Northland Regional Council

Kaihu Flood Management Scheme Stage 3C

September 2010

BARNETT & MACMURRAY
LIMITED
Computational Hydraulics Specialists

Quality Assurance Statement	
Northland Regional Council	**Prepared by: Vicki Henderson and Hugh MacMurray**
Kaihu Flood Management Scheme Stage 3C	**Reviewed by: Hugh MacMurray**
Project Manager: Hugh MacMurray	**Approved for issue by: Hugh MacMurray** *September 2010 – BM1-238-3C*

Barnett & MacMurray Ltd
318 Grey St
P O Box 9463
Hamilton
Tel: 64-7-859 3663
Fax: 64-7-859 3662

Cover picture:

Northland Regional Council
Kaihu Flood Management Scheme Stage 3C

Contents

1. Introduction

Barnett & MacMurray Ltd (B&M) was commissioned by Northland Regional Council (NRC) to undertake refinement of the hydrological and hydraulic models for the Kaihu River catchment, and to use the refined models to produce a preliminary plan of a flood management scheme for the Kaihu valley. The contract for the project was based on an offer of service from B&M in October 2009 in two separate agreements for the hydrology and hydraulics, and both were signed on 30 October 2009 for NRC by Bruce Howse (Land/Rivers Senior Programme Manager) and by Hugh MacMurray for B&M. This work forms Stage 3C of the Kaihu Flood Management Scheme Investigation.

1.1 Scope of work

The work is divided into two distinct parts:

<u>Hydrology refinement.</u>

1. Construct and calibrate Kaihu catchment hydrology model to observed events at the Kaihu Gorge.
2. Use upper catchment hydrology to derive design rainfalls working backwards from the design flood hydrographs. Selected design events are the 0.5, 1 and 1.5 year average recurrence interval events, together with the 2, 5 10 and 100 year events already investigated in earlier project stages.
3. Apply constructed design rainfalls to the lower catchment hydrology model to produce design hydrographs for all 21 Kaihu valley subcatchments.
4. Apply two historic rainfall events to hydrology model to produce estimated historic flood hydrographs.

<u>Hydraulic model update and benchmark simulations</u>

1. Use bank survey and water surface gradient compensation to represent existing stopbanks more accurately in the model. Use LiDAR survey data to estimate levels of natural unmodified river banks, and apply water surface gradient compensation so as to represent them accurately in the model.
2. Review and update floodgate locations and dimensions using new survey data provided by NRC.
3. Simulate the June 2002 event using the updated hydraulic model and hydrographs produced with the refined hydrologic model. Adjust where necessary to achieve good agreement with recorded data.
4. Simulate scheme design floods to provide river water level data for design of the flood management scheme (expected to be between 6 months to 1 year ARI)
5. Provide a sketch plan of proposed flood management scheme, showing stopbank alignments, overtopping crests and floodgates and provide to Council for discussion and consultation.
6. Produce prefeasibility rough order cost estimates of scheme works.
7. Using the unmodified river bank profiles established in Task 1, create two "benchmark system" models, with estimated unmodified river banks, with State

Highway 12 and other road embankments, and in versions with and without the old railway embankment.

8. Simulate the 1 year ARI design flood in the benchmark system models, and calculate the volumes and durations of flooding in the drainage areas proposed to be protected by the flood management scheme. The results are to be used in a further stage of the investigation, in which overflow crests will be designed to preserve the proportions of natural flooding in a range of floods greater than the scheme design flood.

1.2 Background

Stages 1 and 2 of the Kaihu Flood Control Scheme Investigation were completed in November 2008. 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 design events of 2, 5, 10, and 100 year ARI, against which the effectiveness of proposed flood control measures could be assessed. A flood frequency analysis was performed to estimate the peak flow and volume for the above events at the Kaihu Gorge. Runoff from the subcatchments downstream of the gorge was estimated by scaling the derived gorge hydrographs by an area ratio raised to the power 0.8. A hydraulic model of the Kaihu valley was constructed, including the main river channel, stopbanks, floodgates and floodplain storage areas. The hydraulic model was calibrated and the base case scenario for each event was simulated and the flooding mapped.

In Stage 3A this initial model was used to investigate fundamental questions arising before evolving the flood control scheme. These were - how much difference do the existing stopbanks make to the flood pattern in different events? And- how much effect would widening the lower river have on flooding?
The main conclusions arising from Stage 3A were:

- Widening the lower river would decrease the flood duration on the floodplain downstream of Maitahi to the mouth, in some areas up to 3 days. Major improvements in flooding duration were observed downstream of Rotu Stream in the 2 and 5 year ARI simulations, and downstream of Parore Rd in the 10 and 100 year ARI simulations. However, it is doubtful whether the large excavation and maintenance required to achieve this is practical or cost effective,
- The existing stopbanking in the valley has an effect on flooding in all the events investigated,
- The effect of stopbanking is greatest in the upper and lower parts of the valley and less in the middle reaches, between Waihue Rd and Parore Rd.
- The hydraulic and hydrologic models of the catchment would need further refinement in order to establish a benchmark state and test proposed flood control schemes against it.

In Stage 3B specific questions relating to the alignment of the Kaihu outlet on the Northern Wairoa River were investigated.

Stage 3C is the natural progression from these earlier stages, where both the hydrologic and hydraulic models are refined in order to model a base case more accurately and with reference to this, evolve a sound flood management scheme for the valley.

2. Hydrology

As no hydrological model of the Kaihu catchment existed, refinement of the hydrology included creating a model and calibrating it. This involved several steps, which are outlined in Section 2.1 below. Each step is then described in more detail in the following sections.

2.1 Hydrology overview

A hydrologic model of the upper catchment to the Kaihu Gorge was developed based on physical parameters such as catchment area, length, slope, soil type and vegetation cover. Selected rainfall events recorded in the upper catchment were applied to the model and catchment parameters adjusted until there was reasonable agreement between the model runoff and that recorded at Kaihu Gorge for the same events. Then the calibrated model of the upper catchment was used to build rainfall events that produce the design hydrographs at Kaihu Gorge generated during Stage 1 and 2 of the investigation.

A hydrologic model of the ungauged Kaihu valley catchments was developed, taking into account any particular parameter adjustments that were required in the calibration of the upper catchments. The design rainfall events were translated to the lower catchments incorporating any differences in rainfall at the gauges across the catchment, and applied to the Kaihu valley hydrologic model to generate new design flood hydrographs for each subcatchment. The new hydrographs were compared to the old. Rainfall from two historic storm events was also applied to the hydrologic model and flood hydrographs obtained for each subcatchment. As a check, these were applied as boundaries to the hydraulic model and simulation results were compared with observed flood level and duration data from these events.

