Hydraulic Modelling of the Kaituna River

Report prepared for Environment Bay of Plenty
by

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1 Introduction

Environment Bay of Plenty regularly reviews river channel hydraulics of its major rivers, in order to ensure that stopbanks continue to meet design standards specified in the river scheme asset management plans. This report describes the investigations and findings of the river channel capacity review for the Kaituna River, undertaken in 2007/08.

Together with recent reviews of hydraulics of the Raparapahoe, Waiari and Kopuaroa (major tributaries of the lower Kaituna), this report is therefore an update of the 1999/2000 Kaituna River Major Scheme Hydraulic Review (Environment Bay of Plenty, 2000).

This most recent review has been carried out in two parts. The first has been a review of the Kaituna River downstream of the Te Matai Recorder at the State Highway 2 bridge (Figure 1). The recorder gives well-defined design flows and calibration hydrographs to work with, and downstream of this point the stopbank design capacity is 1% AEP[1] plus 500mm freeboard.

The second part has been a review of the river upstream of Te Matai, to just beyond the upstream extent of the stream stopbanks (Figure 2). Over this reach the stopbank design capacity is 10% AEP, with no freeboard (Environment Bay of Plenty, 2003). As is explained later, the model used for this part actually also covered the lower portion of the river.

The project brief required that higher flows also be assessed; specifically the 200 year (0.5% AEP) and the 500 year (0.2% AEP) flood flows. Furthermore, the possible impact of climate change had to be assessed, by considering such floods, and sea level conditions, in 2030 and 2080 (as well as under present day conditions). Although not explicitly part of the brief, 10% AEP floods have been assessed as well, in order to review the stopbanks upstream of Te Matai.

At the same time as these investigations, Beca has been working on the design of the Tauranga Eastern Motorway (TEM), which is proposed to cross the Kaituna River near Bell Road. An assessment of the hydraulic impacts of the bridge crossing, and of options to lower flood levels by using some of the right bank floodplain as storage, is summarised in Appendices V and VI.

[1] Annual Exceedence Probability. By definition, a 1% AEP flood has a 1% probability of occurring in any one year. This is also known as a 1 in 100 year flood or simply as a 100 year flood. Likewise a 10% AEP flood has a 10% probability of occurring in any one year, and is also known as a 10 year flood.

Figure 1 Kaituna River and cross-section location, Te Matai to Te Tumu, plus close-up of Fords Cut area

Figure 2 Kaituna River and cross-section location, Maungarangi Rd bridge to Te Matai

2 Previous Modelling

A MIKE 11 model of the Kaituna River was first developed by Environment Bay of Plenty in 1991 (Titchmarsh, 1993), to assess inflows into the Maketu Estuary. MIKE 11 is a software program developed by the Danish Hydraulic Institute (DHI) that has been used in New Zealand for river and floodplain hydraulic analysis for a number of years. MIKE 11 uses 1-dimensional flow equations and allows a network of connected channels to be modelled; the flow within each is assumed to be one-dimensional.

In 1999-2000, as part of a capacity review of the scheme stopbanks, Matthew Surman updated the model and expanded it to cover the river upstream to Maungarangi Bridge, the canals and drains on the lower floodplain and the Maketu Estuary (Environment Bay of Plenty, 2000). Data used in the model included river cross-sections surveyed in 1997 and estuary bathymetry obtained in 1996.

The model was calibrated to floods of July 1998 and May 1999. File notes also suggest that data gathered by the University of Waikato in 1996 were used to help calibrate the model in the estuary area.

Between 2001 and 2007, that model was updated and used for further investigations into the flow between the Kaituna River and the Maketu Estuary (Wallace, 2007).

Models concentrating on each of the Waiari Stream, Raparapahoe (and Ohineangaanga) Canal and Kopuaroa Stream were developed in 2007 (Environment Bay of Plenty, 2007a, 2007b and 2007c). Those models used 2006 cross-sections for these branches, and were connected to the then Kaituna model (to provide a downstream boundary condition at the sea). Where possible, the models were calibrated to observations from the August 2006 flood.

3 Current Modelling

3.1 Model Software

MIKE 11 remains the standard modelling program used by Environment Bay of Plenty, and many other organisations in New Zealand. It has therefore been used as the principal modelling tool for this current capacity review. However, in recent years, the use of the software program MIKE FLOOD, developed by DHI to allow combined 1-D and 2-D flow modelling, has become more common. MIKE FLOOD incorporates MIKE 21 (i.e. 2-D flow equations) and MIKE 11 (1-D flow equations), allowing them to be dynamically linked during a simulation. This program is better suited to floodplain modelling than MIKE 11 alone. It still however allows the use of MIKE 11 in well-defined flow channels such as the Kaituna River. (DHI, 2008)

MIKE FLOOD is used here to model the upper part of the river, as there are large areas of floodplain that are inundated in flood events. That inundation causes marked attenuation of flows and it is desirable that this be modelled accurately.

3.2 Model data

River cross-sections were resurveyed in April 2005. As the lower part of the study catchment was affected by storms in May 2005, some of the sections were resurveyed in 2007 to determine whether the river bed had changed significantly and whether a complete resurvey was necessary. It was concluded that the 2005 bed survey would be adequate. Nonetheless, where they existed, the 2007 survey results were used in the model. That survey also included a few additional Kaituna cross-sections (near the Mangorewa confluence, near the Kopuaroa confluence and upstream of the mouth).

LIDAR spot height data for the floodplain upstream of the railway, from airborne laser surveys in 2006 and 2008, have been used to define the floodplain topography.

The cross-section at the Te Matai gauge site is from a recent (2007) gauging, supplemented on the berms with LIDAR data.

Hydrological data used in the model have also been reviewed, as discussed below.

4 Hydrology

4.1 Calibration Events

Flood hydrology information exists for the May 1999 and August 2006 flood events chosen as calibration events. At Te Matai, these are estimated to have been about 2.5% AEP (40 year) and 20% AEP (5 year) events respectively. Although the former is closer to the design flows (at least in the lower reach), it is further beyond gauged flows and so more extrapolation of rating curves is needed to estimate the peak flows in the event. Thus both have been treated as calibration events rather than one as calibration and the other as verification.

Assumptions about the inflows for each of these events are detailed in Appendix I. In brief though, the May 1999 peak flow used at Te Matai is as rated ($268m^3/s$), and differs slightly from what Surman used. The previously adopted flow estimate ($834m^3/s$) for the Mangorewa recorder site in that event has always been subject to much uncertainty and has not been used in final simulations. A value of $600m^3/s$ was used. The only other recorder site that was in use at the time was the Raparapahoe, meaning that all the other inflows are based on assumptions.

For the August 2006 event, the Waiari recorder was also in use. The peak flow at Te Matai was estimated at $158m^3/s$.

The sea level record for the May 1999 event is as used by Surman, and is from the Moturiki Island recorder. For the August 2006 event, a sinusoidal time series was fitted to high and low tide values taken from the NIWA tide forecaster[2]. This therefore will not allow for any storm surge that might have been present at the time but this is not expected to have a significant impact beyond the mouth area.

4.2 Design Scenarios

Several design scenarios have been modelled, each consisting in a particular combination of Kaituna River flow, tributary inflows, sea levels and climate change scenario. The combinations are summarised in Table 1 below, and a discussion of how the flows were estimated follows. Further details are provided in Appendix I.

[2] http://www.niwa.co.nz/services/free/tides

	Kaituna River Flow	Tributary Flow	Sea Level	Climate Change
Lower Model	1%	5%	5%	none
	5%	5%	1%	none
	0.5%	5%	5%	none
	0.2%	1%	5%	none
	1%	5%	5%	2030, mid
	5%	5%	1%	2030, mid
	1%	5%	5%	2030, high
	5%	5%	1%	2030, high
	1%	5%	5%	2080, mid
	5%	5%	1%	2080, mid
	1%	5%	5%	2080, high
	5%	5%	1%	2080, high
Upper Model	10%[1]	5%[2]	5%	none

Notes 1. Including Mangorewa River

2. 10%AEP for Parawhenuamea

Table 1 Design scenarios modelled (AEP)

4.2.1 Te Matai Flows

Surman's 1999 analysis concluded that the design 1% AEP flood at Te Matai should be reduced from 580m^3/s to 500m^3/s, largely on the basis of a regional flood curve he derived. He noted several reasons why the figure should not be reduced further, including

- The largest floods were from the 1950s and 1960s, and the accuracy of the information was not clear.
- More recent floods were from a more benign period.
- Loss of storage due to bed down-cutting and stopbank construction will tend to increase flood peaks.

In reaching this conclusion, he noted that for the period 1956 to 1999, the 1% AEP flow using an EV2 distribution was 519m^3/s. Adding the 1951 flood estimate gave a figure of 526m^3/s. He also noted that the data did not fit well with a EV1 distribution.

Available data for the intervening years 2000-2006 years has now been added to the record. Analysis of 1956 to 2006 gives EV1 and GEV flow estimates of 299m^3/s and 386m^3/s respectively for the 1% AEP flood, using L-moments to plot the curves (Figure 3). (Plotting of the data manually on a log-normal plot gives a 1% flow of about 390-400m^3/s) Reanalysis of the 1956-1999 data using this method gives EV1 and GEV values of 306m^3/s and 409m^3/s respectively. This suggests that the design flood could be brought down from the 1999 estimates. However, in view of the comments Surman made at the time, the design flow will be left at 500m^3/s.

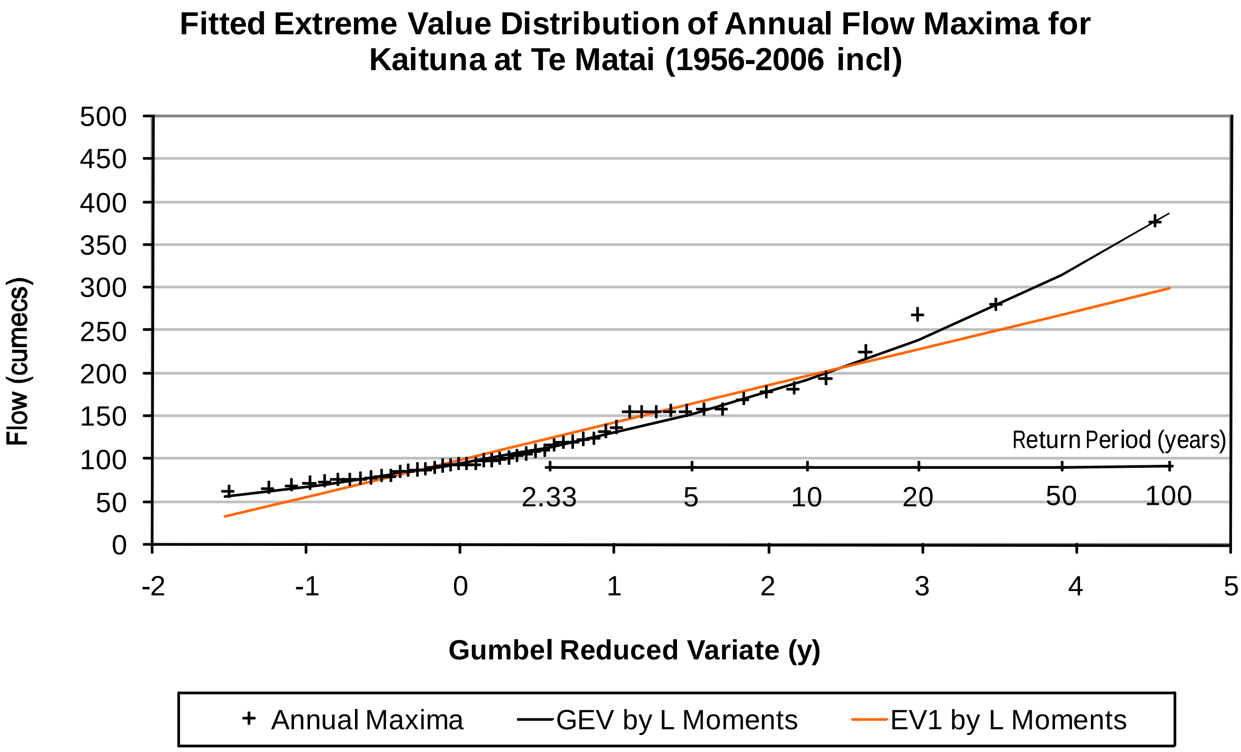

Figure 3 Analysis of Te Matai Flows, 1956-2006.

The 10% AEP Te Matai flow estimate has also been taken from Surman's analysis. In that case, he preferred the EV2 distribution results. The value adopted was 206m^3/s. As above, reanalysis of the data to 1999 and then with the addition of seven more years of data (to 2006) suggests that the value could be lowered slightly. However, the difference is small and Surman's figure has been adopted.

The 5% AEP (20 year), 0.5 % AEP (200 year) and 0.2% AEP (500 year) flows have been estimated using the same method Surman used to estimate the 1% AEP flow i.e. from the equation he fitted to the regional flow data (the graph on p7 of his report). The resulting values are 298m^3/s, 679m^3/s and 755m^3/s respectively.

(As Surman used the EV2 result for the 10% AEP flow, one could argue that the EV2 result may also be better for the 5% AEP also, considering the graph on p7 of his report. However, the result is not critical since the design stopbank heights are almost all determined by the 1%/5% AEP flow/tide combination rather than the reverse combination.)

4.2.2 Tributary Flows

The 5% AEP flows for the major tributaries as reported by Rachael Medwin (Environment Bay of Plenty 2007a, 2007b, 2007c) have been applied in conjunction with the 1% AEP Kaituna flows (Table 1). These flows are generally less than Surman used, but the hydrograph shapes he used have been retained in the design simulations. The inflows are applied at the upstream end of the relevant branch shown in Figure 5.

A number of miscellaneous inflows applied by Surman in the lower part of the catchment (e.g. Bell Road, Ford Road, Singletons Drain) have been retained in the model for the 1% AEP Kaituna flows and higher, but these do not have a significant impact on river levels. They have been ignored completely for the smaller simulations.

For the 0.2% AEP Kaituna scenario, the 1% AEP flows from the tributaries have also been taken from the 2007 Medwin reports.

In the 10% AEP Kaituna scenario (for the capacity of the upper reach stopbanks), 10% AEP flows from the tributaries have also been used. (Sea level conditions have been left at 5% AEP as these have no impact on the river levels in the upper reach.)

4.2.3 Upper River Flows

The design (and calibration) flows for the reach between Maungarangi Bridge and Te Matai are not well-defined and are very difficult to establish. The nearest recorder on the Kaituna branch is at Taaheke, about 22km upstream, and in any case flow at Taaheke is regulated by the Lake Rotoiti control gates. A recorder exists on the Mangorewa at Saunders Farm, 8km upstream of the Kaituna confluence. Other than the flow from that catchment, little is known of the flood flows entering the study area. As outlined in Appendix I, the 10% flow scenario is assumed to have an inflow of 246m^3/s to the Mangorewa branch and an inflow of 78m^3/s at Maungarangi Road Bridge which, with the model as calibrated, leads to the accepted 10% design flow of 207m^3/s.

The peak flows used in the model runs are summarised in Table 2.

Scenario	May 1999	August 2006	10% AEP	5% AEP	1% AEP	0.5% AEP	0.2% AEP
Tributary							
Kaituna @ Te Matai	268	158	207	300	500	679	755
Kaituna @ Maungarangi Bridge	180	84	78				
Mangorewa @ model upstream end	600	142	246				
Parawhenuamea	65	21	29				
Rangiuru South	12	4	7				
				5% AEP (used with Kaituna 5%, 1%, 0.5% AEP)			1% AEP (used with Kaituna 0.2%AEP)
Kopuroa	34	36	35	46			76
Raparapahoe	86	71	66	79			136
Waiari	119	38	72	86			148
Ohineangaanga	43	36	20	30			52

Table 2 Peak flows modelled (current climate)

4.2.4 Sea Levels

Sinusoidal tide time series have been applied at the sea boundaries in the model. The time series is based around a normal spring tide cycle, but is raised over the period of the flood to peak values of 1.62m and 2.06m for the 5% AEP and 1% AEP sea levels[3] respectively (in the case without climate change), as illustrated in Figure 4.

The tides have been phased so that the peak sea levels occur at around the same time as the arrival of the flood peak.

[3] All height values in this report are to Moturiki Datum.

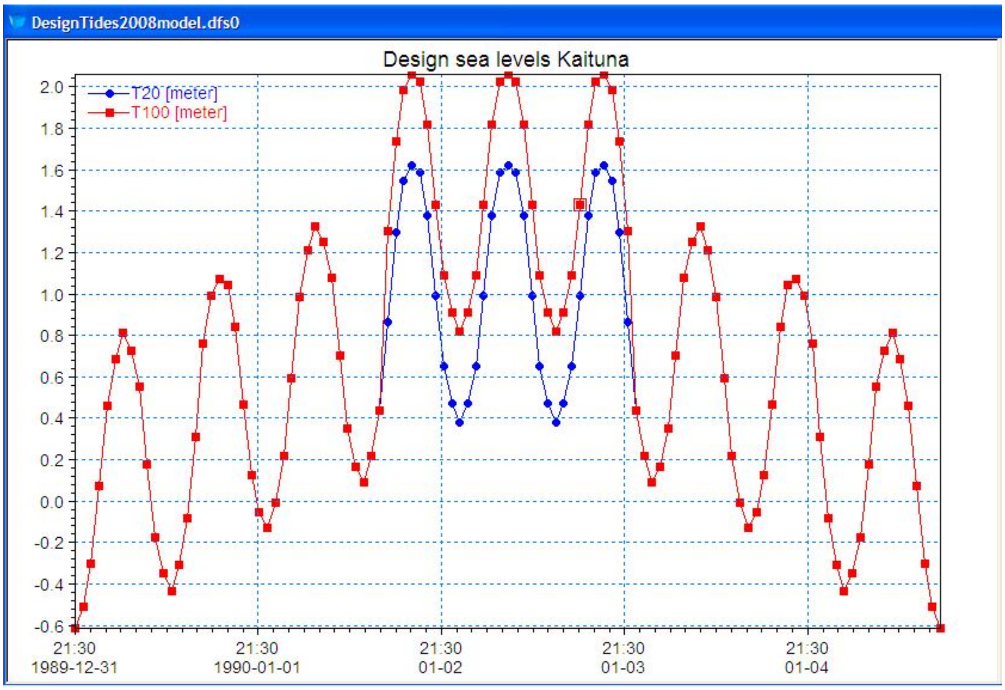

Figure 4 Design sea levels (current climate)

4.2.5 Climate Change

Climate change over the next century is likely to increase sea levels. Guidance from the Ministry for the Environment in 2004 recommended that future sea level rises of 0.2 m by 2050 and 0.5 m by 2100 (relative to 1990 levels) are used (NZ Climate Change Office, 2004c). In this exercise, it is assumed that the 2080 rise is 0.49m (previous Environment Bay of Plenty practice) and the 2030 rise is 0.2m.

(Subsequent to the modelling exercise, further guidance was released by the Ministry for the Environment (2008b). That recommended that a base sea level rise of 0.5m should be used for planning and decision timeframes out to the 2090s (2090–2099). The guidance also recommended that the potential consequences of at least 0.8m sea level rise should be assessed. This is perhaps the subject of further investigations.)

A second, but less quantified adverse effect of global warming is that the frequency and magnitude of high intensity rainfalls are expected to increase. The Ministry for the Environment also provides guidance, based on NIWA research, for estimating future design flows under climate change scenarios (New Zealand Climate Change Office, 2004b).

Most recent estimates of increase in rainfall depths per ºC recommend:

- 100 year return period events: 8% (independent of duration)
- 20 year events, 48hr duration (assumed for Kaituna river, as per Surman’s report): 7.1%
- 20 year, 6 hour duration (assumed for tributaries): 7.4%
  (Ministry for the Environment, 2008a)

Two flow change scenarios - medium and high, based on the “medium” and “high” scenarios for increase in mean annual temperature - have been evaluated.

According to the 2004 guidance, the predicted increases in mean annual temperature to 2030 in the Bay of Plenty area are 0.7°C and 1.3°C for the medium and high scenarios respectively. For 2080, the corresponding figures are 2°C and 3.8°C.

(More recently, the 2008 guidance uses mid-range projections of 0.9°C and 2.1°C from 1990 to 2040 and 2090 respectively. The range of projections is from 0.2°C to 2.4°C for 2040 and from 0.6°C to 5.5°C for 2090. These were not available at the time of modelling, so the 2004 figures have been used.)

Normally, one could expect the resulting percentage increase in runoff, i.e. in design flows, to be slightly more than the increase in rainfall. As the catchment geology here is largely pumice, it is assumed that the increase in design flow equals the increase in runoff. Design flows for each of the boundary conditions have been increased accordingly.

5 Lower Model Network

The lower model extends from the Te Matai recorder to the sea at Te Tumu and at the Maketu Estuary mouth.

The MIKE 11 model network is based on Surman's model and so includes the main tributaries as well as a number of floodplain, drain and pump branches. Some of these smaller branches have been removed to simplify the model (and as they are not the focus of this study), but further simplification would also be possible.

The Maketu Estuary has been retained as a separate channel in the branch, even though clearly in reality the flows in it would not be one-dimensional, as earlier modelling indicated that whether a 1-d or 2-d representation of the estuary was used made little difference to flood levels in the Kaituna River (Wallace, 2007). Initial modelling also showed that the river flood levels were not dependent on the alignment of the estuary mouth (which changes over time).

Nonetheless, a more detailed 2-d model of the estuary and lower river is being developed by DHI as part of further Maketu Estuary investigations, and that will most likely be better suited to predicting design levels for the Estuary. (Hence, for now, the capacity of the Maketu Estuary stopbanks has not been finalised.)

Model branches for the Kopuaroa, Waiari, Raparapahoe and Ohineangaanga have been taken from the separate and recent studies of those tributaries. These replace the model branches in the Surman model.

Areas on the left bank downstream of Bell Road (cross-section 12) and on the right bank near section 3 are not adequately defined in the river cross-sections. With the aid of 1m contours from photogrammetric work (2006), cross-sections have either been extended or had storage added to them or, on the left bank from sections 11 to 9, a separate floodplain branch added.

Branches representing spillage to the floodplain have also been modified or added. The level at which water spills has been determined from the surveyed stopbank profiles, rotated to allow for the slope of the river flood level profile, for the existing situation.

A separate version of the model has also been built, representing the design situation where stopbanks are topped up to prevent water spilling in the 1% AEP event. For this, the links between the river and the floodplain have been set much higher, so that water would never spill.

Figure 5 shows the MIKE 11 model used in the combined upper and lower model (see section 6). The lower model alone is that portion downstream of the point marked (Te Matai Recorder).

6 Upper Model

The upper model is an extension of the lower model, upstream to Maungarangi Road bridge (Figure 5). The model also extends 2km up the Mangorewa River and the Parawhenuamea Stream.

Cross-sections upstream of Te Matai are relatively sparse and do not necessarily represent the floodplain and berm flow well. For this reason, and as LIDAR data cover the reach, a MIKE FLOOD (1-D + 2-D) model has been built, with the 2-D expected to better represent the berm flow and more accurately represent the storage available. The MIKE 21 component covers the floodplain upstream of the railway at Te Matai (Figure 6). A cell size of 10m has been used. As the floodplain inundation is not the focus of this study, a constant floodplain resistance is adequate (a value of 0.077 has been assumed).

The right bank between the State Highway and the railway, modelled as a storage pond in the lower model, is included in the MIKE 21 component in the upper model. Other than that, and some insignificant adjustments required in some of the lower floodplain branches (certain warning messages received in MIKE 11 simulations become fatal in MIKE FLOOD simulations), the lower part of the upper model is the same as the lower model.

