Northland Regional Council

Kaihu Flood Control Scheme Investigation
Report on Stages 1 and 2

November 2008

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LIMITED
Computational Hydraulics Specialists

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Northland Regional Council
Kaihu Flood Control Scheme Investigation Stages 1 and 2

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1. Executive Summary

The purpose of Stages 1 and 2 of the Kaihu flood control scheme investigation was to establish a base case of flooding under existing conditions in events of 2, 5, 10, and 100 year average recurrence interval, against which the effectiveness of proposed flood control measures could be assessed.

A new frequency analysis of the Kaihu Gorge flow data established both peak flood flow and flood volume for average recurrence intervals of 2, 5, 10 and 100 years. A design flood hydrograph shape was selected based on previous work by NIWA. The catchment area downstream of Kaihu Gorge was divided into 21 subcatchments with areas up to 28km^2, whose runoff hydrographs were assumed to have the same shape as the Kaihu Gorge hydrograph, but with peak flows scaled so that total catchment runoff varied as total catchment area to the power 0.8.

The tide record at Dargaville was examined by Mulgor Consulting Ltd in a subcontract that produced tide heights for return periods up to 100 years, by both Monte Carlo simulation and frequency analysis. Mulgor Consulting also produced tidal time series with peaks of 10, 2, and 1 year return period, and mean high water springs, as boundary conditions for the design flood simulations.

A hydraulic model of the river and floodplains was constructed in the Mike 11 software. The Kaihu River part of the model was based on cross sections surveyed between 1993 and 2008, and the floodplains were based on cross sections extracted from a LIDAR survey flown in March 2006. The river and floodplain branches are linked by drain outfalls, and by link channel which allow flood flows over the river banks or stopbanks.

The hydraulic model was calibrated against flood events in June 2000, August 2003, and June 2002, with the last event having by far the best set of peak flood level data. The upstream and downstream boundary conditions were the measured Kaihu Gorge flow and Dargaville tide level, respectively. The runoff from the ungauged catchment area was assumed to follow the empirical catchment area rule described above. Flood levels and durations were reasonably satisfactorily reproduced.

Design floods of 2, 5, 10, and 100 year ARI were simulated in conjunction with tidal time series having peak levels of mean high water springs, and 1, 2, and 10 year return period, respectively, following an approximate pragmatic rule established by Hong Kong Government for coastal flood studies. However this investigation does not deal with flooding primarily caused by high tide levels. Flood maps were produced as a base case reference for evaluation of flood control scheme options in later stages of the project.

2. Introduction

Barnett & MacMurray Ltd (B&M) was commissioned by Northland Regional Council (NRC) to undertake modelling of the Kaihu River and its floodplains to establish a base case for investigation of an overall flood protection scheme for the Kaihu valley. The contract for the project was based on an offer of service from B&M dated 12 May 2008, and was signed for NRC by Tony Phipps (Operations Manager) on 18 July 2008, and by Hugh MacMurray for B&M on 22 July 2008.

This project forms Stages 1 and 2 of the investigation of options for an overall flood protection scheme for the Kaihu valley. It includes upgrading the existing hydrological models, calibrating hydrological and hydraulic models of the Kaihu valley, and establishing the flooding under existing conditions in floods of 2, 5, 10 and 100 year average recurrence interval (ARI).

In an extension of the original contract, B&M was commissioned to engage Mulgor Consulting Ltd (MC) to provide a tidal level analysis for Dargaville. This study; “Extreme sea levels at Dargaville” supports this report and has been supplied to NRC.

2.1 Scope

The scope of the investigation is:

Stage 1: Upgrade the Kaihu hydrology

- Select suitable calibration floods from the historic record
- Perform a flood frequency analysis to generate design floods
- Generate hydrographs for ungauged catchment area
- Analyse tidal record to produce tidal boundary conditions (in conjunction with analysis from MC [2008])

Stage 2: Build and calibrate hydraulic model

- Produce a hydraulic model of the existing river and floodplains
- Calibrate the hydraulic model to 3 calibration floods
- Simulate 2, 5, 10 and 100 year ARI design floods to establish the base case
- Provide flood maps of the design flood simulations

2.2 Background

2.3 Site visit

Hugh MacMurray and Vicki Henderson from B&M visited the site on the 17th-18th September 2008. Numerous sites around the Kaihu valley were visited in order to gather information on critical linkages for the model. This included drainage channels, culverts, flood gates and bridges. Dimensions were measured, photos taken and assessments made of possible overflow paths and channel roughness. Suitable locations were also noted for later placement of flood monitoring sticks, in order to gain a better understanding of flood behaviour in the valley. The visit was also useful to get an understanding of the topography, floodplain vegetation, land use and channel condition and to pick up any relevant connections, structures or embankments not already listed for inclusion in the model. Valuable background information was also gathered from discussions about historic floods and land management with NRC staff and local farmers.

The Kaihu River undergoes a striking transition from a relatively steep, wide channel with coarse bed sediment and high flow capacity at Kaihu, to a narrower channel of lower gradient, fine bed sediment, and much lower flow capacity in the lower valley. Relatively frequent flooding of the lower reaches is inevitable in such a system. A suitable flood management scheme should reduce the frequency of flooding, but it seems likely that the level of service will still be relatively low compared with most other New Zealand flood control schemes, because of the constraints imposed by the natural topography.

2.4 Source data

The following data used in this investigation were supplied by NRC:
LIDAR survey of the floodplains flown in March 2006
Survey of river cross sections in 1993 (2 sections upstream of Mamaranui), 1999 (26 sections from Dargaville to Mamaranui), 2001 (four sections between Mamaranui and Kaihu Gorge), 2005 (three sections in the vicinity of the Rotu Bottleneck), and 2007-08 (seven whole valley cross sections upstream of Ahikiwi where the LIDAR coverage ends, and 16 sections between Dargaville and the upstream end of Valley Road)
A survey of various bridge waterways and culverts in the Kaihu valley
Water level record at Dargaville on the Northern Wairoa River, station 46655.
Flow record at Kaihu Gorge on the Kaihu, station 46611.
Rainfall records at these locations: Waipoua at NZMS (station 536501), Ty Ranch at Tutamoe (station 536611), Waima at Tutamoe (station 536613), Brookvale at Oputeke (station 536812), Trounson Park at Kaihu (station 537611), Coates at Whatoro (station 537614) and Mamaranui at Mamaranui (station 538801)

2.5 Software

The LIDAR data was processed to a 2mx2m grid format, and contour maps were created, using the Surfer software produced by Golden Software. Cross sections were extracted from the gridded LIDAR data using the AULOS software produced by Hydra Software Ltd. The Danish Hydraulic Institute Mike 11 software was used for the hydraulic modelling, and flood maps were produced using AULOS and Surfer.