The design flood hydrographs will be used in the hydraulic model to create a baseline case for flooding against which flood control measures can be evaluated.

2.2 Catchment

The Kaihu catchment is situated in Northland extending from near the west coast to the Tutamoe Range on its eastern side, and from the locality of Tutamoe in the north to its outlet on the Northern Wairoa River at Dargaville. The catchment has an area of 35,512ha, ranging in elevation from approximately 540m in its headwaters to 3m at the outlet. Approximately $^{1}/_{3}$ of the catchment lies upstream of the Kaihu Gorge flow recorder and $^{2}/_{3}$ downstream. Above the Kaihu Gorge the catchment is steep hill country, with a mixture of native bush, regenerating native forest, pine plantations and pasture. The lower catchment is flatter, swampy in places and predominantly in pasture.

2.3 Rainfall

Long term rainfall records for local gauges were supplied by NRC. The gauges used are listed in Table 2.1 below. Gauge locations are shown together with the catchment boundaries in Figure 1 in Appendix A.

Site	Reference	Frequency	Record start	Record end	Notes
Waima at Tutamoe	536613	15 min	Dec 2003	Current	Top end catchment
Coates at Whatoro	A53661	Daily			Near gorge recorder
Mamaranui	A53881	Daily	Jan 1951	Current	Mid catchment
Dargaville	53986, 539807	Hourly	Oct 1997 Nov 2003	Feb 2001 Current	Bottom of catchment

Table 2.1: Rain gauges used in hydrology

For the upper catchments, the relevant gauges were Waima at Tutamoe, near the headwaters of the Waima river, and Coates at Whatoro, a couple of kilometres upstream of the Kaihu Gorge flow recorder. The lower Kaihu contains the Mamaranui gauge in its mid reaches, and Dargaville at the outlet.

For all the catchments, Thiessen polygons were used to calculate gauge weightings. This means the rainfall applied to each catchment is a scaled combination of rainfall from the nearest gauges. Because of the wide separation of the rain gauges, applied rainfall was from either one or a combination of two gauges for all Kaihu catchments. The same gauge weightings have been used in applying both historic and design rainfalls.

Where catchments were greater than 1000ha, rainfall has also been multiplied by an Areal Reduction Factor (ARF) as recommended in TP108 (ARC, 1998). This helps to account for the uncertainty in applying the point rainfall from a gauge to the entire catchment area. The same approach has been used for the design rainfalls, so that they are directly comparable to the gauged rainfalls. Gauge weightings and ARF for each catchment are given in Table 9.1 in Appendix B .

2.4 Rainfall-runoff model

HEC-HMS v3.2 modelling software was used to create the hydrologic model. This software is developed by Hydrologic Engineering Centre of the U.S. Army Corps of Engineers. HEC-HMS approximates each catchment as an 'open book' with two hill slopes falling to a central channel. Physical characteristics such as area, slope, length and roughness are supplied for each hill slope. The channel is also assigned dimensions, length, slope, and roughness. There are a number of model options for transforming applied rainfall into runoff on the hill slopes, and also for routing of runoff. After tests with the intial and constant, SCS curve number and Green Ampt models, the SCS curve number method was chosen to represent the loss processes for the catchments. For routing of runoff down hill slopes and in channels the kinematic wave method was used.

The parameters required for the SCS curve method are:

- Initial abstraction
- SCS curve number
- Percentage impervious area

Initial abstraction has little effect on significant events and was set to 5mm for all catchments. A curve number was calculated for each subcatchment, based on the soil type and vegetation. Guidelines for this calculation were taken from TP108 (ARC, 1999). Percent impervious area was taken from land use data supplied by NRC, but is 5% or less in 3 catchments and zero in the rest of the catchments. For the upper catchment model a seasonally varied baseflow of up to $4m^3/s$ was applied, based on recorded Kaihu Gorge low flows. No baseflow was applied to the lower subcatchments.

2.4.1 Upper catchment calibration

The upper catchment has been divided into two subcatchments, one for each tributary; the Waima and the Mangatu River. Physical catchment parameters have been measured using topographical maps, and soil and land use data supplied by NRC staff in GIS format. Using the GIS land use data from NRC, the contributing areas of pasture and forest were found for each catchment. These were used in turn to estimate an initial CN for the loss model and a hill slope roughness. Curve numbers are calculated for an area weighted land use type, divided into 3 broad categories depending on the soil type. The categories are

A. volcanic granular loam,
B. alluvial,
C. mud/sandstone.

Infiltration rates range from highest for group A, to lowest for group C. Based on soil notes provided by NRC (Cathcart, 2009) the soils tend to be clay and silt clay loams, with low infiltration rates. As a starting point, CN values for group C soils were used, with CN allowed to vary during the calibration between values for groups B and C.

Within the upper catchment, Tutamoe has the highest definition rainfall record but the shortest. From within the period of record at Tutamoe, three suitable storm events were selected for calibration purposes. Two were used for the calibration and one for verification. The events are listed in Table 2.2 below.

Event	Qp	ARI	Vol (1000m3)
March 2006	38.47	0.2	3172.3
February 2004	240.71	6.9	12444.0
July 2007	294.64	17.4	19100.2

Table 2.2: Upper catchment calibration events

As much as possible, the events selected span the range of design events that the model is expected to simulate. The flood in July 2007 was the largest event occurring during this period. A better fit could have been achieved by calibrating to just one event, but the goal is a model which will produce a reasonable runoff simulation across a range of events.

This has resulted in a calibration with some compromises, but a reasonable fit to two of the three selected events.

Rainfall events applied in the model used the rainfall pattern from Tutamoe, which has a record frequency of 15min. This temporal pattern was used to shape the total daily rainfall occurring at Whatoro. After studying the rainfall records, rain was assumed to occur at the same time at both gauges, except for the July 2007 event, when it was estimated that the rainfall at Whatoro occurred 8hrs earlier than at Tutamoe. Applying the rainfall pattern from Tutamoe at Whatoro introduces uncertainty, because the rain pattern is not necessarily the same and it may not occur at the same time. However, it is better to have an estimate of the temporal rainfall distribution than just daily rainfall totals for the calibration.

Combined runoff from the upper catchments was compared to the gauged flow at Kaihu Gorge for each event. The model was optimised for least root mean square error integrated over the entire hydrograph and peak flow agreement. Calibration variables were curve numbers and slope roughness. All other parameters were fixed. The optimum solution for each event is plotted in Figure 2- Figure 4 in Appendix A . The simulated peak discharges and runoff volumes are compared to the actual flows at the gorge in Table 2.3 following.