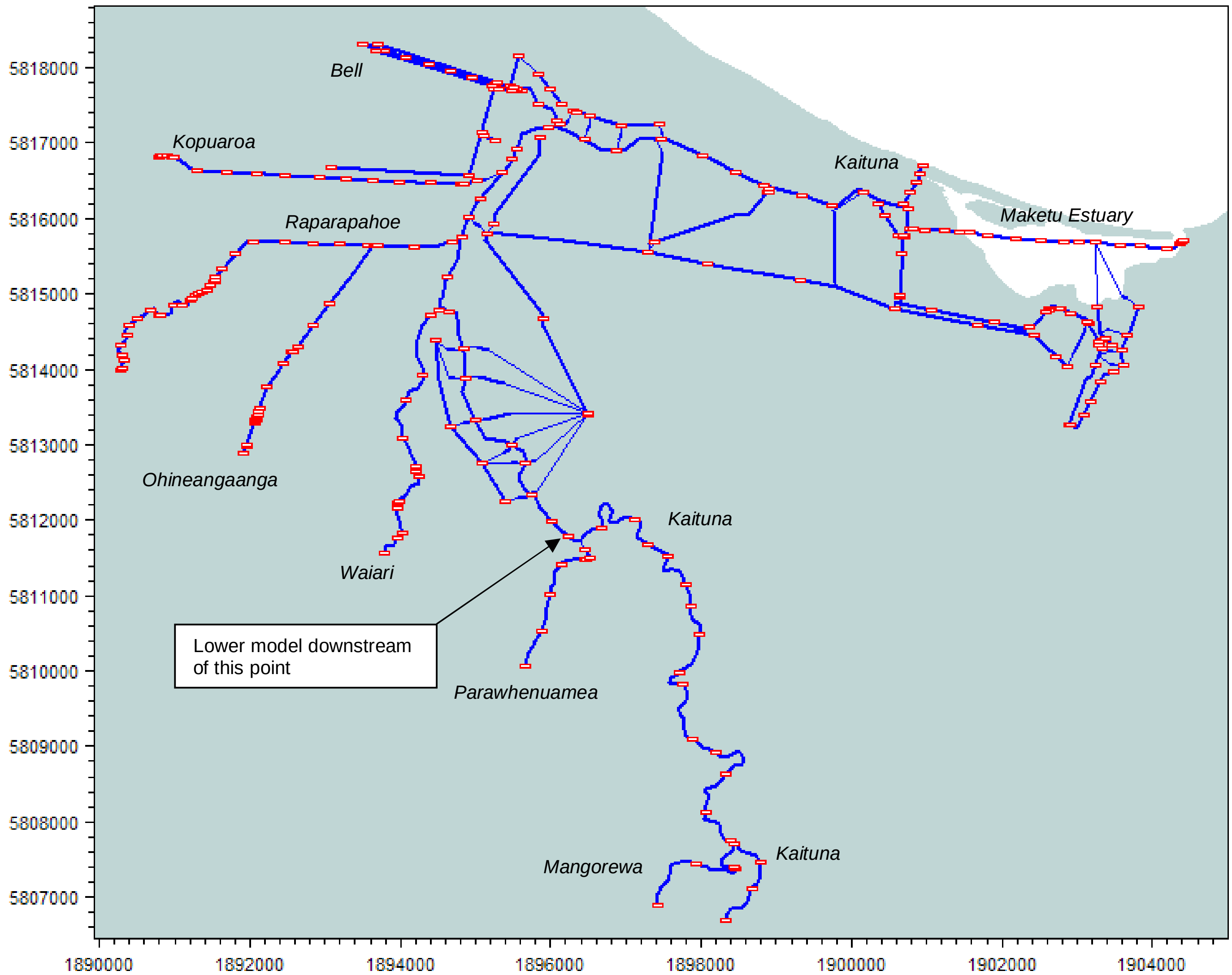

Figure 5 MIKE 11 model network (combined upper and lower model)

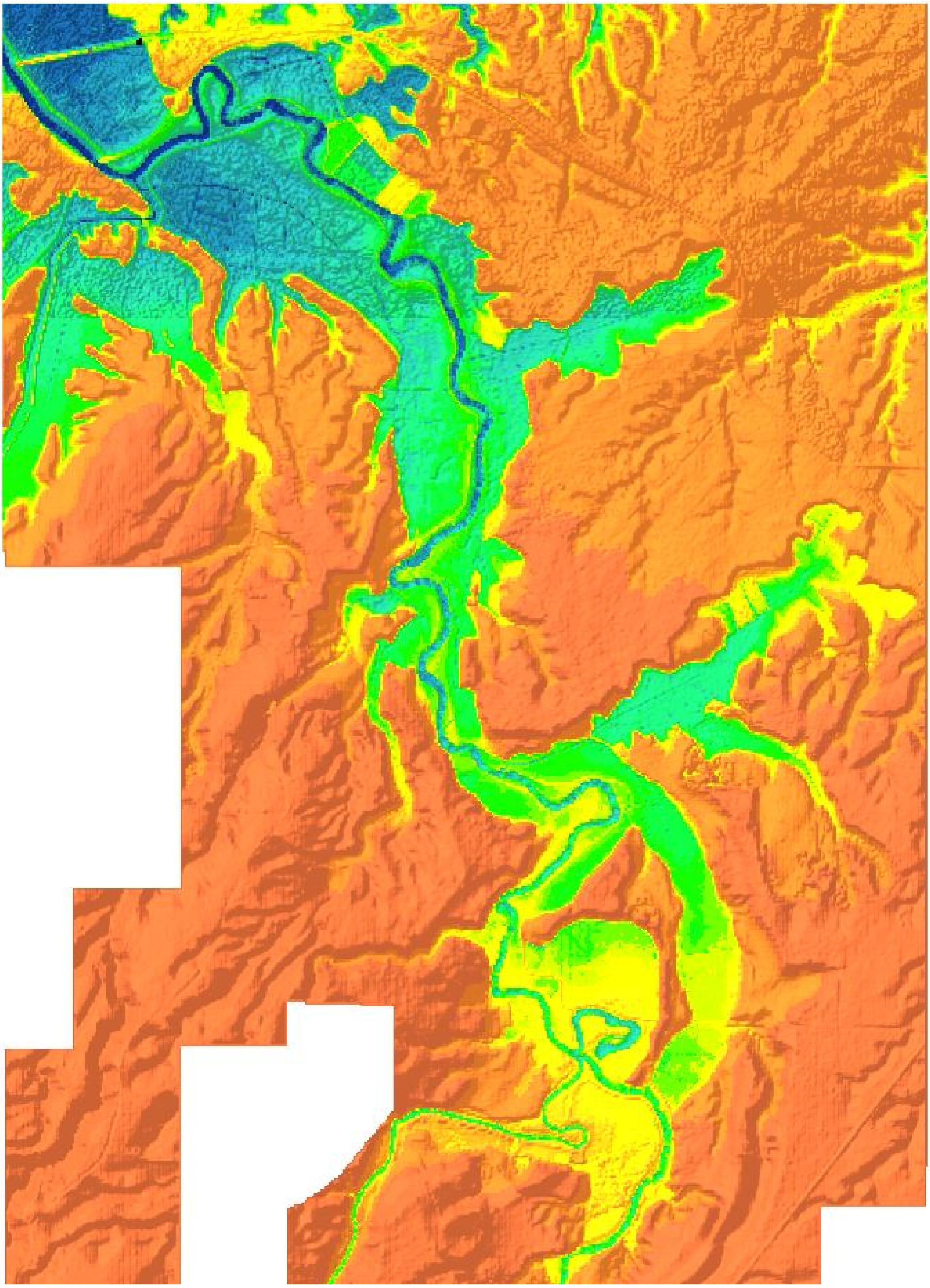

Figure 6 MIKE 21 model area, upper model (colour coded topography, from blue (low) to orange (high))

7 Calibration Results

7.1 Lower Model

The lower reach of the river has been easier to calibrate than the upper reach, as the flows at Te Matai are better defined. Nonetheless, there have been no high flow gaugings. Furthermore, calibration results show some sensitivity to the assumptions regarding the ungauged tributary inflows (e.g. Kopuaroa).

Some uncertainty also surrounds the model upstream of the railway. The May 1999 event required a high Mannings n to fit the observed data (Figure 5), yet the August 2006 event was overpredicted there using the same roughness values. One possibility is that the railway bridge causes upstream levels to rise in large flood events. However, according to Environment Bay of Plenty hydrology staff, this is not thought to be the case. Surman, in noting the significant drop across the bridge in May 1999 indicated by debris levels, also concludes that there is no reason to suspect that the bridge is a particular constriction. Further flood observations and data would be useful in resolving this.

Good flood level information exists for both the May 1999 and August 2006 events. Figures 7-9 show the model peak level predictions compared to measured peak levels down the river. Just considering the peak debris levels (i.e. ignoring the values recorded by the automatic level/flow recorder, but including those debris levels noted as being of lesser confidence), the model gives an average absolute error of 0.08m for the May 1999 event and 0.09m for the August 2006 event.

Figure 7 shows the Mannings n values adopted in the calibration. It can be seen that generally, slightly higher values were used than Surman adopted.

Recorded water level hydrographs Clarkes (upstream of the Kopuaroa Canal) compare well with the model predictions (taking the average of the results at cross-sections 15 and 16). Predictions at Ford's Cut are not quite as good, but reasonable nonetheless. One likely reason for the underprediction in the August 2006 event is that no storm surge was applied to assumed sea levels, as noted earlier (Figures 10-13).

Further plots of calibration results, for water levels recorded on the recession of the August 2006 event, are given in Appendix II. Model predictions generally show good fit to these data.

Note that the May 1999 simulation used the 1997 Kaituna River cross-sections. It was not considered important to replace the 2006 tributary cross-sections in the model with earlier ones however.

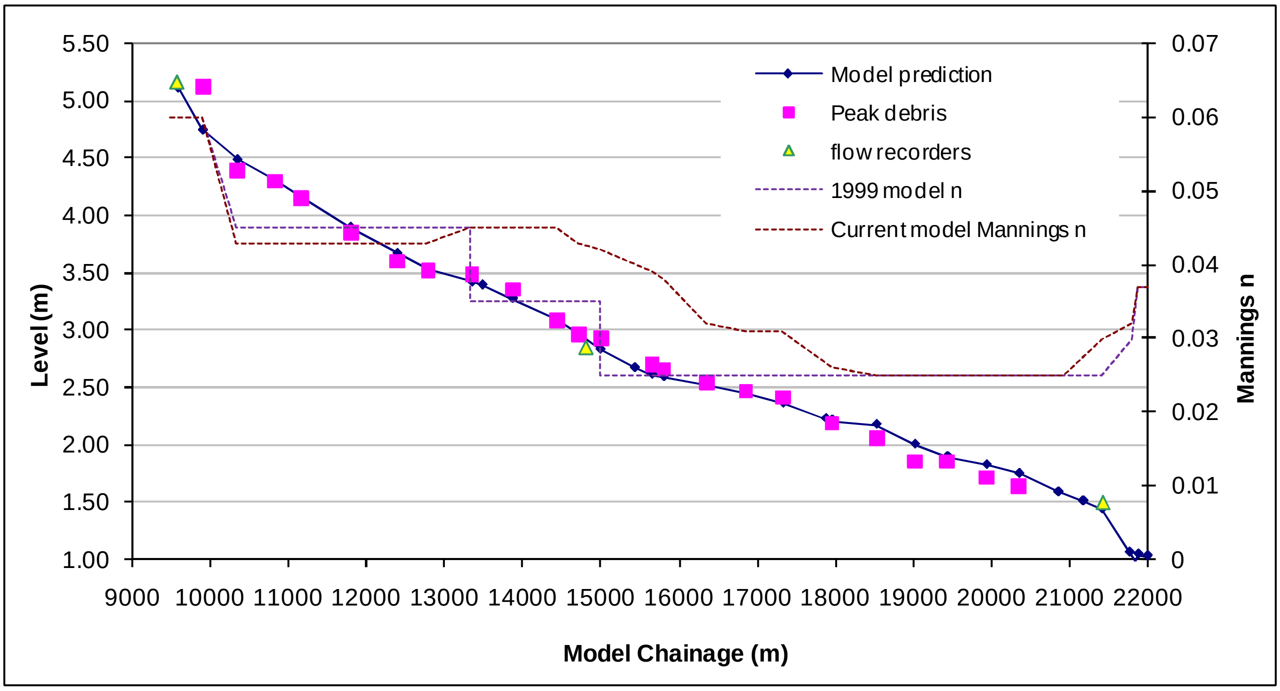

Figure 7 Model calibration, peak flood levels, May 1999 event.

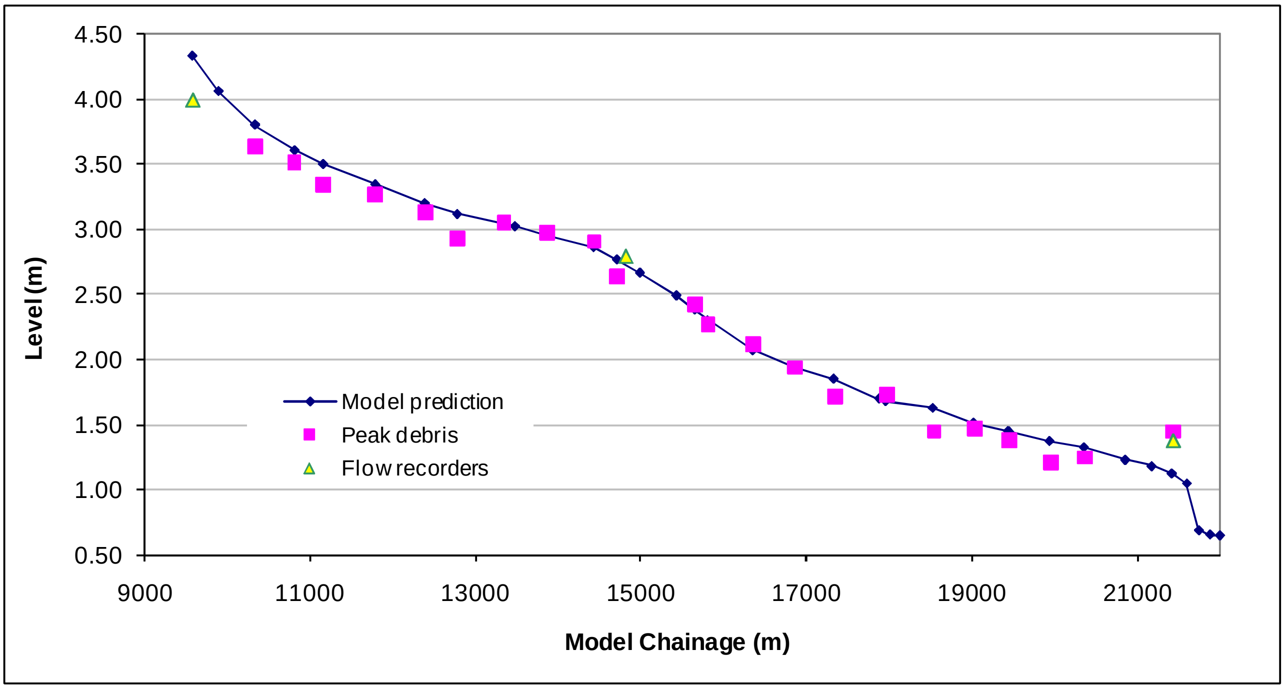

Figure 8 Model calibration, peak flood levels, August 2006 (all recorded points)

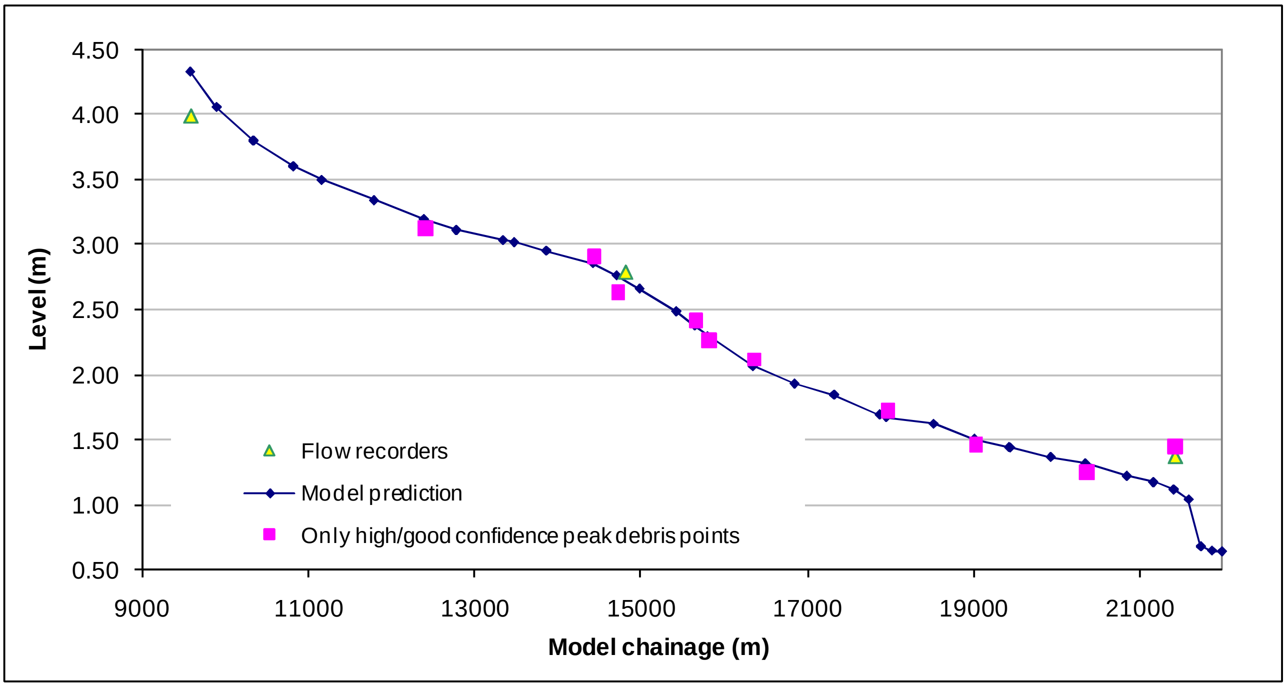

Figure 9 Model calibration, peak flood levels, August 2006 (only those debris marks noted as being at least good confidence).

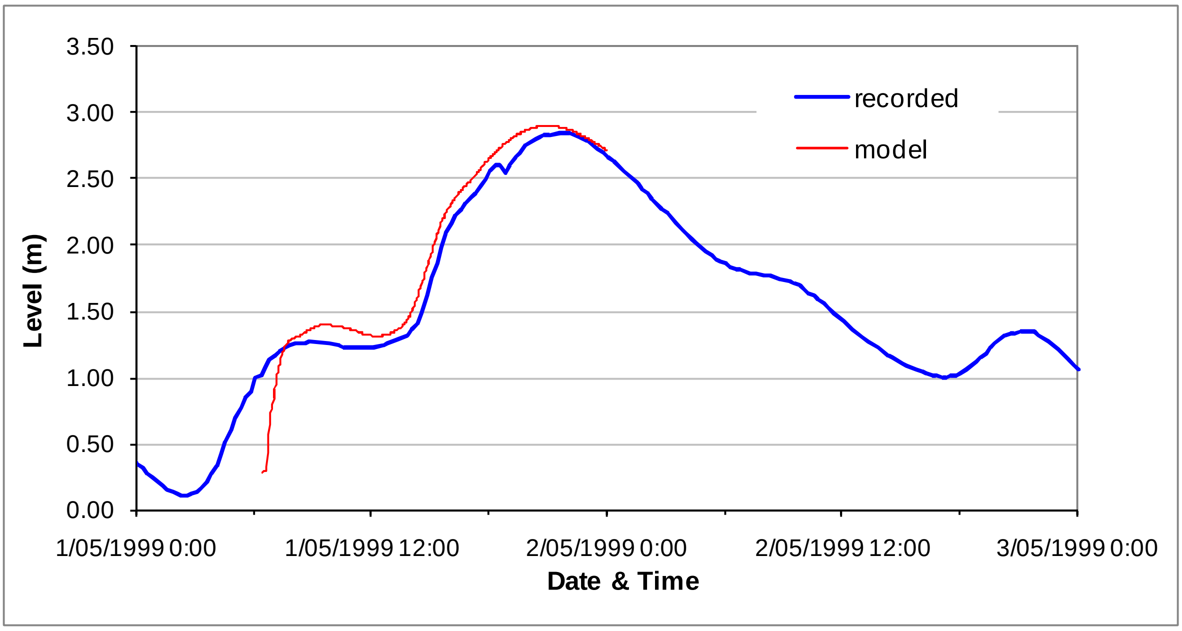

Figure 10 Model calibration, Clarke's Recorder site, May 1999 event.

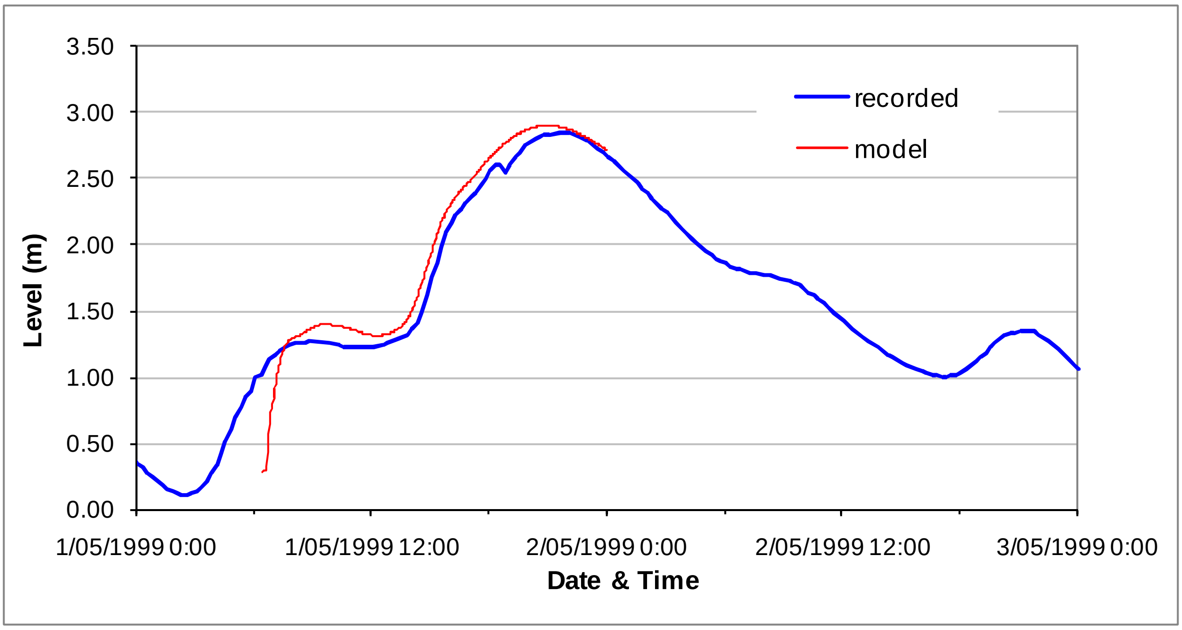

Figure 11 Model calibration, Fords Cut Recorder site, May 1999 event.

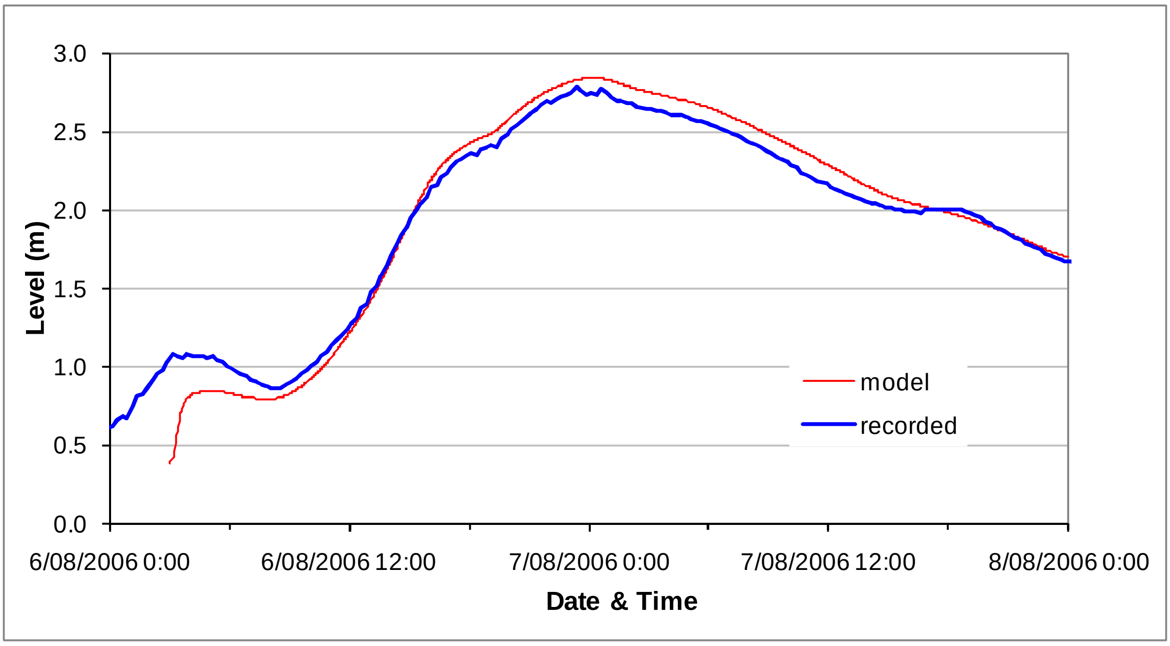

Figure 12 Model calibration, Clarke's Cut Recorder site, August 2006 event.