3. Hydrology

3.1 Flood frequency analysis

3.1.1 Flow data

Flow data for the Kaihu Gorge (station 46611) has been used for this analysis. The flow record spans 39 years, extending from 1970 to the present, though there are some gaps in the record. The flow record was examined together with rainfall data from local rain gauges. After examination of the data, data from the years 1983 and 1999 was discarded, as both had gaps during which significant rainfall fell. It was possible that the annual peak flows had occurred during the gaps, so these years have not been included in the analysis. 2008 was also discarded since the year is not complete, though there was a significant event in July (peak flow 228m^3/s) which if included, would have ranked 6th largest among the annual maxima. This resulted in 36 years of data.

In their 2005 report NIWA discussed a possible shift in the flow data. A period of higher floods until 1977 followed by lower floods until about 1999 fits with longer periods of predominantly El Niño and La Niña climatic conditions in the Pacific. In El Niño conditions the North and North East of New Zealand tends to be drier, and during La Niña episodes this region tends to be wetter. These conditions are influenced by the Inter Decadal Pacific Oscillation, which is a long term pattern of anomalous sea surface temperatures in the Pacific Ocean. Since 1999 conditions have switched between El Niño and La Niña episodes, and are currently neutral. There have been several large floods in this period but it is not yet clear whether we are entering a dominant period for La Niña. The current flow record then, consists of at least 19 years through a dominant El Niño period when rainfall and flows on the Kaihu were lower than average. Flow estimates made using this full data series may tend to underestimate the actual distribution. This may be particularly true during a La Niña period. However, as the flow record extends over time, the anomalous lower and higher than average flow years will tend to average out. This point should be kept in mind when using the estimated flood flows for design purposes.

The peak discharge and maximum flood volume were selected from each year. Maximum flood volume was determined as the total flow volume from the time of local minimum before the flood until the rate of change of volume reached a minimum, or the initial minimum flow was attained after the flood peak, which ever occurred first. Any peak occurring within 3 days of the first flood peak was included in the flood. Based on these criteria, peak discharge and maximum flood volume occurred in the same event 22 out of the 36 years. Peak discharge is plotted against corresponding maximum volume in Figure 1. With a Pearson correlation coefficient of 0.70 there is some correlation between the two, but it is not strong. On this basis it has been decided to fit a distribution to peak discharge and maximum volume separately, since a distribution which fits both at once will be at best a compromise.

3.1.2 Peak discharge distribution

For peak discharge, the Extreme value type 1 (EV1) distribution as selected by NIWA (2005) will be used. A good fit was found with location and scale parameters u = 127.17

and α = 58.66 m^3/s respectively and is shown in Figure 2 together with 95% confidence limits.

3.1.3 Flood volume analysis

A statistical analysis has been carried out on the maximum volume sample values, using the method of linear moments as detailed by Hosking (1990). Parameters have been estimated for various likely distributions.

From this the Log Pearson III (LP3) distribution was chosen as a suitable fit. The distribution has parameters α = 5.066, β= 1.277 and γ= 5.777 (for volume in 10^6 m^3), where α, β and γ are respectively the location, scale and shape parameters for the distribution. The chosen distribution is plotted in Figure 2 together with the estimated 95% confidence intervals. The confidence intervals for the volume distribution are wider than those for the discharge because a three parameter distribution such as the LP3 tends to have a greater uncertainty than a two parameter distribution such as EV1 when all the parameters are estimated from a sample. While the 36 year data series is an admirable length for a New Zealand river, a sample size of 36 is, in statistical terms, rather short and increases the uncertainty in both distributions.

The maximum peak discharge in any one year is obvious, but volume is more difficult to quantify. Criteria must be developed, which means the selection of sample values is subject to some constraints; a fixed duration or what determines the end of the flood, or when multiple peaks are included; any of these may affect the viability of volume as a random, independent variable. If different criteria were used to define flood volume it would be possible to select a different set of sample values from the same data, which would lead to estimation of different distribution parameters or even be better fit by a different distribution altogether. The chosen distribution is only valid for volumes fitting the criteria used here for flood volume.

3.1.4 Historical return periods

Using the above distributions return periods have been estimated for all the annual peak discharge events. This supplies two separate estimates of return period, T, for each event. One is in terms of peak discharge, and the other in terms of maximum flood volume. These are summarised in Table 3-1 below. Note that T = 1/AEP, the Annual Exceedance Probability, which is not the same as ARI, Average Recurrence Interval, used to size the design floods in later sections.

	Peak discharge		Maximum volume	
Event	Discharge (m^3/s)	T	Vol($x10^6m^3$)	T
Aug-1970	197	3.8	12.63	2.8
Jul-1971	147	1.9	8.03	1.3
Oct-1972	223	5.6	18.45	7.8
Jul-1973	162	2.3	12.60	2.8
Apr-1974	65	1.1	7.40	1.2
Sep-1975	89	1.2	10.66	2.0
Aug-1976	148	2.0	9.98	1.8
Jun-1977	157	2.2	6.88	1.1

