

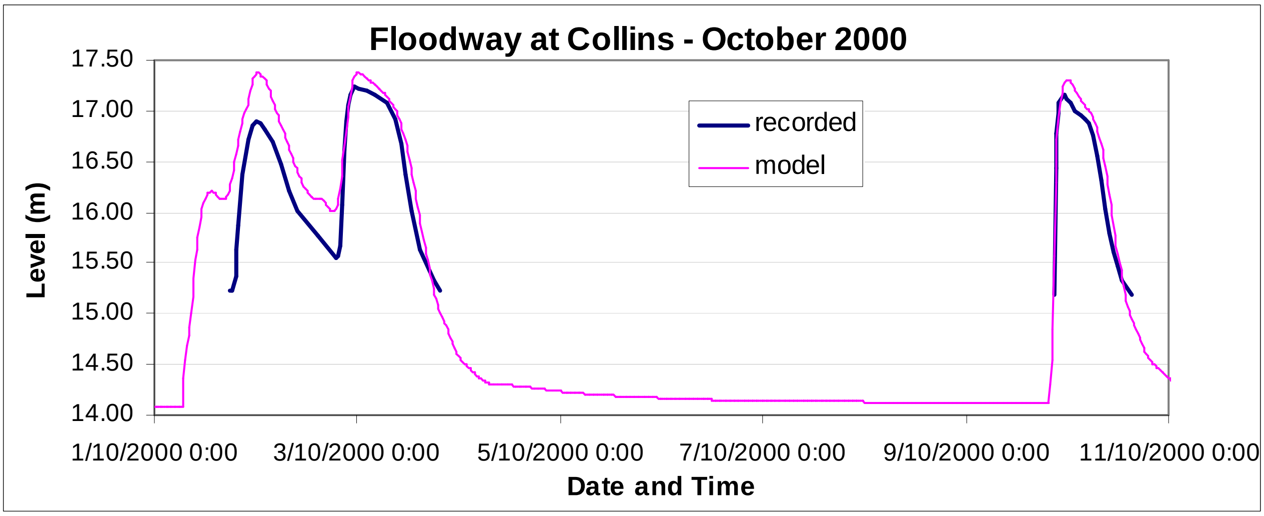


Figure 42

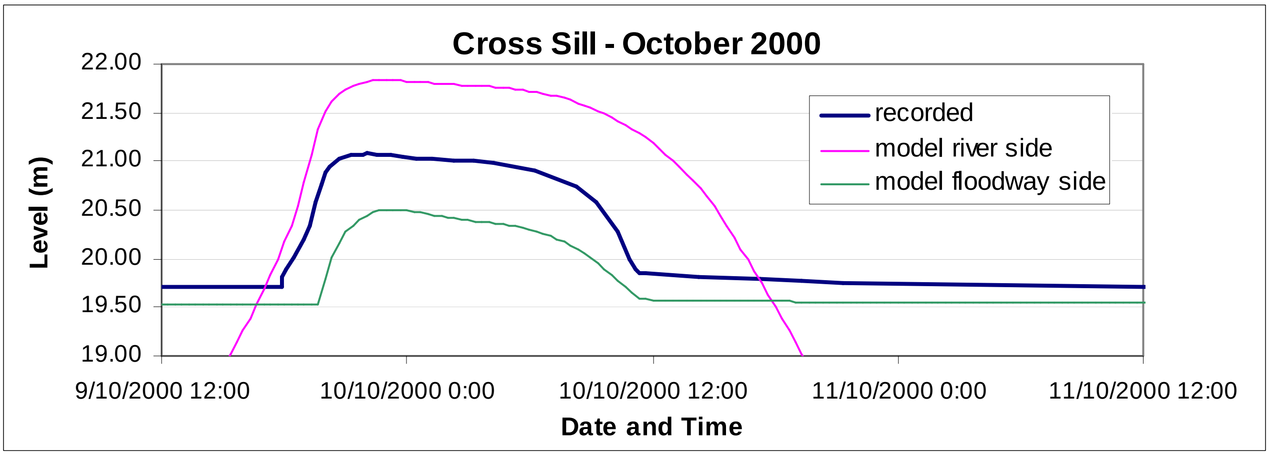


Figure 43

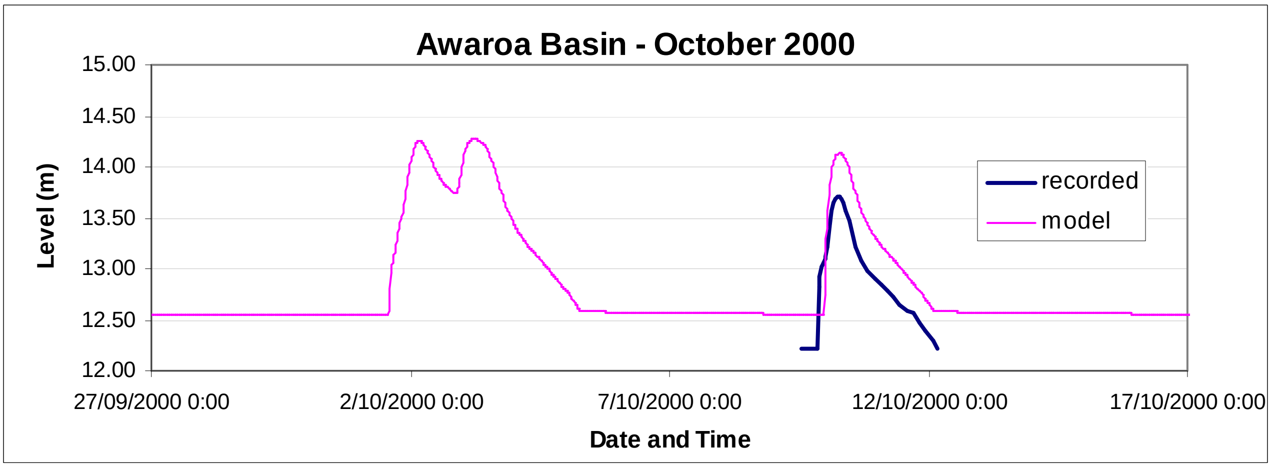
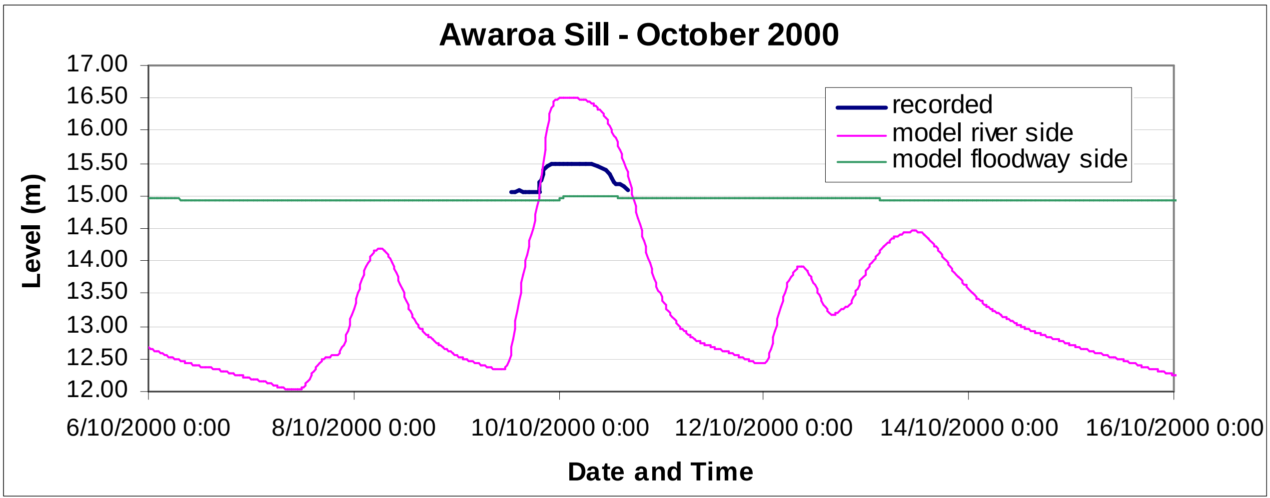


Figure 44



5.3.5 June 2003

(Refer to Figures 45-49 and June2003-calibration.xls)

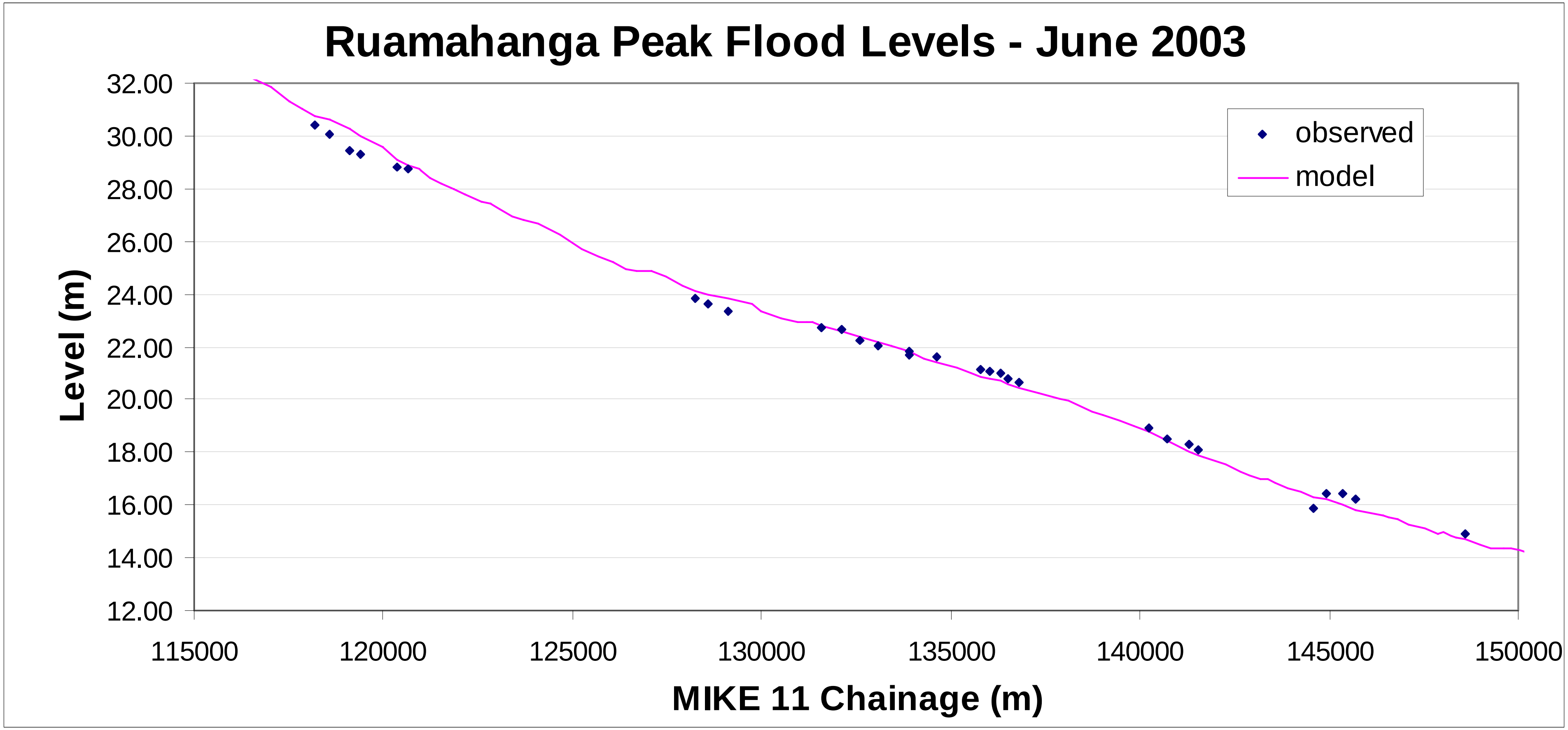
The flood event occurred after the model had been calibrated, during the capacity and design simulation stage of the project. However a good set of peak level data was collected, with the knowledge of where the model had not matched observations from previous events. This flood event can be regarded as the verification flood.

Results are encouraging, and no changes to the model parameters were subsequently made. As with the previous calibration simulations however, there were a few discrepancies between observations and the model results that were noted.

As before the model overpredicted the peak and recession limbs of the water level hydrographs at Waihenga Bridge. In this case, as the Jenkins Dip flow observed was not great (most likely due to raised sill levels), it is suggested that there was more attenuation of the flood between Waiohine and Waihenga than predicted or too much inflow at Huangarua assumed. As there is no certainty in inflows in any of these calibration events, no changes have been made to the model however. It was considered better to at least keep a consistent approach rather than to adjust the inflows somewhat arbitrarily.

Peak water levels are predicted to within an average of 250mm, with greater errors above the Jenkins Dip sill (as was the case for the 2000 flood). Comparing the peak water level profile along the Ruamahanga with that from the 1992 flood, the underprediction around Waihenga Bridge and the overprediction around 128-129km (i.e. cross-sections 108-110) are similar to the 1992 results, the predictions improved around 141km (sections 73-75) and were not quite as good around 145km (sections 61-62).

Figure 45



Ruamahanga Peak Flood Levels - June 2003
32.00
30.00
28.00
26.00
24.00
22.00
20.00
18.00
16.00
14.00
12.00
Level (m)
115000
120000
125000
130000
135000
140000
145000
150000
MIKE 11 Chainage (m)
observed
model