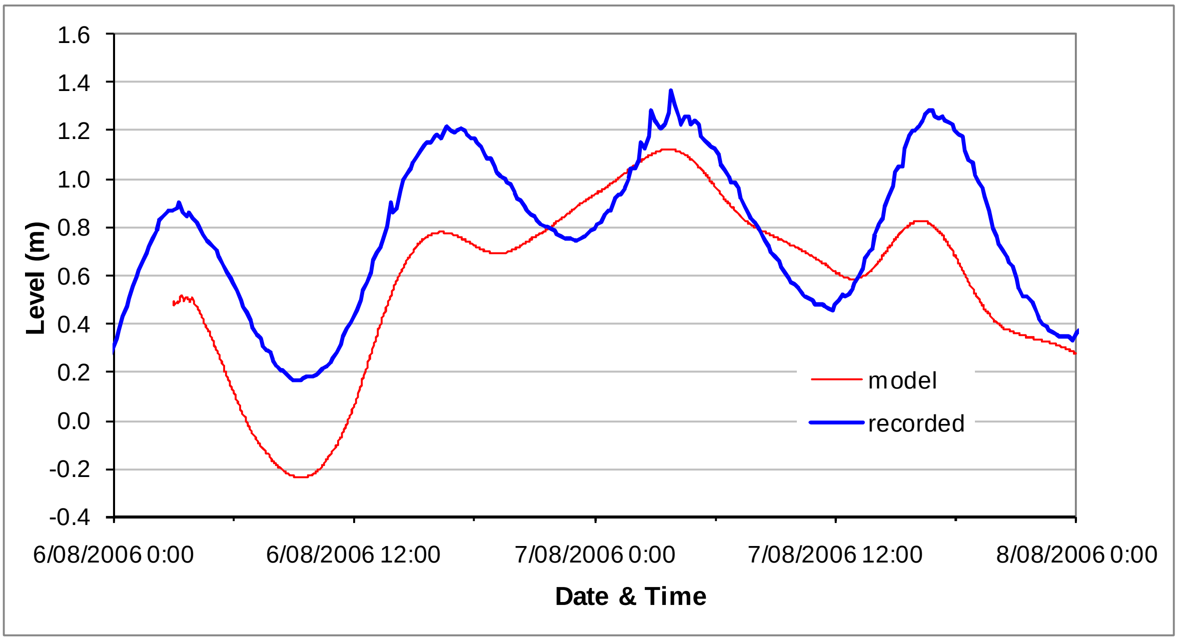

Figure 13 Model calibration, Fords Cut Recorder site, August 2006 event.

7.2 Upper Model

Again, good peak flood level information exists for both the May 1999 and August 2006 flood events in the upper reaches. Unfortunately, as discussed earlier, this is not matched with reliable information on flows. An attempt at calibrating the model was made, but clearly the results are subject to uncertainty.

The flows adopted, after several trials with both flow and channel resistance, are as described in Section 4.2.3 and Appendix I.

Figure 14 shows the observed and predicted peak levels for the 1999 event. Ignoring the recorded values at cross-sections 53 and 48, which appear to be low, the average absolute error is 0.16m. The observed and predicted peak levels for the 2006 event are shown in Figure 17. Ignoring the recorded value at cross-section 48 which is clearly wrong, the average absolute error for that event is 0.19m.

For both flood events, the predicted hydrographs at Te Matai show the peak somewhat earlier than actually occurred (Figures 15, 16, 18 and 19). As the floodplain storage was modelled (excepting spillage into the Waiari catchment in May 1999), this error is not likely to be due to attenuation of the floodwave by floodplain storage. It is more likely to be due to the assumed Kaituna inflows at Maungarangi being too early. Those assumed hydrographs were based on scaled versions of the Mangorewa, even allowing for some travel time, with the implicit assumption that the rainfall and runoff peaked in the catchment to Maungarangi at the same time as in the Mangorewa. Results suggest that this may not have been the case. Hyetographs from the few rainfall stations nearby do not reveal any particular trend to the timing of the rainfalls over the area in those storms however, and in the absence of any more information no change has been made to the timings of the model inflows.

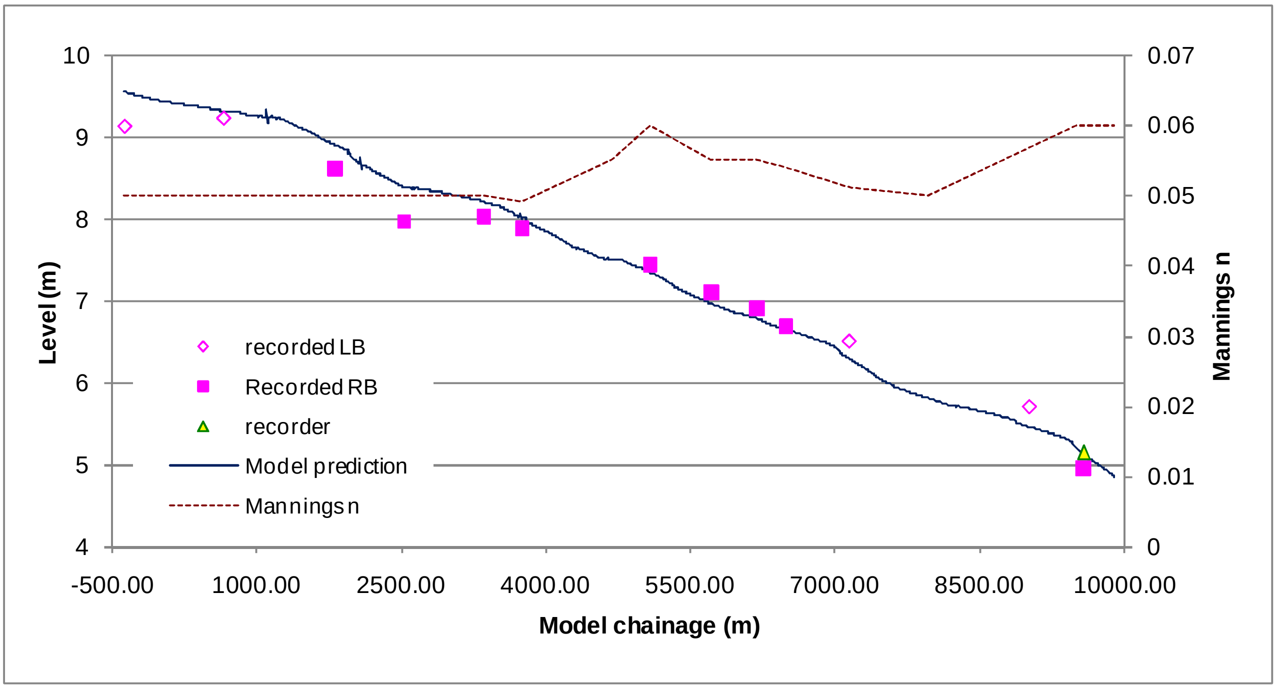

Figure 14 Upper Model calibration, peak flood levels, May 1999 event.

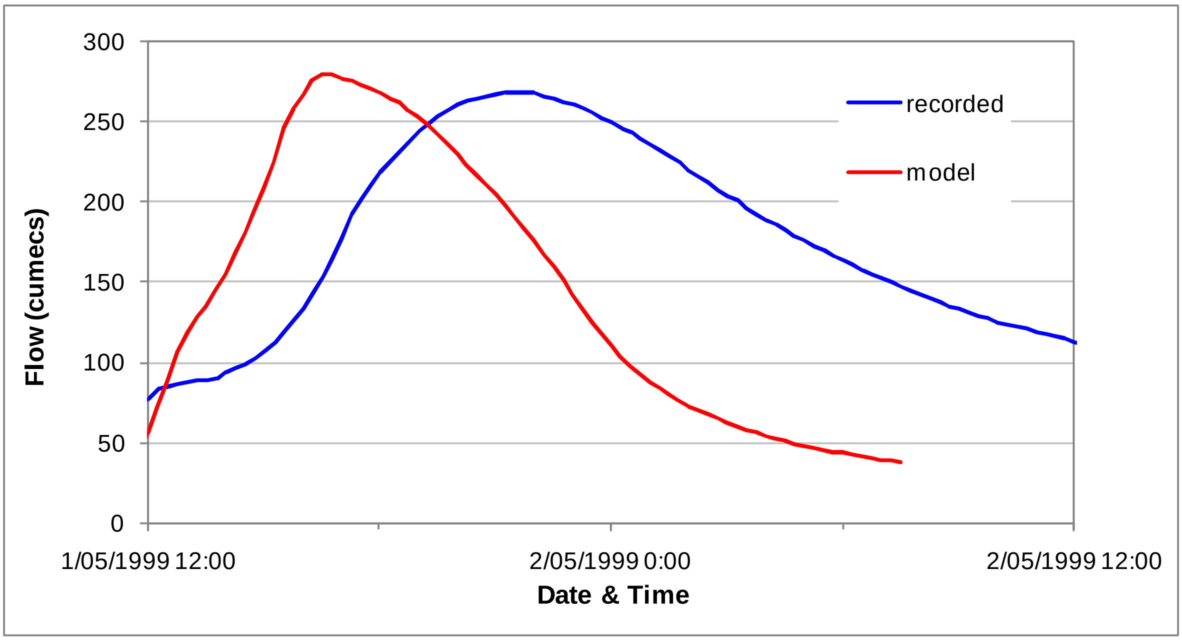

Figure 15 Upper Model calibration, flows at Te Matai, May1999 event

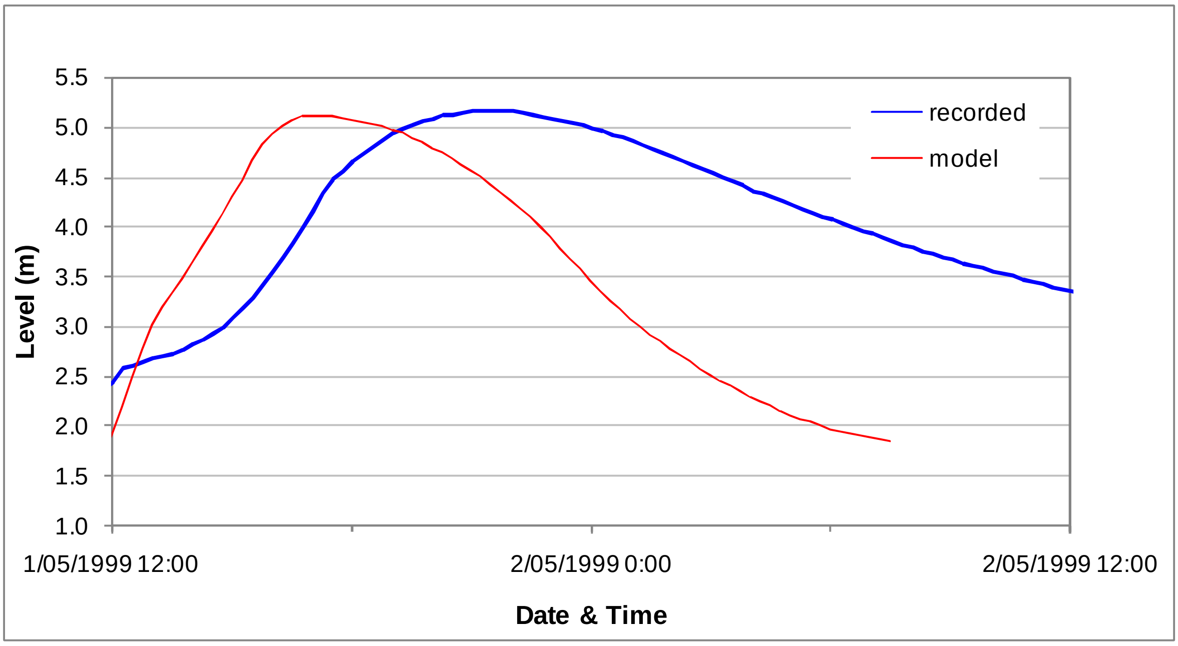

Figure 16 Upper Model calibration, levels at Te Matai, May1999 event

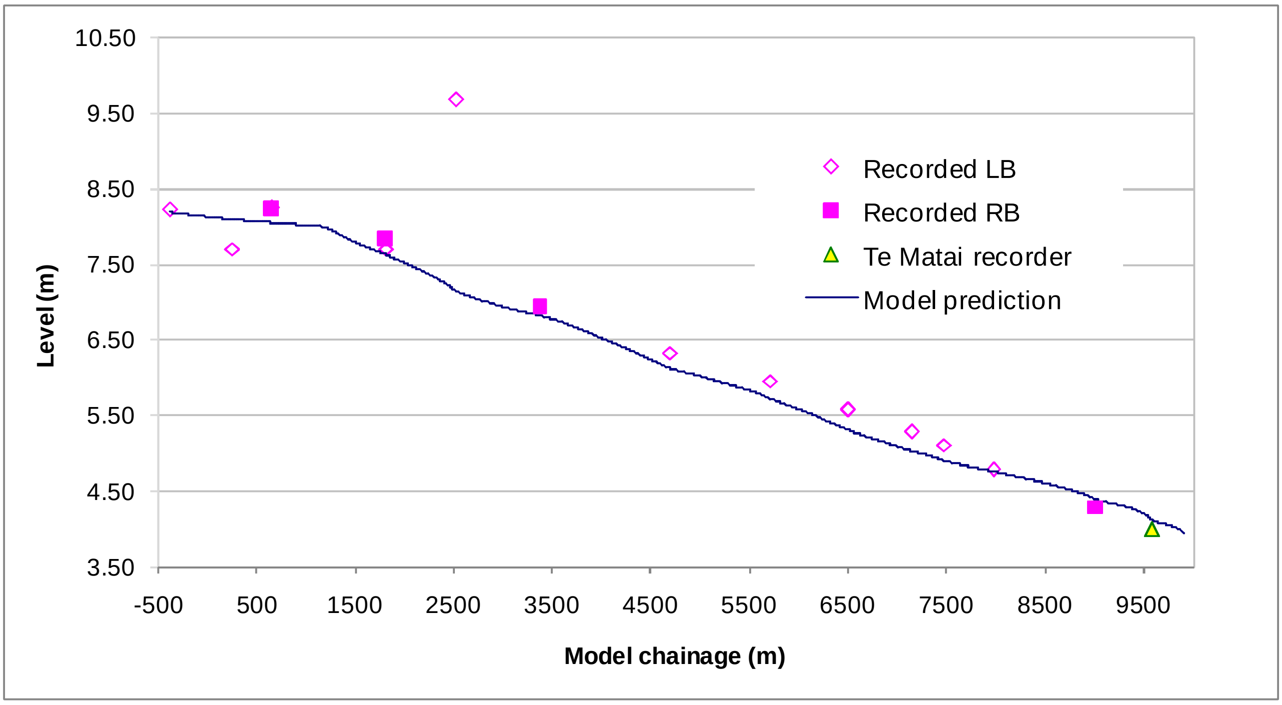

Figure 17 Upper Model calibration, peak flood levels, August 2006 event.

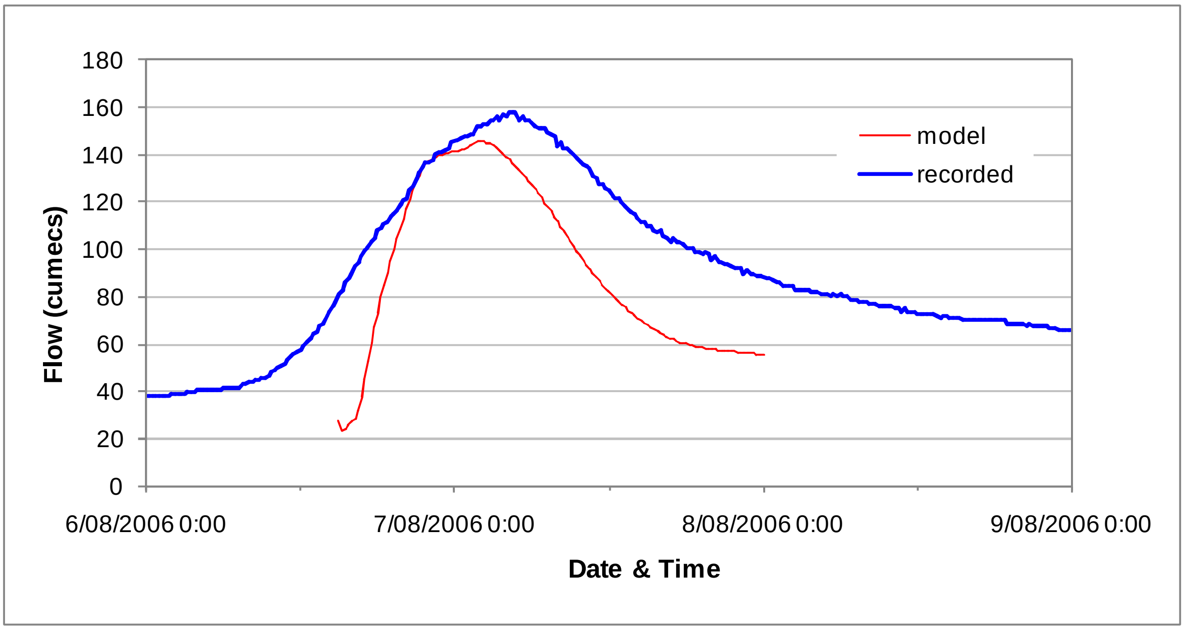

Figure 18 Upper Model calibration, flows at Te Matai, August 2006 event.

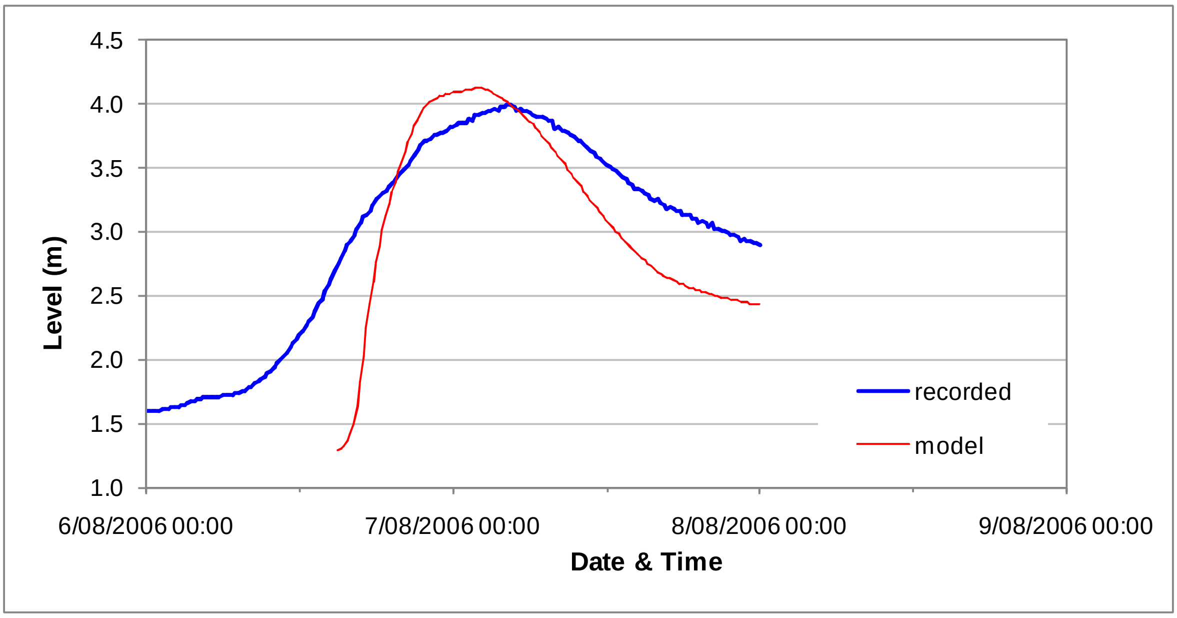

Figure 19 Upper Model calibration, levels at Te Matai, August 2006 event.

8 Design Results

8.1 Lower Model

In running the design flow scenarios for the lower reach, the calibrated roughness values have been increased by 10% to allow for additional turbulence that large floods can cause. For simplicity, this was also done for the 5% AEP flow/1% sea level scenarios. (However this was not done for the 10% flow scenario in the upper reach as that flow is of the same order as the calibration events).

Table 3 summarises the results for the various scenarios. Further results in Appendix III show that the 1% design levels are determined by the 1% AEP flow/5% AEP sea level from cross-section 2 upstream.

Design flood scenario results have been plotted in Figures 20 and 21, against existing stopbank levels. Also plotted are the results with the design 500mm freeboard applied, and with half this freeboard. As the Asset Management Plan states that the stopbanks should be topped up to the full design height once freeboard drops to below half the design amount, top-ups are required now over the following reaches:

- Left bank:
  - midway between sections 18 and 19 (around Waiari confluence) to the railway
  - isolated areas near sections 15, 17 and 18

- Right bank:
  - Section 2 to 4 (note that at the culverts to Fords Cut, the bank only needs topping up to 2.2m RL (Figure 21).
  - an isolated area between sections 5 and 6
  - isolated areas between sections 16 and 18
  - section 19 to the recorder at SH2

Upstream of the Waiari, the design levels of Surman cannot be reproduced, even using the 1997 cross-sections. It can also be seen from Figure 21, and Figure 20 to a lesser extent, that the current stopbank profile does indeed appear to drop upstream of section 19 (relative to the general slope). Hence the required top-ups are higher than previously might have been expected.

(Figures 22 and 23 do not show a marked difference in settlement between upstream and downstream of section 19. Hence, although it has not been checked, the most likely explanation for the drop in the bank profile is that design predictions changed. In general, stopbanks downstream of section 19 were built between 1979 and 1982 while those upstream were built from 1984 onwards. Thus it is conceivable that the later design estimates for the river levels were lower than the earlier ones that were accepted at the time the downstream stopbanks were designed.)

Model results have been used to estimate the cost of upgrading stopbanks to the 1% AEP design levels, for both the current climate conditions and the medium forecast for 2030 (the latter to avoid having to raise banks again only about 20 years hence). Only those lengths of stopbank where the existing freeboard is less than half the design amount have been included in the costings. Results are provided in Appendix IV; in summary the estimated costs are $2.8m for the current climate design levels and $4.3m for the 2030 levels. (These figures do not include any allowance for raising stopbanks in the tributary canals if required; these canals also have a 1% AEP design standard and the backwater from the Kaituna River may determine design levels.)

Figures 22 and 23 show that the stopbanks have settled since the time of the last long-section surveys in 1998 and 2002. Presumably this is ongoing post-construction settlement, even though it has been 20-25 years since the stopbanks were built. Notwithstanding that the rate could decrease over time, this information could be used to refine the estimates of stopbank depreciation (and the funding requirements) outlined in the Asset Management Plan.

Results for modelling of the proposed TEM bridge and for modelling of options of using the Kaituna wetland area as flood storage are given in Appendices V and VI respectively.