	Peak discharge		Maximum volume	
Event	Discharge (m^3/s)	T	Vol($x10^6m^3$)	T
Jul-1978	102	1.3	19.48	9.4
Jun-1979	142	1.9	10.98	2.1
Jun-1980	199	3.9	14.26	3.7
May-1981	189	3.4	9.97	1.8
Apr-1982	79	1.1	6.84	1.1
Mar-1984	148	2.0	5.97	1.0
Feb-1985	178	2.9	5.73	1.0
Jan-1986	99	1.2	6.24	1.0
Jul-1987	109	1.3	9.91	1.8
Mar-1988	395	96.1	37.12	253.9
Jan-1989	130	1.6	6.39	1.1
Aug-1990	46	1.0	4.49	1.0
Jun-1991	67	1.1	3.56	1.0
Jul-1992	227	6.0	7.81	1.3
May-1993	177	2.9	9.32	1.6
Jul-1994	58	1.0	6.96	1.1
Sep-1995	132	1.7	7.65	1.2
Aug-1996	70	1.1	8.97	1.5
Jun-1997	196	3.8	6.19	1.0
Jul-1998	205	4.3	11.22	2.2
Jun-2000	191	3.5	12.02	2.5
Dec-2001	206	4.3	8.87	1.5
Jun-2002	250	8.7	20.85	12.1
Mar-2003	265	11.1	10.86	2.1
Feb-2004	241	7.4	14.56	3.9
Jul-2005	79	1.1	7.38	1.2
Apr-2006	135	1.7	4.97	1.0
Jul-2007	295	17.9	18.64	8.1

Table 3-1: Estimated return periods for annual peak discharge events

The event in March 1988 was the largest overall in the 36 year time series, with a return period close to 100 years in terms of discharge, but a much higher return period of approximately 250 years for volume. Otherwise there are only 2-3 other events with a return period of about 10 years or greater. For many of the events, the return period for both discharge and volume is quite similar, but sometimes they are very different, as in the March 2003 event, where return period for discharge was 11 years compared to just 2 years for volume. This occurs when the flood rises very sharply to a high peak discharge, then falls just as sharply, generating a relatively small volume during the flood.

3.2 Design floods

3.2.1 Hydrograph shape

The NIWA (2005) investigation derived a representative hydrograph shape from the 10 largest floods by peak discharge in their data set. Each hydrograph was normalised so that the peak flow became 1, and centred with its peak at time zero. The median flow value each hour for the 24 hours before the peak and 24 hours after the peak formed the design hydrograph. This design hydrograph cannot be used directly as a basis for the design floods in this study because it is too short. Grass death starts after about 72 hours under water, so the hydrographs should be at least this long in order to see the effects of ponding in the latter stages of a flood. Also, based on the criteria for calculating flood volume, all of the annual maximum volume floods found were longer than 48 hours. Therefore the NIWA representative hydrograph was compared to a range of normalised historical floods to find a match. The selected flood is from the end of July 1998. A normalised plot of this flood with the NIWA representative hydrograph is shown in Figure 4. As can be seen, the two are a close match in form, and the July 1998 flood has a tail which will allow simulations past the critical duration for grass damage.

3.2.2 Hydrograph size

Using the frequency distributions outlined in Section 3.1, peak discharge and maximum flood volume have been calculated for the 2, 5, 10 and 100 year ARI event. These are summarised below along with their respective 95% confidence intervals.
The chosen design hydrograph shape shall be scaled to fit these peak discharge and maximum volume values.

ARI	Q	95% CI	Volume	95% CI
Years	m^3/s	+/-	10^6 m^3	+/-
2	168	13%	12.09	+25 / -23%
5	222	14%	16.51	+26 / -20%
10	262	15%	20.08	+28 / -20%
100	397	20%	32.22	+60 / -39%

Table 3-2: Peak discharge and volume for selected ARI events

3.2.3 Ungauged catchment hydrographs

The contributing catchment area between the Kaihu Gorge and the Kaihu outlet on the Wairoa River has been divided into 21 subcatchments. The subcatchment division is shown in Figure 5. The subcatchment areas range from $1.5km^2$ for Mamaranui to $28.3km^2$ for Babylon.

Design flood hydrographs for the ungauged catchments have been based on the Kaihu Gorge design hydrograph derived above. The flows for the ungauged catchments are related to the Kaihu Gorge design flows by the following equation:

$$\frac{\sum Q_{DownToSubcatchment}}{Q_{KaihuGorge}} = \left(\frac{\sum Area_{DownToSubcatchment}}{Area_{KaihuGorge}} \right)^{0.8}$$

This is an area ratio relationship, with summing from the upstream end of the catchment (including the catchment area upstream of the Kaihu Gorge recorder) down to the subcatchment in question giving a slightly greater weight to the upstream catchments. This assumes that the rainfall-runoff characteristics of the most upstream subcatchments will be most similar to those for the catchment upstream of the Kaihu Gorge and downstream valley areas may have a lower rainfall or be less similar to the catchment upstream of the Gorge. The peak flows for all the subcatchments are shown for the design events in Table 3-3. The maximum discharge from any single ungauged catchment is approximately 16% of the Kaihu Gorge peak discharge.

	Peak discharge (m^3/s)			
Catchment	2yr ARI	5yr ARI	10yr ARI	100yr ARI
Kaihu Gorge	168	222	262	397
Waingarara	22.2	29.3	34.6	52.5
Waipapataniwha	18.9	25.0	29.5	44.7
Ahikiwi	9.6	12.7	15.1	22.8
Te Kawa	12.6	16.7	19.7	29.9
Maropiu	21.4	28.2	33.4	50.6
Mamaranui	1.6	2.1	2.5	3.7
Waihue	17.5	23.1	27.3	41.3
Waiatua	13.8	18.2	21.6	32.7
Taita	18.7	24.6	29.2	44.2
Maitahi	6.4	8.4	10.0	15.1
Dip	6.9	9.1	10.7	16.3
Frith	11.0	14.5	17.1	26.0
Pouto	2.3	3.1	3.6	5.5
Rotu	4.6	6.1	7.3	11.0
Parore	7.5	9.9	11.7	17.8
Babylon	27.1	35.8	42.4	64.2
Scottys	4.3	5.7	6.8	10.3
Baylys	3.2	4.2	5.0	7.5
Mangatara	23.3	30.7	36.4	397.3
Valley	8.3	10.9	12.9	52.5
Okahu	3.5	4.7	5.5	44.7

Table 3-3: Ungauged catchments design peak flows

3.3 Historical calibration floods

The recorded floods have been reviewed in order to select 3 to be used for calibration of the model. The criteria were that they should be approximately 2, 5 and 10 year ARI events and that some calibration data should exist. This has meant that the selected floods all occurred since 2000. This also avoids differences between the topography existing at the time of an older flood and the LIDAR data used for the hydraulic model, which was flown in March 2006. The selected events are shown in Table 3-4 below.