	Peak discharge (m^3/s)			Volume ($1000m^3$)		
Event	Gauge	Model	Ratio	Gauge	Model	Ratio
March 2006	38.5	47.5	1.23	3172	4019	1.27
February 2004	240.7	127.6	0.53	12444	6665	0.54
July 2007	294.6	256.5	0.87	19100	14436	0.76

Table 2.3: Calibration results upper catchments

The model tends to overestimate flow in small events and underestimate flow in large events. For the largest and smallest events, the error range in peak discharge was +/-23% and the volume error was $^+$/- 27% . For the February 2004 event, the error was greater at around 47%. A greater runoff volume for this event could not be achieved for CN within acceptable limits. Possible reasons why the match was not as good for this event are:

- Assumptions made about the rainfall pattern and timing at Whatoro,
- Inability to monitor storm movement and intensity in detail with current rain gauges,
- SCS loss model not as good at simulating events which begin with a period of low intensity rainfall, as it depends on a fixed loss curve,
- Model has assumed uniform antecedent conditions; if conditions in February 2004 were significantly different from those during the other events, higher runoff may have resulted. It would be difficult to address this without running a long term rainfall runoff model, or investigating many more events. Such a detailed study is outside the scope of the current investigation.

The calibration resulted in curve numbers 3% lower than group C values originally estimated from catchment soils and vegetation. Slope roughness varied between 0.25 and 0.33 for the Waima catchment, and between 0.4 - 0.6 for the more heavily forested Mangatu catchment. Based on the relative proportions of each land use type in the two catchments, average roughness values of 0.53 for scrub and forest, and 0.17 for pasture were calculated. These were used to generate roughness values for the lower catchments (see Section 2.6). Catchment parameters used in the hydrologic model are summarised in Table 9.2-Table 9.4.

2.5 Design rainfall generation

The calibrated hydrologic model for the upper catchments can be used to construct design rainfall events using a 'bottom up' process. Estimated design rainfall events are applied to the model until the simulated runoff at the gorge matches the design floods defined in Stages 1 and 2. This is a trial and error process, with no precise right answer as it is possible to generate similar runoff from slightly different rainfall patterns. However given the uncertainty in the model calibration and other inputs, design rainfalls created in this way should give reasonable results when applied to the lower catchment.

Using this process, design rainfalls have been constructed for the 2, 5, 10 and 100 year ARI events previously investigated. Rainfalls have also been constructed for the smaller 6 month, 1 and 1.5 year ARI events, as these will be required for the scheme design and evaluation.

The same gauge weightings as for the calibration were used for each catchment, and the rainfall depths at Tutamoe and Whatoro were related using a ratio from HIRDS V.2, produced by NIWA. Across a range of ARI, for storms of a 72 hour duration, this ratio remained fairly constant with Whatoro depths = 0.87 x Tutamoe depths. All the design rainfalls are plotted against a common time scale in Figure 5. With the constructed design rainfalls very good agreement with design runoff was achieved for all events. Peak discharge was within 1 % of the target peak and volume also agreed well within the main flood. Base flow before and after the event was not as well matched, but this is not significant during flood events. The simulated and target flows at the gorge for the 100 year ARI event are displayed in Figure 6. Only one event is plotted, as the design hydrographs all share the same form and the results for the rest are very similar to this.

As a check, constructed design rainfalls for Tutamoe have been compared to corresponding depths estimated by HIRDS. The rainfall depths are compared in Table 2.4 below.

	Event ARI		
	2y	10y	100y
Constructed design rainfall (mm)	161.9	233.4	337.8
HIRDS design rainfall (mm)	144.6	201.1	319.2
Constructed / HIRDS	1.12	1.16	1.06

Table 2.4: HIRDS and constructed 72 hour design rainfalls at Tutamoe

There is good agreement between the HIRDS and constructed design rainfall depths, with constructed depths being higher but 16% higher at the most. This may be due to a larger fraction of the constructed rainfall having an intensity too low to contribute actively to the event runoff. Also, for simplicity of design, the constructed rainfall consists of blocks of steady and smoothly changing rainfall. Actual rainfall (on which the HIRDS figures are based) is likely to be more variable and 'peaky', and therefore more rainfall may be needed to achieve the same runoff if a smoother rainfall pattern is applied.

To transfer design rainfalls to the lower catchment, HIRDS rainfall depth ratios were used to scale the rainfall for the remaining 2 gauges. The ratios relative to Tutamoe are given in Table 2.5.

Gauge	Tutamoe	Whatoro	Mamaranui	Dargaville
Rainfall depth ratio	1.0	0.87	0.72	0.65

Table 2.5: Rain gauge depth ratios from HIRDS

Points to note about the design rainfalls:

- All rainfall is assumed to fall at the same time at each gauge.
- The same ARF are used as for actual storms, for catchment areas greater than 1000ha.
- The gauge weightings are as detailed in Section 2.3.

Because of the gauge weightings and ARF, most of the catchments in the model receive differing amounts of rainfall. Catchments higher up the valley tend to receive the most rainfall and those lower in the valley less. The total rainfall for each of the design events is summarised for each gauge in Table 2.6 below. The design rainfall time series for every catchment will be supplied in a spreadsheet accompanying this report.

	Event average recurrence interval (years)						
Gauge	0.5	1	1.5	2	5	10	100
Tutamoe	112.4	131.4	149.6	161.9	200.5	233.4	337.8
Whatoro	97.8	114.3	130.1	140.8	174.4	203.1	293.9
Mamaranui	70.4	82.3	93.7	101.4	125.6	146.2	211.6
Dargaville	45.8	53.5	60.9	65.9	81.6	95.0	137.5

Table 2.6: Design 72 hour event rainfall depths (mm)

2.6 Lower catchment rainfall- runoff model

The lower Kaihu catchment is made up of the 21 subcatchments between the Kaihu Gorge and the outlet of the Kaihu onto the Northern Wairoa River. These are outlined in blue in Figure 1. Main physical catchment parameters have been measured from data supplied by NRC. Curve number and ground roughness were adjusted based on the upper catchment calibration results. Curve number was calculated using percentages of pasture and scrub/forest present for group C soils, then reduced by 3% to match results from the upper catchment calibration. The upper catchment calibration resulted in average ground roughness values for pasture and scrub/forest as 0.17 and 0.53 respectively. Urban areas were assigned a roughness of 0.1. Roughness per catchment was calculated as an area weighted sum of these values, depending on percentage of different land types. The catchment parameters used in the hydrologic model are summarised in Table 9.2 to Table 9.4 in Appendix B

2.7 Historical floods

Before running the design floods, two historic floods were applied to the updated lower catchment hydrologic model to see how it performed. The chosen events were floods occurring in June 2000 and June 2002. The resulting runoff hydrographs were applied to the Kaihu hydraulic model as inflow boundaries, and peak water levels and flood durations compared to observed data for these floods. The hydraulic model used for the flood simulations was the most current as described in Section 3. While there is not a great deal of observed data, it is possible to get an indication of how well the updated model reflects actual flood runoff.