Figure 46

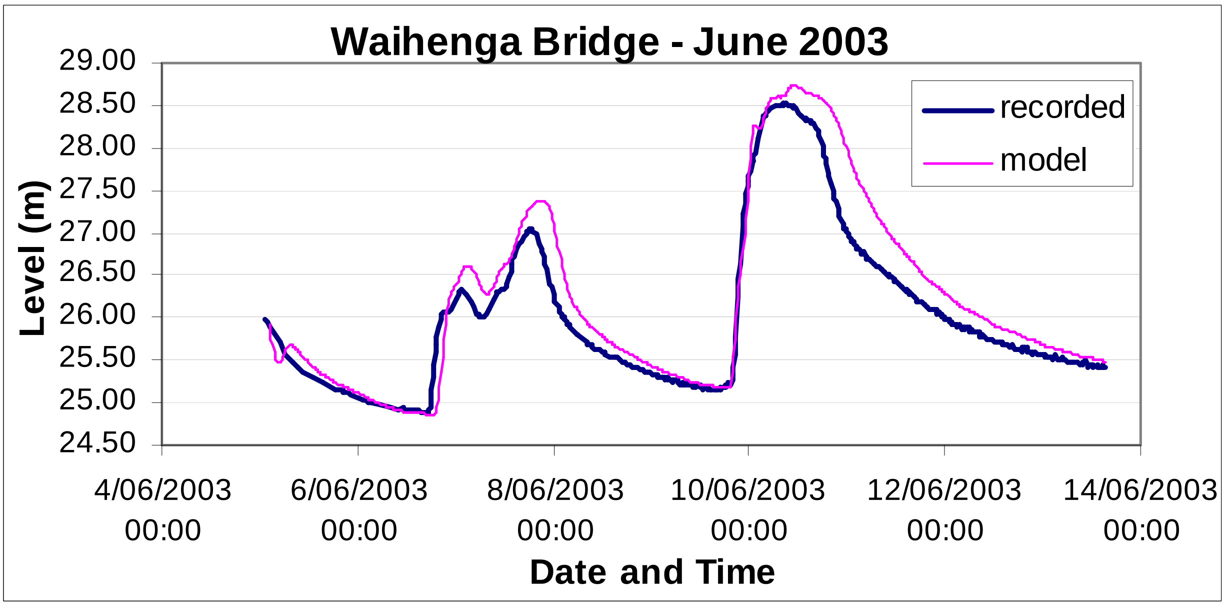


Figure 47

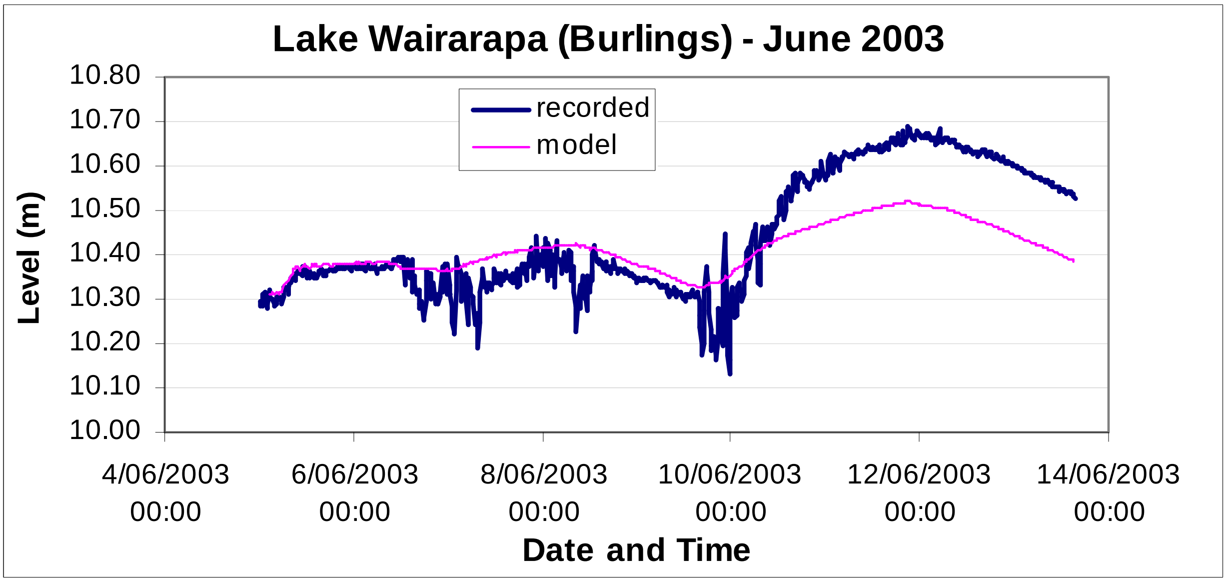


Figure 48

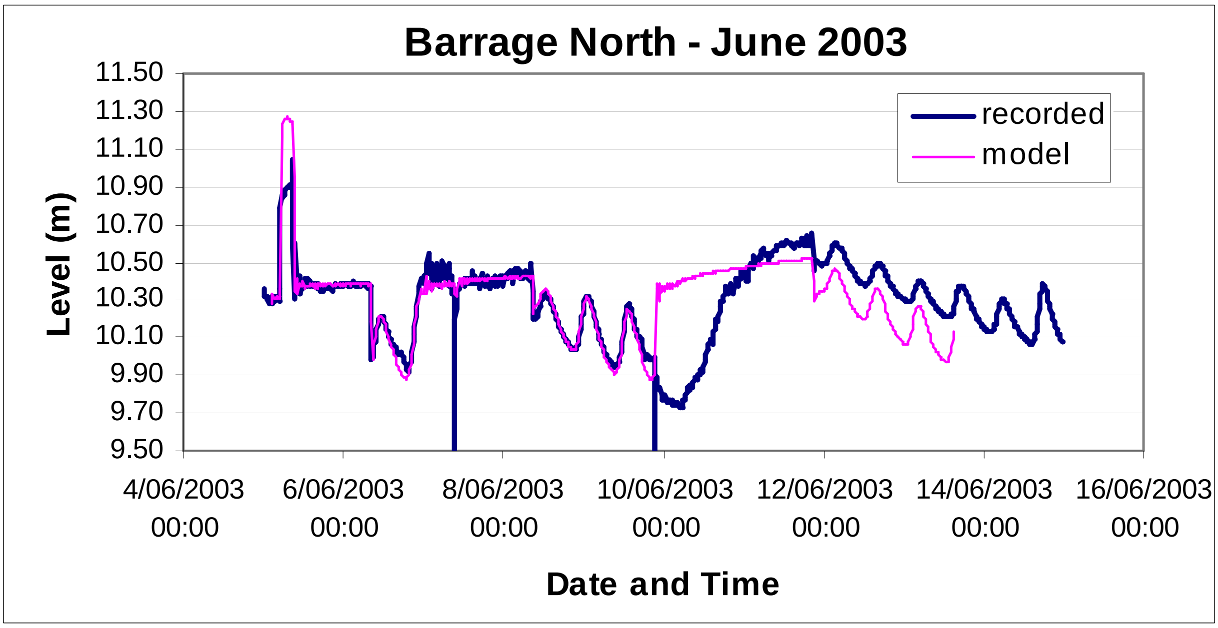
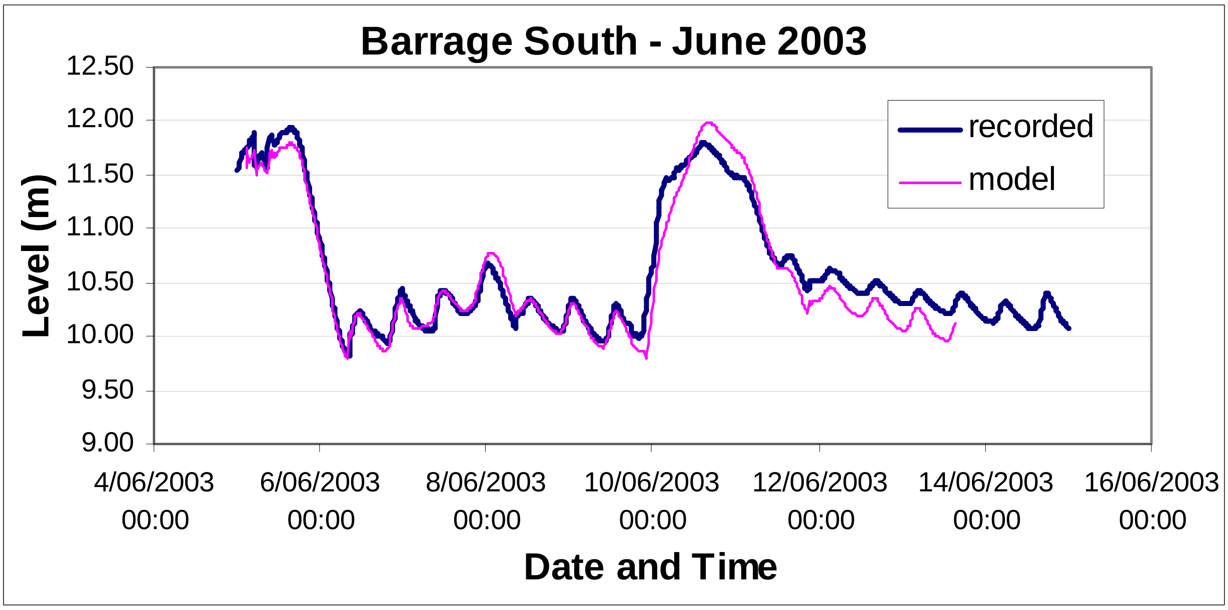


Figure 49



5.4 Summary

Despite the reasonably extensive calibration data set, there have been several difficulties in the calibration process.

- As noted earlier, only one cross-section set has been used. Over the eight year calibration period, changes in cross-sections locally can be expected to have some effect, even though in general the cross-sections are reasonably stable. Also, even during each month long calibration period, the riverbed will change between the individual floods in that period.

- Sill levels will also change over time, generally being raised as successive floods deposit silts. This could help explain why the predicted water levels in the floodways are slightly higher than observed in the 2000 flood event.

- The relative inflow contributions of each ungauged subcatchment will vary over time, between each event.

- The sensitivity of modelled peak levels to changes in Mannings n (traditionally the main calibration parameter) in many locations is not great, as overflows from the main channel occurred. Raising Mannings n, for instance, may in some situations simply push more water out of the main channel and so have a reduced effect on main channel peak water levels.

Nevertheless, a satisfactory calibration has been achieved, sufficient to have confidence in the capacity and design simulations.

6 Sensitivity Tests

Several sensitivity tests were carried out in developing the model. These have included sensitivity to the following parameters.

6.1 Channel Roughness (Mannings n)

During the calibration process, numerous Mannings n profiles (versus river chainage) were trialled. The Q_{50} design scenario has been run with the final adopted profile and the initial trial, to show the sensitivity of results to Mannings n. Results are plotted below in Figures 50 and 51. The maximum variation in the peak water level in the river between the two simulations was 0.24m. (Note that subsequently some modifications were made to the barrage gate rules, and Figures 50 and 51 do not show the final design results. Rather they only illustrate the effect of Mannings n on peak water levels).

Figure 50

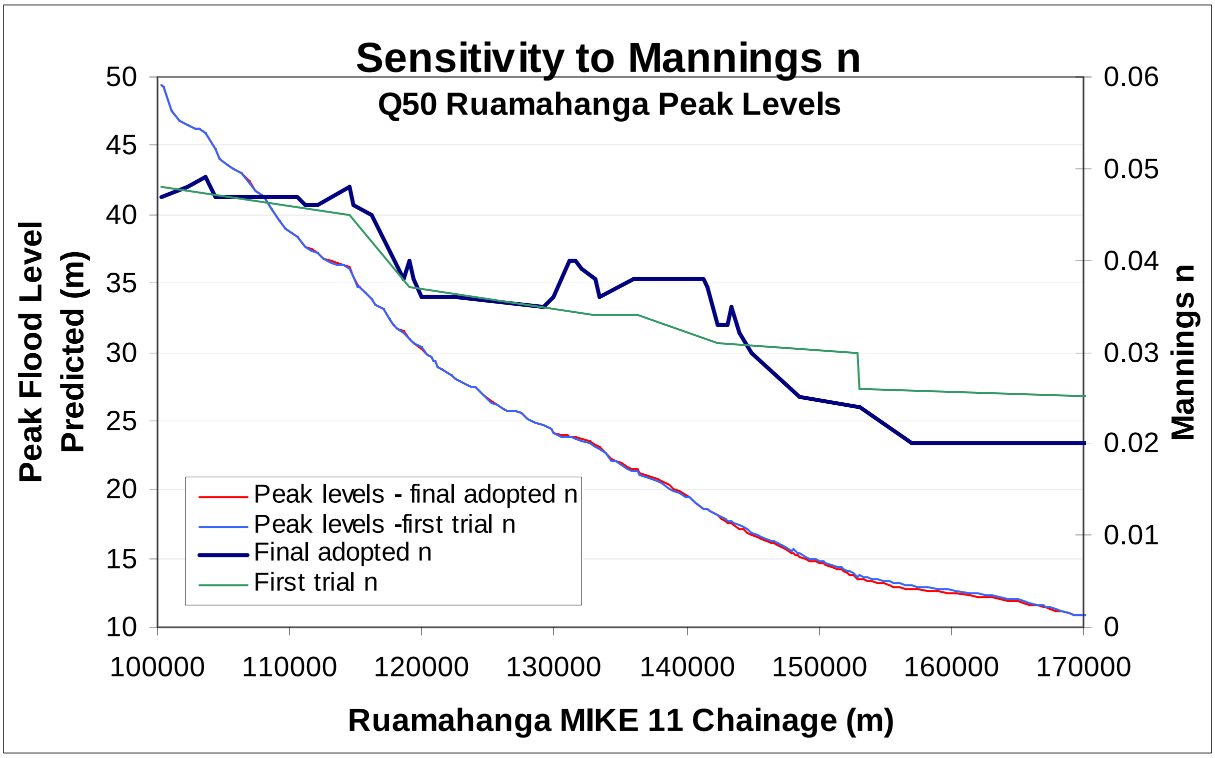
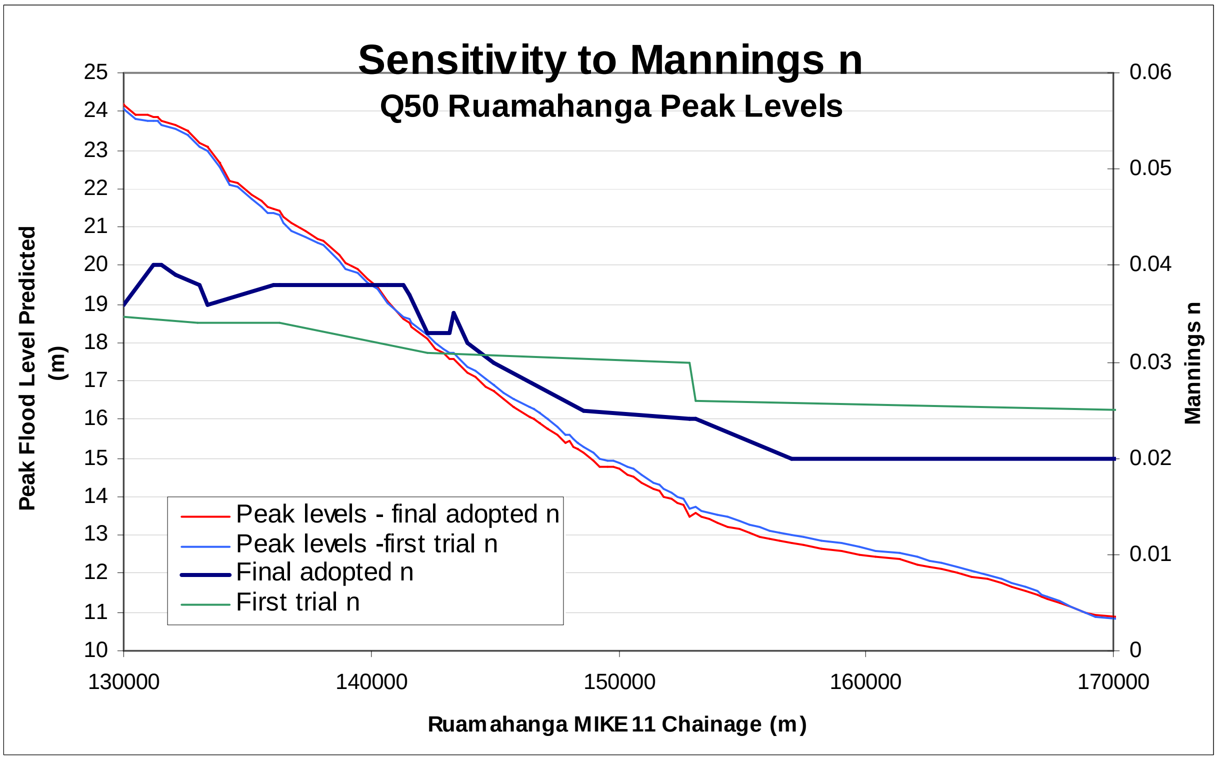


Figure 51



6.2 Delta (Δ)

The time weighting of the finite difference scheme used by MIKE 11. A value of 0.5 gives in general the most accurate results, but the model can become unstable in complex situations. A higher value (up to 1) can damp instabilities, and a value of 0.6 to 0.7 tends to be a good compromise. Initial runs were all done with $\Delta = 0.6$, but a later trial using $\Delta = 0.75$ was trialled on the November 1994 and Q_{50} simulations. The change in Δ made very little difference, and thus $\Delta = 0.6$ was used for all final simulations.

6.3 Link Geometry and Roughness

The setup for the river/berm/floodway link connections requires the user to specify a length (i.e. parallel to the link flow directions) and a roughness value. (As noted earlier, weir crest profile and width are also needed). Tests were done on the length and roughness values, with link lengths between 50 and 250m and the Mannings n roughness between 0.050 and 0.150. While some differences in results did show, these were not significant compared to the effect that the level of the weir crest would have, and in general a length of 50m and a roughness of $n = 0.050$ were used. In several locations however, aerial photographs or site inspections suggested a higher roughness would be required and n values of 0.075 to 0.125 were used as considered appropriate.

As discussed in more detail in section 8.4 below, sensitivity tests were also run with the Tawaha Sill raised by 300mm, as the sills are expected to have aggraded since the 1998 survey (on which the model sill levels are based). Results showed that peak water levels were not affected significantly, but that the Waihenga rating curve would shift.

6.3 Cross-section Aggradation

The effect on design levels of channel aggradation in the vicinity of Waihenga was also assessed. The bed of the main channel between sections 127 and 136 was raised by 300mm for these sensitivity tests.

A further set of tests was made with the raised bed levels combined with the raised sill levels as described in 6.2 above.