	Existing Situation (spillage allowed)			If Stopbanks Topped up									
	Existing Climate			Existing Climate		Climate change2030				Climate change to 2080			
						med		high		med		high	
Cross-section	1% AEP	0.5% AEP	0.2% AEP	1% AEP	+freeboard	1% AEP	+freeboard	1% AEP	+freeboard	1% AEP	+freeboard	1% AEP	+freeboard
1a	2.14			2.15	2.65	2.48	2.98	2.50	3.00	2.60	3.10	2.62	3.12
2	2.63	2.65	2.78	2.72	3.22	3.03	3.53	3.07	3.57	3.07	3.57	3.17	3.67
FordLoop500	2.62	2.65	2.78	2.72	3.22	3.02	3.52	3.06	3.56	3.06	3.56	3.17	3.67
FordLoop100	2.62	2.65	2.78	2.72	3.22	3.02	3.52	3.06	3.56	3.06	3.56	3.17	3.67
3	2.73	2.75	2.90	2.83	3.33	3.11	3.61	3.15	3.65	3.17	3.67	3.28	3.78
4	2.89	2.92	3.10	3.00	3.50	3.27	3.77	3.32	3.82	3.35	3.85	3.50	4.00
5	2.93	2.96	3.15	3.04	3.54	3.30	3.80	3.35	3.85	3.39	3.89	3.54	4.04
6	3.03	3.06	3.28	3.15	3.65	3.40	3.90	3.46	3.96	3.50	4.00	3.67	4.17
7	3.12	3.15	3.37	3.24	3.74	3.48	3.98	3.53	4.03	3.58	4.08	3.76	4.26
8	3.34	3.39	3.66	3.49	3.99	3.72	4.22	3.78	4.28	3.86	4.36	4.07	4.57
9	3.38	3.42	3.69	3.52	4.02	3.74	4.24	3.81	4.31	3.88	4.38	4.09	4.59
10	3.54	3.59	3.89	3.70	4.20	3.90	4.40	3.97	4.47	4.06	4.56	4.29	4.79
11	3.59	3.64	3.94	3.75	4.25	3.95	4.45	4.02	4.52	4.11	4.61	4.34	4.84
12	3.66	3.70	4.01	3.82	4.32	4.01	4.51	4.09	4.59	4.18	4.68	4.41	4.91
13	3.92	3.97	4.31	4.09	4.59	4.27	4.77	4.36	4.86	4.46	4.96	4.71	5.21
14	4.14	4.19	4.55	4.32	4.82	4.49	4.99	4.58	5.08	4.70	5.20	4.96	5.46
15	4.37	4.42	4.80	4.57	5.07	4.73	5.23	4.83	5.33	4.95	5.45	5.24	5.74
16	4.50	4.56	4.94	4.71	5.21	4.88	5.38	4.97	5.47	5.11	5.61	5.40	5.90
17	4.63	4.69	5.08	4.85	5.35	5.01	5.51	5.11	5.61	5.25	5.75	5.55	6.05
18	4.85	4.91	5.28	5.09	5.59	5.24	5.74	5.35	5.85	5.49	5.99	5.80	6.30
19	5.01	5.09	5.43	5.27	5.77	5.43	5.93	5.54	6.04	5.69	6.19	6.00	6.50
20	5.12	5.22	5.50	5.39	5.89	5.54	6.04	5.66	6.16	5.80	6.30	6.11	6.61
21	5.26	5.37	5.60	5.55	6.05	5.70	6.20	5.82	6.32	5.96	6.46	6.27	6.77
22	5.51	5.60	5.74	5.82	6.32	5.97	6.47	6.09	6.59	6.23	6.73	6.55	7.05
23	5.79	5.89	5.95	6.10	6.60	6.24	6.74	6.36	6.86	6.50	7.00	6.82	7.32
24	5.97	6.11	6.15	6.26	6.76	6.41	6.91	6.53	7.03	6.66	7.16	6.98	7.48
25	6.25	6.48	6.54	6.50	7.00	6.64	7.14	6.76	7.26	6.90	7.40	7.22	7.72
26	6.74	7.19	7.37	6.91	7.41	7.07	7.57	7.19	7.69	7.33	7.83	7.66	8.16
Te Matai Gauge	7.21	7.84	8.08	7.34	7.84	7.50	8.00	7.63	8.13	7.78	8.28	8.13	8.63

Table 3 Summary of flood levels, design scenarios, lower reach.

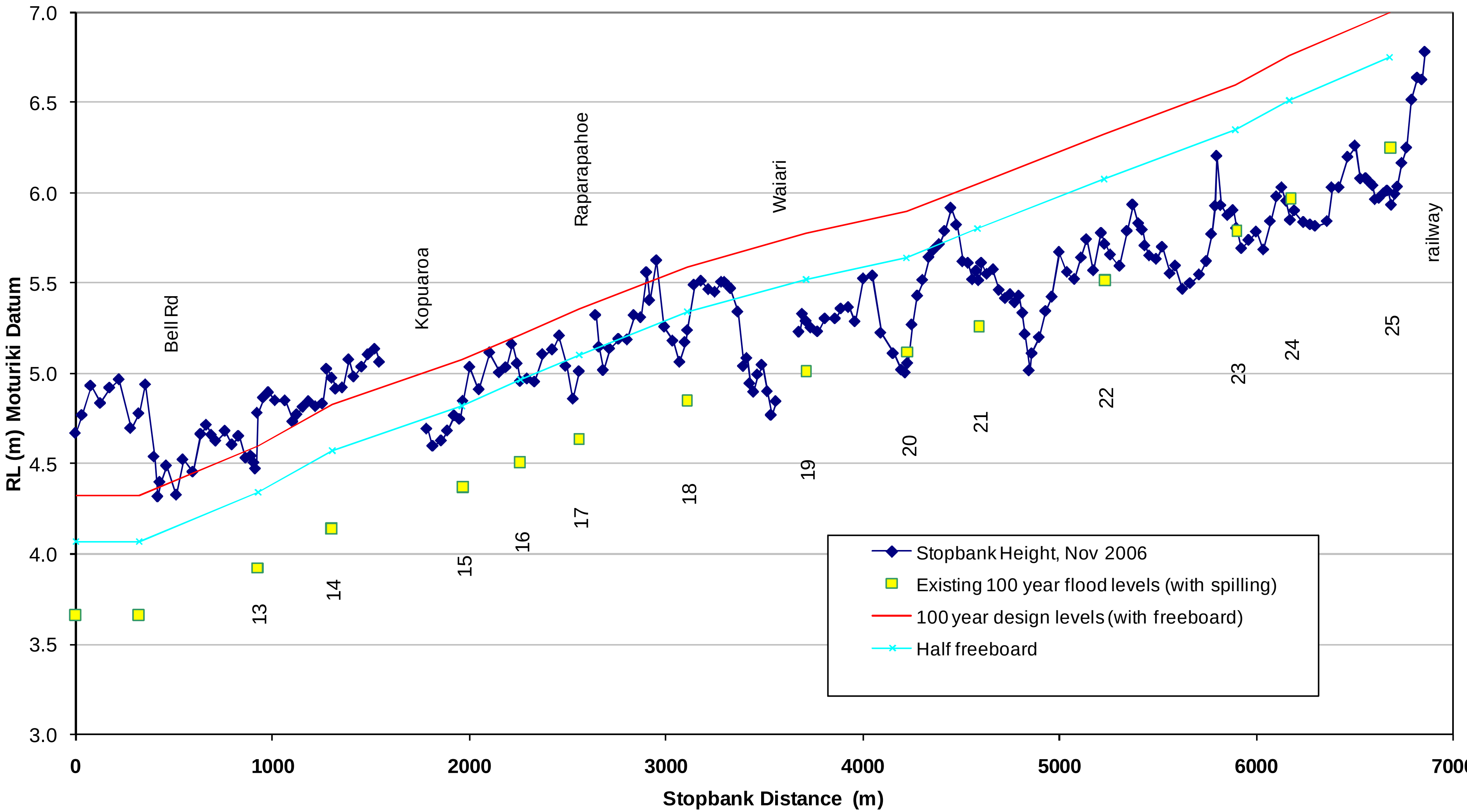

Figure 20 Kaituna River left bank stopbank crest and design profiles from Bell Road area to end of stopbank at Te Matai (Cross-section locations shown).

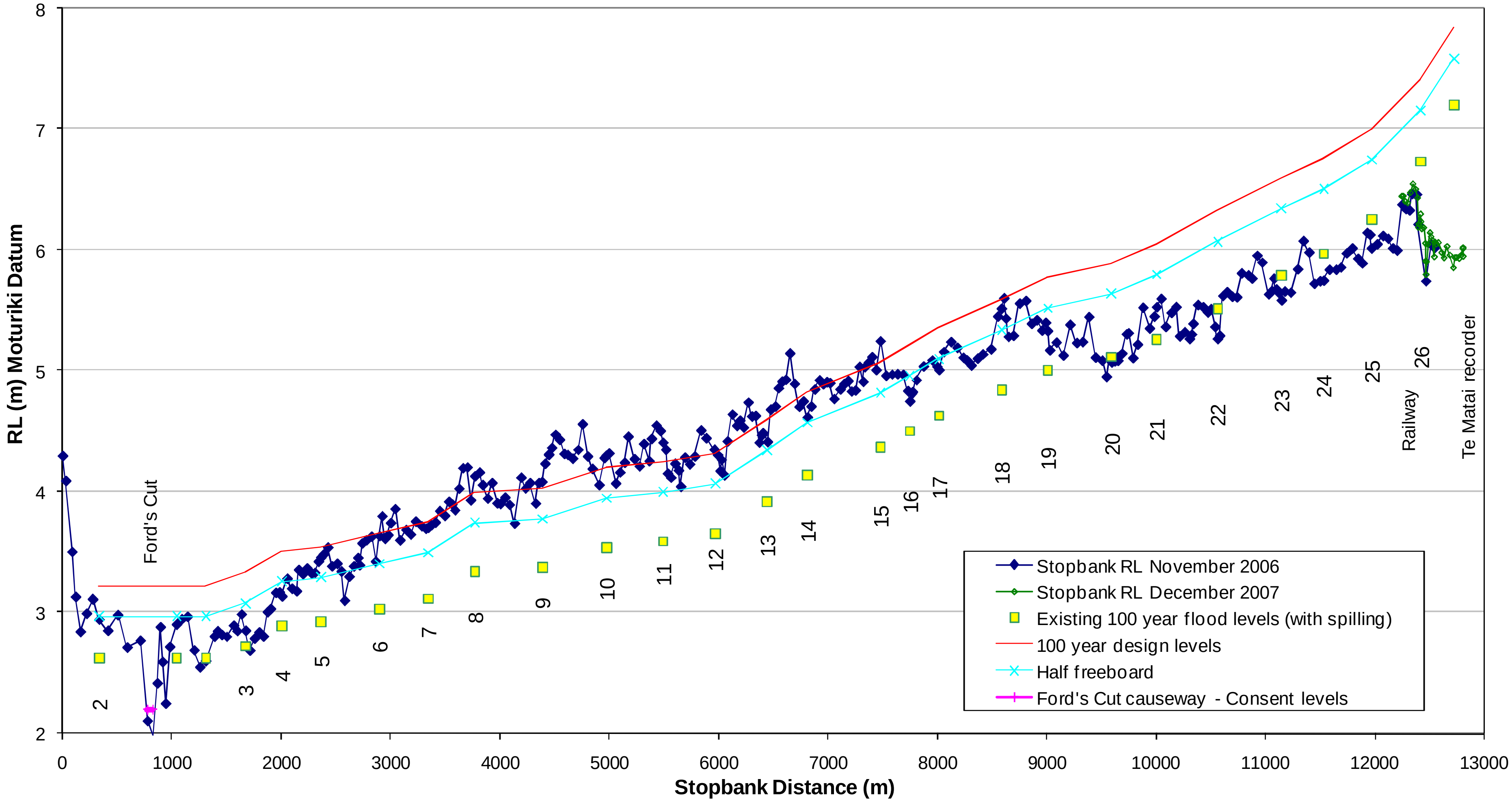

Figure 21 Kaituna River right bank stopbank crest and design profiles from mouth to Te Matai (Cross-section locations shown)

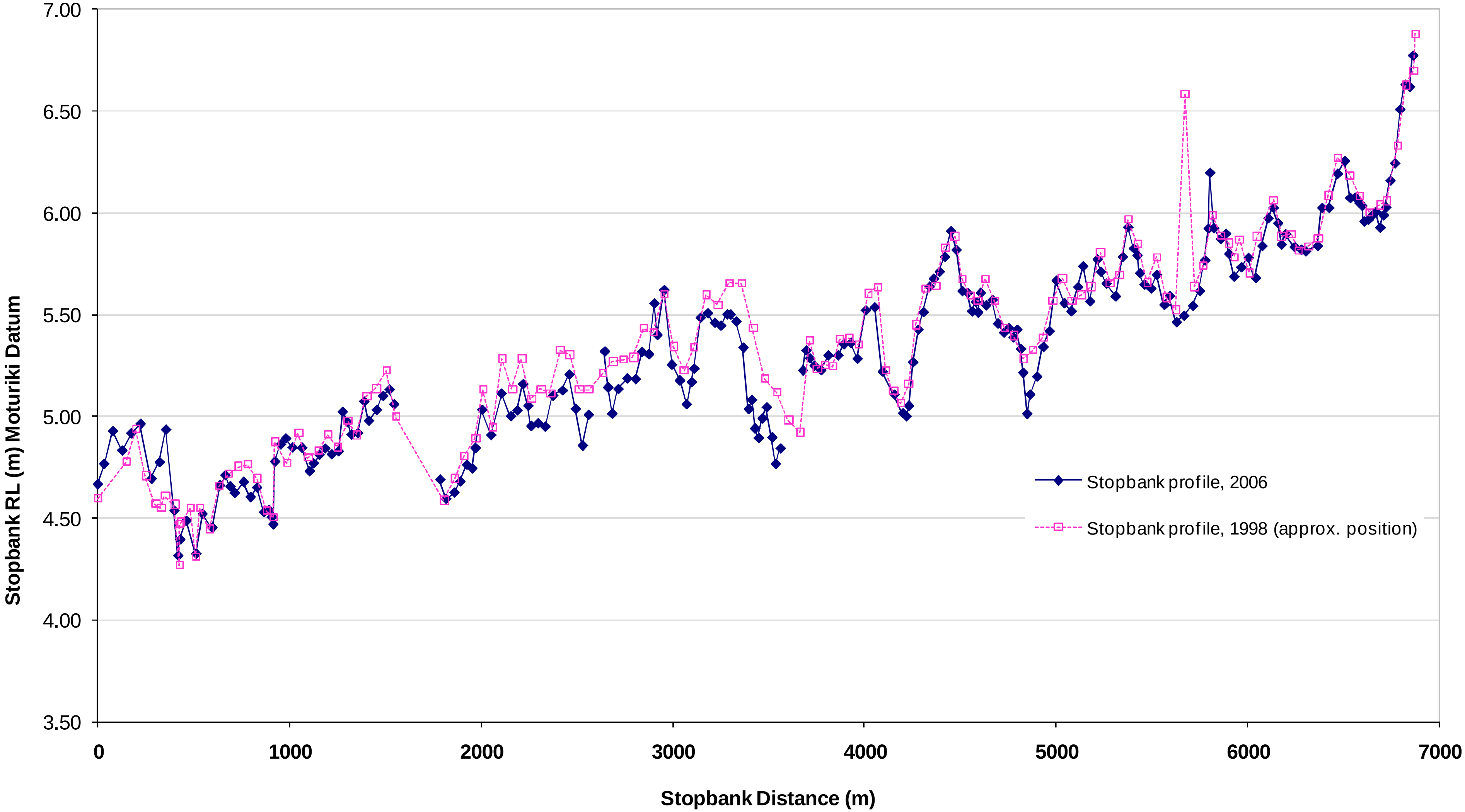

Figure 22 Kaituna River left bank 1998 and 2006 stopbank crest profiles from Bell Road area to end of stopbank at Te Matai

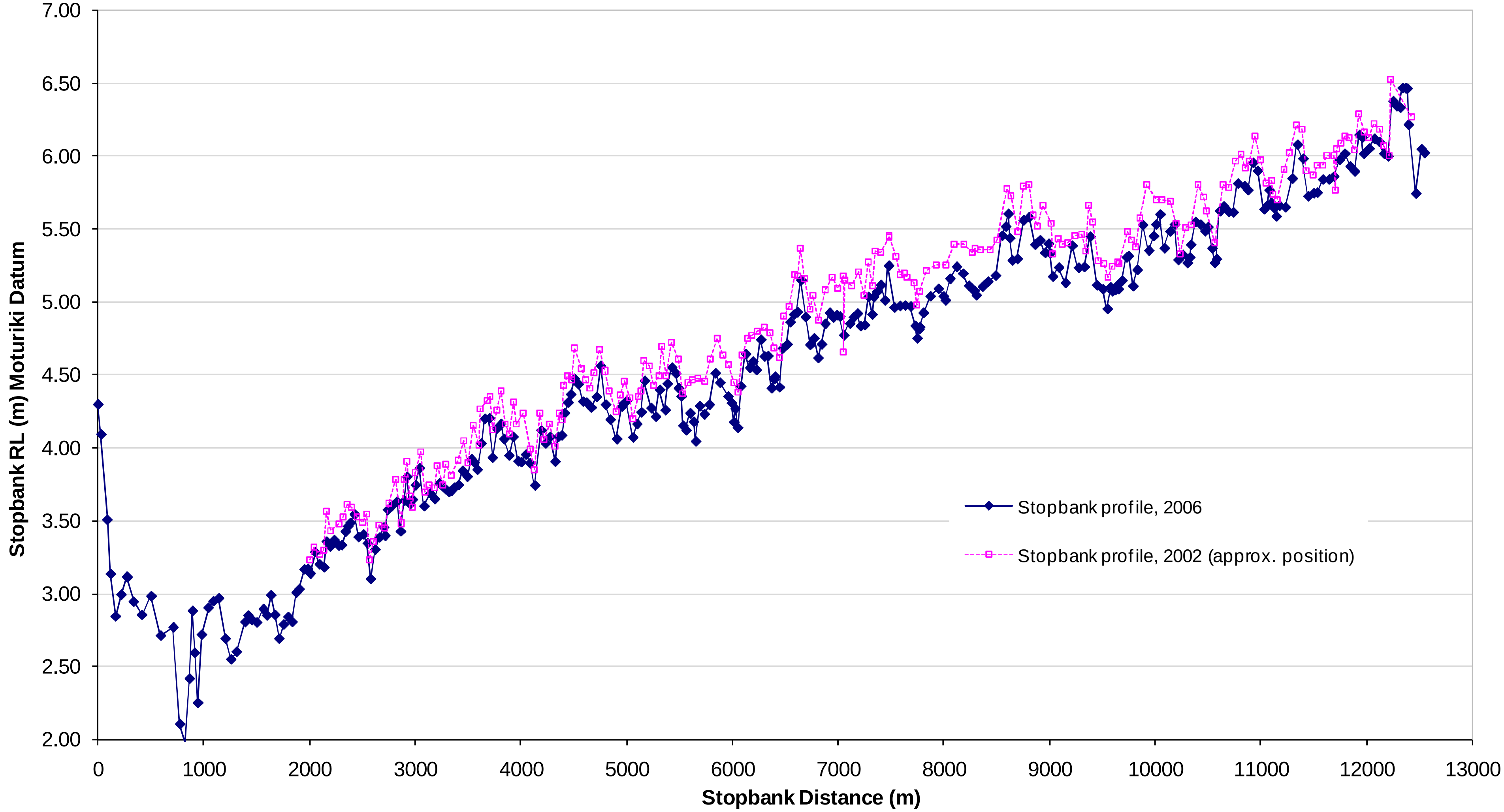

Figure 23 Kaituna River right bank 2002 and 2006 stopbank crest profiles from mouth to Te Matai

8.2 Upper Model

The design scenario modelled was a 10% AEP flood. Design flood levels are given in Table 4 and plotted in Figures 24-29. According to the model predictions, the stopbank crest levels exceed the design levels (the design standard is defined as 10% AEP without freeboard).

Note that these results are based on the assumed proportions of inflows from the Kaituna and Mangorewa Rivers. Results upstream of the confluence in particular are subject to some uncertainty. If and when a flow recorder is installed at the Maungarangi Road bridge on the Kaituna, the estimates may be able to be refined.

Cross-section	10% AEP
37	5.21
38	5.53
39	5.67
40	5.82
41	6.08
42	6.23
43	6.41
44	6.68
45	6.82
46	7.27
47	7.46
48	7.76
49	8.17
51	8.59
52	8.61
53	8.67

Table 4 Design flood levels, upper reach.

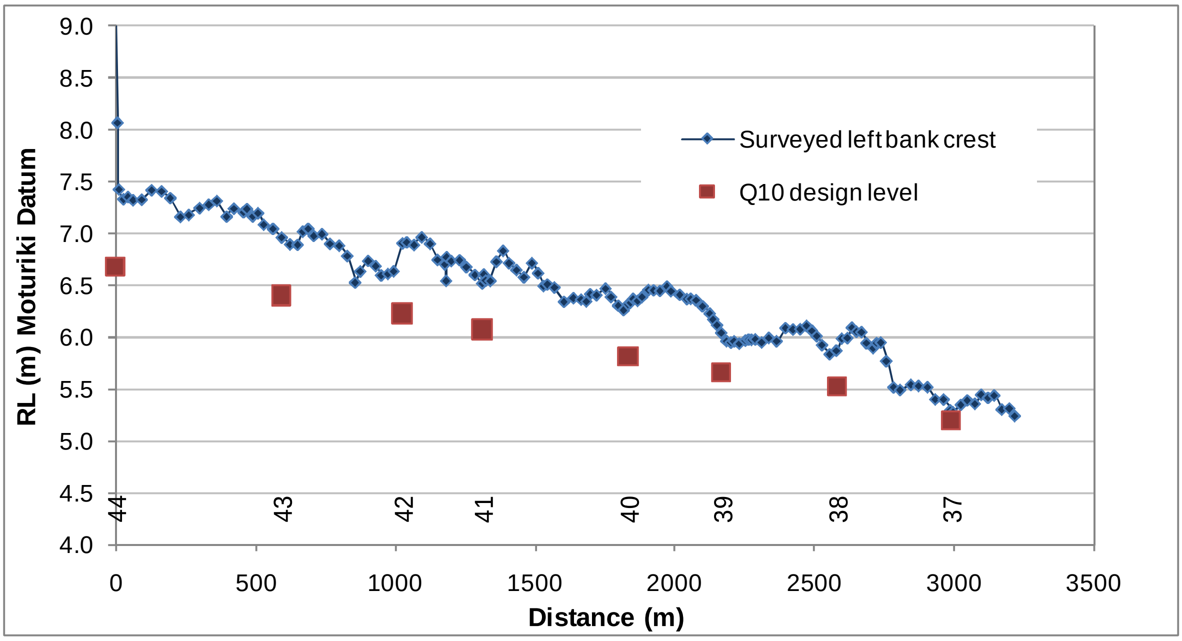

Figure 24 Kaituna River left bank stopbank crest and design profiles, from cross-section 44 to Te Matai (Cross-section locations shown).

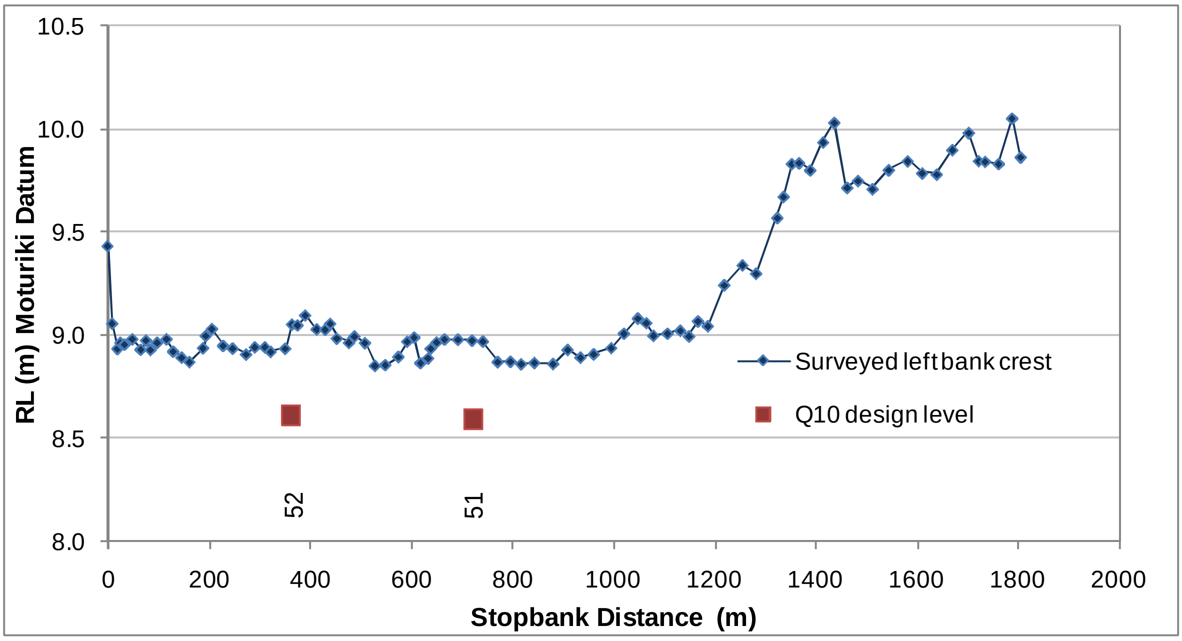

Figure 25 Kaituna River left bank stopbank crest and design profiles, near Mangorewa (Cross-section locations shown).