Event	Qp	ARI(Q)	Volume	ARI(Vol)	Mean ARI
	m^3/s	years	10^6 m^3	years	
June 2000	191	2.9	12.017	2.0	2.5
August 2003	176	2.3	17.825	6.5	4.4
June 2002	250	8.2	20.85	11.6	9.9

Table 3-4: Historic calibration floods

As peak discharge and maximum volume have separate estimated distributions, a different ARI is attributed to each event in terms of each variable. Based on the mean ARI shown in the last column, these events cover the 2-10 year ARI range reasonably well. After adjusting the events in time so that each has its peak occurring at time =0, they are plotted together in Figure 6. The calibration events also cover a range of shapes; June 2000 approaches a traditional single peaked hydrograph though it is slower to peak than the other two, while August 2003 displays an initial lower peak half a day before the main flood peak and another afterwards. June 2002 has a wider shape, with multiple peaks before and after the main peak.

Matching hydrographs for the ungauged catchments were generated as for the design floods, using the calibration flood flows as a base.

3.4 Tidal record analysis

3.4.1 Calibration event tides

For the calibration flood simulations the downstream boundary condition has been extracted from the water level record on the Northern Wairoa at Dargaville. For the chosen events the maximum water levels at Dargaville were:
June 2000 2.739m
June 2002 2.295m
August 2003 2.360m

There is a gap of 27 hours in the tidal record during the August 2003 flood. The gap occurs approximately 2 days after the flood peak at the Kaihu Gorge. It has been filled by interpolating high and low tide levels from existing levels either side of the gap.

3.4.2 Design event tides

In determining the sea level to be used with the design events, a guideline developed by the Drainage Service Department, Hong Kong Government (1994) was used. This refers to a technical study which considers the combined influence of heavy rain and high sea level in stormwater drainage analysis. An approximate pragmatic rule is given for determining a return period for sea level, X, given a return period for rainfall, T, which we will take as the return period for the river flood.

An X year sea level in conjunction with a T year river flood
T=10 and X=2
T=50 and X=10
T=100 and X=10

This rule covers the more extreme cases to be investigated. For the 2 and 5 year ARI events, mean high water springs (MHWS) and the 1 year return period sea level respectively were used.

Note that the guideline deals with return periods, while the design events are described in terms of ARI. While these are not the same for smaller values of T, given the broadness of the guideline, the sea level return period has been selected by assuming that ARI ≈ T.

Mulgor Consulting Ltd (MC) was commissioned to provide a detailed analysis of the sea levels at Dargaville; "Extreme Sea Levels at Dargaville" (2008). In this analysis a relationship between peak sea level and return period has been derived by Monte Carlo simulation and frequency analysis methods. Sea levels for the return periods above have been estimated using an average from the two methods (for more detail see MC [2008]).

3.4.3 Storm surge and river flood

Mulgor (2008) noted that storm surge effects at Dargaville differ from those on the open coast. In major storm surge events, the water levels at Dargaville are elevated for an extended period either side of the peak, rather than rising and falling in 5-7 days. Possibly this is caused by interaction with the river flow, but it is difficult to separate the two effects.

For each of 24 annual peak discharge events on the Kaihu since the tidal record began in 1981, the maximum water level at Dargaville over the duration of the flood event has been extracted. The relative times of the river peak discharge at the Gorge and the maximum tidal level at Dargaville were compared. In 83% of cases the maximum tide occurred after the flood peak. This may be due to the slower reaction of storm surge to atmospheric low pressure associated with the storm causing the flood rainfall in the Kaihu Valley.

In 46% of the events the maximum tide level occurred within 1 day of the flood peak, and in 25% of events maximum tide level was within 6 hours of the flood peak. This implies that there is a reasonable chance that a local maximum tide will occur at Dargaville in the day either side of a major flood peak at the Kaihu Gorge.

4. Hydraulic model

4.1 Hydraulic model construction

4.1.1 Overview

Except in a few places, the Kaihu River and its floodplains are represented by separate branches in the model. This approach was adopted so that the observed behaviour of the river system during floods could be realistically simulated, and so that the effect of providing more or less conveyance to the river floodway by means of adjusting berm widths and stopbank heights could be accurately quantified. These two points are discussed further in the next two paragraphs.

The LIDAR survey shows that in many parts of the valley the river banks are significantly higher than the floodplains further away from the river. This topography is common even in river and floodplain systems formed entirely by natural processes, and the raised banks are sometimes known as natural levees. In some parts of the Kaihu valley, the natural levees have been further raised by addition of material dredged from the river or drains (particularly beside the main river). As a result of the raised river banks, a model that treats the river and its floodplains as a single channel is not accurate with regard to the process of filling and emptying the floodplains, although such a model may be reasonably accurate at the peak of a flood.

Looking ahead to the next stages of the project, we envisage that flood management options will largely consist of setting stopbanks back from the river to produce a floodway with greater hydraulic capacity than the existing river channel, so that farm land can be protected to a higher level of service than at present. With the model configuration adopted, new stopbank alignments can be tested relatively easily by taking land from the floodplain branches and incorporating it in the river and floodway branch.

Overflow crests with typical stopbank or river bank levels taken from the LIDAR survey were included in the model to provide hydraulic links between the river and floodplain branches. Many of the floodplain branches also have a major drain with an outfall on the Kaihu river, which provides free interaction between river and floodplains at some places.

The model includes many of the piped and floodgated outfalls, but there are some for which details were not available, and probably some which we simply do not know about. For this reason the model in its present level of detail is more reliable up to the peak of a flood than on the recession of the flood when drainage details become more important. This is consistent with the scope of work of this stage of the project, in which the main output is base case mapping of peak flood levels. As further details of the drainage system come to hand in future, these can be included in the model.