With the sills alone raised by 300mm, the peak water levels for the 50 year (short hydrograph) flood simulation were 120mm lower in the upper part of the Tawaha floodway, 45mm lower at the Jenkins Dip and 45mm higher at Waihenga Bridge. Further upstream and downstream the differences quickly disappeared. Peak flow at Jenkins Dip was $20m^3/s$ (3%) lower.

With the river bed raised by 300mm, but the sills left as originally modelled, the peak flow at Jenkins Dip was $50m^3/s$ (7%) higher than with the standard model. With both the river bed and the sills raised by 300mm, the peak flow at Jenkins Dip was $40m^3/s$ (6%) higher than with the standard model.

Figures 52 and 53 show the predicted effect of such aggradation scenarios on peak water levels, while further results are discussed in sections 8.3 and 8.4.

Note however that while the thalweg has risen about 1m under the Waihenga Bridge, and the thalweg in other sections from 128 to 130 has risen by 200-400 mm, the actual decrease in cross-section area between 1998 and 2003 has been small. Figure 54 presents a comparison of the bridge cross-section between 1998 and 2003, as surveyed.

Figure 52 Effect of 300mm Rise in Cross-sections and/or Tawaha Sill Levels, Q_{20} Flood

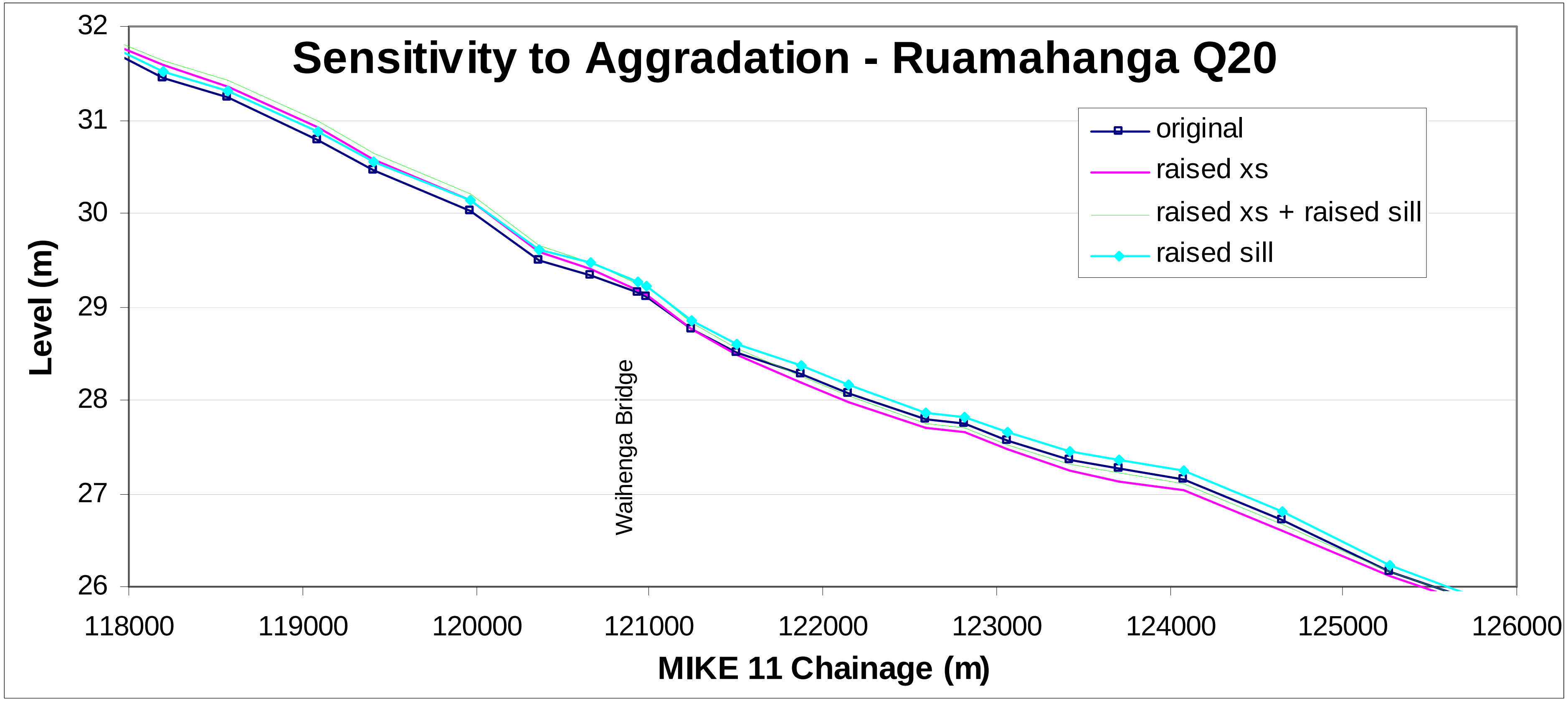


Figure 53 Effect of 300mm Rise in Cross-sections and/or Tawaha Sill Levels, Q_{50} Flood

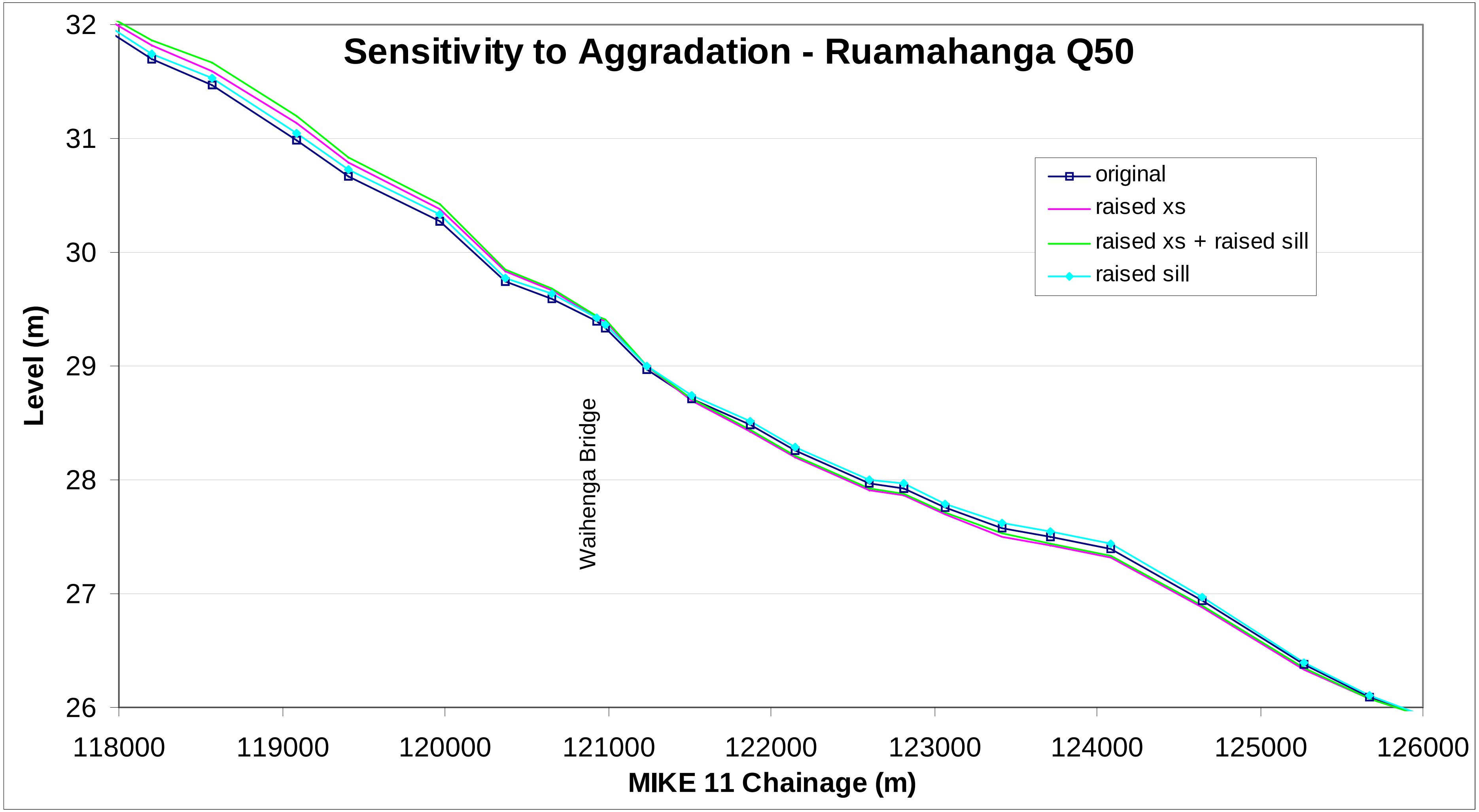
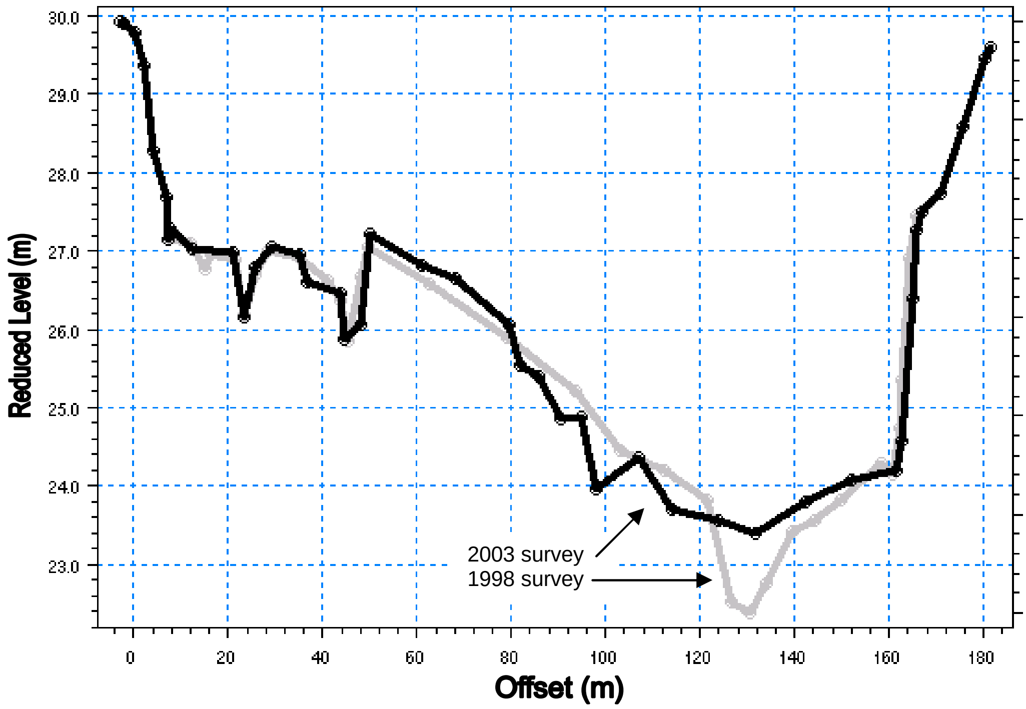


Figure 54 Surveyed Aggradation at Waihenga Bridge



6.4 Lake Wairarapa Representation

As discussed above, various trials were used in determining an appropriate representation of Lake Wairarapa. It was found that the treating the lake as a series of cross-sections gave the most reasonable and stable results. However, using a one-dimensional model for the lake has its limitations; a two-dimensional model with good lake bathymetry data would likely be an improvement.

6.5 Lake Onoke Representation

Also as noted above, trials with the representation of Lake Onoke showed little difference in results. As with Lake Wairarapa, the one-dimensional representation of Lake Onoke will nonetheless have limitations.

6.5 River Mouth Dimensions

Following low flow simulations (Section 9 of this report) with a tidal range in Palliser Bay as suggested by the NIWA tidal model and with the “short mouth” model, Greater Wellington staff noted that in Lake Onoke both the minimum levels were lower and the tidal range was greater than what would be expected. A trial with the mouth opening invert raised by 1m was then made, and results showed minimum lake levels in Lake Onoke would be raised by approximately 300mm and peak flows in and out of the mouth would be halved.

However, during a flood the mouth would be expected to scour and thus the initial mouth dimensions assumed were kept for the final design flood simulations.

7 Capacity Runs

Capacity is defined as the maximum flow that stopbanks could hold without overtopping (assuming no breach failure), subject to a given freeboard. Capacity was assessed for three freeboards:

- 600 mm - this is the defined scheme standard (originally 2')
- 1000 mm – this allows extra for the effect of winds in the lower reaches (wind waves and set up will be greater in the lower reaches in times of high winds, particularly from the south)
- 0 mm – this defines the maximum possible capacity

Stopbank crest levels, as taken from cross-sections, were plotted against river chainage. (This assessment does not consider the possibility of lower stopbank levels between cross-sections).

A range of steady state inflow scenarios were simulated to determine capacity. A nominal inflow for Lake Wairarapa was also included, although that does not have any significant effect on the results. For simplicity it was assumed that the barrage gates were closed, as would usually be the case in flood conditions.

Capacity runs for both a "short mouth" and a "long mouth" opening to the sea were modelled, but the "blocked mouth" scenario was not as this would be rather meaningless in a steady state condition (water would continue to rise until the mouth unblocked itself).

The river system has been broken down into five main reaches, as follows:

- Tawaha/Jenkins Dip upstream sill to Hikinui north sill
- Hikinui north sill to Awaroa south sill
- Awaroa south sill to Tuhitarata
- Tuhitarata to barrage channel exit
- Barrage channel exit to Lake Onoke

The capacity flows have generally been measured at the top of these reaches, so that in effect the capacity refers to the total river plus parallel floodway flow in the reach. The exception is for the Awaroa to Tuhitarata reach, where putting more flow in above simply lead to more flow being diverted down the floodway and having very little effect on the downstream river channel freeboard. Results for that reach are therefore given in terms of the flow in the river.

The resulting freeboard at each section for each simulation was plotted and the general capacity over a reach assessed visually. Inevitably there has been some smoothing or generalising of capacity results. An example of the plots is given in Figure 51, with a summary in Table 1. A full set of results can be found in spreadsheets CAPACITY.xls and CAPACITY2.xls.

Note that the river stopbank levels rather than the floodway stopbank levels determined the capacity. That is, for a given scenario the general freeboard was greater in the floodways than in the river.

Figure 55   Example of Capacity Results

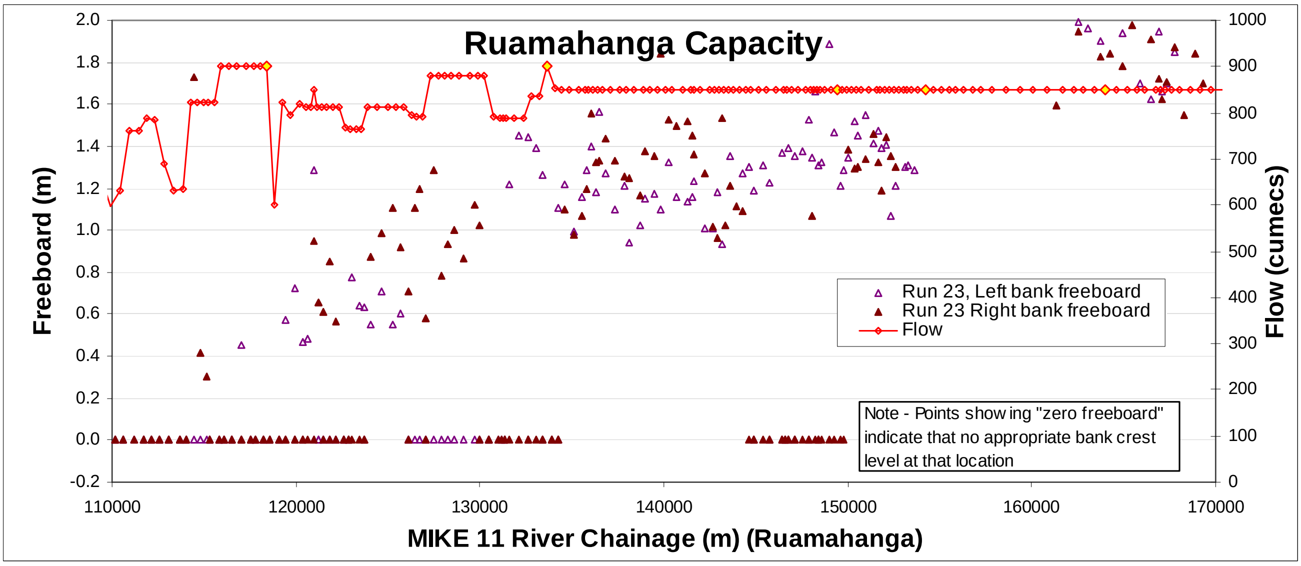


Table 1 Capacity

Reach	Freeboard (mm)	Flow (cumecs)					
		Short mouth			Long Mouth		
		0	600	1000	0	600	1000
Upstream end of Tawaha Sill to Hikinui Sill		1550	800				
Hikinui Sill to Awaroa		2000	1130				
Awaroa to Tuhitarata		1380	1030				
Tuhitarata to Barrage Channel exit		2100	1640	1430	1960	1600	1390
Barrage Channel exit to Lake Onoke		2050	1485	1240	1820	1390	1190

8 Design Simulations

8.1 Design Hydrological Conditions

Two design sea level conditions have been modelled. The first assumes a constant sea level of 10.5m datum. This is approximately 0.5m above high tide.