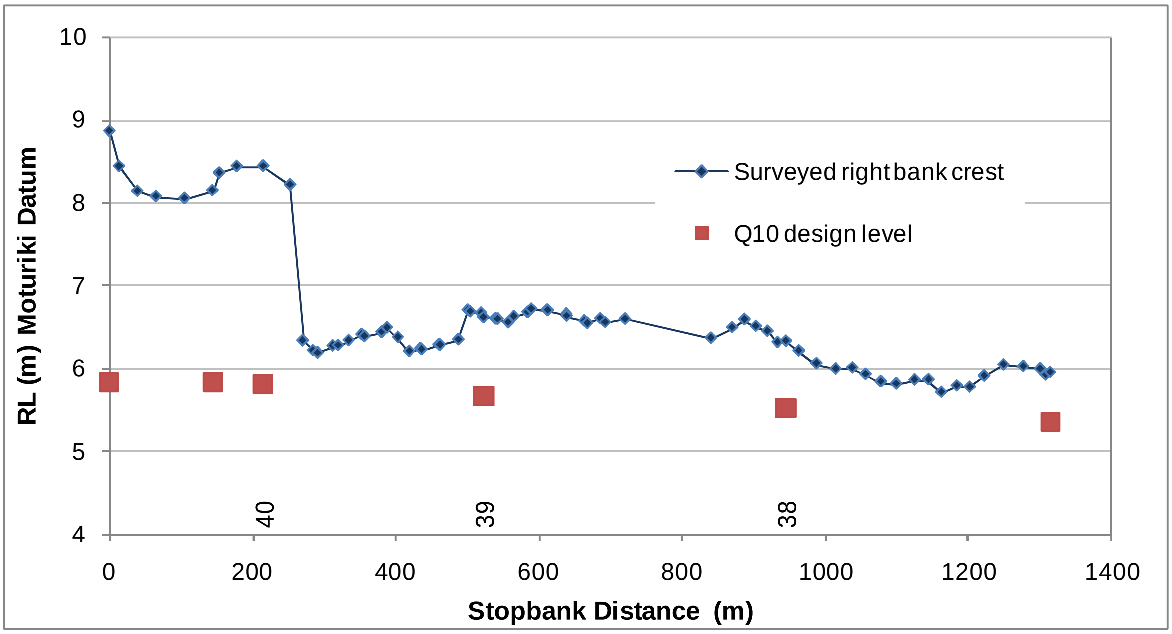

Figure 26 Kaituna River right bank stopbank crest and design profiles, cross-section 42 to Te Matai (Cross-section locations shown).

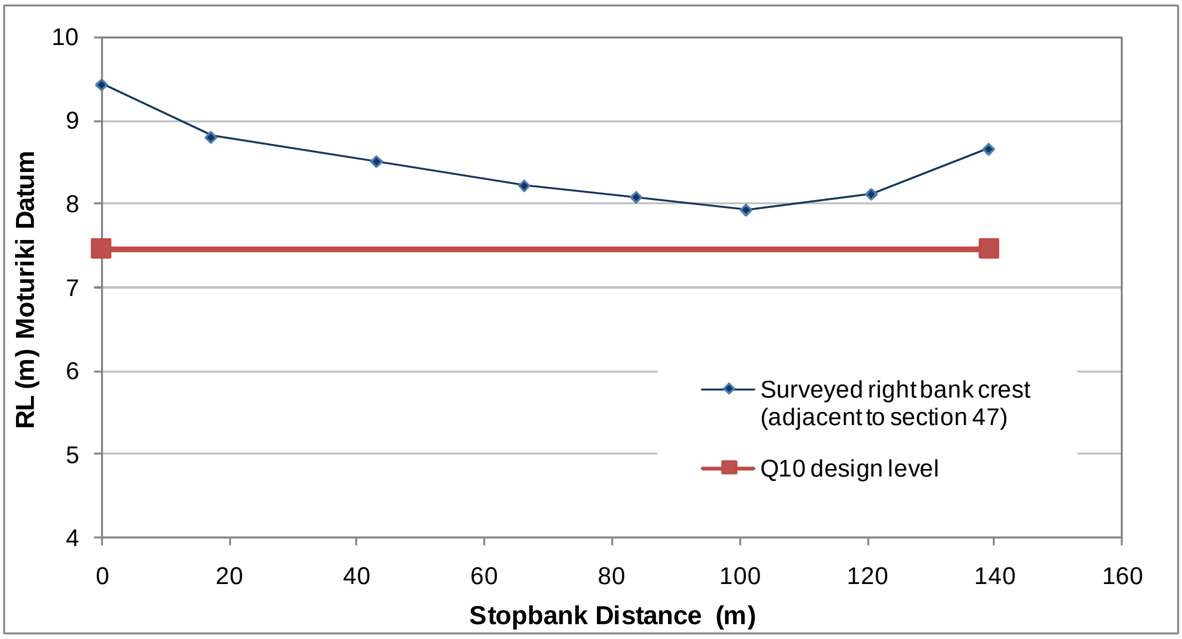

Figure 27 Kaituna River right bank stopbank crest and design profiles, adjacent to cross-section 47.

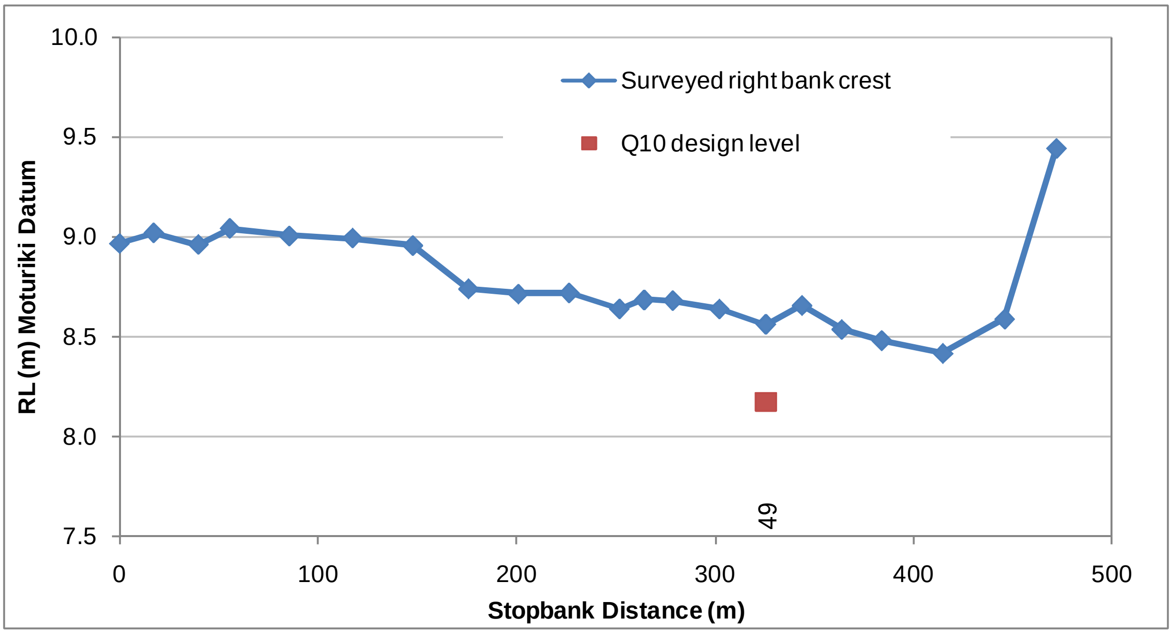

Figure 28 Kaituna River right bank stopbank crest and design profiles, adjacent to cross-section 49.

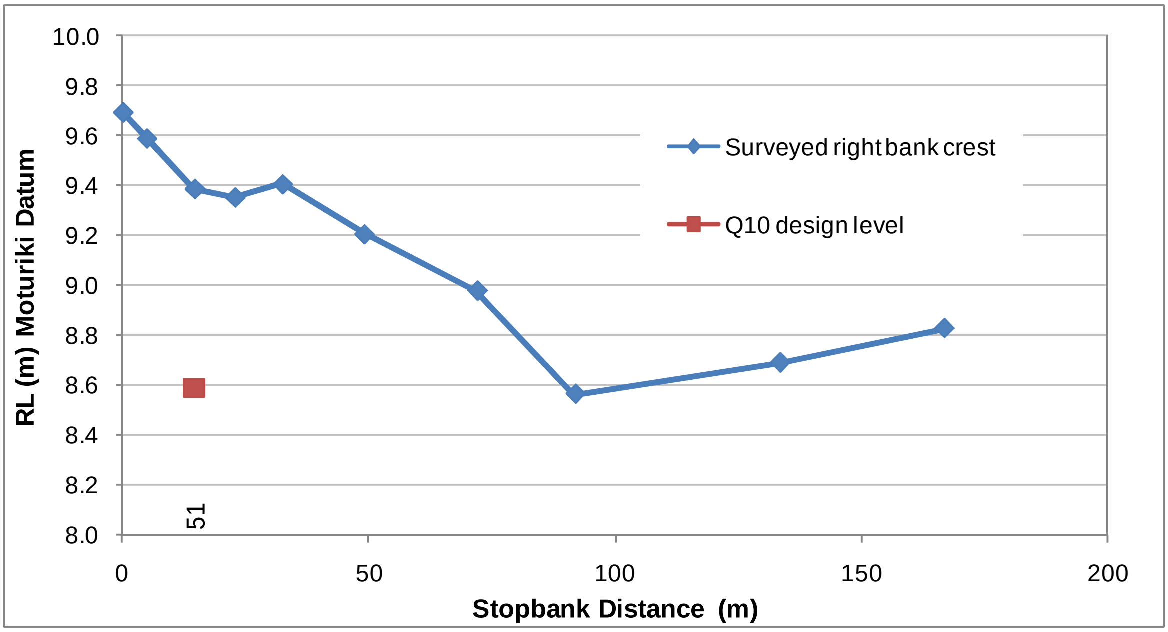

Figure 29 Kaituna River right bank stopbank crest and design profiles, adjacent to cross-section 51.

9 Further Investigations

Surman made the comment that there would be benefit in having more information on flows (their peaks and the timing) and flood levels from flood events. While some progress has been made on this, for instance the reinstatement of the Waiari recorder and the collection of good flood level data from the August 2006 event, more can still be done. Of particular benefit would be:

- Flow records, or at least a water level hydrograph, for the Kaituna River upstream of the Mangorewa. That would allow the calibration and the setting of design flows for the reach upstream of Te Matai, and is perhaps the highest priority of the information requirements.
- Flow and/or water level records for major ungauged tributaries such as the Kopuaroa.
- Further observations of flood levels around the railway bridge, to help confirm or refine the design flood levels upstream of the bridge.

Some benefit to flood level predictions, particularly in the very lower reaches near Te Tumu and adjacent to the Maketu Estuary stopbanks, could occur if the models described in this report are connected to the 2-d model of the estuary being produced by DHI, even though that model will not specifically be for flood investigations.

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Appendix I Flow Assumptions

Calibration peak flows: Note these affect the n values chosen.

May 99:

Raparapahoe

Recorder flow (as rated) gives 73 cumecs. Matt estimated 86, based on slope-area calculations, debris marks, measured velocities, and perhaps up to 6 cumec bypassing the site. (Appendix II of his report). Matt's figures used here.

Ohineangaanga

Matt used 64, calculated as for the Raparapahoe. However there was quite a variability in results of his calculation trials before he arrived at that figure.

Ignoring the flat last $2km^2$, $A = 18.8km^2$. If $Q \alpha A^{0.75}$, $Q_O = 0.508Q_R$. Assume $Q_O = 0.5Q_R$, i.e. a peak of $43m^3/s$. $43m^3/s$ assumed for model (lower than any of Matt's trial calculations).

Waiari

Matt used 175, again based on slope-area calculations as above. He noted that a peak flow of $203m^3/s$ had been estimated, but it is not clear where this figure came from.

Area at gauge = $71.7km^2$. If $Q \alpha A^{0.75}$, $Q_W = 1.389Q_R = 119m^3/s$.

However, the catchment lies between Raparapahoe and Mangorewa. Matt used a combination of the two hydrographs for his model.

Use that shape, but scale to $119m^3/s$. (Again lower than any of Matt's trials.)

Kopuaroa

Using nearby Raparapahoe catchment as basis again. Q_R estimate $86m^3/s$

Kopuaroa catchment $A = 46.36km^2$ at recorder

If $Q \alpha A^{0.75}$, $Q_K = 0.536Q_R$. If $Q \alpha A^{0.80}$, $Q_K = 0.51Q_R$. Kopuaroa is more coastal than Raparapahoe, and the May 99 event was more centred to the south, near Rotorua. Therefore assume that $Q_K = 0.5Q_R$, i.e. a peak of $43m^3/s$.

If just consider the area behind the railway, i.e. ignore lower part of catchment on floodplain, $A = 18km^2$, and if $Q \alpha A^{0.75}$, $Q_K = 0.49Q_R$. If change to 0.4, get better calibration fit for Kaituna levels.

Adopt $Q_K = 0.4Q_R = 34m^3/s$

Parawhenuamea

$A = 32.7km^2$ – If base on Waiari ($A = 71.7km^2$), and if $Q \alpha A^{0.75}$, $Q_P = 0.55Q_W = 65m^3/s$

Rangiuru South (refer to Stephen Parks' report for catchments upstream of Te Matai).

Use 0.1 x Waiari, ie 0.1 x 119m^3/s = 12m^3/s

Mangorewa

Subject to much discussion. Based on debris marks and an estimated velocity of 5m/s, the accepted figure at Saunders has to date been 834m^3/s.

Without knowing the Kaituna flow, and timing of the flow, at Maungarangi Road bridge, model simulations suggest that 834m^3/s is too high. After some trials, a figure of 600m^3/s has been assumed for the Mangorewa (i.e. about 8km downstream of Saunders).

Kaituna @ Maungarangi Road Bridge

A figure of 180m^3/s has been assumed, i.e. 30% of the Mangorewa flow.

August 2006

Assumptions as above, except for following

Waiari

The recorded flow can be used.

Mangorewa

At Saunders recorder, A = 178.7km^2. Mangorewa catchment A = 189km^2. If $Q \propto A^{0.8}$ then use flow of 1.046 x Saunders flow. Thus a figure of 142m^3/s has been assumed. Also assume a 1 hour lag in timing.

Kaituna @ Maungarangi Road Bridge

Assume hydrograph is the sum of Taaheke flow (with 3 hour lag) and 0.5 x Saunders flow (with 1 hour lag). Resulting peak flow is 84m^3/s.

Rangiuru South

Use 0.1 x Waiari, ie 0.1 x 38m^3/s = 4m^3/s

Parawhenuamea

Based on Waiari (and $Q \propto A^{0.75}$), $Q_P = 0.55Q_W$ = 21m^3/s

Q10, Q20, Q100 Flows

Raparapahoe, Ohineangaanga, Waiari & Kopuaroa

Use the estimates in Rachael's reports.

Parawhenuamea

Use Matt Surman's Q10 estimate of 29m^3/s. (Lower than if the area-based method of above was used: that would give 39m^3/s).

Mangorewa

Use 95% of the previously adopted Q10 value. (That previous Q10 of 259m^3/s was influenced by the high reading of May 1999.)

Kaituna @ Maungarangi Road Bridge

For Q10, assume 78m^3/s (30% of previously adopted Mangorewa Q10 value), which together with the assumed Mangorewa flow gives the accepted value of Q10 at Te Matai of 207m^3/s.

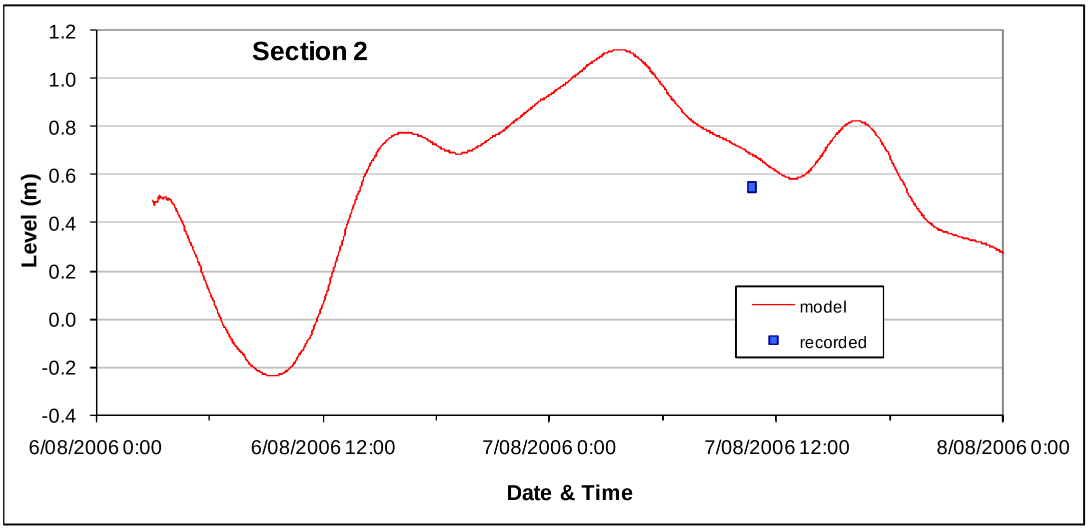
Rangiuru South

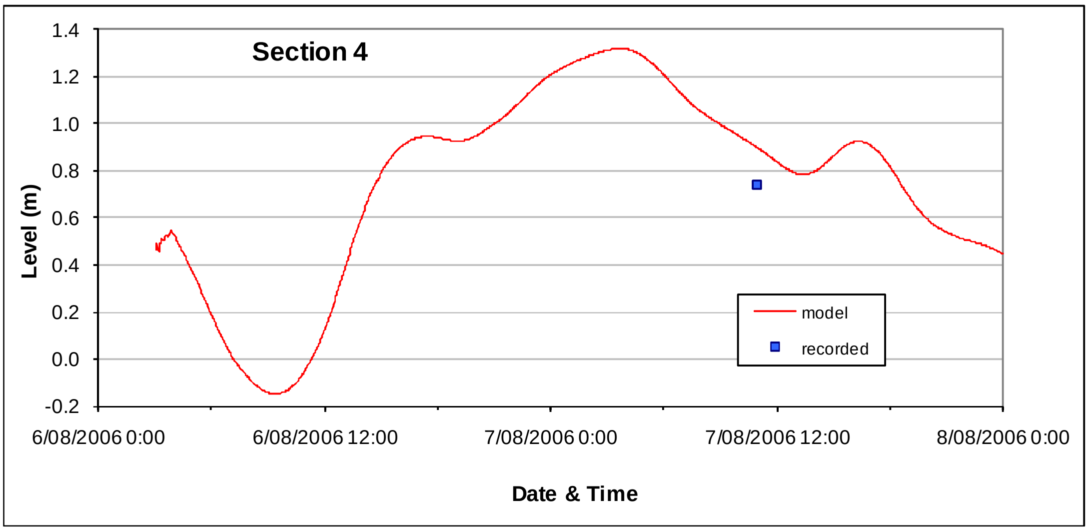
Use 0.1 x Waiari

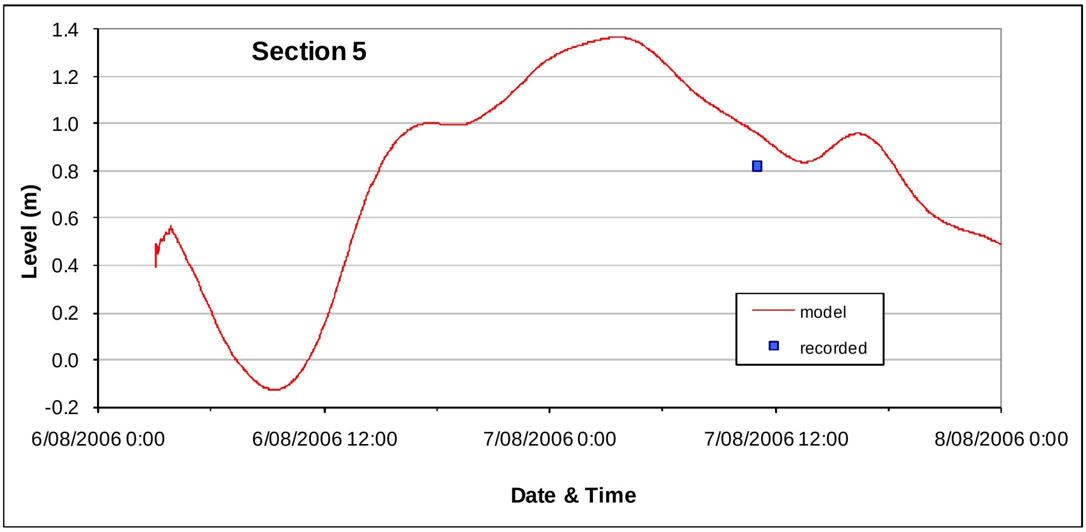
Parawhenuamea

Based on Waiari (and $Q \alpha A^{0.75}$)