4.1.2 Kaihu River

The Kaihu River branch of the model is based on surveyed cross sections, which in some cases were extended using the LIDAR data. The cross sections were located by New Zealand Transverse Mercator coordinates which were supplied by NRC. For the recent surveys, every point was coordinated, while for the cross sections surveyed in 1999, 1993 and 2001, the coordinates of the cross section benchmark were supplied.

The distance along the river channel was calculated by digitising the channel centreline. Topomap tiles were exported from Map Toaster and loaded into Mike 11 as a georeferenced background for the model network diagram. The river channel was digitised on the network diagram, then the Mike 11 points table was used to calculate the distance along the digitised river channel. This calculation was used from the upstream end of the model at Cross section 34 (just upstream of the Opouteke Road bridge) to a point just downstream of Ahikiwi, within the extent of the LIDAR survey. From that point to Dargaville the channel centreline was digitised on contour maps generated from the LIDAR data, and the river centreline distance was calculated by spreadsheet.

To assign river chainages to the cross sections, their locations were plotted on either the Mike 11 map or the LIDAR contour maps as appropriate, and the chainage was taken from the digitising described above. This is a new independent measurement of channel distance. A list of the surveyed cross sections is given in Appendix D. In some cases the cross sections of earlier surveys were clearly superseded by the latest surveys, and were deleted.

In some places the floodplain is so narrow that it seemed unlikely that it would ever be justifiable to protect it by a stopbank, and the narrow floodplain is included in the river cross section. Such areas are noted in Appendix D.

After establishing a base set of cross sections, further intermediate cross sections were created by interpolation using the AULOS software, so that the cross section spacing is 250m or less. The main reason for interpolating extra cross sections was to provide sufficient points for hydraulic connections between the river and floodplains. Including the interpolated sections, there are 175 Kaihu River cross sections.

4.1.3 Floodplains

NRC supplied thinned raw LIDAR topographical data, which was used to generate the floodplain branches. The Surfer software was used to create a 2m grid of ground levels from the raw LIDAR data, by the kriging method. Experiments with the data showed that this method generally produces realistic ground levels, and in particular is less likely to create artificial low points on stopbanks than some other methods such as triangulation. The 2m grid was chosen so that important hydraulic features such as stopbanks and road crowns would be reasonably well resolved. The heights were stored to the nearest 0.1m to reduce data storage requirements, and because the vertical uncertainty of the LIDAR data is about 0.1m.

Contour maps (mostly with 0.25m vertical interval) were created from the gridded data using Surfer, and the desired floodplain cross sections were drawn on the maps. A graphical editor within the AULOS software was then used to extract the floodplain cross sections from the gridded LIDAR data. The spacing of the floodplain cross sections is usually about 100m, but varies considerably depending on the shape of the floodplain area being represented.

There are 34 floodplain branches, containing altogether approximately 800 cross sections taken from the LIDAR data. The locations of the branches are indicated in Figure 7, Figure 8, Figure 9, and Figure 10. Details of the floodplain branches are given in Appendix E.

LIDAR survey tends not to accurately measure the full depth of drains and rivers, because the laser beams sent out by the aircraft are not vertical (except directly under the flight path), and because if there is any water in the drain the water surface is recorded as the ground level. On the other hand there is no other survey data available for the drains. In many cases we have edited the drains on the LIDAR cross sections, to make them deeper.

This means the floodplain branches can accommodate a reasonable baseflow without flooding the land, and it improves the computational stability of the floodplain branches. Baseflows on the floodplain branches are discussed further in Section 4.1.5.

Most of the floodplain branches run more or less in the direction of the main Kaihu valley, and are bounded on one side by the river bank and on the other by high ground (either the toe of the hills or the State Highway 12 embankment or the old railway embankment). Such branches play a dual role of flood conveyance and flood storage, and the floodplain cross sections are selected with a view to realistically simulating these processes.

There are a number of side valleys on the western side of the Kaihu valley that have a significant role in flood detention, which are also included in the model. These include Taita valley, Waitukuhuruhuru valley (west of the dip in State Highway 12 north of the Rotu Bottleneck), Rotu valley (west of Pouto Farms property), Korariwhero valley (Babylon Stream and its tributaries), Scottys valley (south of Scottys Camp Road), Baylys valley (north of Baylys Coast Road), and Mangatara valley (south of Baylys Coast Road). The Mangatara valley is floodgated near the Kaihu River, limiting its flood detention role. These side valleys are included in the model as branches defined by cross sections. The model thus realistically simulates flow into and out of the side valleys.

In setting up the floodplain branches we have used the railway or State Highway 12 embankments as branch boundaries, where appropriate. This means that the model is capable of quantifying the effect of adding or removing floodgates through the embankments, as may be done for the proposed flood control scheme.

On the eastern side of the valley there are two significant side valleys, the Waihue and the Waiatua. The lower parts of these valleys are included in the model.

The floodplain cross sections are the basis of the flood maps. The mapping software reads the simulation result file, and creates a water surface between the cross sections, which is then overlaid on the ground surface to determine flood depths. The mapping occurs within a polygon formed by linking the ends of the cross sections. The cross sections must be straight lines. This can lead to incomplete map coverage, for example where the river loops back on itself and creates a small pocket that is not within the cross section polygon. There are a few such places in the model, and the flood maps should be interpreted with this in mind. While it is usually possible to refine the model to remove such mapping anomalies, we judge that the extra expenditure is not worthwhile at this stage of the project, when the requirement is for a practical tool to evaluate flood control scheme alternatives.

Most of the floodplains are in pasture, which has a relatively low hydraulic resistance. However farms also have fences, which collect flood debris and can become elements of high hydraulic resistance. Some of the floodplains have scrub or bush cover. Considering these various sources of resistance we have selected a Manning n of 0.06 for the floodplains.