The second is a variable tide time series, and has been taken from the NIWA tide predictions in the vicinity of Palliser Bay. (Only the high and low tide peaks for each cycle have been predicted, so that the time series used is of “saw-tooth” rather than sinusoidal shape.) The predictions have been converted to the model datum by the addition of 9.3m. (The model datum is MHWS = 10m (or 100 feet in imperial units), and from the NIWA time series, the MHWS is approximately 0.7m.)

A period starting on 1 January 1995 has been extracted. To this time series, 0.5m has been added to make some allowance for storm surge.

Four flow scenarios have been modelled – nominally a 5% AEP (“20 year”) and 2% AEP (“50 year”) for each of two hydrograph shapes. The first shape is a vertically scaled version of the second peak of the November 1994 event at Waihenga Bridge, which was similar in shape to the second peak of the October 2000 event and the last peak of the

October 1998 event at the bridge (Figure 56). The second hydrograph shape is of longer duration and is a scaled version of the June 1985 flood event as recorded at the bridge.

The 20 year and 50 year return period estimates for total Waihenga Bridge and Jenkins Dip flow are 1611 m^3/s and 1845 m^3/s respectively. The combination of inflows at the top end of the model and from the Huangarua to give the correct total Waihenga Bridge and Jenkins Dip flow in the model were obtained by trial and error for the shorter hydrograph, and are as follows:

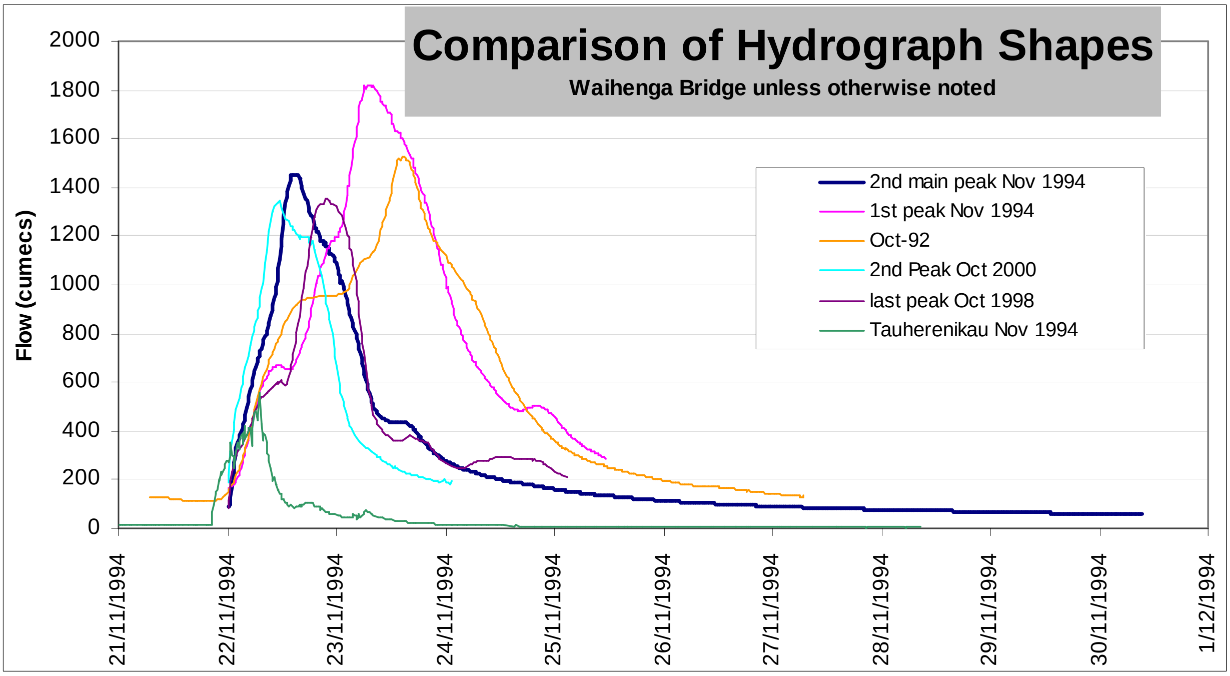
	Ruamahanga @ Waiohine	Huangarua @ Ruamahanga	Waihenga + Jenkins Dip	
			Model	GW Estimate
20 year	1550	100	1878	1845
50 year	1800	100	1616	1611

These values were also used for the longer hydrograph scenarios. Note that the Huangarua flow of 100m^3/s is less than the estimate of the 2 year flood for that river (160m^3/s).

For each scenario, the following additional inflows were included:

- 450 m^3/s into Lake Wairarapa
- 50 m^3/s each from the Turanganui, Tauanui and Whangaehu/Paharakeke Rivers (50 year scenario only)

Figure 56 Comparison of Recorded Hydrograph Shapes (each shifted horizontally)



8.2 Design Water Level Results

Design water level profiles are plotted in Figures 57 to 64 and tabulated in Appendix III. Each of the Q_{20} and Q_{50} profiles plotted show the highest of the various long mouth/short mouth, long/short hydrograph and constant/variable tide combinations possible.

Results indicate that in the lower reaches, the stopbank levels are above the design 50 year design standard, leaving sufficient to provide at least a 1m freeboard in most locations. However in the system between Waihenga and section 67, freeboard in the design 20 year event is compromised over significant lengths. From section 67 to Tuhitarata, stopbank levels are above the 20 year design standard.

The “blocked mouth“ result is given for interest and comparison only; it should not be regarded as a realistic result. The top level of the weir at the mouth in this setup was sufficient to cause blocking but was not based on any topographical data. The model results show water overtopping this weir in the design scenarios, but it would be expected that at the least the weir (i.e. the sand spit) would erode as floods overtopped it and more likely that the mouth would “blow out” before the levels predicted in these simulations would be reached.

Figure 57 Peak Water Level Predictions – Q_{20} without Freeboard, Cross-sections 82 to 175

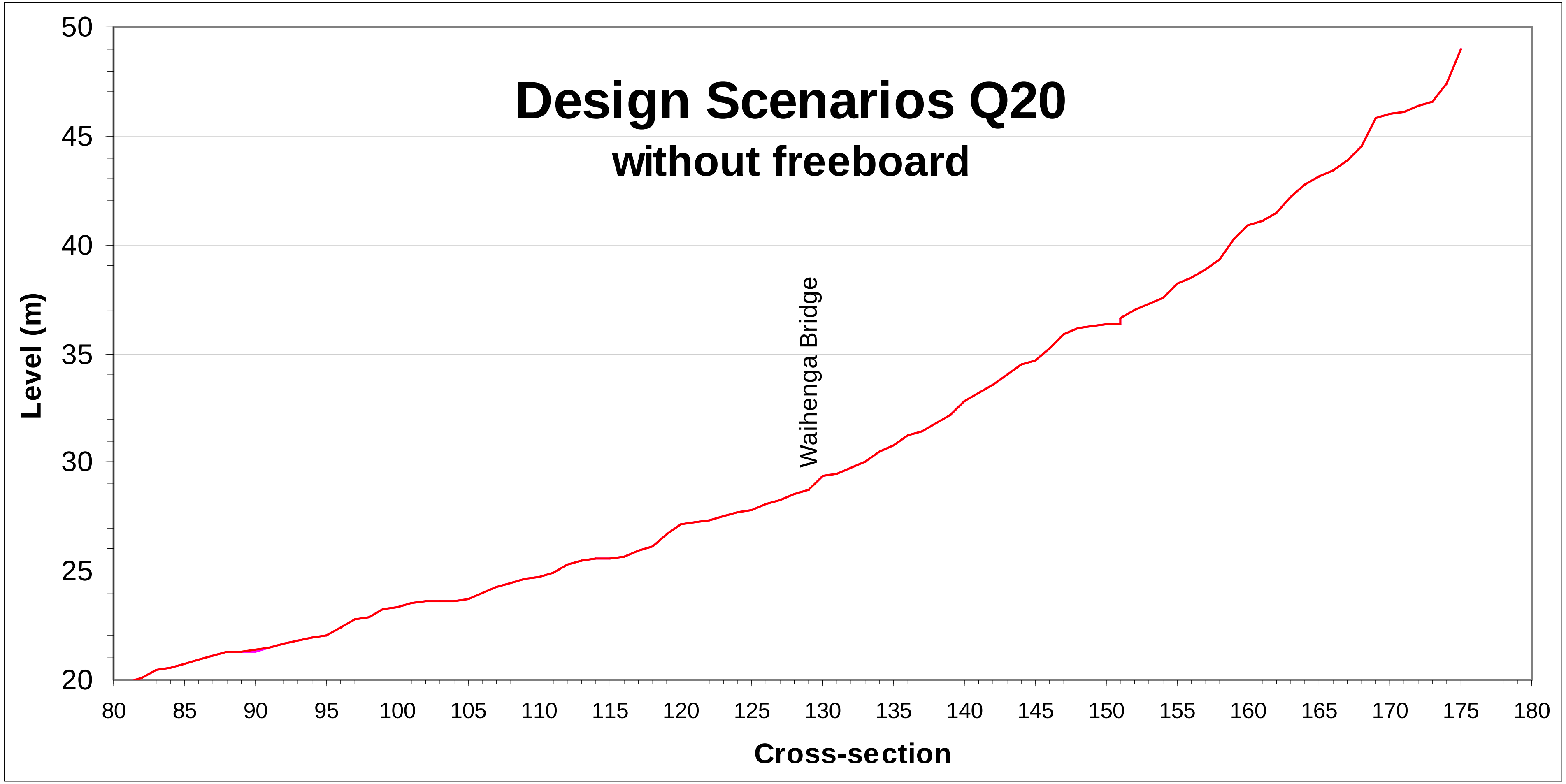


Figure 58 Peak Water Level Predictions – Q_{20} without Freeboard, Cross-sections 1 to 100

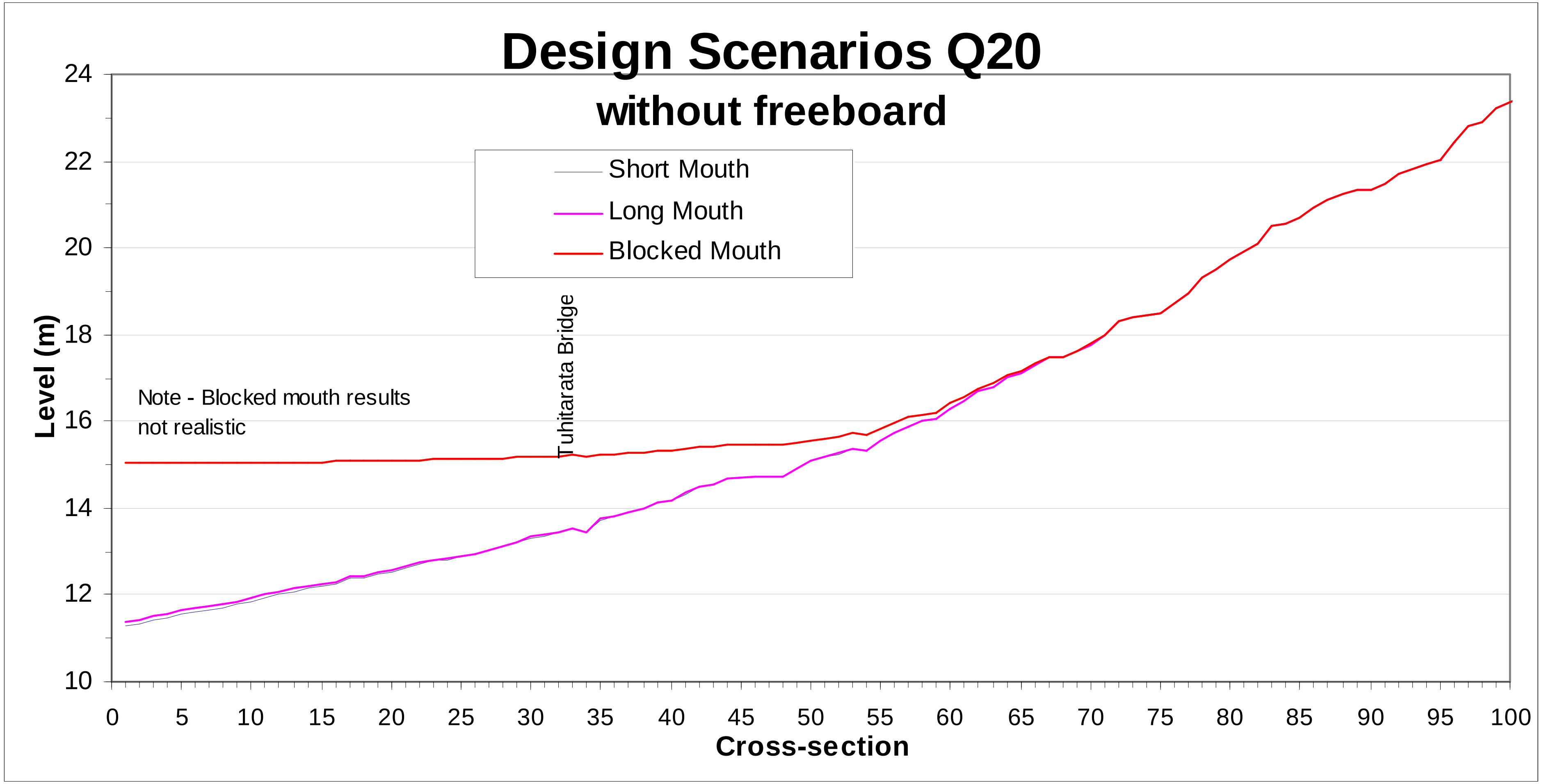


Figure 59 Peak Water Level Predictions – Q_{50} without Freeboard, Cross-sections 82 to 175