Source rainfall for the historic events was formed using recorded data from the Dargaville, Mamaranui and Whatoro gauges. In June 2000 Dargaville was the only gauge providing hourly rainfall, so this was used to shape the temporal pattern of rainfall at the other two gauges. Based on the pattern of daily rainfalls, the Dargaville hourly pattern was applied to the daily rainfall at the other gauges that it best matched. This means the pattern at the other gauges does not always directly mirror the pattern at Dargaville for the same day. The aim is to best match the pattern to the correct day as the storm migrates up or down the catchment.

The situation was more complicated for the June 2002 event, as there was no record for the NIWA Dargaville gauge over this period. Alternative rainfall data was supplied from the NRC Dargaville gauge, but this is non-verified data and did not appear as reliable. For example, on comparison with 3 months of data in common with the NIWA gauge at Dargaville, it only recorded on average about 50% of the rainfall, and had occasional isolated high spikes of rainfall not recorded at the other gauge, which are regarded as errors. As the alternative was to construct a rainfall hyetograph (pattern of rainfall over time) from scratch for the storm, it was decided to use the NRC Dargaville data as a basis for the rainfall, but to moderate the pattern where needed. Two changes were made to the raw hourly data.

1. Because the recorded rainfall was suspected to be lower than actual at Dargaville, the rainfall was scaled up using the HIRDS factor relative to the Mamaranui daily rainfall (see Table 2.5). This resulted in an increase in the Dargaville total event rainfall from 39mm to 162mm. Corresponding total rainfall at Mamaranui was 178mm.
2. The single peak hourly rainfall reading was very high and generated unreasonable peak discharges in some of the catchments. This rainfall was smoothed over 3 hours so that catchment peak discharges became more realistic.

The results from the historic storm hydrologic and hydraulic simulations have been evaluated where observed information was available. For the June 2002 event, peak water levels were recorded and these have been plotted against the calculated Kaihu River values in Figure 7. The water levels from the original calibration in Stage 1 and 2 are included for comparison. The most upstream point has not been plotted to avoid distortions in scale. The water levels are shown in Table 2.7.

Note that plotted levels are for the main Kaihu River only. It was not clear from the flood records whether the recorded flood peak levels were measured in the Kaihu River channel or on the floodplains, so left and right floodplain levels corresponding to that river chainage have also been included in Table 2.7. The observed level may correspond to any one of these locations.

Cross section	Description	Model river branch	Model river chainage	Recorded level	Original simulation	Updated hydrology
			(m)	(m)		
32	Kaihu Gorge	Kaihu	1820	66.82	66.18	66.33
26	Upstream of Mamaranui bridge	Kaihu	16830	8.65	9.66	9.25
		Maitahi	1680		8.55	8.69
24	1200m downstream of Mamaranui bridge	Kaihu	18560	7.7	8.03	7.94
		Maitahi	3265		8.03	7.94
21	4500m downstream of Mamaranui bridge	Kaihu	21230	6.2	6.49	6.29
		Frith	1145		6.17	6.29
		Cemetry	1335		6.54	6.48
20	Upstream of Rotu at dip in SH12	Kaihu	22820	6.06	6.08	6.08
		Frith	1845		6.1	6.14
		Cemetry	2080		6.18	6.14
9	Downstream of Parore Road bridge	Kaihu	32570	3.5	4	3.81
		Brown	2120		4.04	3.83
8	740m downstream of Parore Road bridge	Kaihu	33390	3.44	3.71	3.57
		Parorelb	1130		3.79	3.68
		Parore-rb	558		3.19	3.18
6 to 7	Upstream of Marae between CS 6 and 7	Kaihu	35115	3.28	3.11	3.07
		Parore-rb	1870		3.18	3.11
6	Te Hoanga Marae	Kaihu	35610	3.3	2.98	2.98
		Parore-rb	2530		3.16	3.00

Table 2.7: Observed peak water levels in June 2002 flood, compared with simulated river and floodplain peak levels

The greatest difference in water levels occurs in the upper reaches, the updated hydrology giving levels 0.49m low at Kaihu Gorge and 0.60m high upstream of Mamaranui bridge (although the modelled floodplain water level there agrees well with the measured level). Downstream of this, the simulated water levels are fairly close to observed, agreeing well at CS 21, and then up to 0.31m high downstream of Parore Rd bridge. Overall, the updated hydrology produces a better match to the observed water levels than the original calibration, particularly in the upper reaches. In the lower reaches the results are similar to the original calibration, and the model trend of overestimating the steepness of the water level profile between CS 9 and 6 is reproduced. The Kaihu River resistance was adjusted to attempt to improve this, by increasing Manning n to 0.042 downstream of Okahu Creek (where the heavy rice grass growth starts), and reducing it to 0.027 from

Okahu Creek to downstream of Parore Bridge. Upstream of the Parore Bridge the original Manning n of 0.032 was retained.

Observed flood durations at selected locations were available for both events. These are shown in Table 2.8 and Table 2.9.

Location	Nominal flood level (m)	Model branch and chainage (m)	Flooding duration (h)	
			Observed	Modelled
CS25, 0.6km downstream of Waihue Rd	6.5	Maitahi 2275	72	70
	6.5	Waihue 1365		70
CS20, 1.5km upstream of Rotu Bottleneck	4	Frith 2370	96	87
Rotu stopbank	5.3	Bush 1555	8-12	14

Table 2.8: Observed and simulated flooding duration in June 2002 flood

Location	Nominal flood level (m)	Model branch and chainage (m)	Flooding duration (h)	
			Observed	Modelled
CS25, 0.6km downstream of Waihue Rd	6.5	Maitahi 2275	48	40
	6.5	Waihue 1365		39
CS20, 1.5km upstream of Rotu Bottleneck	4	Frith 2370	96	45
Rotu stopbank	5.2	Bush 1555	No overflow	2

Table 2.9: Observed and simulated flooding duration in June 2000 flood

Flood duration results for the simulated June 2002 flood are close to those observed. The total runoff volume from all of the updated hydrology catchments is 97% that of catchment runoff using scaled Kaihu Gorge hydrographs in the original calibration.