Appendix II Additional Calibration Results, August 2006



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0.6
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0.0
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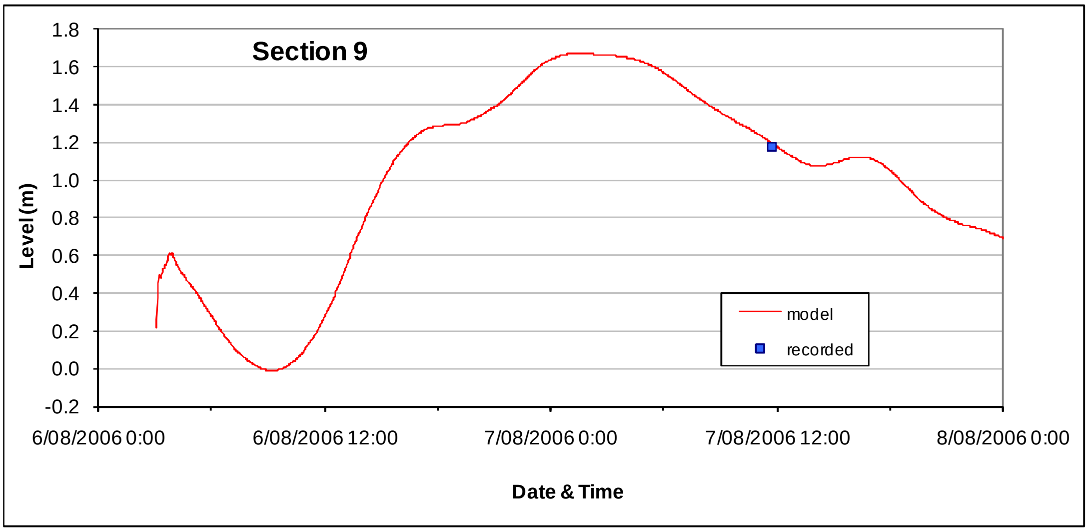
6/08/2006 12:00
7/08/2006 0:00
7/08/2006 12:00
8/08/2006 0:00
Date & Time
model
recorded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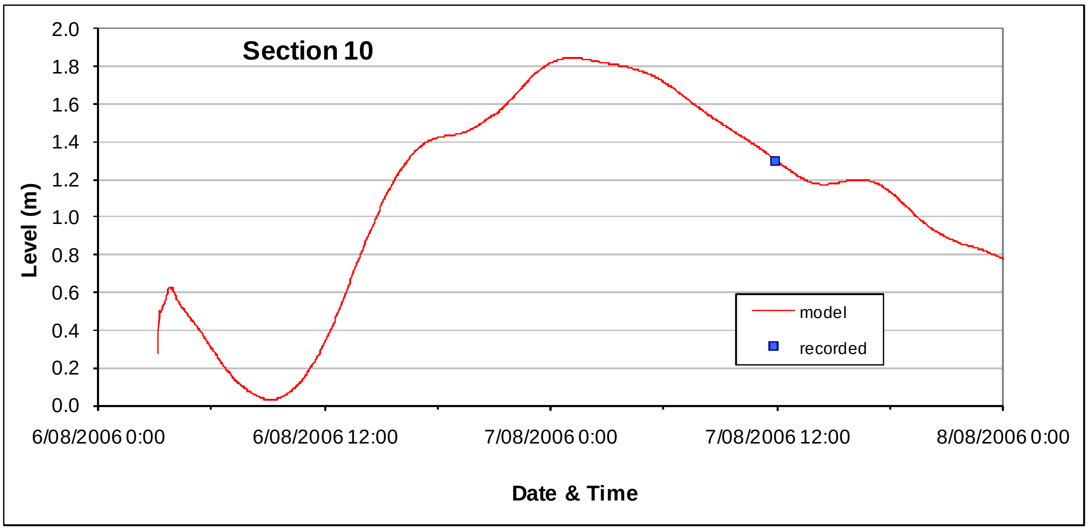
7/08/2006 12:00
8/08/2006 0:00
Date & Time
model
recorded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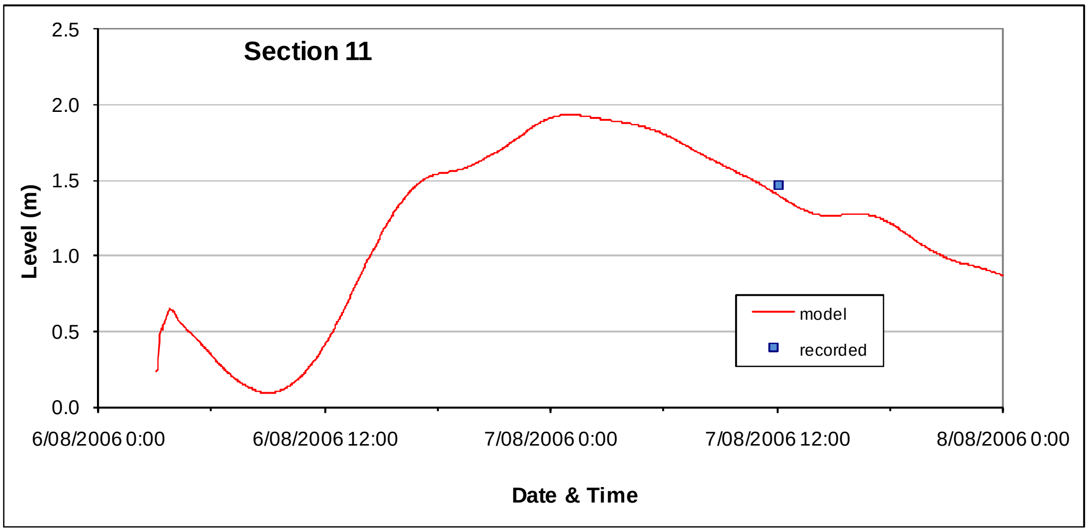
Date & Time
model
recorded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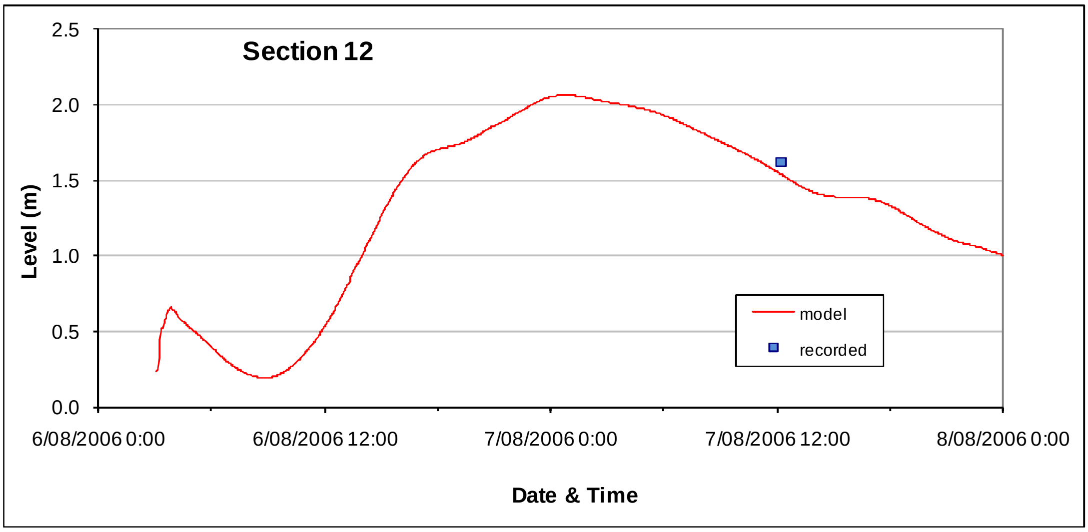
Date & Time
model
recorded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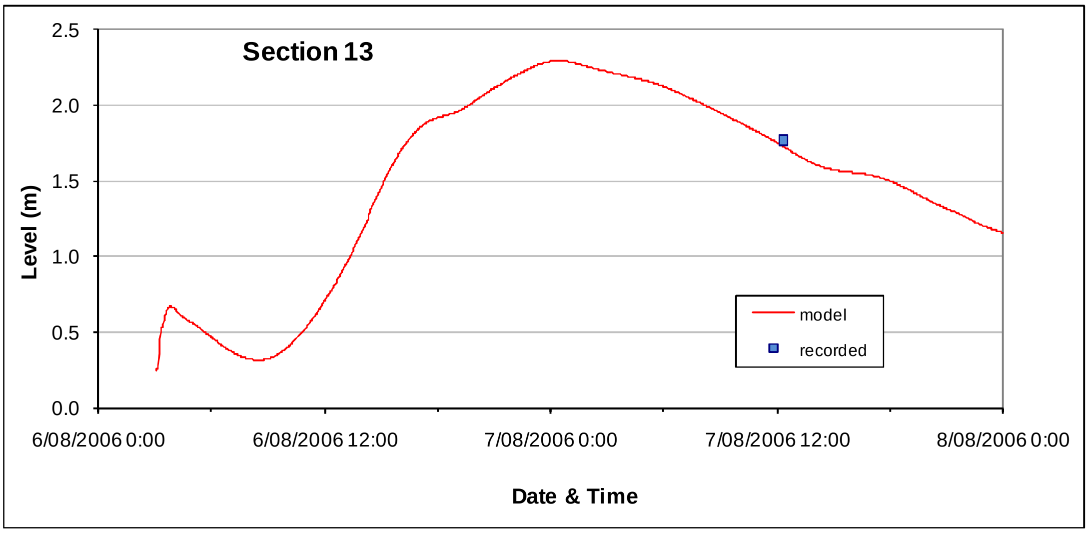
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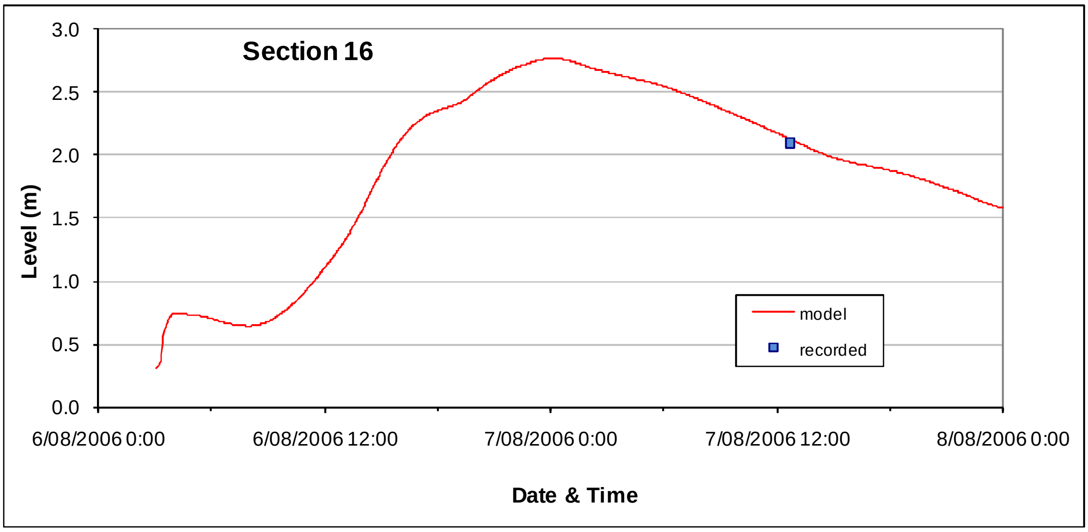
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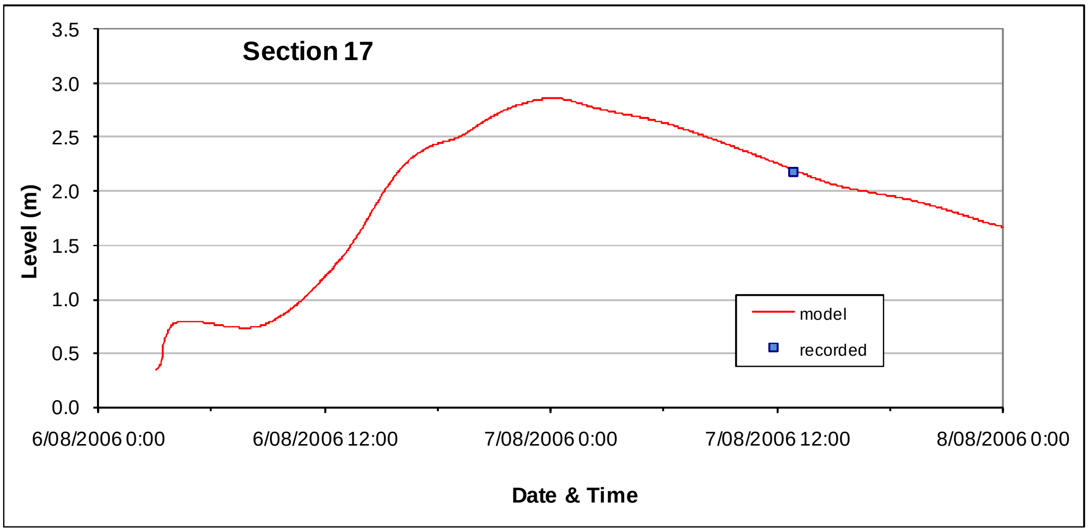
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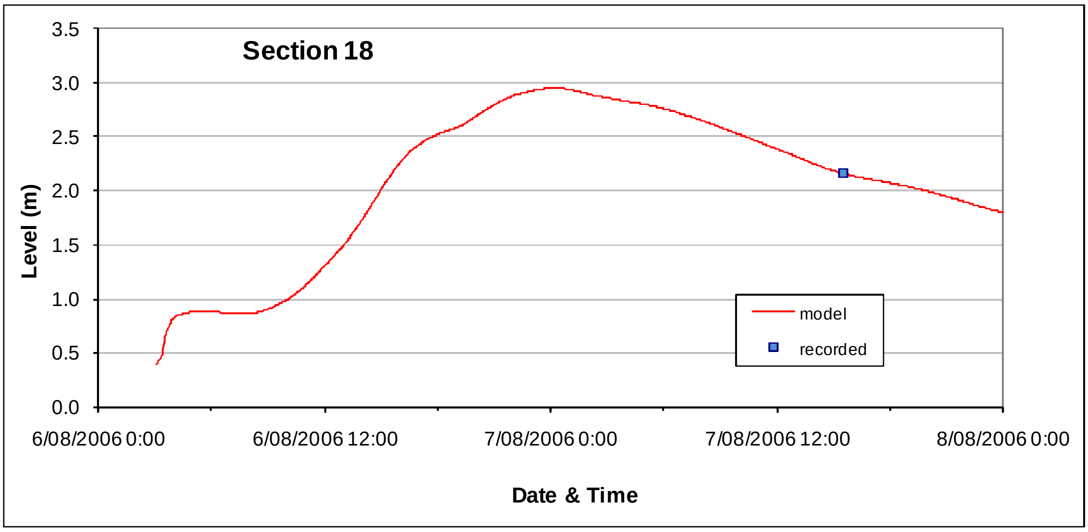
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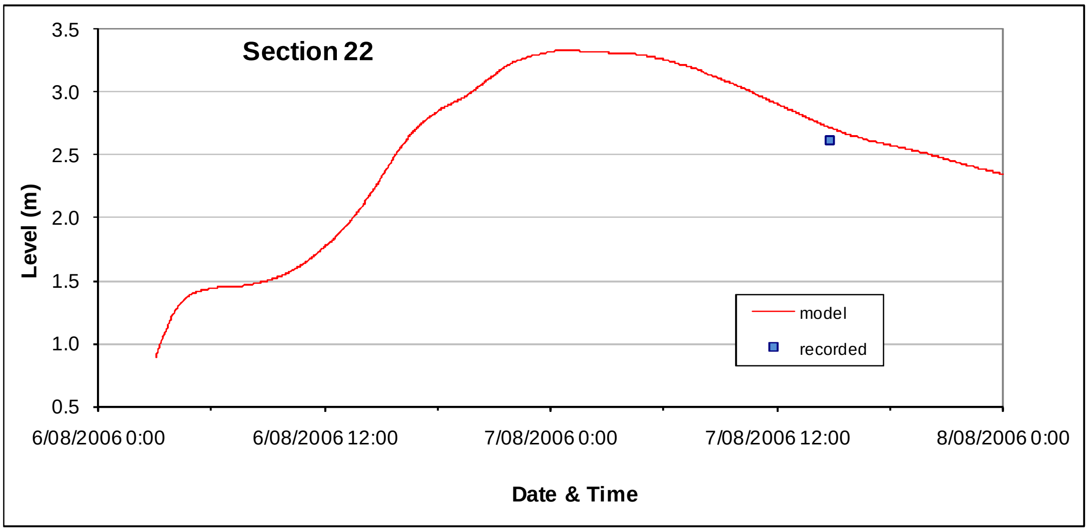
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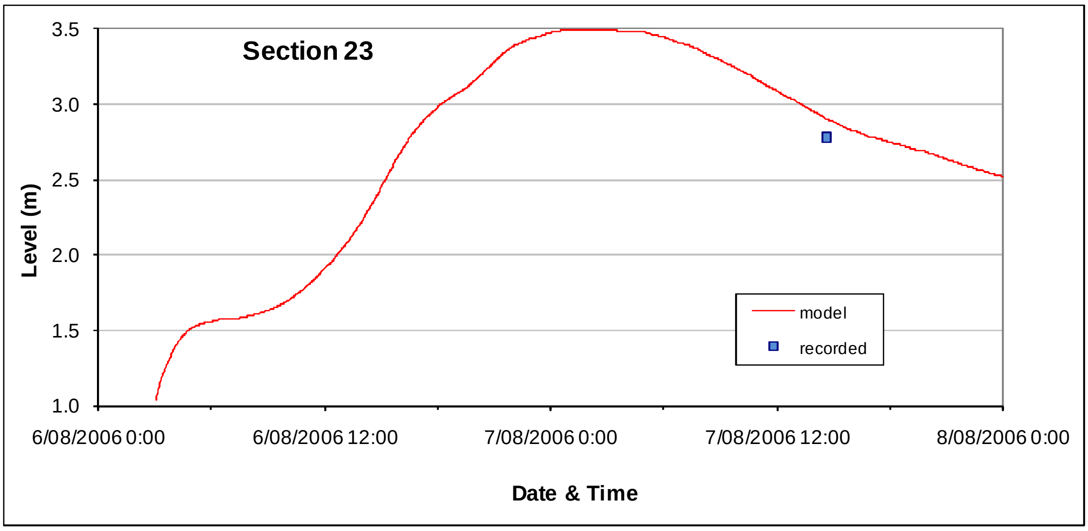
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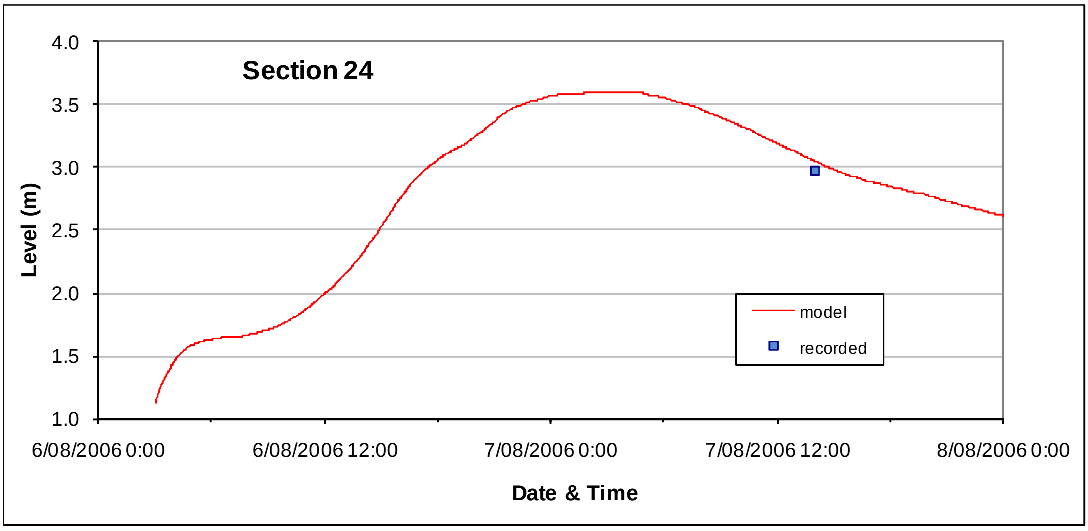
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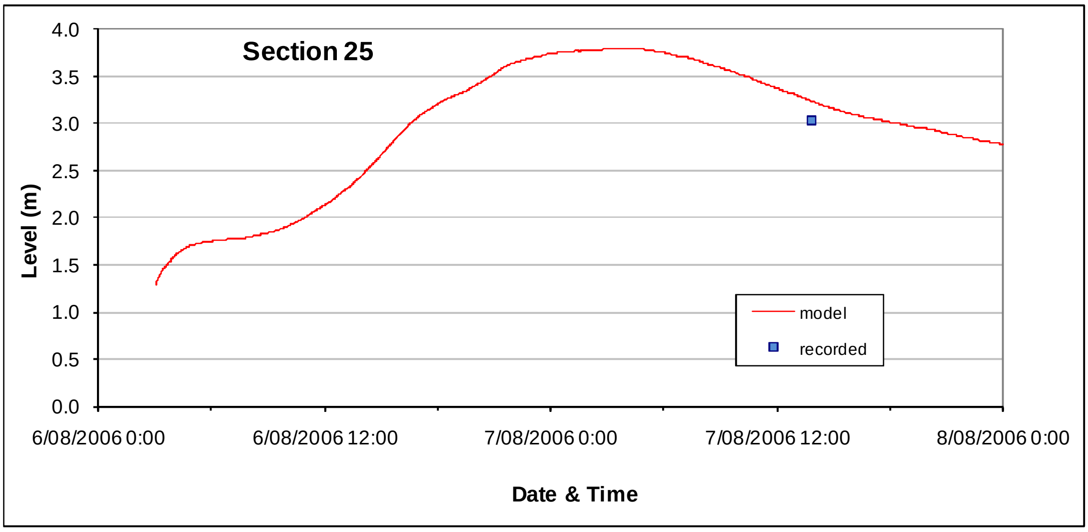

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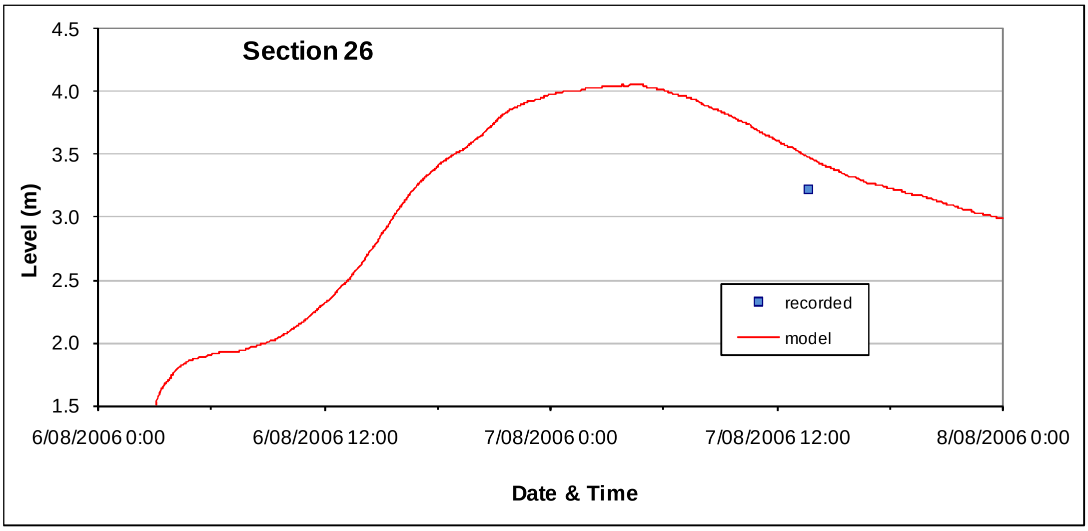

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Appendix III Design Scenario Results

Climate Change Scenario	Existing Climate						climate change 2030 ("med")		climate change 2030 ("high")		climate change 2080 ("med")		climate change 2080 ("high")	
Existing or Design	Existing Situation (spillage modelled)				Design levels i.e. no spillage		Design levels i.e. no spillage		Design levels i.e. no spillage		Design levels i.e. no spillage		Design levels i.e. no spillage	
Flow Scenario	Q100T20	Q20T100	Q200T20	Q500T20	Q100T20	Q20T100	Q100T20	Q20T100	Q100T20	Q20T100	Q100T20	Q20T100	Q100T20	Q20T100
Cross-section														
1a	2.021	2.143	2.046	2.183	2.110	2.146	2.251	2.477	2.298	2.500	2.473	2.598	2.558	2.621
2	2.625	2.593	2.651	2.781	2.716	2.601	2.857	3.029	2.903	3.065	3.069	2.954	3.174	3.034
FORDLOOP 100	2.624	2.593	2.650	2.779	2.715	2.600	2.855	3.024	2.900	3.060	3.064	2.951	3.167	3.030
FORDLOOP 500	2.624	2.592	2.650	2.779	2.715	2.600	2.854	3.024	2.900	3.060	3.064	2.951	3.167	3.029
3	2.726	2.657	2.754	2.898	2.825	2.665	2.960	3.111	3.010	3.152	3.166	3.010	3.284	3.098
4	2.889	2.764	2.921	3.104	3.004	2.773	3.140	3.273	3.200	3.322	3.354	3.120	3.503	3.228
5	2.926	2.790	2.958	3.146	3.041	2.799	3.175	3.301	3.235	3.351	3.385	3.142	3.535	3.251
6	3.028	2.859	3.063	3.275	3.153	2.869	3.288	3.402	3.353	3.457	3.502	3.213	3.671	3.333
7	3.116	2.923	3.152	3.374	3.243	2.933	3.373	3.475	3.440	3.532	3.583	3.269	3.759	3.394
8	3.343	3.082	3.385	3.661	3.492	3.093	3.630	3.715	3.710	3.784	3.857	3.441	4.072	3.594
9	3.377	3.113	3.418	3.692	3.524	3.123	3.657	3.738	3.736	3.806	3.879	3.463	4.090	3.616
10	3.540	3.237	3.585	3.885	3.697	3.248	3.832	3.900	3.916	3.974	4.057	3.589	4.285	3.756
11	3.592	3.281	3.637	3.940	3.750	3.292	3.884	3.948	3.968	4.023	4.108	3.630	4.338	3.799
12	3.658	3.337	3.703	4.012	3.818	3.348	3.950	4.011	4.036	4.087	4.175	3.682	4.409	3.855
13	3.920	3.549	3.968	4.307	4.092	3.561	4.226	4.271	4.319	4.355	4.456	3.896	4.709	4.088
14	4.016	3.627	4.065	4.417	4.194	3.640	4.329	4.370	4.425	4.456	4.563	3.976	4.822	4.176
15	4.368	3.916	4.422	4.797	4.568	3.930	4.708	4.732	4.812	4.827	4.952	4.271	5.242	4.494
16	4.504	4.028	4.560	4.942	4.713	4.043	4.855	4.875	4.963	4.973	5.105	4.387	5.401	4.618
17	4.633	4.136	4.690	5.081	4.851	4.152	4.995	5.010	5.106	5.111	5.247	4.498	5.547	4.737
18	4.846	4.295	4.909	5.277	5.088	4.313	5.236	5.243	5.352	5.349	5.494	4.663	5.801	4.913
19	5.009	4.417	5.092	5.426	5.270	4.438	5.421	5.425	5.542	5.535	5.685	4.790	6.000	5.048
20	5.118	4.496	5.220	5.504	5.390	4.518	5.540	5.540	5.659	5.649	5.800	4.869	6.112	5.128
21	5.260	4.594	5.368	5.602	5.548	4.619	5.698	5.696	5.818	5.805	5.958	4.970	6.273	5.236
22	5.514	4.762	5.597	5.738	5.821	4.796	5.970	5.964	6.089	6.072	6.228	5.149	6.547	5.423
23	5.792	4.957	5.894	5.950	6.096	4.998	6.244	6.235	6.363	6.342	6.500	5.352	6.818	5.633
24	5.972	5.082	6.110	6.153	6.259	5.126	6.407	6.397	6.525	6.503	6.662	5.480	6.981	5.764
25	6.253	5.293	6.476	6.544	6.499	5.339	6.644	6.633	6.761	6.739	6.897	5.691	7.215	5.976
26	6.735	5.624	7.190	7.365	6.913	5.674	7.065	7.051	7.188	7.162	7.329	6.034	7.662	6.328
Te Matai Recorder	7.205	5.979	7.835	8.079	7.341	6.018	7.500	7.485	7.630	7.601	7.780	6.385	8.130	6.687

Note: Q100T20 refers to 100 year (1% AEP) flow in Kaituna and 20 year (5% AEP) sea level conditions, etc

Appendix IV Stopbank Top-up Cost Estimates

MEMORANDUM

To: Robbin Britton
Project Engineer

From: Ingrid Pak
Environmental Engineer

Date: 4 June 2008

File Ref: 5600 05

Copy To: Phil Wallace, Arch Delahunty

Subject: **Cost estimates for Lower Kaituna stopbank topups**

Robbin,

As per your brief from 2 May 2008 I have undertaken a cost estimate for stopbank top-ups on the Lower Kaituna River between Te Matai and the river mouth.