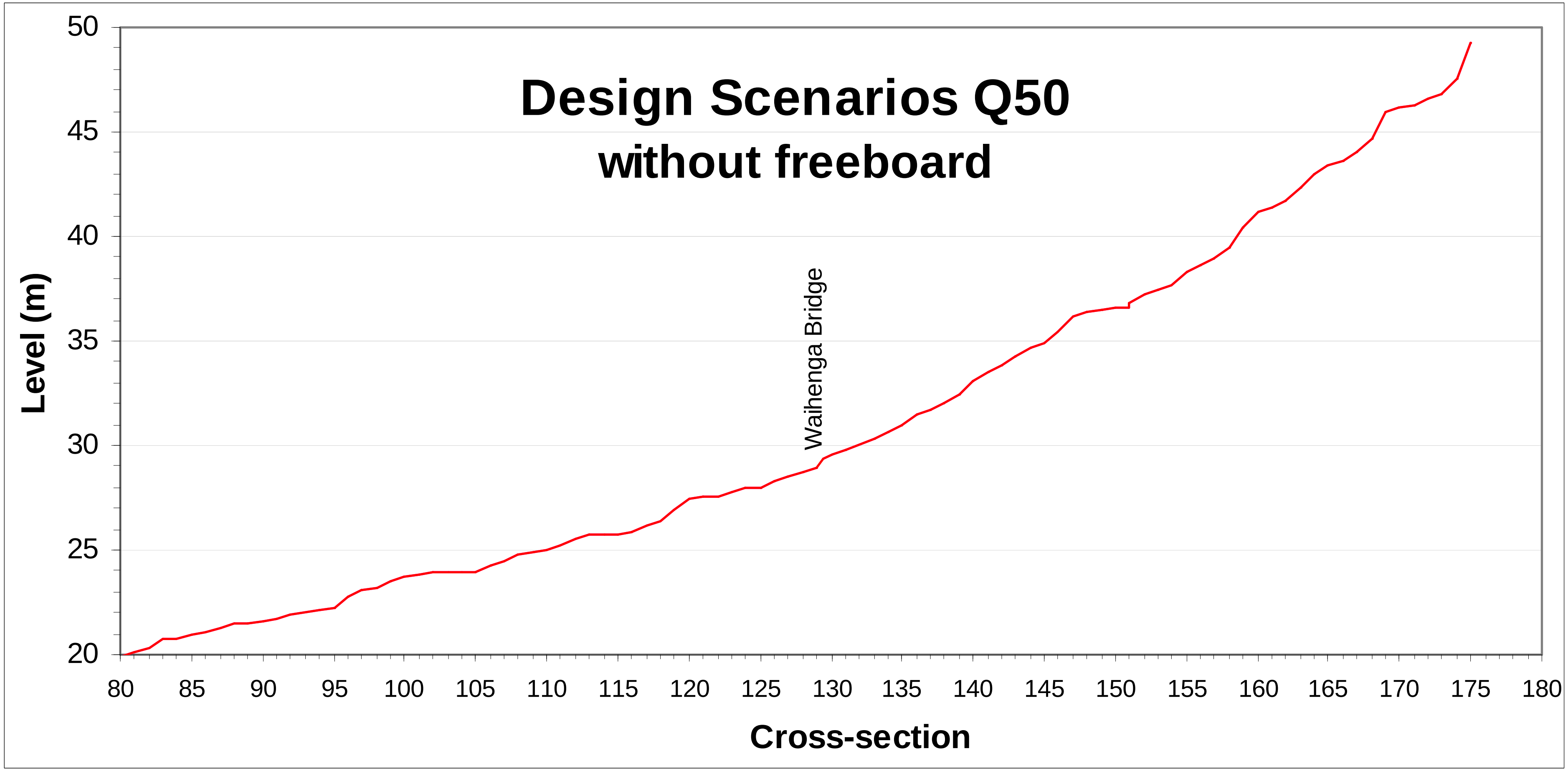


Figure 60 Peak Water Level Predictions – Q_{50} without Freeboard, Cross-sections 1 to 100

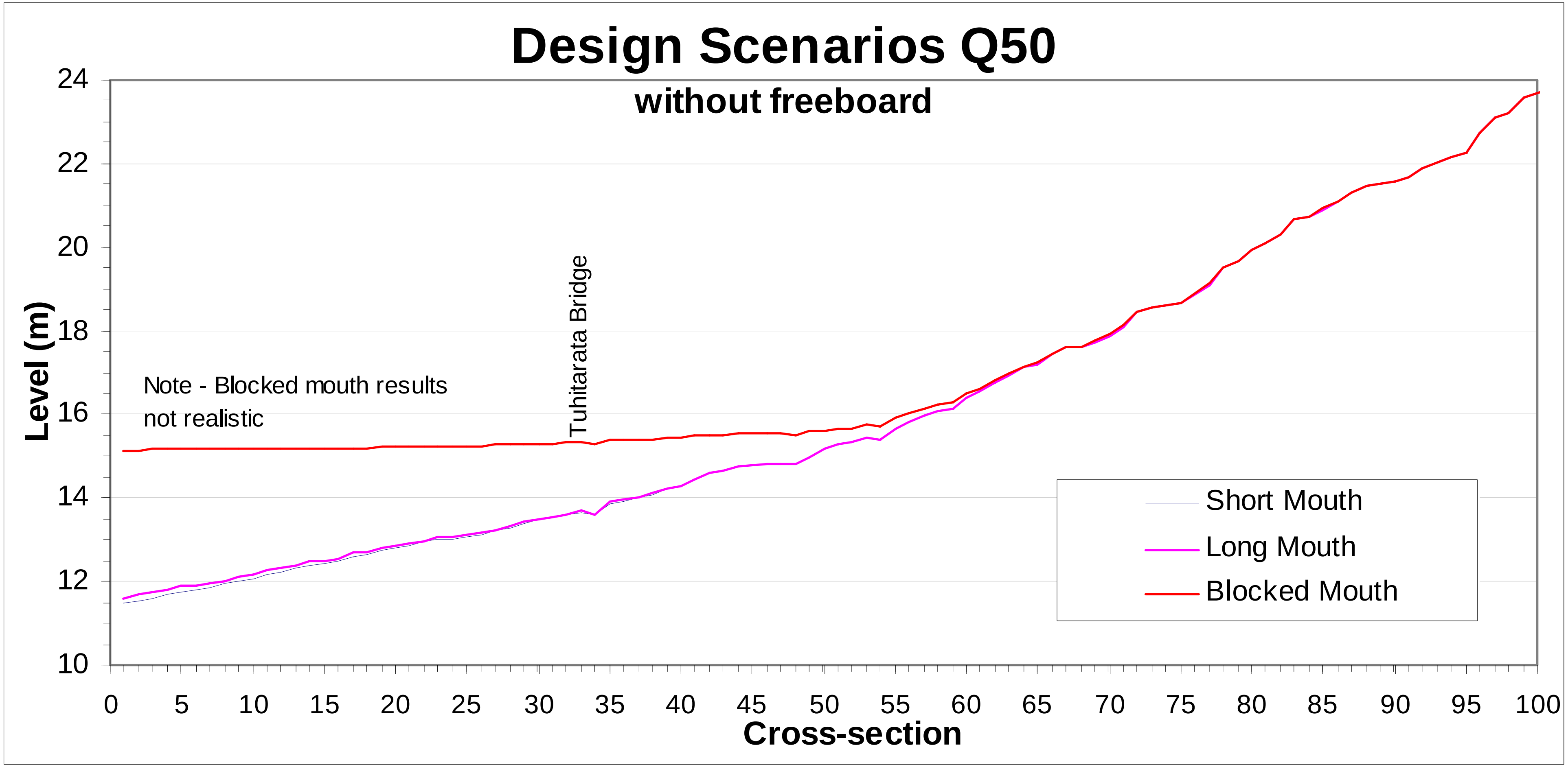


Figure 61 Stopbank Levels and Peak Water Level Predictions –Without Freeboard, Cross-sections 80 to 150

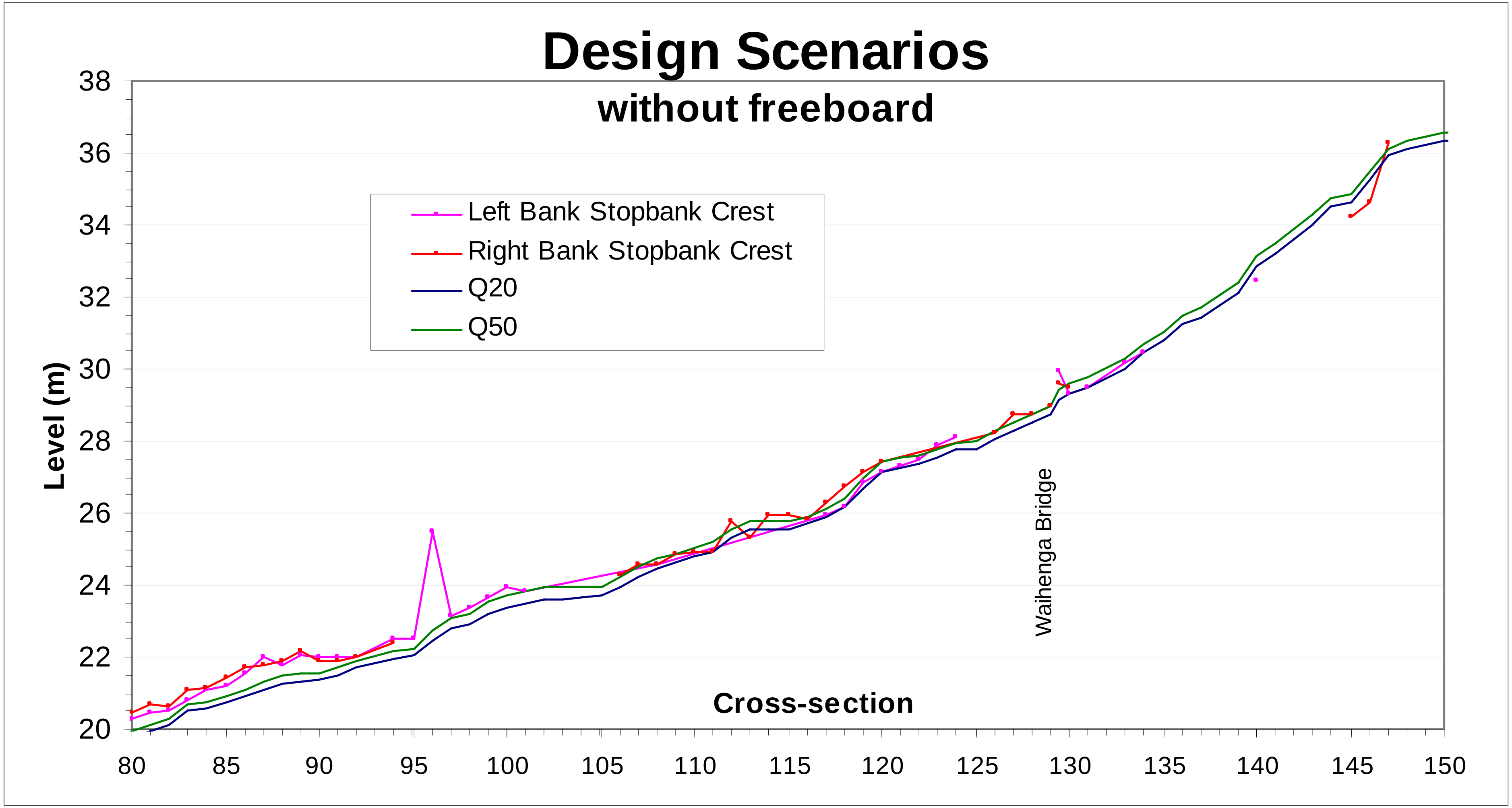


Figure 62 Stopbank Levels and Peak Water Level Predictions –Without Freeboard, Cross-sections 1 to 100

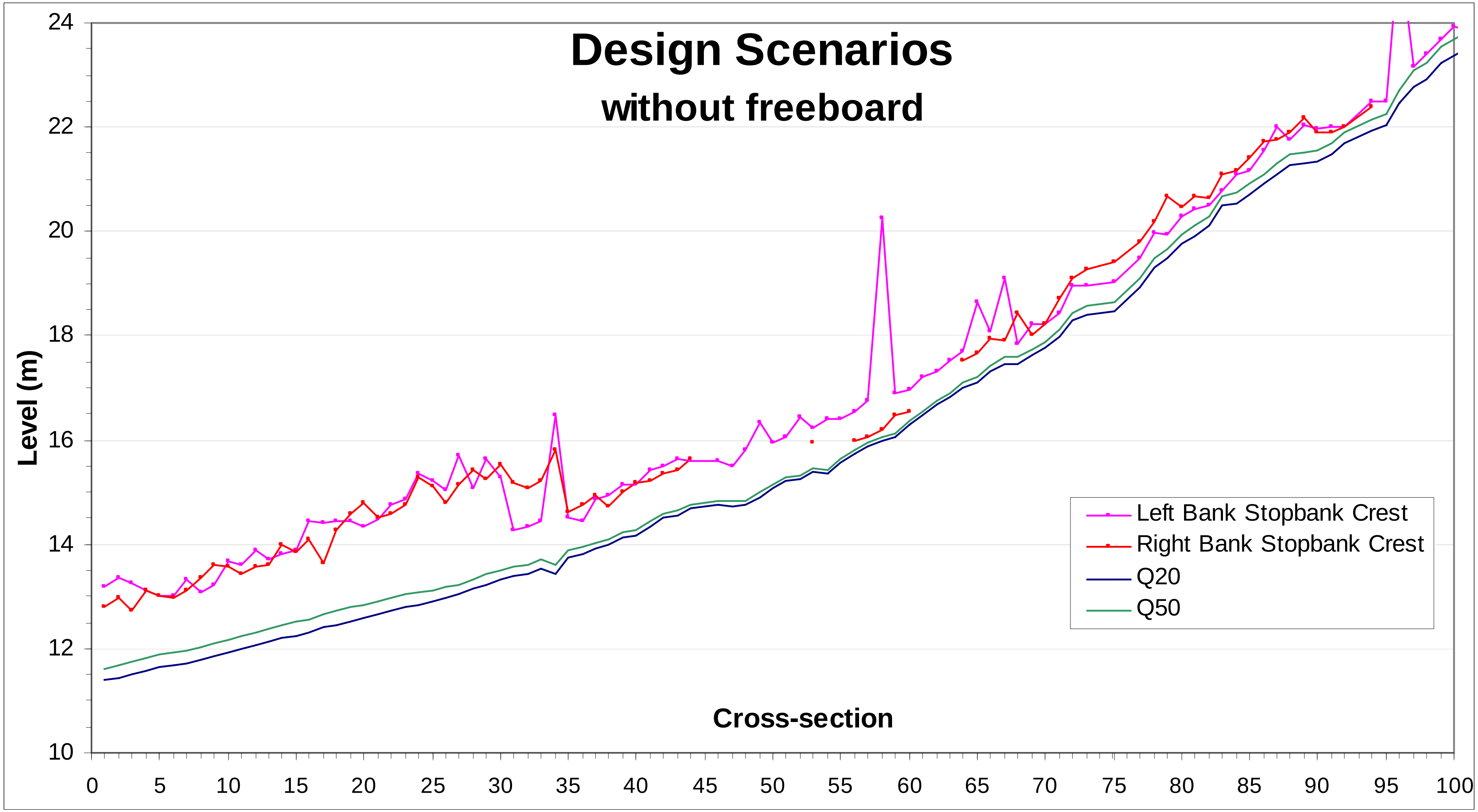


Figure 63 Stopbank Levels and Peak Water Level Predictions –With Freeboard, Cross-sections 80 to 150