For the June 2000 flood, the simulated durations are in fairly good agreement except at CS20, 1.5km upstream of the Rotu Bottleneck. Possible reasons for this include:

- The updated hydrologic model produced only 60% of the runoff estimated in the original calibration, which had better agreement with observed durations. Less water in the system will lead to reduced flooding.
- Assuming that the storm occurs at the same time across the valley may have affected the timing of peak flows such that flood durations were reduced.
- Applying the Dargaville rainfall temporal pattern may have missed more intense rainfall further up the valley which caused higher runoff and possible longer flooding.
- Saturated antecedent conditions in June 2000 may have generated more runoff than the more general model can predict.

General agreement of the updated hydrology model results with observed data is good, and is better than that produced in the original calibration, apart from the CS 20 flood duration for the June 2000 flood. Because there is not much calibration data and it does not cover the whole valley, there is little justification for adjusting the hydrologic parameters for the lower catchment on the basis of the historic event simulations.

2.8 Design flood hydrographs

Design flood hydrographs have been produced for the 21 catchments in the Kaihu Valley. They range in severity from the 6 month to the 100 year ARI event. Hydrographs were produced by applying the design rainfalls as described in Section 2.5 to the hydrologic model for the lower catchment. Hydrographs produced using the new hydrology differ, sometimes significantly from those generated by scaling the Kaihu Gorge flood hydrograph. The 5 year ARI hydrographs by both old and new methods are plotted together in Figure 8 - Figure 28 for all the catchments. Hydrograph characteristics for each catchment in the larger ARI events are compared in Table 2.10 below. The complete new design hydrographs for all events are provided in a spreadsheet with this report.

	New/old peak flow				New/old volume			
Catchment	Event ARI (years)				Event ARI (years)			
	2	**5**	**10**	**100**	**2**	**5**	**10**	**100**
Ahikiwi	1.72	1.64	1.61	1.54	1.19	1.20	1.24	1.29
Babylon	0.80	0.87	0.97	1.03	0.89	0.91	0.96	1.02
Baylys	1.42	1.34	1.31	1.29	0.93	0.95	0.99	1.06
Dip	1.25	1.25	1.24	1.24	0.92	0.95	0.98	1.05
Frith	1.24	1.25	1.26	1.28	0.99	1.02	1.05	1.12
Maitahi	1.56	1.48	1.46	1.42	1.03	1.05	1.09	1.16
Mamaranui	2.03	1.92	1.90	1.85	1.31	1.34	1.39	1.48
Mangatara	1.06	1.09	1.11	1.16	0.88	0.90	0.94	1.01
Maropiu	1.03	1.06	1.08	1.17	0.97	0.99	1.03	1.09
Okahu	1.45	1.40	1.36	1.32	0.94	0.97	1.01	1.08
Parore	1.05	1.11	1.16	1.20	0.96	0.99	1.03	1.11
Pouto	1.58	1.49	1.48	1.41	1.00	1.02	1.06	1.13
Rotu	1.55	1.47	1.43	1.40	0.99	1.01	1.05	1.13
Scottys	1.41	1.33	1.31	1.28	0.90	0.93	0.97	1.03
Taita	0.78	0.88	0.98	1.05	0.90	0.94	0.98	1.05
TeKawa	1.62	1.55	1.53	1.48	1.11	1.13	1.17	1.23
Valley	0.98	1.04	1.08	1.12	0.88	0.91	0.95	1.02
Waiatua	1.28	1.26	1.26	1.25	0.93	0.96	1.00	1.06
Waihue	1.35	1.28	1.26	1.22	0.90	0.92	0.96	1.00
Waingarara	1.65	1.55	1.51	1.45	1.06	1.08	1.12	1.18
Waipapataniwha	0.97	0.97	0.99	1.04	1.07	1.09	1.13	1.19

Table 2.10: Peak flow and volume ratios, design hydrographs

The major changes in going from the old to the updated hydrographs are discussed below.

The original scaled hydrographs meant that all the catchments peaked at the same time. Time to peak now differs per catchment and from peak flow timing at the Kaihu Gorge. Approximately 70% of the catchments now peak before the Kaihu Gorge, and the range of peak timing is almost 4 hours for the 0.5 year ARI, reducing to almost 2 hours for the 100 year ARI event. First catchments to peak are the smallest, like Mamaranui and Pouto, followed by those that are relatively steep. Last catchments to peak are flatter longer catchments, such as Parore or Babylon.

Catchment characteristics other than area now influence hydrograph form. This is demonstrated by some of the smaller (and shorter) catchments having a higher peak discharge and runoff volume, such as Mamaranui or Baylys. This also occurs in steeper catchments, eg, Waingarara. A few of the catchments happen to have very similar runoff to the scaled hydrographs, for example Maitahi and Mangatara. Babylon, with its relatively flat topography and lower rainfall has a lower peak runoff and volume than previously. Many of the catchments now peak before the Kaihu Gorge, but a few like Babylon and Taita peak later, because they are relatively large and flat.

Peak discharge is more responsive to event magnitude. Generally smaller catchments like Scottys have a peak discharge ratio (new:old) which decreases with increasing event magnitude, and peak discharge ratio for larger catchments such as Mangatara increases with greater event magnitude.

The ratio of new design volumes to original design volumes increases with increasing event ARI. It is interesting to note that overall, the total volume of runoff was reduced by 9% in the 1 year ARI event, and increased by 10% in the 100 year ARI event. This result indicates that the scaled area hydrograph approach used in the first stages gave a reasonable estimate of total runoff compared to a hydrologic model, despite limitations in hydrograph form and timing. It is likely that this method is fairly reliable for the total lower valley catchment because it has an area of the same scale as the upper valley. For individual lower valley catchments which are between 1 and 24% of upper valley size, runoff volumes are not as comparable.

3. Hydraulic model improvements

3.1 Additional flood storage in tributary valleys

Some relatively minor areas of flood storage that were not included in the Kaihu valley model to date were added. The locations were selected by inspection of the 100 year ARI flood maps. Off channel storage areas were determined from the LiDAR data using the Surfer software, either using the ground level grids that were prepared for previous stages of the project, or in some cases from new grids where the areas were outside the extent of the previous gridding. The new ponding areas are given in Table 3.1.