4.1.4 River banks, stopbanks, and other embankments

Flow over the river banks or stopbanks provides the hydraulic linkage between the river and the floodplains. The model includes the river banks as link channels, which operate like weirs in allowing flow to pass once the crest level is exceeded. Typical bank crest levels were estimated by inspection of the contour maps with 0.25m vertical interval generated from the gridded LIDAR data. There are 81 link channels in the model, spaced more or less evenly over the length of the river – thus there is a link channel on each side of the river every kilometre on average. The link channels provide a weir crest of nominal width, generally 200m or 300m. The simulated flood levels proved to be relatively insensitive to the link channel widths, provided there is enough width to allow free overflow at reasonably close spacings.

4.1.5 Boundary conditions

The catchment area of the Kaihu River downstream of Kaihu Gorge was divided into 21 subcatchments as described in Section 3.2.3 (and shown in Figure 5). The Kaihu Gorge hydrograph was applied directly to the Kaihu branch of the model at Cross section 34. The runoff hydrographs for the subcatchments downstream of the gorge were applied to the various floodplain and river branches that best correspond to the actual runoff location. The Mike 11 software allows inflow to be distributed along a length of river, and also it allows one hydrograph to be divided between several inflow points. These facilities were used extensively, to make the model more realistic, and to avoid any computational difficulties associated with point inflows. Details of the application of flow boundary conditions to the model are given in Appendix F.

The floodplain branches all have a baseflow applied at their upstream ends, which is taken out of the model either at the lower end of the branch, or at the Kaihu River where the floodplain branch joins it. The purpose of the baseflow is to provide a smooth transition from low flow or no flow conditions to flooding. The flows over the stopbank crests sometimes rise rapidly, and the sudden increase in flow in the floodplain branches can cause computational problems. In branches that enter the Kaihu River through floodgates, the baseflow is taken out just upstream of the floodgates, as otherwise the baseflow gradually fills the floodplain branch before any water enters over the river bank. The baseflows are low enough so that the initial volume of water in the floodplain branches is small compared with the volume that enters in a flood.

The tidal boundary condition was applied to the downstream end of the Kaihu River at its confluence with the Northern Wairoa River. For the calibration floods the recorded tide levels were used. The rationale for selection of tides for the design flood simulation is described above in Section 3.4.2. Mulgor Consulting as part of their subcontract produced tidal time series with maximum levels of mean high water springs, and return periods of 1, 2, and 10 years, which were used as boundary conditions for the 2, 5, 10 and 100 year ARI flood events respectively. The time scale of the design tides was adjusted so that the maximum tidal height occurred at the same time as the maximum discharge at Kaihu Gorge. Table 4-1 gives the maximum levels of the design tide events.

Flood event ARI (years)	Corresponding tide	Maximum tidal level (m)
2	MHWS	1.848

Flood event ARI (years)	Corresponding tide	Maximum tidal level (m)
5	1 year return period	2.265
10	2 year return period	2.537
100	10 year return period	2.688

Table 4-1: Maximum levels of design tides

4.2 Calibration of hydraulic model

NRC supplied nine flood levels between Dargaville and Kaihu Gorge levelled from debris marks after the June 2002 flood. This is the best data available for calibration of the model. Limited information is also available for various other floods, including those selected for calibration in June 2000 and August 2003.

In April 2001 the stopbank running across the floodplain at the Rotu Bottleneck was raised slightly. Soil profiling work undertaken for Pouto Farms Ltd and for Northern Dairylands Ltd was used to estimate the bank level at the time of the June 2000 flood, and a separate model version was created to simulate that event.

At the time of the calibration floods, flow from the Kaihu valley into the Waitukuhuruhuru valley at the dip in State Highway 12 north of the Bottleneck, was restricted by a 1200mm culvert through a stockrace beside the highway. This restriction is included in the calibration models.

The greatest uncertainty in the calibration of the model is the inflow from catchment runoff. The river flow at Kaihu Gorge is reasonably well known, within the limitations of an extrapolated rating curve. Gaugings are generally considered to have an uncertainty of plus or minus 8%. For flood flows which rely on extrapolation of the rating curve from gaugings made at much lower flows, the uncertainty could well be plus or minus 20% or more.

However this uncertainty in the river flow is probably dwarfed by the uncertainty inherent in extrapolating the gauged river flow to the ungauged catchment area. The gauged catchment area at Kaihu Gorge is 116km^2, compared with the total catchment area of 357km^2 at Dargaville. In making the extrapolation we assume that the ungauged two thirds of the catchment behaves hydrologically like the gauged one third. Considering effects due to topography and elevation, soil type, land gradient, and variable storm paths it seems probable that there is significant uncertainty in the estimate of runoff from the ungauged catchment area.

This uncertainty could be reduced in future by recording flood flows further down the valley, for example at the Rotu Bottleneck. For the present investigation there is no option but to extrapolate the runoff from the Kaihu Gorge flow.

According to McKerchar and Pearson (1989) flood flows vary as the catchment area to the power 0.8. This rule was used to extrapolate the Kaihu Gorge flows to the ungauged catchment area, as explained in Section 3.2.3. The way we have interpreted the rule gives the least possible inflow volume. The greatest possible volume would result from scaling the runoff from each subcatchment according to the ratio of its area to the catchment area at Kaihu Gorge, and would be approximately twice as great.

The lowest possible inflow volume according to the catchment area rule, applied to an hydraulic model that is an accurate geometric representation of the Kaihu valley, with what we consider reasonable hydraulic resistance parameters, gives calibration flood levels reasonably near to those recorded. Details are given in Table 4-2.

It is not clear from the information supplied by NRC whether the flood levels are within the main channel, or somewhere on the floodplain. As the main channel capacity was well exceeded in the June 2002 event, it seems likely that most of the debris marks used to establish flood levels were on the floodplain. Accordingly left and right floodplain levels from the model as well as river levels were compared with the measured flood levels. Debris marks are not always reliable measures of peak level owing to wave action, and recession of the debris with the flood. The model tends to overestimate levels in the upper reaches, be fairly accurate at the Rotu Bottleneck, overestimate levels just downstream of Parore Road, and underestimate levels in the lower reaches.