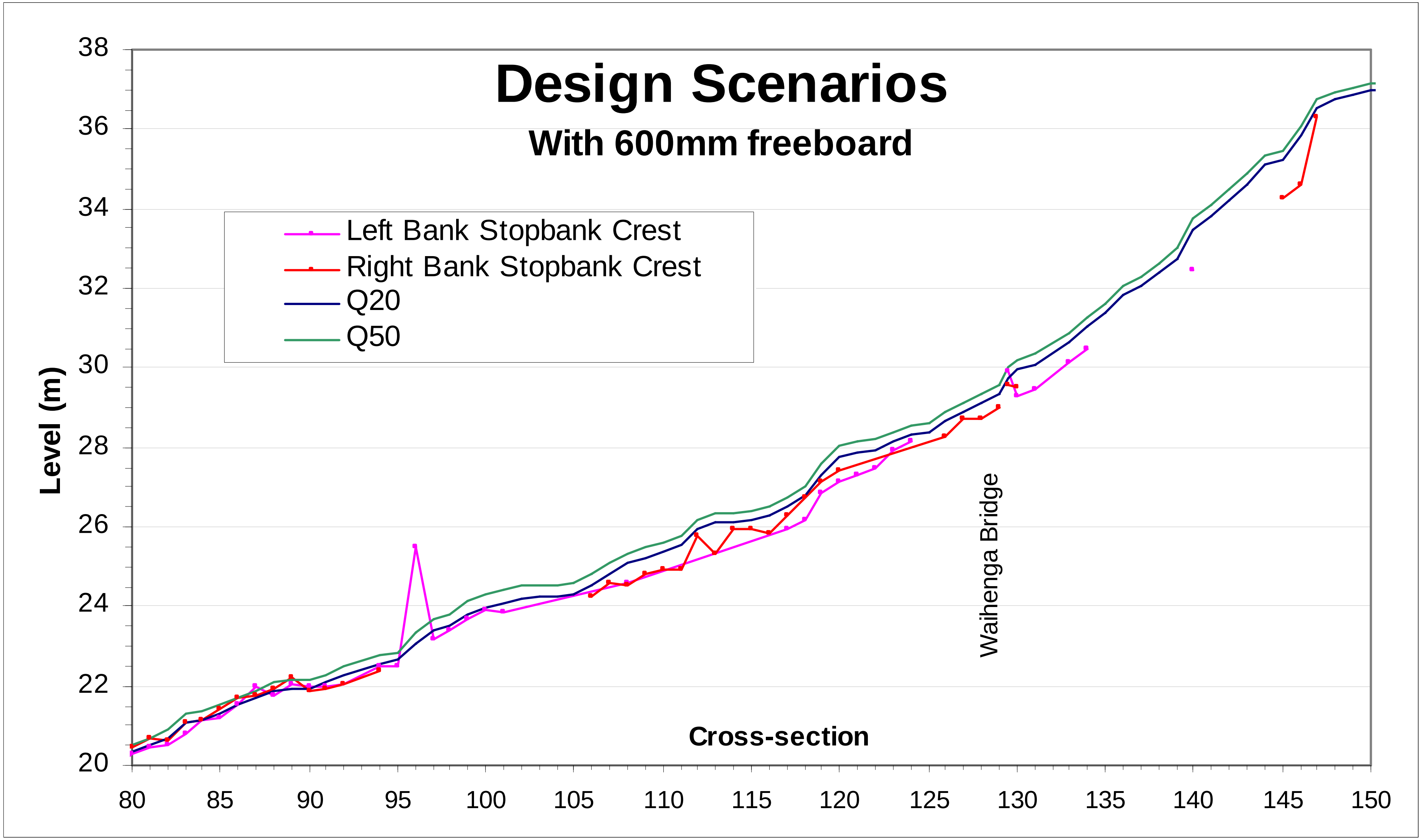
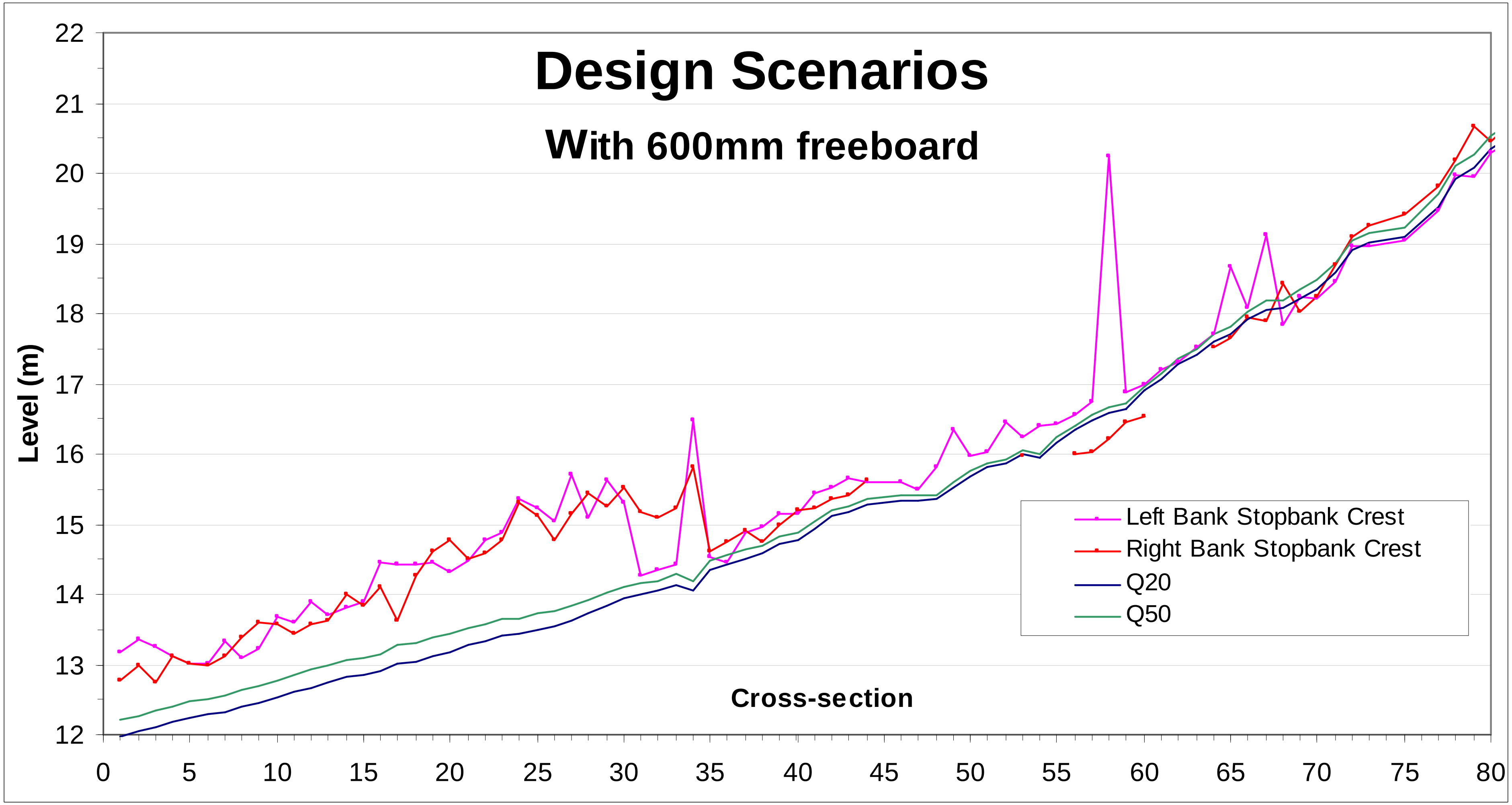


Figure 64 Stopbank Levels and Peak Water Level Predictions –With Freeboard, Cross-sections 1 to 100



8.3 Tawaha Sill Spill

The model predicts overflow into the Tawaha floodway at a Waihenga level of between 27.75m and 28m, i.e. a gauge height of approximately 4 to 4.2m (the lower levels were for the long hydrograph scenario). This is lower than recent observations of Greater Wellington staff, which suggest 4.5 to 4.7m.

As noted above, sensitivity tests were run with the Tawaha upstream sills and the river bed between sections 128 and 130 raised by 300mm. With the sills alone raised by 300mm, the overflow started at 4.2 to 4.5m. With the river bed raised by 300mm, but the sills left as originally modelled, the predicted overflow began at 3.95 to 4.15m. With both the river bed and the sills raised by 300mm, the overflow began at 4.3 to 4.5m.

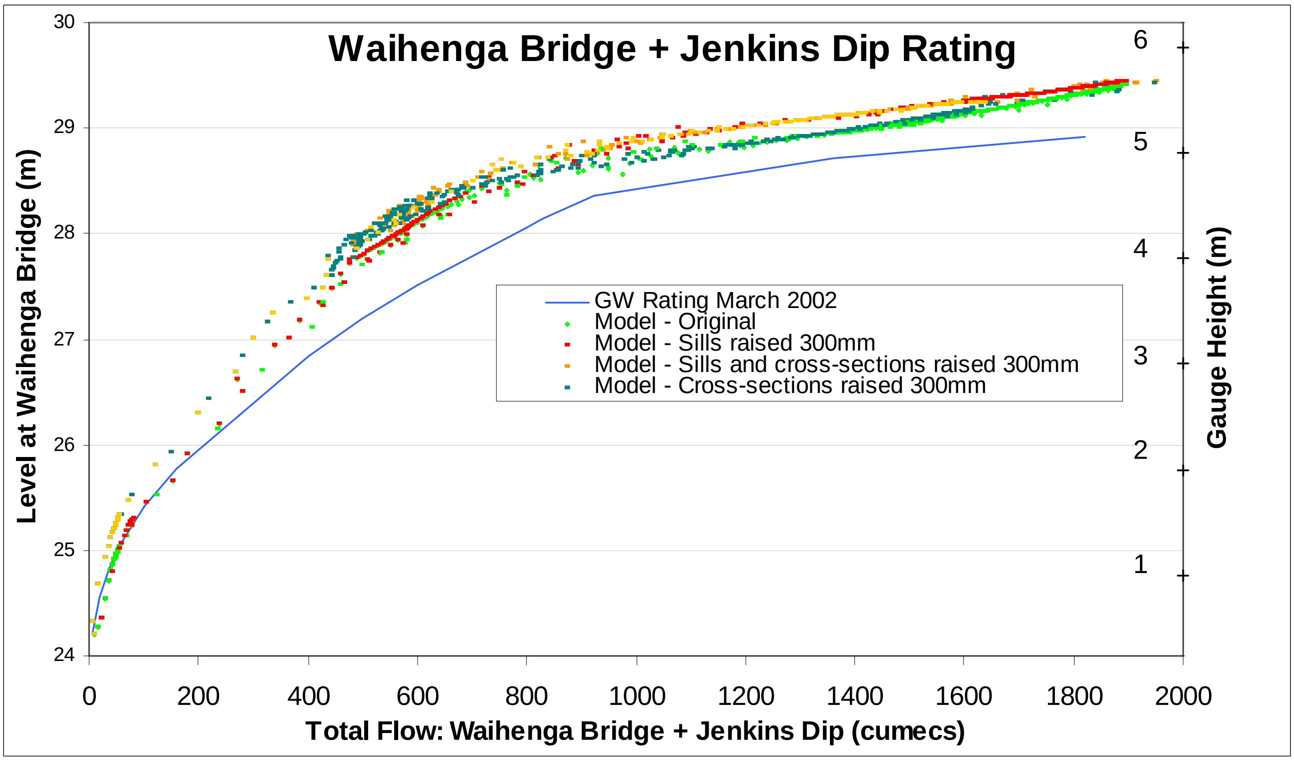
As noted in Section 4.2.4 of this report, a simple geometry has been assumed for the sills given the lack of recent detailed sill surveys. Clearly the Waihenga gauge level at which the overflow starts is sensitive to the sill level, and thus the overflow predictions could be refined with more accurate information on the sill levels.

8.4 Waihenga Rating Curve Prediction

Greater Wellington staff suspect that the Waihenga rating curve (i.e. Waihenga Bridge flow plus Jenkins Dip flow) is not particularly accurate. It is known, for instance, that the bed has been built up under and around the Waihenga Bridge and that the sill level has also been building up (and has become overgrown). Furthermore, it is difficult to perform high flow gaugings.

The model predicts a significantly different rating, as shown in Figure 61. With the sills raised by 300mm, the rating moves even further from the currently adopted Greater Wellington curve. Although the model cross-sections in the vicinity date from 1998, and the model sill levels are also based on these 1998 sections, the results do indicate that the current rating is misleading. However, with the Huangarua flows and the sill levels being not well defined, the model cannot define the rating with certainty.

Figure 61 Waihenga Rating Curve



9 Low Flow Simulations

In early 2003, river levels dropped below the minimum required for irrigation abstractions. Although the model has been designed for flood simulations and has not been calibrated to low flow conditions, some indicative simulations of low flow conditions were made, with various mouth blocking scenarios. The simulations used provisional versions of the model.

The predictions compared well with limited observations, although no detailed quantitative comparison was made, and the exercise served its purpose.

Future low flow simulations could be made with caution, but ideally some recalibration to low flow conditions would be undertaken first. Data for such work could include a series of river levels down the length of the study reach, with the time of observation noted.

10 Conclusions and Further Investigations

The LWVDS is a complex system, and this has been reflected in the modelling.

Recorded data from a period of 11 years has been used to calibrate the MIKE 11 model. Calibration has been reasonably successful, although some anomalies exist between the observations and the predictions. Suspected causes of the anomalies include the generalised assumptions made about the size and timing of inflows from ungauged tributaries. However, as flood levels are a function of the inflows and the sill levels (amongst other variables), both of which are not known with any precision, no changes to the inflow assumptions have been made.

Design flood simulations indicate that the system generally meets expected standards – i.e. stopbanks are not overtopped in a 50 year flood in the lower part of the system and in a 20 year flood in the middle part. However, between Waihenga and section 66, stopbanks are expected to be overtopped in a 20 year event.

Results indicate that the current Waihenga rating curve is in need of revision; they suggest that for a given level at Waihenga, the flow is not as great as expected.

As yet, no optimisation options have been assessed. Changes in overflow sill levels, stopbank levels, barrage rules, culvert sizes, and roadway "weir" levels could be assessed as to their effectiveness in improving the flood capacity of the system. In comparing results from the capacity simulations, in which the barrage gates were shut, with the design simulations, in which the rules of Figures 10 and 11 were in place, it was found that the latter gave lower levels in the river. This was because floodwaters could backflow into Lake Wairarapa through the gates as they were opened on the flood recession. Note also that earlier model simulations assumed the gates would be opened when Lake Onoke reached 10.6m, as opposed 11.3m in the final simulations ("priority 3", Figures 10 and 11) – those earlier trials indicated lower water levels in the river for the same reason.

Having the barrage gate rules in the model also presents the possibility for using the model for improving day to day management of the system, i.e. in low to medium flow conditions. (The model has already proved of some value in modelling low flow conditions, although in that case the gates were assumed shut). If the model is to be used in this way, however, further work to calibrate it to lower flows is recommended.

10.1 LIDAR and 2D Modelling

A one-dimensional model is suitable for modelling the bulk of the system, as the river channels and floodways are well-defined. Possible exceptions are the middle reaches of the Pukio-Awaroa floodway, where the overland flow path is wide and depths relatively shallow, and Lakes Wairarapa and Onoke.

It is possible to combine 1-D and 2-D modelling with the DHI product MIKEFLOOD. This allows 2-D models to be joined to adjacent 1-D models or to be nested within a 1-D model, using MIKE and MIKE 21 software. Given that Greater Wellington has commissioned a LIDAR survey, a detailed ground topography dataset will be available that could be most productively used with such a modelling approach. Certainly, if flooding of the areas beyond the stopbanks was to be modelled (e.g. in a "banks-down" or an overdesign event), then this approach should be considered. While modelling costs would be greater, it is probable that more confidence would be placed in the results.

Even if a 2-D modelling approach was not adopted, LIDAR data could still be used to refine the MIKE 11 model. In particular, more information on the sill geometry and on the floodway topography could be extracted.

10.2 Reporting and Documentation

All models and associated files (e.g. spreadsheets) are stored on the Greater Wellington computer network. Appendix II lists the most relevant files, including the MIKE 11 model and result files.

References

Webby, M. G., Peacock, D. and Forster, D. (2002); "Some Lessons from Modelling the Waipaoa River Flood Control Scheme, Poverty Bay". Presentation given to New Zealand Hydrological Society 2002 Symposium.