Description	Indicative coordinates (NZTM)		Installed in model at	
	E(m)	N(m)	Branch	Chainage (m)
North side of Mangatara valley	1672500	6021500	Mangatara	1790

Description	Indicative coordinates (NZTM)		Installed in model at	
	E(m)	N(m)	Branch	Chainage (m)
True right upstream of Parore Road	1674300	6025300	Brown	1185
Side valley off Korariwhero Flat	1671000	6024750	Korariwhero	1175
Side valley off Korariwhero Flat	1670750	6025250	Korariwhero	1000
Small tributary valley on true left north of Rotu Bottleneck	1672000	6029500	Kaihu	24525
Small tributary valley on true right north of Rotu Bottleneck	1670500	6029250	Bush	575
Small valley on true left upstream of Bottleneck	1671400	6030250	Frith	2530
Pocket west of SH12 upstream of Bottleneck	1670500	6029750	Bush	50
Western end of valley on true right at dip in SH12	1669000	6029750	Dip	400
True right of Waiatua valley	1670200	6033500	Waiatua	860
True right of Waiatua valley	1670900	6033000	Waiatua	490
True left side valley upstream of Waihue Road	1668700	6034900	Kaihu	16552
True left side valley upstream of Waihue Road	1668400	6035400	Kaihu	15719

Table 3.1: Off channel ponding areas added to the Kaihu valley model

3.2 Existing river banks from new survey

A new survey of existing river bank levels was undertaken for this project in December 2009 and January and February 2010. The survey extended from Okahu Creek near Dargaville to Waihue Road. Downstream of Okahu Creek survey was not practical because of the heavy growth of rice grass on both banks. Upstream of Waihue Road the degree of tree cover on the river banks meant that a GPS survey was not feasible, so planned survey between Waihue Road and Ahikiwi was not carried out.

The purpose of the survey was to determine the level of the effective overtopping crest that governs the flow of flood water between the main river channel and the floodplains. In the previous model version this was estimated from the LiDAR survey data.

The new bank survey data was incorporated into the Kaihu valley model as follows:

a. The bank survey points were plotted in plan view on contour maps prepared from the LiDAR data. This provided a check on the alignment chosen by the surveyors. Where the contour maps indicated a different alignment of the effective overtopping crest, the survey was reviewed. In one case the survey was realigned to follow the top of the Edwards stopbank (downstream of the Taita Stream confluence).

b. The bank survey points were identified by number on the contour maps, and the locations of the river cross sections were also shown. Maximum water levels from a simulation of the August 2003 flood were assigned to the bank points nearest to the respective river cross sections.
c. Water levels were interpolated for all the bank points that lay between the bank points corresponding to river cross sections, using an automated spreadsheet process.
d. The locations of the link branches of the Kaihu valley model (which allow flow between the river channel and the floodplains) were associated with the nearest bank survey points.
e. The surveyed bank levels were adjusted as follows:
   - at the point where the link branch joins the river channel, no change;
   - downstream of the link branch joining point, levels raised by the difference between the joining point water level and the interpolated water level at each bank survey point
   - upstream of the link branch joining point, levels lowered by the difference between the joining point water level and the interpolated water level at each bank survey point.
f. The bank level adjustments were checked to ensure that they were reasonable.
g. The adjusted bank survey levels were read into Mike 11 as cross sections, and the cross section processed data table was used to provide link branch geometry as a table of width versus depth.

The reason for undertaking this procedure is that the Mike 11 model uses the water level at the point where the link branch joins the river to calculate the flow over the bank. Bank levels generally more or less follow the gradient of the river, thus bank points that are downstream of the joining point will tend to be overtopped at too low a water level. The procedure described above may be viewed as a method of taking the gradient out of the bank profile, so that a long reach of bank may be attached to particular river and floodplain cross sections with an acceptably small loss of simulation accuracy.

It should be noted that the bank level correction procedure described above will not give perfectly accurate results, because the water surface gradient varies during a flood, being steeper on the front of the flood wave than on its back. The gradient may also vary from flood to flood. The August 2003 and June 2002 maximum flood levels were plotted and indicated that the gradients of maximum water levels were similar in those events. Probably the gradient of maximum water levels would be similar in all floods that overtop the river banks, and would be mainly determined by the overall valley gradient.

In principle the need for the correction procedure described above could be reduced by increasing the number of links between river and floodplain. However the Kaihu valley model contains 74 links between the river and floodplains, and a balance has to be struck between the manageability of the model and the accuracy of the representation of the river banks. It should be noted that installing the links is not a once only task, as for the present Stage 3C project, the links were installed once as described above, adjusted upwards to prevent bank overflows for estimating scheme design flood water levels in the Kaihu River, and presuming that the project continues after the forthcoming consultation phase, will be reinstalled to represent the estimated natural unmodified river banks.

Upstream of Waihue Road where no survey data was available, a new determination of the bank levels was made from the contour maps and the LiDAR data. The alignment of the overflow crests was digitised on the contour maps, and was used to "slice" a 2m grid generated from the LiDAR data using Surfer software. This procedure produces a ground level everywhere the bank crest alignment crosses a grid line. The resulting ground level points were thinned to give a similar spacing of points as was provided by the ground survey. Thereafter the points were treated in the same way as if they were true ground survey points, as described above.

3.3 Natural unmodified banks from LiDAR - procedure

There is no exact way to determine what the river bank levels may have been before modification associated with human settlement. It should also be noted that even in an entirely natural, unmodified Kaihu valley system the river bank levels would change over time with the formation of natural levees. However comparison of soil horizon data from test pits with surveyed bank toe levels in the property of Pouto Farms Ltd indicated that the latter are generally a reasonable indicator of unmodified bank levels. For this project it was assumed that ground levels along a profile 20m on the land side of the surveyed river bank crest represented the natural unmodified bank levels, with a few exceptions as noted below.

Bank toe alignments were generated automatically from the surveyed bank crest alignments by joining points placed 20m on the land side of each bank survey point. The 20m offset was made along the bisector of the angle between the line segments leading to and from each point. The surveyed and bank toe alignments were plotted together in plan view, and the automatically generated alignment was edited as necessary (in some cases oddities in the bank survey alignment meant that the generated alignment did not follow the desired consistent 20m offset).

The bank toe alignment was then used to "slice" a 2m ground level grid generated from the LiDAR data using the Surfer software, thus producing ground level points wherever the bank toe alignment crossed a grid line. Those points were thinned to give a similar spacing between points to that provided by the survey data.

The lengths of the offset profiles were then stretched or shrunk linearly, to be the same as the lengths of the corresponding bank survey profiles. The adjustments required were small. The reason for the adjustment is that the offset profiles are to be used in place of the bank survey profiles, to represent the hypothetical case of all bank modifications being removed.

In a final step of generating estimated natural unmodified bank levels, any levels higher than the corresponding point on the surveyed bank crest were disallowed, and were set equal to surveyed bank crest level. This step of the process used interpolated surveyed bank crest levels at every offset bank point, generated using an automated spreadsheet procedure.