Volume calculations are based on the design levels supplied by Phil Wallace in May 2008. Two scenarios were considered:

1. Existing climate
2. Climate change to 2030 ("medium" scenario)

Some assumptions were required to be made to proceed with the volume calculations. Volume calculations are based on the existing stopbank geometry as far as available from cross-sectional surveys. Generally, assumptions were made to the outside batter as 3:1 as per original design drawings. The geometry of the design stopbank is 3.0m top width and 3:1 batter. Crowning was used where possible, otherwise widening was assumed on the landward side of the stopbank. Ground level on the landward side was assumed to be similar to the river side (berm). For each cross-section an area was calculated based on an average top-up height for the stopbank section immediately upstream. This area was then multiplied by the distance to the next cross-section to gain a volume for each stopbank section.

A unit rate of $25.00/m^3 was applied to the calculated volumes. This rate includes winning and royalties at the quarry, carting, placing, compacting, and shaping of material. In addition to that, a rate of $25.00 per lineal metre is estimated for removal and replacing of vegetation and topsoil, fencing, and grassing. Further costs for establishment (approx. $10,000 per contract) and traffic control (approx. $50,000) will apply. No costs for geotechnical investigations and remedial works (e.g. toe loading, cut-off trenching, etc.) are considered here. It is also noted that the unit rate above is based on the current price regime. Especially with rising fuel costs this rate could potentially be outdated in the not too distant future.

Results of the volume calculations and cost estimates are presented in the attached spreadsheet. The estimated total costs are broken into sub-reaches as requested below.

Sub Reach Description	PDF Colour Code	Current Cost Estimate	Planned Construction Date	Ingrid's cost estimate Existing	Ingrid's cost estimate 2030
Kaituna River – LB d/s of Waiari	Pink line	$113,315	2008/09	$313,548	$699,889
Kaituna River – RB d/s of 3M (i.e. d/s of Kaituna Wetland)	Brown line	$214, 790	2009/10	$227,735	$537,393
Kaituna River – LB & RB Railway to Waiari	Purple box	$260,152	2008/09	$2,199,577	$2,833,668
Kaituna River – RB5268 (i.e. Kaituna Wetland) to Waiari	Green line	$114,244	2009/10	$72,402	$260,655
Total Cost		**$702,506**		**$2,813,262**	**$4,331,605**

Ingrid Pak
Environmental Engineer

Appendix V Hydraulic Assessment of TEM Bridge

TEM BRIDGE – IMPACTS ON KAITUNA RIVER FLOOD LEVELS

Philip Wallace, 24 April 2008

Introduction

A twin bridge to carry the TEM over the Kaituna River is proposed near river cross-section 13. The proposed bridge will have two piers in the river flood channel for each of the two carriageways (Beca/TransitNZ drawings 3932036, sheets SK012, SK013, SK015, S193).

Effect on Flood Levels

From the drawings, the pier width is 1.5m each for the main portion of the pier. Dimensions for the width of the "hammerhead" at the top of each pier were not given, but scaling from the small-scale drawings indicates that they are perhaps about 1.75m wide. As the piers appear to align with the flow, the length of the piers is considered immaterial.

It is assumed that some debris will collect on the piers. However, it is assumed that the bridge soffit will be sufficiently high that debris will not build up on the underside of the bridge.

Without debris, the pier width in total is 2 x 1.5m = 3m. The channel width is about 125m, although the average width (between the base of the piers and the soffit) is $495m^2/5.09m = 97m$.

I have considered three debris blockage scenarios:

- In the first, I have assumed that 0.5m of debris collects on the outside of each pier, so that the total width becomes 2 x (1.5m + 0.5m + 0.5m) = 5m.
- In the second, I have assumed that the pier/width ratio of the no-debris case (i.e. pier/width ratio = 3/97) is increased by 0.1 to 0.13. This follows the practice of Greater Wellington (WRC, 1991).
- Finally, I have used Transit Guidelines for debris rafting as suggested by Jacob Steenkamp in his email of 22 April 2008 to Robbin Britton. This gives a debris raft of 15m wide by 3m deep.

I have used four methods to assess the effect of these two cases:

- Adding the effective piers to the cross-section, and modelling with that section in a MIKE 11 model
- With the Yarnell method, with the aid of HYBRID software (Barnett Consultants, 1993)
- Using HEC-RAS (the results given are for the highest of the energy equation solution, the Yarnell method and the WSPRO method)
- Momentum Balance using Benn et al

Climate change scenarios:

- Existing flows and sea level
- Mid-range estimates for 2030 (sea level and flows)
- High range estimates for 2080 (sea level and flows)

Note that at the bridge site, the Q100 and T20 combination gives higher levels than the Q20/T100 combination. Thus the former has been used in all simulations.[4]

It is also assumed that the stopbanks upstream and downstream are raised to prevent any overflows at the design flows.

Results from the various simulations are summarised in Table 1.

Assumptions	Method	River section		Climate change scenario			
				Current	2030 (mid)	2080 (mid)	2080 (high)
Existing Situation (no bridge)			MIKE 11 chainage	Peak Flood Level (m)			
	MIKE 11	14	KAITUNA 15655.00	4.19	4.33	4.56	4.82
		13	KAITUNA 15805.00	4.09	4.23	4.46	4.71
		12	KAITUNA 16353.00	3.82	3.95	4.18	4.41
				Increase in peak flood level (m)			
no debris on bridge	MIKE 11						
	HYBRID (drop across bridge), W/S = 0.024			0	0		0
	HEC-RAS			0.01	0.01		0.02
0.5m debris each side	MIKE 11	14					0.061
		13 (bridge)					0.038
	HYBRID (drop across bridge), W/S = 0.05			0.01	0.01		0.01
	HEC-RAS			0.01	0.01		0.02
Increase pier width ratio by 0.10	MIKE 11, (5.3m debris each pier)	14			0.076		0.09
		13 (bridge)			0.041		0.047
	HYBRID (drop across bridge), W/S = 0.13			0.02	0.02		0.03
	HEC-RAS			0.02	0.02		0.02
Transit Guideline, 15 x 3 m raft per pier	MIKE 11, 15 x 3m debris each pier	14					0.401
		13 (bridge)					0.282
	MIKE 11 method 2	14					0.133
		13					0.144
		bridge					0.041
	HYBRID (drop across bridge), W/S = 0.18			0.03	0.03		0.05
	HEC-RAS	14		0.05	0.04		0.05
		13		0.05	0.05		0.05
	Benn et al Figure 6.3	bridge					0.05

Table 1 Model Results for Flood Levels at Bridge Site and Impacts of the Bridge

The impact of climate change is substantial – potentially raising flood levels around the proposed bridge site by 600mm by 2080.

The impact of the bridge is less, with results suggesting that flood levels would rise by of the order of 50-100mm for the most conservative debris blockage assumption. (For each debris blockage assumption, the HYBRID and HEC-RAS results are similar while the more simplistic MIKE 11 method used gives higher results. As outlined in Appendix 1, the MIKE 11 predictions of flood level increase are probably in general too high, and for the final debris assumption a second refined MIKE 11 prediction was made. The theoretical approach given in figure 6.3 of Benn et al also suggests an afflux of 50mm if a drag coefficient of 1.33 is used. This is probably unrealistic and greater drag for a debris raft would be likely, leading to a greater afflux.)

[4] In hindsight, water levels for the 2030 (mid) climate change scenario are higher for the Q20/T100 scenario at the bridge site. Results given here however are for the Q100/T20 scenario.

River Velocity

Results from the various methods and scenarios indicated that the channel average Q100 velocity is in the order of 1.5m/s to 2m/s (Table 2). The highest velocities are given by the HYBRID method. For the low tide Q100 case, with the highest debris blockage assumption and for the high climate change scenario in 2080, a velocity of 2.2m/s is predicted. The effect of climate change is to increase velocities by around 10-15%. The velocity adjacent to the bank will likely be higher – maybe a factor of 1.5 would be appropriate when designing bank protection under the bridge.

Bank protection is also required to protect against the rise and fall of tides – these plus wind waves may cause ongoing frittering away of the bank.

		Q100T20			Q100, low tide		
Assumptions	**Method**	Current	2030 (mid)	2080 (high)	Current	2030 (mid)	2080 (high)
no bridge	MIKE 11	1.56	1.60	1.75			
0.5m debris each side	MIKE 11	1.61	1.64	1.80			
Increase pier width ratio by 0.10	MIKE 11	1.68	1.72	1.88	1.72		1.93
Transit Guideline, 15 x 3 m raft per pier	HYBRID	1.93		2.17	1.93		2.21
	HEC-RAS	1.56	1.59	1.71			

Table 2 Peak Q100 Channel-Average Velocity Estimates (m/s)

General Comment

The bridge drawings show a single crossing initially, with a second crossing immediately upstream at some date in the future. Each crossing has two piers. Having two sets of piers increases the chances of debris becoming trapped. It would therefore be preferable to have a single, larger pier that could support both crossings.

References

Barnett Consultants (1993); HYBRID User Manual

Wellington Regional Council (1991); Hutt River Flood Control Scheme Review: Modelling of Debris Blockage at Bridge Waterways. Philip Wallace

J R Benn, P Mantz, R Lamb, J Riddell, C Nalluri (2004); Afflux at bridges and culverts - Review of current knowledge and practice: Chapter 5 (Factors that contribute to afflux) and Chapter 6 (Methods of estimating afflux). R&D Technical Report W5A-061/TR1, for Defra/Environment Agency

Appendix 1 Methods

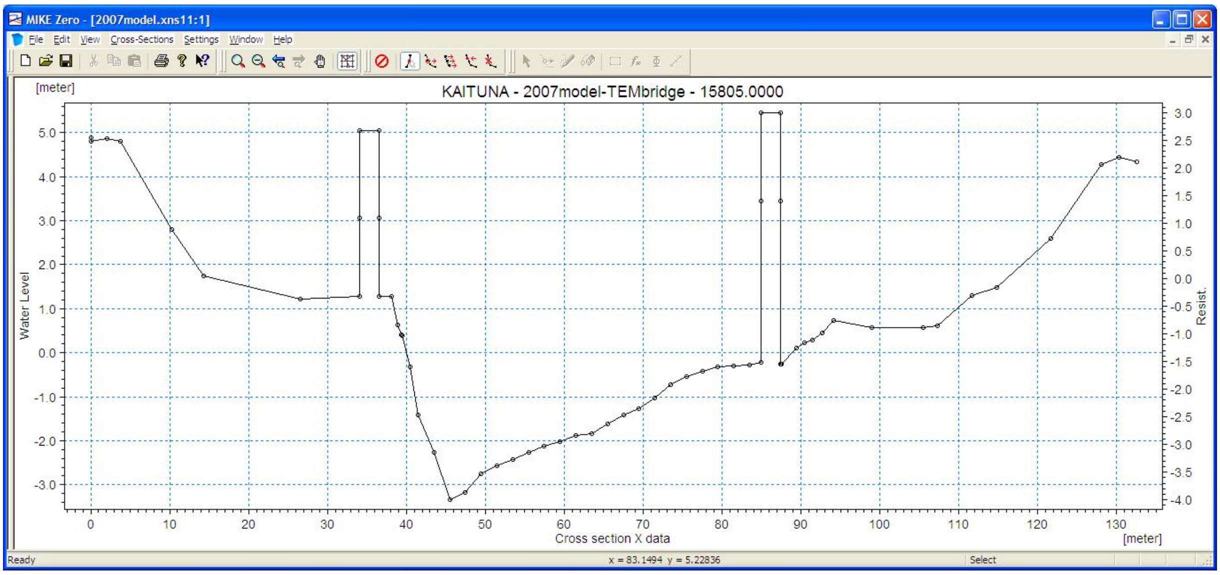
1. MIKE 11 Section

The bridge section has been added as an additional cross-section in the MIKE 11 model, as shown below.

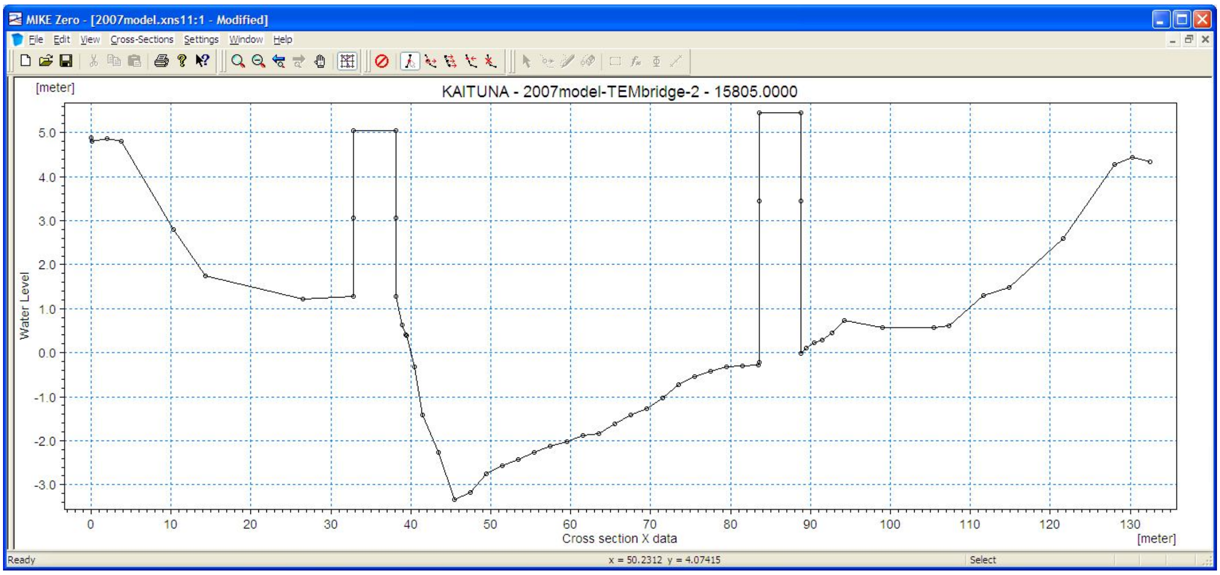
The section is a modified version of cross-section 13, and in most of the simulations it replaces section 13. However, this method predicts greater impacts of the bridge than the HYBRID and HEC-RAS assessments, particularly for the 15m x 3m debris raft case. Part of the reason for this would be the large distance between sections and the bridge section 13 therefore extending too far upstream and downstream.

An alternative approach was modelled for the 15m x 3m debris raft case: copies of the existing section 13 were applied upstream and downstream of the bridge. The use of such intermediate sections is more consistent with the assumptions made in the HYBRID and HEC-RAS assessments.

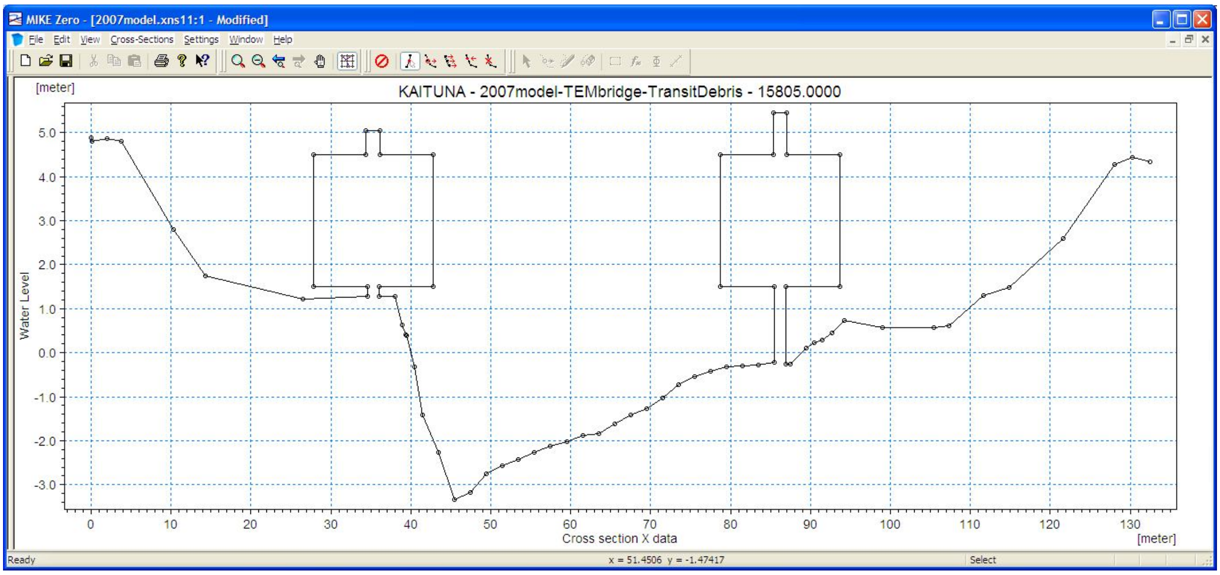
0.5m debris either side of piers



5.3m wide debris raft on piers (Note: To be consistent with the HYBRID results, this should have been a 6.3m wide raft)



Transit Guideline (15m wide and 3m high debris raft per pier)



2. HYBRID Software

Based on the Yarnell Method.

Assume that Yarnell coefficient for semi-circular pier ends (K = 0.90) still applies for the debris raft.

The effect of debris has been included by altering the pier-width ratio. The following cases have been assessed:

No debris. Pier width ratio ("W/S" in the input file) = 0.024 as supplied by Peter West.

5m wide total debris (0.5m of debris either side of each pier). W/S = 0.04 (if take total top width of 125m), and W/S = 0.05 (if use average width of 97m)

Transit Guideline debris raft (15m wide and 3m high debris raft per pier). W/S = 0.18 (blocked area = 90m^2. Section area at RL = 4.4m is 500m^2 approx. Assuming the water level is at about this level, the effective pier-width ratio becomes 90/500 = 0.18.)

3. HEC-RAS Software

Bridges can be included in HEC-RAS models. A short reach of the Kaituna River has been modelled, from section 10 to section 14. At the downstream end, the water level from section 10 from the MIKE 11 modelling has been used as a boundary condition. Flows have been taken from the MIKE 11 model at section 14. A steady state simulation has been made.

The left berm between sections 10 and 12 has not been included in the HEC-RAS model, whereas it has been included in the MIKE 11. (This area is does have some conveyance but is generally storage.) As a result the levels predicted by HEC-RAS upstream of section 12 are higher than those of MIKE 11. This is likely to mean that the velocities predicted by HEC-RAS will be slightly lower, and hence the predicted bridge losses will probably be slightly low.

The results given by HEC-RAS are for the highest energy solution of the energy equation, Yarnell's method and the WSPRO method. (Note that the WSPRO method does not calculate for the 2080 High flow scenario. In any case, the WSPRO method does not seem to be the critical case for the other two flow scenarios).

4. Momentum Balance

Figure 6.3 from Benn et al relates the dimension bridge afflux to downstream Froude number and pier width/spacing and drag coefficient.

C_d = 1.33 for an elongated pier with semi-circular ends. Assume that this still applies for a debris raft (questionable).

Pier width = 15m for the debris case. Pier spacing = 50m approx. Thus $0.5C_dT/B = 0.2$

From the no-bridge case, Mike 11 results at section 13 give $Fr = v/\sqrt{(A/width)}$ = 0.27 and also give y = 4.88m (for the 2080 high scenario).

Thus Fig 6.3 gives an afflux of 0.01y = 0.05m.

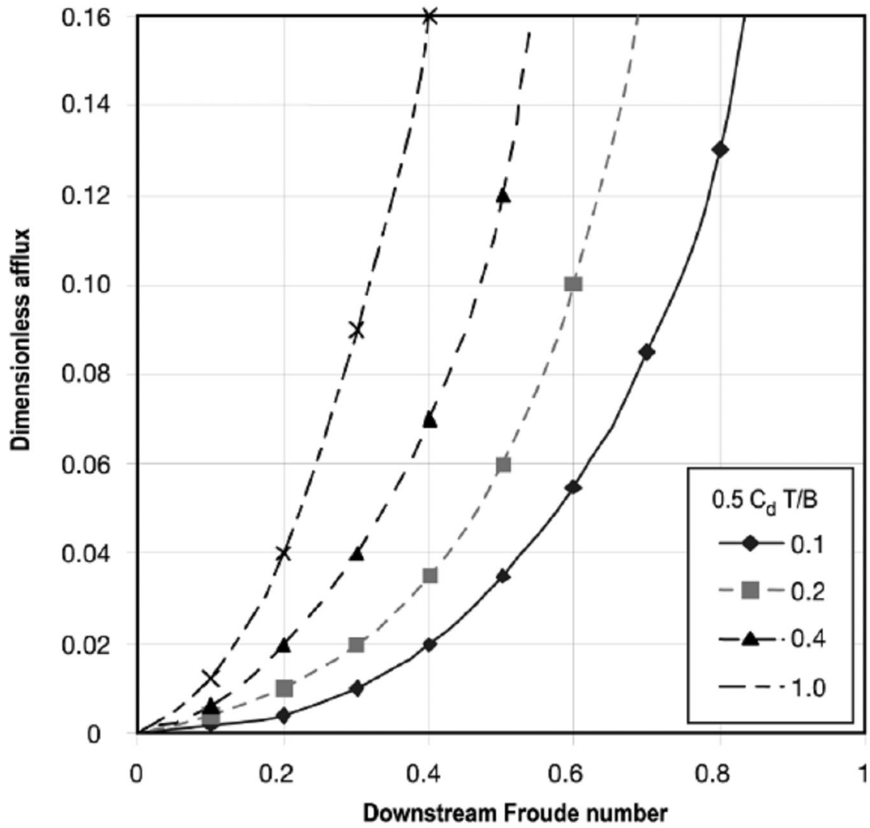


Figure 6.3: Dimensionless afflux as a function of downstream Froude number and contraction parameter

Figure 6.3 from Benn et al

Appendix VI Floodplain Storage Options

Realignment of Kaituna Stopbank downstream of TEM
Modelling Options
27 May 2008

Stopbank realignment options were discussed at the EBOP/BECA meeting held at BECA's Harrington House office on 1 May 2008.