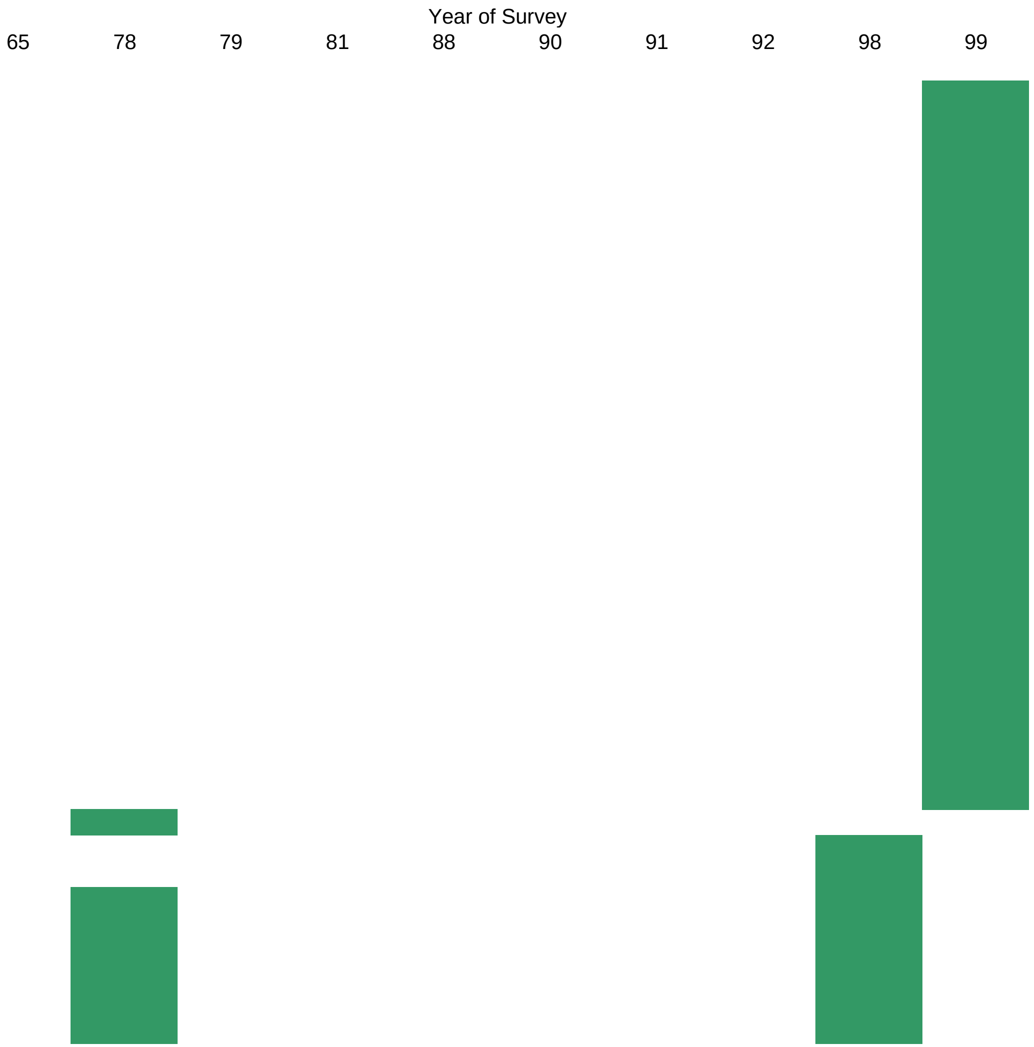
Chow V. T (1959); Open Channel Hydraulics. McGraw-Hill, Tokyo.

Wairarapa Catchment Board (1960); Lower Wairarapa Valley Development Scheme

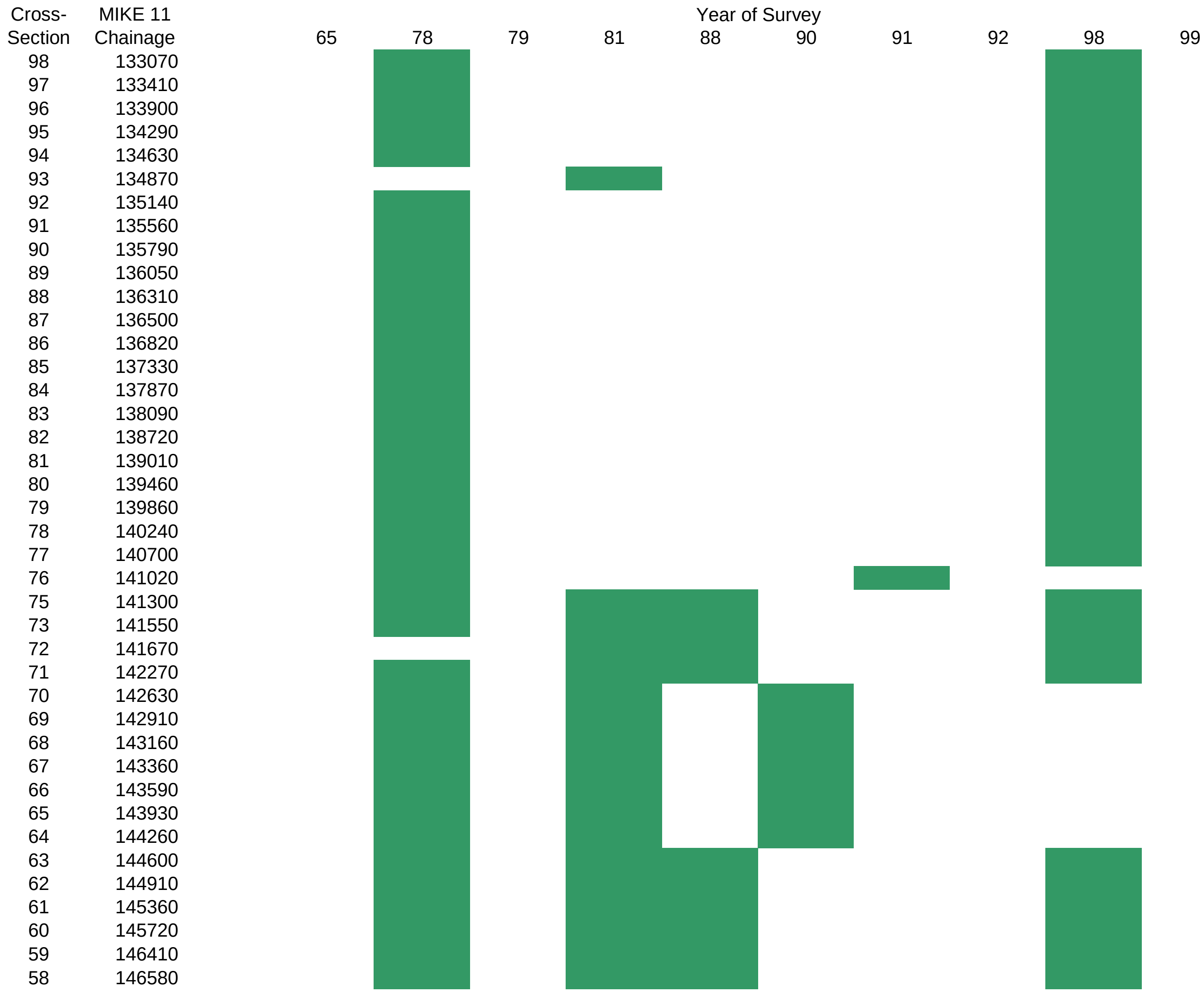
Wellington Regional Council (2001); Annual Hydrology Report 2000. Wairarapa Division

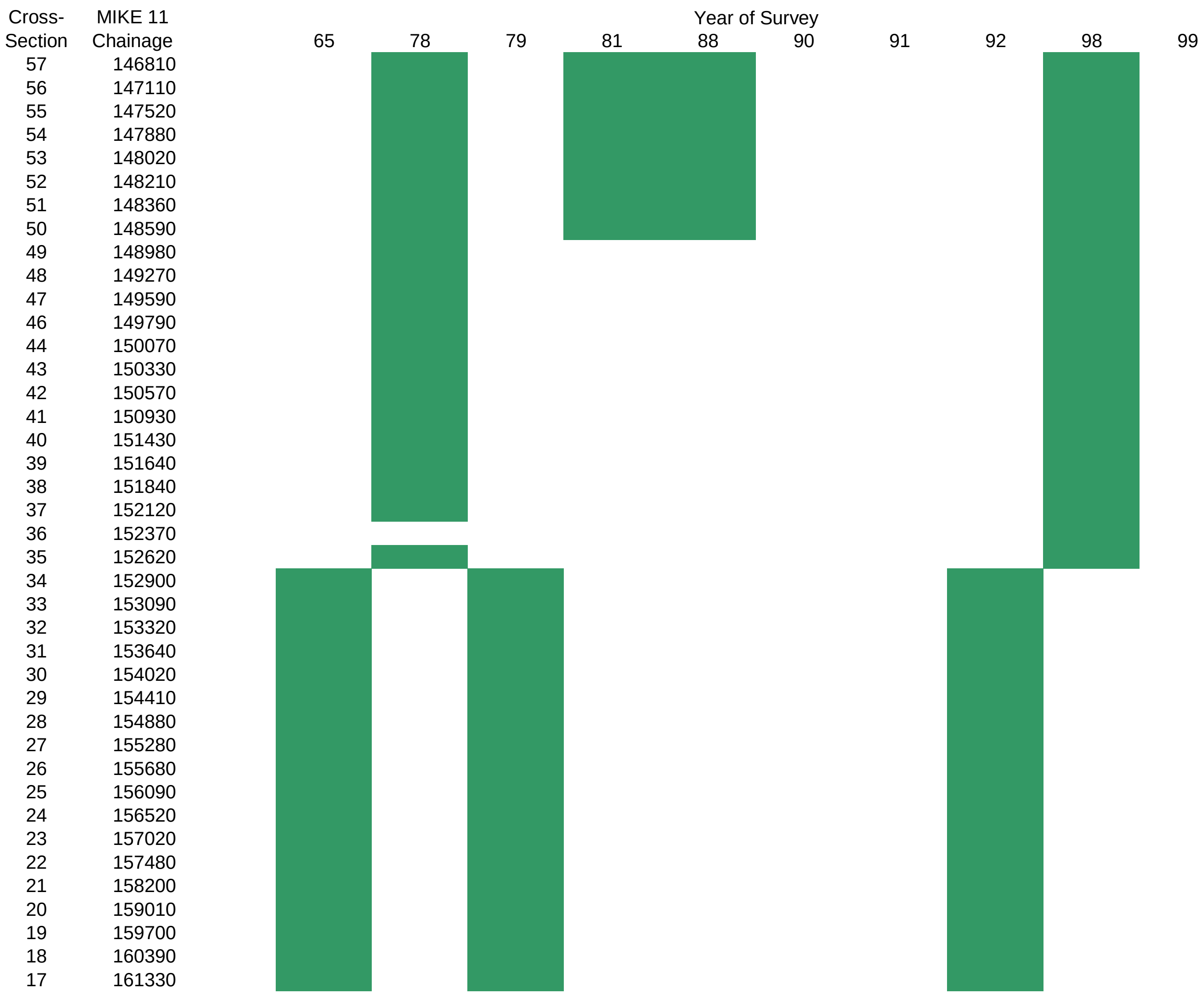
Appendix I Ruamahanga Cross-Sections

Cross-Section	MIKE 11 Chainage
181	100000
175	100400
174	101020
173	101670
172	102250
171	102870
170	103240
169	103700
168	104400
167	104730
166	105320
165	105780
164	106350
163	106950
162	107470
161	107900
160	108230
159	108630
158	109190
157	109700
156	110170
155	110620
154	111180
153	111700
152	112100
151	112600
150	113090
149	113670
148	114020
147	114470
146	114790
145	115130
144	115310
143	115790
142	116110
141	116510
140	117010
139	117530



Cross-Section	MIKE 11 Chainage	Year of Survey									
		65	78	79	81	88	90	91	92	98	99
138	117840										
137	118200										
136	118570										
135	119090										
134	119410										
133	119970										
131	120370										
130	120660										
129.5	120930										
129	121240										
128	121510										
127	121870										
126	122150										
125	122600										
124	122820										
123	123070										
122	123420										
121	123710										
120	124080										
119	124650										
118	125270										
117	125670										
116	126090										
115	126420										
114	126720										
113	127080										
112	127480										
111	127930										
110	128270										
109	128600										
108	129110										
107	129740										
106	129980										
105	130500										
104	130970										
103	131190										
102	131340										
101	131560										
100	132120										
99	132600										


Cross-Section
MIKE 11 Chainage
Year of Survey
65
78
79
81
88
90
91
92
98
99
98 133070
97 133410
96 133900
95 134290
94 134630
93 134870
92 135140
91 135560
90 135790
89 136050
88 136310
87 136500
86 136820
85 137330
84 137870
83 138090
82 138720
81 139010
80 139460
79 139860
78 140240
77 140700
76 141020
75 141300
73 141550
72 141670
71 142270
70 142630
69 142910
68 143160
67 143360
66 143590
65 143930
64 144260
63 144600
62 144910
61 145360
60 145720
59 146410
58 146580


Cross-
Section
MIKE 11
Chainage
Year of Survey
65
78
79
81
88
90
91
92
98
99
57 146810
56 147110
55 147520
54 147880
53 148020
52 148210
51 148360
50 148590
49 148980
48 149270
47 149590
46 149790
44 150070
43 150330
42 150570
41 150930
40 151430
39 151640
38 151840
37 152120
36 152370
35 152620
34 152900
33 153090
32 153320
31 153640
30 154020
29 154410
28 154880
27 155280
26 155680
25 156090
24 156520
23 157020
22 157480
21 158200
20 159010
19 159700
18 160390
17 161330

Cross-	MIKE 11	Year of Survey									
Section	Chainage	65	78	79	81	88	90	91	92	98	99
16	162060										
15	162530										
14	163050										
13	163700										
12	164230										
11	164940										
10	165490										
9	165910										
8	166460										
7	166940										
6	167080										
5	167360										
4	167810										
3	168320										
2	168900										
1	169310										
Onoke 1	170030										
Onoke 2	170620										
Onoke 3	171340										
Onoke 4	171940										

Appendix II – Files Used

Scenario	.BS11	.sim11	.nwk11	.xns11	.bnd11	.HD11	.RES11
October 1992 flood		Oct92	calib-92-98-2000	wairarapa	Oct92	Oct92-calib	Oct92
November 1994 flood		Nov94	calib-nov94	wairarapa	Nov94	Nov94-calib	Nov94
October 1998 flood		Oct98	calib-92982000	wairarapa	Oct98	Oct98-calib	Oct98
October 2000 flood		Oct2000	calib-92982000	wairarapa	Oct2000	Oct2000-calib	Oct2000
June 2003 flood		June2003	calib-92982000	wairarapa	June2003	June2003-calib	June2003
Low flow, with short mouth and variable tide		irrigation6	shortmouth	wairarapa	irrigation6	irrigation6	test
Low flow, with short mouth (raised by 1m) and variable tide		irrigation6 Mouth2	shortmouth	wairarapa-mouth2	irrigation6	irrigation6	testMouth2
Other low flow (irrigation) Simulations *(* note files used were based on state of model at the time – Feb-March 2003)*		irrigation*	irrigation*	wairarapa-2	irrigation*	irrigation*	IRRIGATION*
Capacity runs – short mouth		capacity *(overwritten)*	shortmouth *(gates assumed closed)*	wairarapa	capacity-shortmouth-*	irrigation5	CAPACITY-SHORTMOUTH-*
Capacity runs – long mouth		capacity *(overwritten)*	longmouth *gates assumed closed)*	wairarapa	capacity-longmouth-*	irrigation5	CAPACITY-LONGMOUTH-*
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge	Design		shortmouth-B *(-B indicates that barrage gate rules included)*	wairarapa	shortmouthQ50	Q50v	SHORTMOUTHQ50
Short mouth, shorter design hydrograph, constant 10.5m sea level	Design		shortmouth-B	wairarapa	shortmouthQ50-shortHG-105tide	Q50v	SHORTMOUTHQ50-SHORTHG-105TIDE
Short mouth, longer design hydrograph, variable tide with 0.5m storm surge	Design		shortmouth-B	wairarapa	shortmouthQ50-longHG	Q50v	SHORTMOUTHQ50-LONGHG
Short mouth, longer design hydrograph, constant 10.5m sea level	Design		shortmouth-B	wairarapa	shortmouthQ50-longHG-105tide	Q50v	SHORTMOUTHQ50-LONGHG-105TIDE
Blocked mouth, shorter design hydrograph, variable tide with 0.5m storm surge	Design		BlockedMouth-B	wairarapa	shortmouthQ50	Q50v	BLOCKEDMOUTHQ50
Blocked mouth, shorter design hydrograph, constant 10.5m sea level	Design		BlockedMouth-B	wairarapa	shortmouthQ50-shortHG-105tide	Q50v	BLOCKEDMOUTHQ50-SHORTHG-105TIDE
Blocked mouth, longer design	Design		BlockedMouth-B	wairarapa	shortmouthQ50-longHG	Q50v	BLOCKEDMOUTHQ5