Upstream of Mamaranui the only flood level available is from the recorder at Kaihu Gorge. Based on observations on site, a Manning n value of 0.05 seems appropriate for the rough river bed of the upper reaches. We have assumed that the transition from a relatively steep rough river bed to a narrow channel with fine sediment occurs where the sinuosity of the river course increases markedly, about 2km downstream of Ahikiwi. In the reach upstream of Ahikiwi (where LIDAR data is not available, and the floodplains are modelled as an extension of the main channel) the relative roughness for the floodplains is set to 1.5. The model underestimates the flood level at the recorder site, but in the absence of other flood levels upstream of Mamaranui we prefer to strike a balance between matching the one recorded level and choosing reasonable resistance settings. The Manning n of 0.075 for the floodplains and 0.05 for the river channel is considered believable, but higher values would be difficult to justify.

We have assumed that the Manning n for the Kaihu River channel changes from its upper reach value of 0.05, to its lower reach value of 0.032 through the most upstream of the big oxbow loops, at Maropiu. Manning n of 0.032 is probably at the low end of the range of values expected for the most of the lower Kaihu River. The approach in calibration was to leave the floodplain resistance fixed at Manning n of 0.06, and to adjust the river Manning n to improve the agreement with the measured levels. We understand that it is proposed to install flood peak level recording sticks at a range of sites over the Kaihu valley, and when flood data from these is available, it is recommended that the calibration of the model, including the balance between river channel and floodplain resistance, be reassessed.

Cross section	Description	Model river branch	Model river chainage (m)	Recorded level (m)	Model levels (m)
32	Kaihu Gorge	Kaihu	1820	66.82	66.33

Cross section	Description	Model river branch	Model river chainage (m)	Recorded level (m)	Model levels (m)
26	Upstream of Mamaranui bridge	Kaihu	16830	8.65	9.57
		Maitahi	1680		8.54
24	1200m downstream of Mamaranui bridge	Kaihu	18560	7.7	7.98
		Maitahi	3265	7.7	7.97
21	4500m downstream of Mamaranui bridge	Kaihu	21230	6.2	6.45
		Frith	1145		6.11
		Cemetry	1335		6.51
20	Upstream of Rotu at dip in SH12	Kaihu	22820	6.06	6.03
		Frith	1845		6.05
		Cemetry	2080		6.14
9	Downstream of Parore Road bridge	Kaihu	32570	3.5	3.96
		Brown	2120		3.97
8	740m downstream of Parore Road bridge	Kaihu	33390	3.44	3.66
		Parorelb	1130		3.74
		Parore-rb	558		3.1
7	Upstream of Marae between CS 6 and 7	Kaihu	35115	3.28	3.07
		Parore-rb	1870		3.09
6	Te Hoanga Marae	Kaihu	35610	3.3	2.94
		Parore-rb	2530		3.07

Table 4-2: Comparison of simulated and measured flood levels for the June 2002 flood

Observed data for the June 2000 and August 2003 floods is more limited. Tables 4.3, 4.4, and 4.5 give flood duration observations for all three calibration floods (collated for the legal action of Northern Dairylands Ltd against Pouto Farms Ltd) and the corresponding model results. The model agrees reasonably well with the observations in most cases.

Location	Nominal flooding level (m)	Model branch & chainage (m)	Flooding duration (h)	
			Observed	Modelled
CS25, 0.6km downstream of Waihue Rd	6.5	Maitahi 2275	72	80
	6.5	Waihue 1365		74
CS20, 1.5km upstream of Rotu Bottleneck	4.0	Frith 2270	96	103
Rotu stopbank	5.3	Bush 1555	8-12	16

Table 4-3: Observed and simulated flooding duration in June 2002 flood

Location	Nominal flooding level (m)	Model branch & chainage (m)	Flooding duration (h)	
			Observed	Modelled
CS25, 0.6km downstream of Waihue Rd	6.5	Maitahi 2275	72	71
	6.5	Waihue 1365		41
CS20, 1.5km upstream of Rotu Bottleneck	4.0	Frith 2270	96	92
Rotu stopbank	5.3	Bush 1555	4-8	14

Table 4-4: Observed and simulated flooding duration in August 2003 flood

Location	Nominal flooding level (m)	Model branch & chainage (m)	Flooding duration (h)	
			Observed	Modelled
CS25, 0.6km downstream of Waihue Rd	6.5	Maitahi 2275	48	40
	6.5	Waihue 1365		34
CS20, 1.5km upstream of Rotu Bottleneck	4.0	Frith 2270	96	62
Rotu stopbank	5.2	Bush 1555	No overflow	12

Table 4-5: Observed and simulated flooding duration in June 2000 flood

In the August 2003 flood the dip in State Highway 12 at the Waitukuhuruhuru Stream crossing was flooded to a small depth. The road level at the time was 5.46m (Judd July 2002), and the modelled peak flood level was 5.76m, which is consistent with this observation. However according to anecdotal evidence (R. Woodcock 2004) the flooding of the road occurred in the early evening of 22 August. At that time the modelled level is only about 5.2m, or just below the road level at the time. This discrepancy may well be attributable to the assumptions we have made about the amount and timing of runoff from the ungauged catchment area downstream of Rotu Gorge, as discussed at the beginning of this section.

4.3 Existing case simulations

The dip in State Highway 12 at the Waitukuhuruhuru Stream crossing was raised in late 2005, and the 1200mm stockrace culvert has been replaced with one of 2400mm diameter. The LIDAR survey was flown in March 2006, therefore it was used to give the present road levels in the dip. The existing case version of the model includes these changes.

In 2008 a section of the Rotu stopbank near the Kaihu River was removed, as part of the legal action taken by Northern Dairylands Ltd against Pouto Farms Ltd. This change was not included in the existing case model, because it is understood to be an interim solution, pending construction of a new stopbank set further back from the river than the existing one.

The inflow, tidal and baseflow boundary conditions are discussed above. The resistance settings are as in the calibration versions of the model.

The 2, 5, 10 and 100 year ARI events were simulated to establish the base case for design of flood control measures. The maximum water levels on the floodplains were used to produce flood maps for the area covered by LIDAR survey, which are presented in the next section.