Exceptions to this procedure for estimating the natural unmodified bank levels were made at old river meander cutoffs, and at drain outfalls. Drain outfalls are treated separately in the model, and they would allow flow onto the floodplains at unrealistically low levels if included in the bank profiles. At old meander cutoffs, there is usually very low ground 20m on the land side of the present day river bank crest. It would be unrealistic to include these areas in the unmodified bank crest, because in a truly unmodified system the bank crest controlling flow onto the floodplain would be that beside the cutoff meander loop. These two kinds of exceptions were identified by inspection of the bank longitudinal sections (supplied with this report) and the floodplain contour maps, and the associated artificially low levels were excluded when the model version with the unmodified banks (the benchmark model, described in Section 6 of this report) was created.

A particular exception occurs at the Edwards stopbank, which is a reach of formal stopbank on the true right bank downstream of the Taita Stream confluence. The stopbank was constructed with a 40m set back from the river bank (unlike other stopbanks in the Kaihu valley). Because of the topography of the floodplain, with natural levees close to the river, ground levels 20m on the land side of the Edwards stopbank would be significantly lower than the natural unmodified river bank crest. On the other hand it is likely that the berm between the river channel and the stopbank was substantially modified during stopbank construction, so the present day river channel edge levels are probably also not a good indicator of unmodified bank levels. Interpolated unmodified bank levels from points upstream and downstream of the Edwards stopbank were used in the benchmark model (see also Section 6 of this report).

3.4 Natural unmodified banks from LiDAR - comments

As noted above, a previous investigation in the Kaihu valley has indicated that stopbank toe levels give a reasonably good indication of the natural unmodified river bank crest levels. However there are few reaches of formal constructed stopbank. In some places there are discrete mounds of river cleaning spoil with gaps between the heaps. In other places such heaps have been joined together to form reaches of continuous stopbank. Such reaches of informal stopbank do not have a consistent cross section. Considering these factors, a rather large offset of 20m was chosen, with the intention of placing the offset alignment clear, in most places, of any formal or informal bank works.

The floodplain contour maps show that the Kaihu River builds marked natural levees, with the river bank often being significantly higher than the average floodplain level. The crest of a natural levee is typically very close to the main channel edge. Thus in the process of mechanical cleaning of the river channel, the digger typically puts the spoil more or less on top of the natural levee (within one digger boom length of the river bank). The surveyed bank crest alignment normally passes along the top of the spoil heaps, so the 20m offset alignment is usually some distance on the land side of the natural levee crest. Natural levee side slopes are typically fairly gentle, but are steepest close to the river. Consideration of these points suggests that the ground levels along the offset bank alignment will tend to be a little lower than the true natural unmodified bank crest.

It would also be possible to define an offset bank alignment by inspection of the contour plans. However this would involve subjective judgement. For the present purpose it was considered better to use the consistent automated procedure described above.

3.5 Additional drain outfalls from new survey

A survey in December 2009, and January and February 2010, identified and measured sixty culverts, floodgates, drain outfalls and bridges, close to or on the Kaihu River banks. Photos were also provided for most of the outfalls. The positions of all the surveyed outfalls were plotted on the contour maps, and 22 were identified as being relevant and necessary for the Kaihu valley model. Table 3.2 gives details of those outfalls. Some important culverts and floodgates were not included in the recent survey but were already included in the Kaihu valley model.

Coordinates		Soffit level (m)	Surveyor's details	Notes on model installation	Location in Kaihu valley model	
NZTM E (m)	NZTM N(m)				Branch	Chainage (m)
1669301	6032644	5.339	gate 1000h x 2000w	Floodgate	Maitahi	3775
1669318	6032601	5.297	gate 1000h x 2000w	Floodgate	Maitahi	3775
1669852	6031059	5.457	cv2500	Floodgate	Cemetry	1220
1671014	6029784	3.317	cv1300	Floodgate	Bush	320
1671507	6029635	2.826	cv650	Floodgate	Bush	975
1671612	6029497	2.969	cv1300	Floodgate	Bush	975
1673008	6028900	2.007	gate 750	Floodgate	PoutoEast	625
1673290	6027845	2.073	gate 2000	Floodgate twin 1.1mx1.5m in model from previous survey	Pouto	1605
1673923	6026206	1.027	cv 525	Floodgate	Brown	640
1674731	6026195	1.591	gate 1500	Floodgate	Spillway	2405
1674547	6026143	2.026	gate 1500	Floodgate	Spillway	2405
1673992	6026063	1.07	gate 525	Floodgate	Brown	640
1674426	6026043	2.54	cv 1300	Floodgate	Brown	1185
1674052	6026002	2.142	cv 425	Floodgate	Brown	640
1675007	6025954	1.401	gate 625	Floodgate	Brown	640
1675311	6024998	1.431	Gate	Floodgate 900mm in model from previous survey	Antibrown	560
1675438	6024465	2.921	gate 2000	Floodgate	Parorerb	650
1675499	6024442	2.202	Gate	Floodgate 1mx1m in model from previous survey	Parore-lb	1180
1675804	6024061	1.734	Gate	Floodgate twin 900mm in model from previous survey	Valley	805
1675975	6023861	2.663	Gate	Floodgate 900mm	Parorerb	1380
1676583	6022919	2.119	gate unknown	Floodgate 900mm	Parorerb	2265
1676455	6021631	1.853	gate 2000	Floodgate 3mx1.5m in model from previous survey	Mangatara	5350

Table 3.2: New outfall survey information included in Kaihu valley model

3.6 Adjustment of Frith stopbank

In the previous version of the Kaihu valley model the level of the Frith stopbank (the ring bank on the property of Northern Dairylands Ltd) was estimated from inspection of the contour maps generated from LiDAR data (in the same way as river bank levels were estimated in that model version). The peer review of the Stage 1 and 2 report noted that the bank level appeared to be too low. This bank was included in the survey specification for the summer of 2009 / 2010, but the surveyor was not able to fit it into his programme. Therefore a more detailed analysis of the LiDAR data was made.

Grids of 1m and 2m spacing were made covering the area of the Frith stopbank, and were sliced along the alignment of the stopbank (determined from the contour map). The profiles generated showed several low points of about 5m elevation. Plotting all the LiDAR survey points on the contour map showed that there were several places where there were few points on or near the bank crest. In such places the gridding algorithm had been influenced by the lower ground levels on either side of the bank, generating the low points.

It is understood that the peer reviewer, following detailed analysis of flooding on the Northern Dairylands property, considers that the Frith stopbank overtops at water levels of about 6m (Wilson 2007). Accordingly a simplified bank crest was installed in the model with levels varying between 6m and 6.5m. This takes into account the observation of the peer reviewer and the profiles generated from the LiDAR survey data.