Aim:
Reduce design flood levels by investigating alternative stopbank realignment options for the Kaituna River reach downstream of TEM Bridge.

Refer to aerial map attached or refer to R:\Robbin:\TEM: Kaituna_TEM Modelling Options v1.pdf that shows options layout.

The Options

Option 1: Shift the Kaituna stopbank to the southern extent of the Kaituna Wetland and waterproof the eastern batter face of the TEM between the two locations. Method might be to covert TEM into a stopbank (ROC $45M) or convert proposed sound barrier into a stopbank (ROC $30M).

Option 2: remove shorter length of true right stopbank – between downstream face of TEM Bridge to point where existing Kaituna stopbank intersects Kaituna wetland. Provide waterproofed bund along eastern side of TEM from river to Weld Family Trust property southern boundary and construct new stopbank from this point eastwards to river edge (following DOC wetland northern boundary).

Option 3: as for Option 2 but extend waterproofed bund further along the eastern side of TEM (into Pamment Trust Property)

Estimated Changes in Kaituna Design Flood Levels

Phil Wallace is in the process of completing modelled for the options. Flood level reductions available to date are as follows:

Cross-section	Option 1 (mm)	Option 2 (mm)	Option 3 (mm)
XS 8	-128		
d/strm of TEM	-475		
XS 13 u/stm of TEM	-442	-113	
XS 15 Kopuaroa Stream Outlet	-324		
SH2 at recorder site	-24		

Modelling is for current situation and does not take into account bridge head losses or climate change effects.

Modelling shows although we get the best reduction from Option 1 at XS13 (coming in at -442mm) the biggest reductions will be in areas where stopbanks are quite high anyway WITHOUT any significant reductions where we really need them i.e. upstream of XS19 on both sides.

Phil will complete his options report/memo shortly and Ingrid expects to have cost estimates for stopbank top-ups ready by the end of May 2008. I'm not sure having Option 3 modelled is required at this stage as we know results will lie somewhere between Options 1 and 2.

Information will be given to BECA's who will cost Option 1 and 2.

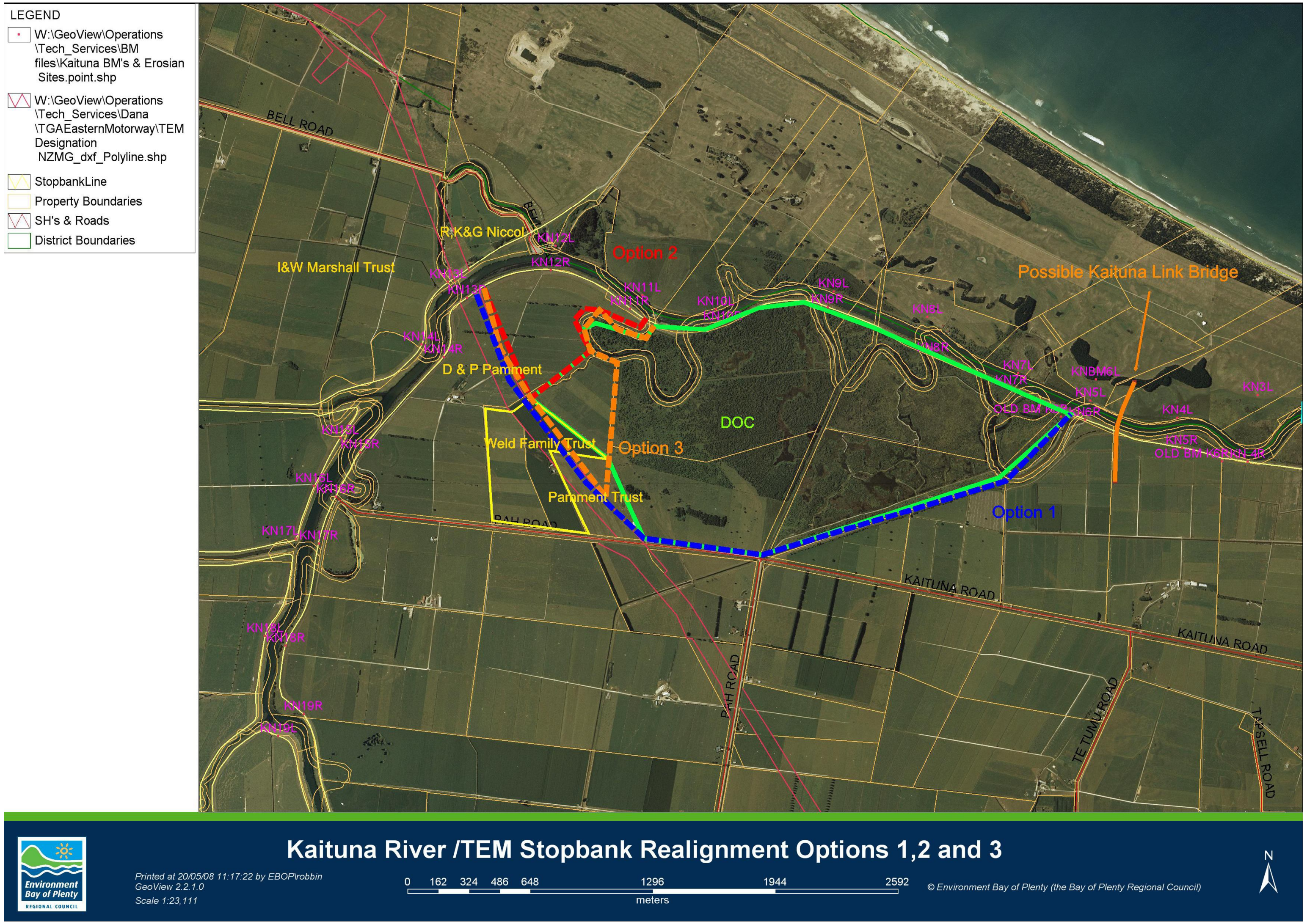


Figure VI-a Stopbank realignment options

Method

A MIKE FLOOD model has been created for the lower reach, i.e. a MIKE 11 component and a MIKE 21 component.

The MIKE 11 component is a modified version of the design model that has been described in the main text of this report. The "design" model, i.e. with no spillage allowed over the stopbanks (excluding the wetland area). The proposed bridge has not been included in the model as earlier modelling has indicated that it will have only a minor impact.

The MIKE 21 component covers the right bank floodplain from the bridge site (downstream of section 13) to section 6, over the area shown in Figure x above. Topographical data for the floodplain area is based on available LIDAR data. Unfortunately the LIDAR data do not cover the northern half of the area, and the ground levels between the LIDAR area and the river berm have been interpolated. The MIKE 21 model uses a 10m grid size.

Three cases have been modelled: Option 1, Option 2 and the current stopbank alignment. In each, the flow scenario considered is the 1% AEP Kaituna River flow, under current climate conditions.

<u>Option 1</u> A copy of section 13 has been placed at the downstream side of the bridge site, and the stopbanks from there to section 6 removed. The MIKE 11 model dx value has been set to 10m for the Kaituna River, to ensure good alignment with the MIKE 21 model. The link between the MIKE 11 and MIKE 21 models has been defined by the existing berm level.

<u>Option 2</u> is as for Option 1 but with the floodplain flow only allowed over the Option 2 area shown in Figure VI-a.

<u>Current stopbank alignment.</u> To ensure a direct comparison with the Option 1 and 2 models, a copy of section 13 has also been inserted at the downstream side of the bridge site and the MIKE 11 dx value has been set to 10m. (Therefore results will not be exactly the same as those previously shown in the main text of this report.)

Results

Results show that in Option 1, water spills downstream of the bridge site onto the floodplain and fills the area before then re-entering the river near section 6 (Figure VI-b). Peak 1% AEP flood levels in the river drop by over 400mm near section 13, with lesser improvements upstream and downstream (Table VI-a, Figures VI-c,d). Option 2 results show only a minor improvement over the current stopbank alignment. Option 3 has not been modelled; this is expected to provide a small improvement over Option 2.

Figures VI-e and VI-f show that although Option 1 does lower levels, it does not substantially change the lengths of stopbank over which immediate top-ups are required.

These results do need to be qualified: the topography over the initial flow path to the wetland is assumed rather than actual. However, if need be that area could be physically regraded to achieve the same result.

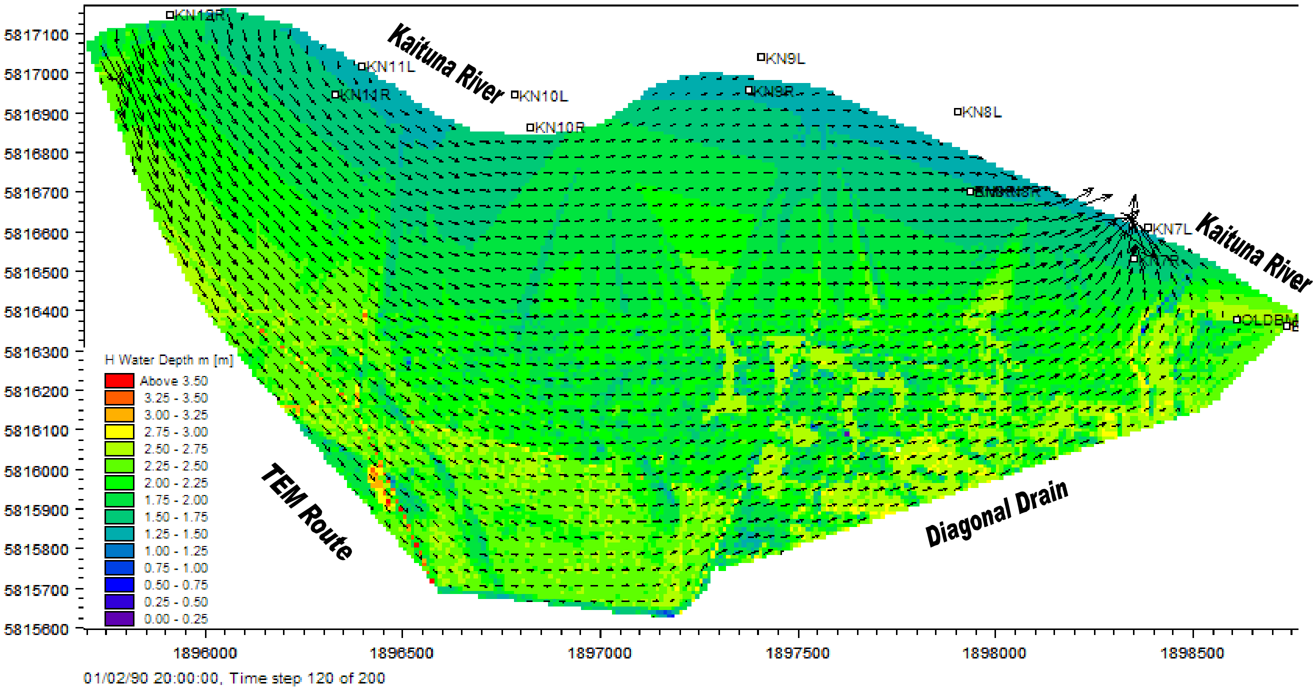


Figure VI-b Flood depths and flow patterns, Option 1, near flood peak.

Location	Peak Water Level Change (m)	
	Option 1	Option 2
FordLoop500	-0.158	-0.002
FordLoop100	-0.158	-0.001
BM3	-0.155	-0.001
BM4	-0.160	-0.001
BM5	-0.156	-0.001
BM6	-0.157	-0.001
BM7	-0.168	-0.021
BM8	-0.128	-0.014
BM9	-0.130	-0.054
BM10	-0.242	-0.049
BM11	-0.273	-0.048
BM12	-0.315	-0.043
BM13	-0.442	-0.109
BM14	-0.416	-0.100
BM15	-0.324	-0.075
BM16	-0.291	-0.067
BM17	-0.258	-0.060
BM18	-0.199	-0.048
BM19	-0.160	-0.039
BM20	-0.136	-0.034
BM21	-0.105	-0.026
BM22	-0.072	-0.018
BM23	-0.057	-0.015
BM24	-0.049	-0.013
BM25	-0.040	-0.011
BM26	-0.029	-0.008
Te Matai Gauge	-0.024	-0.007

Figure VI-b Flood depths and flow patterns, Option 1, near flood peak.

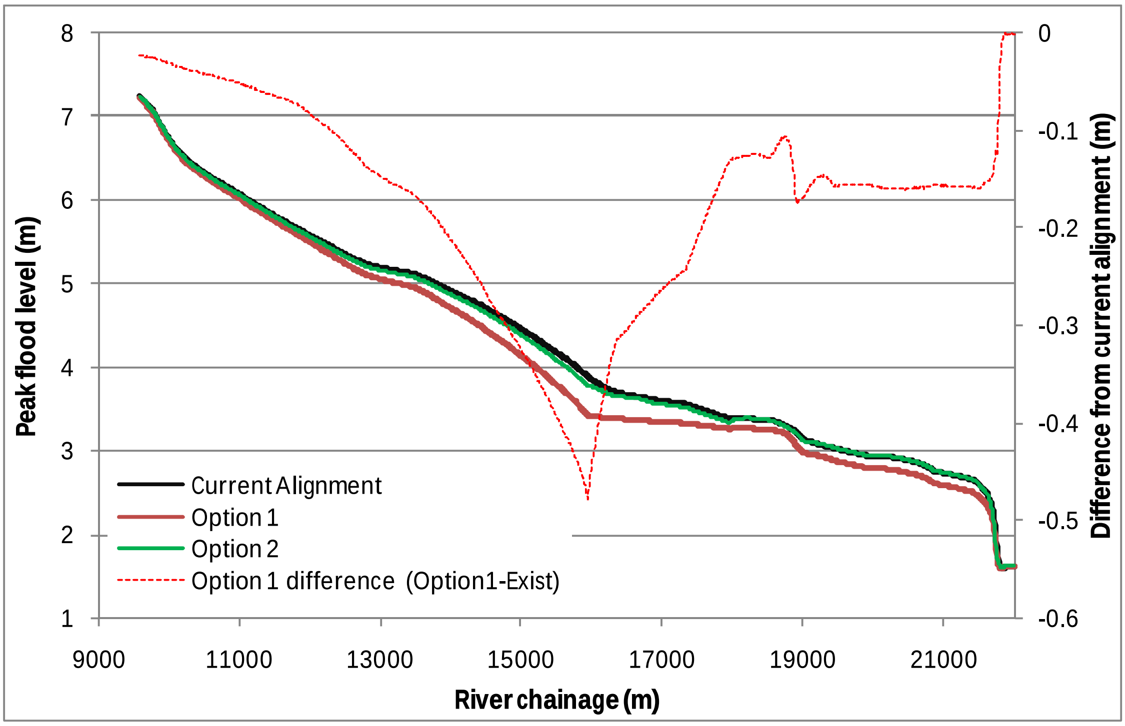


Figure VI-c Peak flood level profiles and effect of Option 1 compared to current alignment.

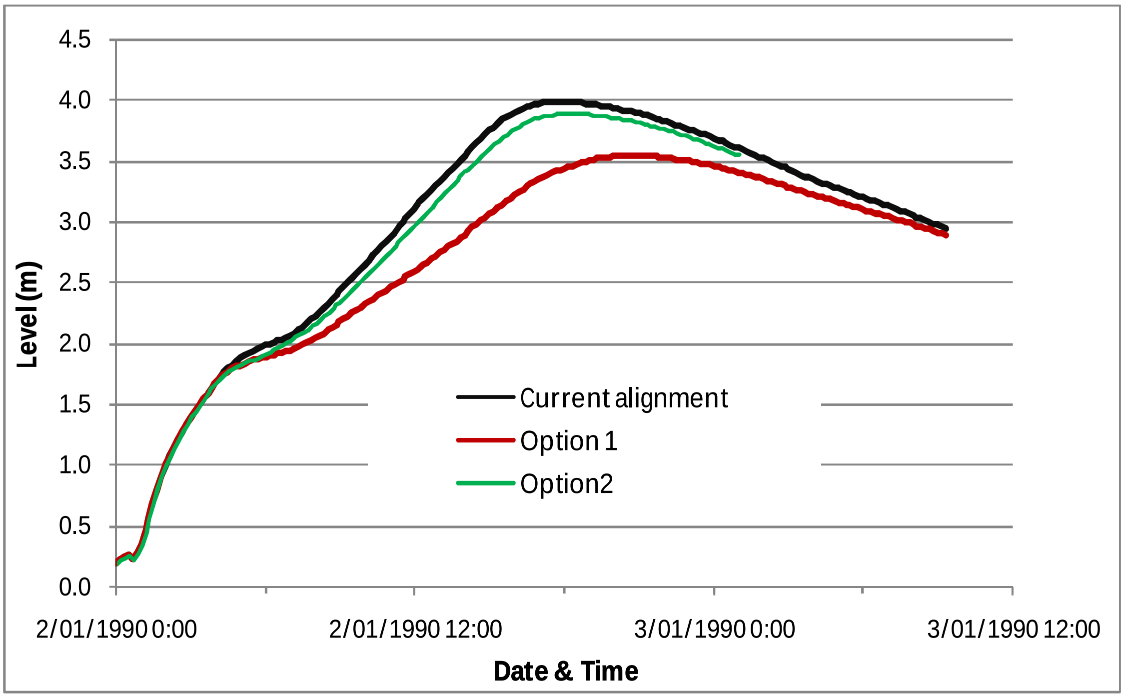


Figure VI-d Flood levels at section 13.

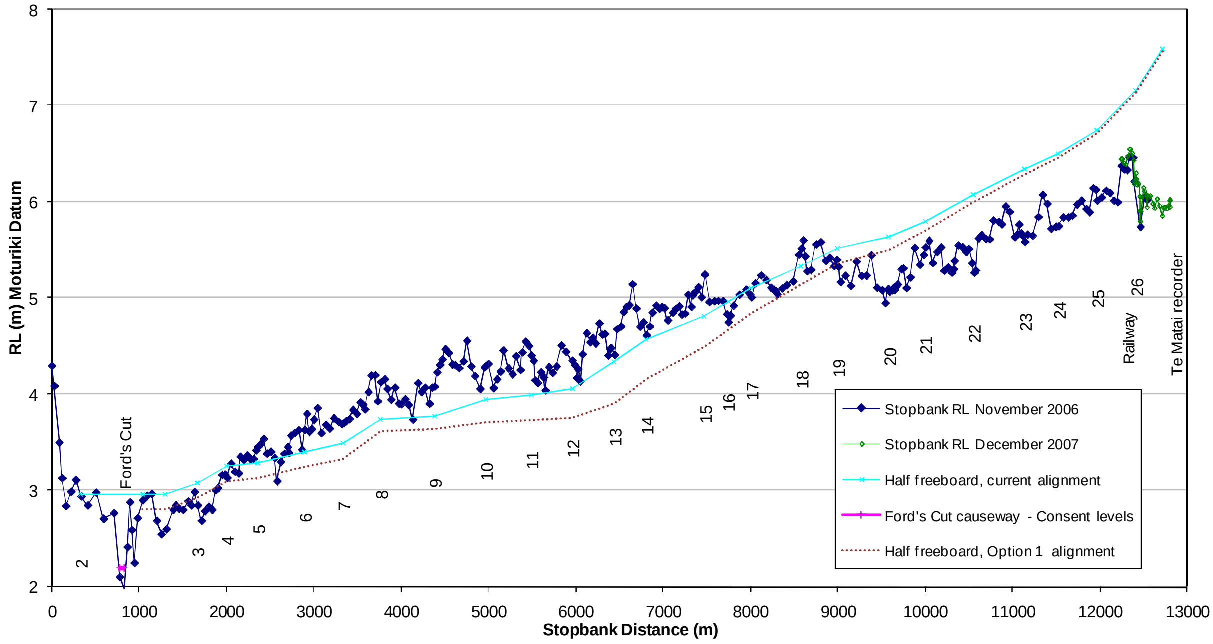


Figure VI-e Half freeboard levels, current alignment and Option 1, right bank.

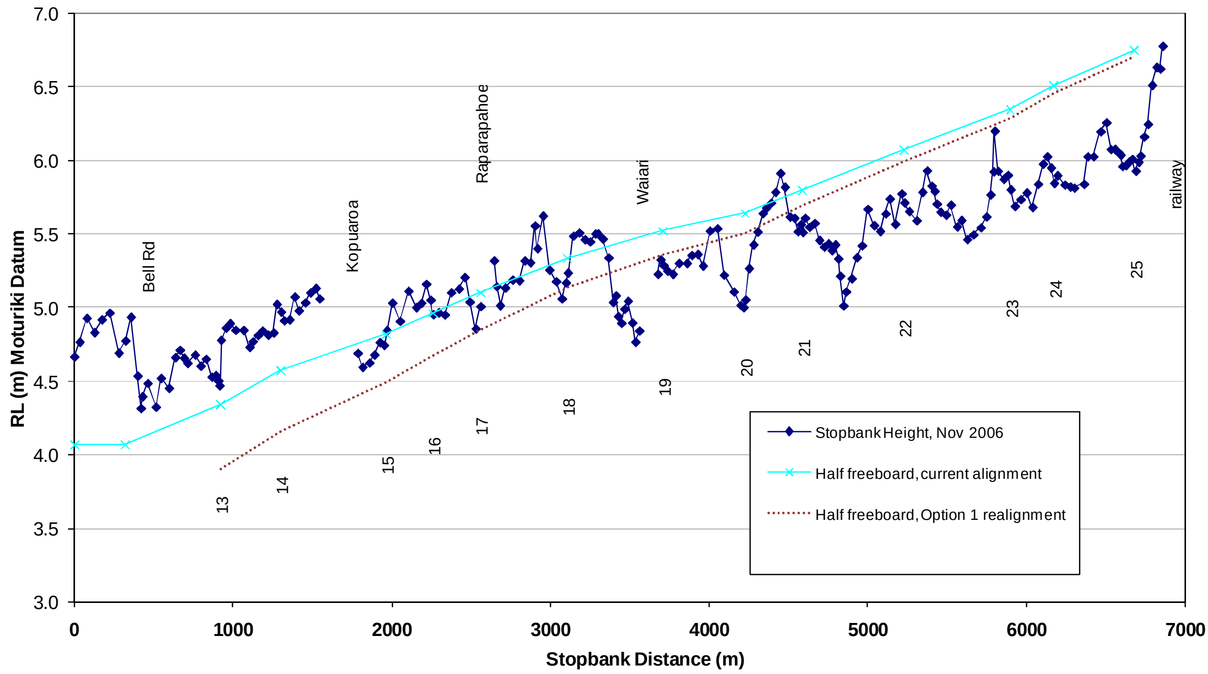


Figure VI-f Half freeboard levels, current alignment and Option 1, left bank.

Appendix VII Model Files

All model input files can be tracked via the following *.sim11* and *.couple* files.

Lower Model

- Stopbanks at current levels
  Aug06event.sim11
  May99-lower.sim11
  kaitunaQ100.sim11
  kaitunaT100.sim11

- Stopbanks raised to prevent overtopping
  kaitunaQ100-design.sim11
  kaitunaT100-design.sim11
  kaitunaQ200.sim11
  kaitunaQ500.sim11
  kaitunaQ100-design-highCC-2030.sim11
  kaitunaT100-design-highCC-2030.sim11
  kaitunaQ100-design-highCC.sim11
  kaitunaT100-design-highCC.sim11
  kaitunaQ100-design-medCC-2030.sim11
  kaitunaT100-design-medCC-2030.sim11
  kaitunaQ100-design-medCC.sim11
  kaitunaT100-design-medCC.sim11

Upper Model

KaitunaMay99.couple
KaitunaAug06.couple
KaitunaQ10-3.couple

Appendix VI Floodplain Storage Options

Q100-RealignSBtoTEM.couple
Q100-RealignSBtoTEM-option2.couple
kaitunaQ100-design-NoRealign.sim11 (effectively the same as *kaitunaQ100-design.sim11*, but allows more precise comparison with the above two floodplain storage options)