Scenario	.BS11	.sim11	.nwk11	.xns11	.bnd11	.HD11	.RES11
hydrograph, variable tide with 0.5m storm surge							0-LONGHG
Blocked mouth, longer design hydrograph, constant 10.5m sea level	Design		BlockedMouth-B	wairarapa	shortmouthQ50-longHG-105tide	Q50v	BLOCKEDMOUTHQ50-LONGHG-105TIDE
Long mouth, shorter design hydrograph, variable tide with 0.5m storm surge	Design		longmouth-B	wairarapa	longmouthQ50	Q50v	LONGMOUTHQ50
Long mouth, shorter design hydrograph, constant 10.5m sea level	Design		longmouth-B	wairarapa	longmouthQ50-shortHG-105tide	Q50v	LONGMOUTHQ50-SHORTHG-105TIDE
Long mouth, longer design hydrograph, variable tide with 0.5m storm surge	Design		longmouth-B	wairarapa	longmouthQ50-longHG	Q50v	LONGMOUTHQ50-LONGHG
Long mouth, longer design hydrograph, constant 10.5m sea level	Design		longmouth-B	wairarapa	longmouthQ50-longHG-105tide	Q50v	LONGMOUTHQ50-LONGHG-105TIDE
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge	Design Q20		shortmouth-B	wairarapa	shortmouthQ20	Q50v	SHORTMOUTHQ20
Short mouth, shorter design hydrograph, constant 10.5m sea level	Design Q20		shortmouth-B	wairarapa	shortmouthQ20-shortHG-105tide	Q50v	SHORTMOUTHQ20-SHORTHG-105TIDE
Short mouth, longer design hydrograph, variable tide with 0.5m storm surge	Design Q20		shortmouth-B	wairarapa	shortmouthQ20-longHG	Q50v	SHORTMOUTHQ20-LONGHG
Short mouth, longer design hydrograph, constant 10.5m sea level	Design Q20		shortmouth-B	wairarapa	shortmouthQ20-longHG-105tide	Q50v	SHORTMOUTHQ20-LONGHG-105TIDE
Blocked mouth, shorter design hydrograph, variable tide with 0.5m storm surge	Design Q20		BlockedMouth-B	wairarapa	shortmouthQ20	Q50v	BLOCKEDMOUTHQ20
Blocked mouth, shorter design hydrograph, constant 10.5m sea level	Design Q20		BlockedMouth-B	wairarapa	shortmouthQ20-shortHG-105tide	Q50v	BLOCKEDMOUTHQ20-SHORTHG-105TIDE
Blocked mouth, longer design hydrograph, variable tide with 0.5m storm surge	Design Q20		BlockedMouth-B	wairarapa	shortmouthQ20-longHG	Q50v	BLOCKEDMOUTHQ20-LONGHG
Blocked mouth, longer design hydrograph, constant 10.5m sea level	Design Q20		BlockedMouth-B	wairarapa	shortmouthQ20-longHG-105tide	Q50v	BLOCKEDMOUTHQ20-LONGHG-105TIDE

Scenario	.BS11	.sim11	.nwk11	.xns11	.bnd11	.HD11	.RES11
Long mouth, shorter design hydrograph, variable tide with 0.5m storm surge	Design Q20		longmouth-B	wairarapa	longmouthQ20	Q50v	LONGMOUTHQ20
Long mouth, shorter design hydrograph, constant 10.5m sea level	Design Q20		longmouth-B	wairarapa	longmouthQ20-shortHG-105tide	Q50v	LONGMOUTHQ20-SHORTHG-105TIDE
Long mouth, longer design hydrograph, variable tide with 0.5m storm surge	Design Q20		longmouth-B	wairarapa	longmouthQ20-longHG	Q50v	LONGMOUTHQ20-LONGHG
Long mouth, longer design hydrograph, constant 10.5m sea level	Design Q20		longmouth-B	wairarapa	longmouthQ20-longHG-105tide	Q50v	LONGMOUTHQ20-LONGHG-105TIDE
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, bed xs128-130 raised 300mm	SillXSs ensitivit y	Q50raise dxs	shortmouth-B-raisedXS	wairarapa	shortmouthQ50	Q50v	SHORTMOUTHQ50-RAISEDXS
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, Tawaha sills raised 300mm	SillXSs ensitivit y		shortmouth-B-raisedSill	wairarapa	shortmouthQ50	Q50v	SHORTMOUTHQ50-RAISEDSILL
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, bed xs128-130 and Tawaha sills raised 300mm	SillXSs ensitivit y		shortmouth-B-raisedXS-raisedSill	wairarapa	shortmouthQ50	Q50v	SHORTMOUTHQ50-RAISEDXS-RAISEDSILL
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, bed xs128-130 raised 300mm	SillXSs ensitivit y		shortmouth-B-raisedXS	wairarapa	shortmouthQ20	Q50v	SHORTMOUTHQ20-RAISEDXS
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, Tawaha sills raised 300mm	SillXSs ensitivit y		shortmouth-B-raisedSill	wairarapa	shortmouthQ20	Q50v	SHORTMOUTHQ20-RAISEDSILL
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, bed xs128-130 and Tawaha sills raised 300mm	SillXSs ensitivit y		shortmouth-B-raisedXS-raisedSill	wairarapa	shortmouthQ20	Q50v	SHORTMOUTHQ20-RAISEDXS-RAISEDSILL
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, bed xs128-	SillXSs ensitivit y		shortmouth-B-raisedXS	wairarapa	shortmouthQ50-longHG	Q50v	SHORTMOUTHQ50-LONGHG-RAISEDXS

Scenario	.BS11	.sim11	.nwk11	.xns11	.bnd11	.HD11	.RES11
130 raised 300mm							
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, Tawaha sills raised 300mm	SillXSsensitivity		shortmouth-B-raisedSill	wairarapa	shortmouthQ50-longHG	Q50v	SHORTMOUTHQ50-LONGHG-RAISEDSILL
Short mouth, shorter design hydrograph, variable tide with 0.5m storm surge, bed xs128-130 and Tawaha sills raised 300mm	SillXSsensitivity		shortmouth-B-raisedXS-raisedSill	wairarapa	shortmouthQ50-longHG	Q50v	SHORTMOUTHQ50-LONGHG-RAISEDXS-RAISEDSILL

Associated Files

Spreadsheets

Superelevation.xls Superelevation calculations. Lengths of connections between main channel and berm or floodway
CAPACITY.xls, CAPACITY2.xls Full results and graphs from capacity simulations
design results.xls Full results and graphs from design simulations
sensitivity.xls Results from various sensitivity tests
Tawaha Sill overtopping and Waihenga rating.xls Estimates of overtopping levels (relative to Waihenga gauge), rating curves and sensitivity tests
irrigation.xls, irrigation-more output.xls Low flow simulations, including sensitivity tests on mouth dimensions and mouth roughness

Arc shape & apr/mxd files, etc

Shape files : *storage_areas* The extent of the ponding (no flow) areas
rua waiohine moiki sm & bm Ruamahanga benchmarks, Waiohine to Moiki
lower valley bms Ruamahanga benchmarks, Tuhitarata to Lake Onoke
cross-sections Model cross-sections
Wairarapa.mxd Working ArcMap file
Directory *M11GIS\Wairarapa* Files used and created with MIKE 11 GIS, to create localised DTMs and generate ponding area volumes

Appendix III – Design Levels

MIKE 11 Chainage (m)	Comment	Cross-section	Left Bank (m)	Right Bank (m)	Q20 (m)	+0.6m freeboard	Q50 (m)	+0.6m freeboard
118570		136			31.23	31.83	31.48	32.08
119090		135			30.78	31.38	31.00	31.60
119410		134	30.46		30.46	31.06	30.68	31.28
119970		133	30.15		30.03	30.63	30.29	30.89
120370		131	29.46		29.50	30.10	29.76	30.36
120660		130	29.30		29.34	29.94	29.61	30.21
120930		129.5	29.93	29.59	29.14	29.74	29.41	30.01
121240	Right bank lower standard	129		29.00	28.77	29.37	28.98	29.58
121510	Right bank lower standard	128		28.72	28.51	29.11	28.72	29.32
121870	Right bank lower standard	127		28.72	28.28	28.88	28.50	29.10
122150	Right bank lower standard	126		28.24	28.08	28.68	28.28	28.88
122600		125			27.79	28.39	27.99	28.59
122820		124			27.74	28.34	27.95	28.55
123070		123	27.91		27.56	28.16	27.78	28.38
123420		122	27.48		27.35	27.95	27.61	28.21
123710		121	27.32		27.26	27.86	27.53	28.13
124080		120	27.12	27.44	27.14	27.74	27.42	28.02
124650		119	26.84	27.12	26.70	27.30	26.96	27.56
125270		118	26.17	26.72	26.17	26.77	26.40	27.00
125670		117	25.94	26.26	25.91	26.51	26.12	26.72
126090		116		25.81	25.70	26.30	25.90	26.50
126720		114		25.96	25.53	26.13	25.76	26.36
127080	on edge of Tawaha Floodway	113		25.30	25.52	26.12	25.76	26.36
127480		112		25.79	25.32	25.92	25.56	26.16
127930		111		24.94	24.94	25.54	25.18	25.78
128270		110		24.92	24.78	25.38	25.03	25.63
128600		109		24.83	24.62	25.22	24.87	25.47
129110		108		24.55	24.48	25.08	24.74	25.34
129740		107		24.58	24.23	24.83	24.50	25.10
129980		106		24.26	23.95	24.55	24.20	24.80
130500		105			23.70	24.30	23.97	24.57
130970		104			23.64	24.24	23.93	24.53
131190	within wider s/b	103			23.62	24.22	23.92	24.52
131340	within wider s/b	102			23.61	24.21	23.92	24.52
131560		101	23.85		23.50	24.10	23.81	24.41
132120		100	23.92		23.37	23.97	23.70	24.30
132600		99	23.68		23.21	23.81	23.55	24.15
133070		98	23.40		22.90	23.50	23.22	23.82
133410		97	23.16		22.79	23.39	23.10	23.70
133900	RB spillway sill	96	25.47		22.45	23.05	22.71	23.31
134290	RB spillway sill	95	22.50		22.05	22.65	22.25	22.85
134630		94	22.50	22.38	21.94	22.54	22.15	22.75
135140		92	22.02	22.01	21.69	22.29	21.89	22.49
135560		91	22.00	21.91	21.49	22.09	21.69	22.29
135790		90	21.98	21.89	21.35	21.95	21.55	22.15
136050		89	22.03	22.19	21.32	21.92	21.52	22.12
136310		88	21.77	21.91	21.26	21.86	21.47	22.07
136500		87	21.99	21.76	21.09	21.69	21.29	21.89
136820		86	21.55	21.71	20.92	21.52	21.11	21.71
137330		85	21.18	21.41	20.71	21.31	20.91	21.51
137870		84	21.11	21.15	20.55	21.15	20.74	21.34
138090		83	20.78	21.09	20.49	21.09	20.69	21.29
138720		82	20.49	20.63	20.10	20.70	20.29	20.89
139010		81	20.43	20.66	19.92	20.52	20.10	20.70
139460		80	20.28	20.46	19.75	20.35	19.93	20.53
139860		79	19.94	20.68	19.48	20.08	19.67	20.27
140240		78	19.98	20.18	19.31	19.91	19.49	20.09
140700		77	19.47	19.81	18.93	19.53	19.10	19.70
141300		75	19.04	19.42	18.49	19.09	18.64	19.24
141550		73	18.97	19.26	18.40	19.00	18.56	19.16
141670		72	18.96	19.09	18.30	18.90	18.45	19.05
142270		71	18.44	18.70	17.97	18.57	18.11	18.71
142630		70	18.22	18.23	17.76	18.36	17.88	18.48
142910		69	18.24	18.02	17.61	18.21	17.74	18.34
143160		68	17.83	18.43	17.47	18.07	17.59	18.19

MIKE 11 Chainage (m)	Comment	Cross-section	Left Bank (m)	Right Bank (m)	Q20 (m)	+0.6m freeboard	Q50 (m)	+0.6m freeboard
143360		67	19.11	17.90	17.46	18.06	17.59	18.19
143590		66	18.08	17.94	17.31	17.91	17.43	18.03
143930		65	18.66	17.65	17.11	17.71	17.21	17.81
144260		64	17.71	17.53	17.01	17.61	17.11	17.71
144600		63	17.53		16.81	17.41	16.90	17.50
144910	RB spillway sill	62	17.31		16.68	17.28	16.76	17.36
145360	RB spillway sill	61	17.21		16.46	17.06	16.54	17.14
145720		60	16.98		16.30	16.90	16.37	16.97
146410		59	16.88		16.05	16.65	16.11	16.71
146580		58	20.25		15.99	16.59	16.06	16.66
146810		57	16.75		15.89	16.49	15.96	16.56
147110	RB spillway sill	56	16.56		15.73	16.33	15.80	16.40
147520	RB spillway sill	55	16.42		15.56	16.16	15.64	16.24
147880	RB spillway sill	54	16.39		15.34	15.94	15.41	16.01
148020		53	16.24	15.96	15.39	15.99	15.46	16.06
148210	RB spillway sill	52	16.44		15.26	15.86	15.32	15.92
148360		51			15.20	15.80	15.27	15.87
148360	RB spillway sill	50	16.04		15.20	15.80	15.27	15.87
148590	RB spillway sill	50	15.96		15.09	15.69	15.16	15.76
148980		49	16.35		14.91	15.51	14.99	15.59
149270		48	15.80		14.75	15.35	14.82	15.42
149590		47	15.50		14.73	15.33	14.81	15.41
149790		46	15.59		14.75	15.35	14.83	15.43
150070		44	15.59	15.63	14.69	15.29	14.77	15.37
150330		43	15.64	15.42	14.56	15.16	14.65	15.25
150570		42	15.51	15.36	14.51	15.11	14.60	15.20
150930		41	15.44	15.23	14.35	14.95	14.44	15.04
151430		40	15.15	15.19	14.17	14.77	14.27	14.87
151640		39	15.15	15.00	14.12	14.72	14.23	14.83
151840		38	14.95	14.74	13.99	14.59	14.10	14.70
152120		37	14.88	14.92	13.91	14.51	14.03	14.63
152370		36	14.46	14.75	13.82	14.42	13.95	14.55
152620		35	14.53	14.62	13.75	14.35	13.89	14.49
152900		34	16.48	15.82	13.44	14.04	13.60	14.20
153090		33	14.43	15.23	13.54	14.14	13.69	14.29
153320		32	14.35	15.09	13.44	14.04	13.60	14.20
153640		31	14.26	15.17	13.39	13.99	13.56	14.16
154020		30	15.30	15.52	13.33	13.93	13.51	14.11
154410		29	15.62	15.25	13.23	13.83	13.42	14.02
154880		28	15.08	15.43	13.13	13.73	13.33	13.93
155280		27	15.72	15.15	13.04	13.64	13.24	13.84
155680		26	15.04	14.79	12.96	13.56	13.17	13.77
156090		25	15.23	15.11	12.90	13.50	13.12	13.72
156520		24	15.35	15.30	12.84	13.44	13.06	13.66
157020		23	14.88	14.76	12.81	13.41	13.05	13.65
157480		22	14.78	14.59	12.74	13.34	12.98	13.58
158200		21	14.49	14.51	12.67	13.27	12.92	13.52
159010		20	14.33	14.78	12.58	13.18	12.84	13.44
159700		19	14.44	14.60	12.51	13.11	12.79	13.39
160390		18	14.43	14.27	12.45	13.05	12.72	13.32
161330		17	14.42	13.64	12.42	13.02	12.67	13.27
162060		16	14.44	14.09	12.31	12.91	12.56	13.16
162530		15	13.88	13.84	12.25	12.85	12.50	13.10
163050		14	13.82	13.99	12.22	12.82	12.47	13.07
163700		13	13.69	13.62	12.15	12.75	12.39	12.99
164230		12	13.89	13.57	12.08	12.68	12.32	12.92
164940		11	13.59	13.43	12.00	12.60	12.25	12.85
165490		10	13.69	13.56	11.93	12.53	12.17	12.77
165910		9	13.22	13.59	11.85	12.45	12.09	12.69
166460		8	13.08	13.38	11.80	12.40	12.03	12.63
166940		7	13.34	13.12	11.72	12.32	11.96	12.56
167080		6	13.02	12.98	11.68	12.28	11.92	12.52
167360		5	13.02	13.03	11.64	12.24	11.88	12.48
167810		4	13.11	13.13	11.58	12.18	11.81	12.41
168320		3	13.27	12.75	11.51	12.11	11.74	12.34
168900		2	13.37	12.98	11.44	12.04	11.67	12.27
169310		1	13.18	12.78	11.39	11.99	11.61	12.21