In the area outside the LIDAR coverage upstream of Ahikiwi, flood mapping is not feasible. Water level time series for this reach are presented in Figure 11, Figure 12, Figure 13, and Figure 14.

4.4 Flood mapping

Flood maps were produced using the maximum water levels on the floodplains. The flood mapping facility provided in the AULOS software was used. The procedure is to export the cross sections from Mike 11 as a text file, and read it into the AULOS cross section data base. A quasi-AULOS result file is then created from the Mike 11 maximum water levels output, using specially developed software. The AULOSGrid software creates a gridded surface of water levels, using the cross section coordinates to locate the water level data. AULOSLay then overlays the water surface on the ground surface (represented as a grid of levels), and thus determines flood depth.

AULOSGrid generates a warped surface of water levels between the cross sections, which experience has shown to be reasonably realistic. The flood mapping facilities included in Mike 11 were tried for this project, but produced some anomalous results. We were advised by Danish Hydraulic Institute that a significantly shorter distance between grid points would be required to achieve better results. Considering that the model runs are already fairly slow, and that the selected floodplain cross sections resolve the topography reasonably well, we preferred not to increase the number of grid points significantly. To achieve a reasonable outcome in a reasonable time, it was decided to use the AULOS mapping routines.

The flood maps cover the flood plains but not the Kaihu River. To map the river would have required assigning coordinates to all the cross section ends, which would have been a significant extra task. Mapping the river would have limited practical value because it is not used for farming, and it is obvious that when the floodplains start to be flooded, the river stage is somewhere above bankfull. If the chosen flood control measures include a wide floodway beside the Kaihu River, it may be worthwhile to map water levels in the floodway, particularly if it is to be used for farming in normal flow times.

Owing to the sometimes convoluted planform of the river channel and valley sides, there are a few places that are not covered by the flood maps. The link branches between the Kaihu River and the floodplain branches are also not mapped, and nor are some of the various road and embankment crowns which act as hydraulic controls. These small gaps in the flood maps will have no practical effect on the trials of various flood control measures. However for final presentation flood maps for public reference, it may be worthwhile to aim for complete coverage.

The flood maps are produced on a 10mx10m grid, to keep the size of the computer files within practical limits. This is considered an appropriate level of resolution considering the purpose of the project and the size of the floodplains. Reduced copies of the maps are included in Appendix C as Figure 15 to Figure 30.

5. Conclusions and recommendations

1) Peak discharge and flood volume have been fitted to separate distributions because the correlation between them was not strong.
2) Peak discharge is well fit by an Extreme Value type 1 distribution and maximum flood volume by a Log Pearson III distribution.
3) These distributions have been used to estimate peak discharge and flood volume for the following design events:

ARI	Q	95% CI	Volume	95% CI
years	m^3/s	+/-	10^6 m^3	+/-
2	168	13%	12.09	+25 / -23%
5	222	14%	16.51	+26 / -20%
10	262	15%	20.08	+28 / -20%
100	397	20%	32.22	+60 / -39%

4) The total catchment area of the Kaihu River at Dargaville is approximately 3 times the gauged catchment area at Kaihu Gorge. The Kaihu Gorge flows were extrapolated to the rest of the catchment (subdivided into 21 subcatchments) using

the empirical rule that the total runoff varies as the total catchment area to the power 0.8.

5) Extrapolating the Kaihu Gorge flows to the ungauged catchment area is one of the main uncertainties of the investigation. It is recommended that a flow recorder be set up at a site further down the valley such as the Rotu Bottleneck, so that flood flows at this point can be recorded in future.
6) After review of the Dargaville tidal record in conjunction with the Kaihu Gorge flow record, an approximate pragmatic rule given by the Drainage Services Department, Hong Kong Government (2004) regarding the conjunction of floods and high tides was adopted. In this rule the 10 year and 100 year ARI floods are assumed to occur with tidal levels of 2 year and 10 year ARI, respectively. For the Kaihu valley investigation, the rule was extended to include floods of 2 year and 5 year ARI, for which tidal levels of mean high water springs and 1 year return period were used, respectively.
7) A hydraulic model that treats the Kaihu River and its floodplains and side valleys as separate elements has been built using the Mike 11 software. This model will be suitable for investigating flood control options, such as the creation of a floodway beside the river. The hydraulic model is based on LIDAR survey of the floodplains and conventional survey of river cross sections.
8) The hydraulic model, using the recorded inflows from one third of the catchment area, and estimated inflows from the remaining two thirds, satisfactorily reproduces observed flood levels and flooding durations from the floods of June 2000, June 2002, and August 2003.
9) The data available for calibration is sparse and it is recommended that a series of peak flood level recording sticks be set up over the length of the valley. In conjunction with a new flood flow recorder at the Rotu Bottleneck as recommended above, these will enable the calibration of the model to be improved in future.
10) It is recommended that flow recording at Kaihu Gorge should continue.
11) Design floods of 2, 5, 10, and 100 year ARI have been simulated, in conjunction with tidal water level series having peak levels of mean high water springs, 1 year, 2 year, and 10 year return period respectively. The corresponding flood extents have been mapped.
12) Contour maps produced from the LIDAR data indicate a rather dynamic floodplain in the Kaihu valley, with clear evidence of the river becoming perched above the floodplain by building of natural levees, followed by total avulsion of the river channel. The dynamic nature of the floodplain is confirmed by anecdotal evidence of ground levels rising by approximately 1m over several decades, and by our site inspection which showed a high energy river capable of carrying a large flow and large sediment load undergoing a rapid transition to a sinuous channel of low gradient, limited flow capacity, and limited sediment transport capacity.
13) Design of flood control measures on such a dynamic floodplain will require particular care to achieve a system that can be sustained in the long term for a reasonable maintenance cost.
14) It is recommended that the existing set of river cross sections be extended to provide adequate resolution over the whole of the Kaihu River downstream of Kaihu Gorge, and that these be resurveyed approximately every 10 years, so that changes in the river bed can be accurately monitored.

6. References

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