3.7 Adjustment of tide timing

The peer review report considered that it may be too favourable to assume that in the design events, the maximum tide level at Dargaville coincides with the maximum flood level at Kaihu Gorge. The records from 1981 to 2007 were analysed, and it was found that the maximum tide level followed the maximum flood level at Kaihu Gorge by 1.8 days on average. The data are probably not sufficient to indicate any particular trend. It was considered that making the highest tide level coincide with the highest flow at Dargaville would make the overall probability significantly smaller than the nominal design event probability. Accordingly the tide times were adjusted so that the maximum tide level followed the maximum Kaihu Gorge flow by the average of 1.8 days.

For the new design events of 6 months, 1 year and 18 months ARI, tide time series with maximum levels equal to the Mean High Water Springs level were used.

3.8 Calibration check

The June 2000 and June 2002 floods were simulated with the upgraded model, and the results compared with observed data. The results are described in Section 2.7. The calibration simulation is a check of both the hydrological and hydraulic models.

3.9 Estimate of model uncertainty

The uncertainty associated with the calculated water levels in the model varies depending on the location in the model, and the magnitude of event. Relative uncertainties, indicating the effect of some change, are usually smaller than absolute uncertainties. In simulation of design events, the definition of the event is itself uncertain, being based on statistical analysis of peak flows and volumes of historical floods. The estimates of uncertainty given below are not based on rigorous analysis, but are derived from observations of the behaviour of the models.

In simulations of historical events, the Kaihu Gorge discharge record gives a reasonably accurate indication of the discharge from about one third of the total catchment area. There is some uncertainty associated with the discharge record, as river gaugings are normally considered to have an uncertainty of plus or minus 8%. The rating curve from which flood discharges are based is normally extrapolated from lower flow gauging. Without making a detailed examination of the gauging records, it is estimated that the uncertainty of the Kaihu Gorge flow is about 20% near the peak of significant floods. At lower flows within the range of gaugings the uncertainty is probably less than 10%.

In the present model version the flood flows from the tributaries downstream of Kaihu Gorge, which represent two thirds of the total catchment area, are estimated by hydrological modelling. The uncertainty of these flows is estimated to be plus or minus 30% in a historical event. The rainfall runoff process is very variable depending on the antecedent moisture levels and other catchment related factors, and the spatial and temporal distribution and amount of rainfall is only approximately known. These are the main reasons for the uncertainty. There is also uncertainty due to the simplifications inherent in the model, and in the imperfect nature of the process of calibrating a hydrological model against historical events.

The definition of a design event at Kaihu Gorge of a particular ARI is uncertain as noted above. In addition the estimation of runoff of the tributaries is subject to the same plus or minus 30% uncertainty as in the case of historical floods.

With respect to the uncertainty of the hydraulic model, important factors are the accuracy of representation of the conveyances and volumes of the river and floodplain branches, and of the hydraulic characteristics of various important structures such as major floodgates. Based on previous analysis of a cross section based model compared with a digital elevation model derived from the LiDAR data, it is considered that the floodplain volumes in the model are accurate to within a few percent.

During this project it was found that with default settings for the cross section processed data, there were major mass conservation errors in some floodplain branches. A mass conservation error means that the flows onto the floodplains over the river banks or by backflow through drains or stream channels, were not consistent with the volumes of water on the floodplains. The problem was investigated by Danish Hydraulic Institute, and found to be related to the automatic selection of processed data levels. By experimentation with the Kaihu valley model, it was found that the error could be reduced to within reasonable limits by setting the software to produce 100 equidistant processed data levels at all the floodplain cross sections. At the time of writing this report Danish

Hydraulic Institute is still investigating to find the full reason for the mass conservation error. Based on detailed tests of three of the floodplain branches, the error is thought to be relatively minor in the present version of the model – at two of those branches the agreement between volumes calculated from flow cross sectional areas, and volumes calculated from discharges, is very small up to the time of maximum flooding, while at a third branch the error is varies with time, reaching about 10% at the maximum water levels. This discussion of uncertainty assumes that mass conservation errors are small, but when a full explanation of the error is available, this assumption should be reviewed.

The river channel is based on surveyed cross sections, some of which are now over 10 years old (most of the river was surveyed in 1999). The lower reach up to Parore Bridge was resurveyed recently, and is considered to be quite accurately represented in the model. Upstream of Waihue Road the cross sections are widely spaced. The uncertainty of the conveyance in this reach is considered significant, perhaps plus or minus 20%. This uncertainty has been noted elsewhere in this report as it affects the confidence level of our assessment that stopbanking on the true right bank is not required from about 1km upstream of Waihue Road.

Downstream of Waihue Road the uncertainty of the river channel conveyance is considered to be plus or minus 10% or better. Thus the estimates of stopbank raising required to contain the design flood are considered to be reasonably accurate.

Calibration simulations are useful as indicators of the uncertainty of the simulations. However it should be noted that a calibration simulation is simultaneously a check of both hydrological and hydraulic models. As such it is in principle an under determined problem, in which a range of different combinations of hydrological and hydraulic parameters may give similar results. However most of the uncertainty is associated with the hydrological model. The best calibration data available for the Kaihu valley model was obtained in the June 2002 event. This was the only event for which reasonably accurate peak flood levels covering the whole of the valley were available. June 2002 was a large flood, with general flooding of the floodplains. In such a case, the details of the hydraulic model linkages and main channel conveyances are probably not particularly important, and the most important element of the model is its representation of the floodplain topography, which determines the available floodplain storage capacity. As noted above, this is considered to be quite accurately represented in the model, so the June 2002 calibration is probably mainly a test of the hydrological model.

Considering the factors discussed above, the uncertainty of calculated water levels in the Kaihu valley model between Ahikiwi and Dargaville is estimated to vary between between plus or minus 0.2m and plus or minus 0.5m, depending on the size of the flood, and the location of the calculated water level.

4. Preliminary scheme layout

4.1 Method of developing the scheme layout

The concept of the flood management scheme is to provide protection for most of the floodplains against a "nuisance flood". This was taken to mean a flood in the range of 6 months to 1 year ARI, in combination with a maximum tide level of Mean High Water Springs.

To determine what stopbank levels would be necessary to provide this level of service, a new version of the Kaihu valley model was created in which flow from the Kaihu River onto the floodplains was prevented, by raising the existing banks clear of any conceivable flood, and by installing floodgates to prevent backflow onto the floodplains where there are presently no floodgates. Some iteration was required to define the layout of the scheme, because the water levels in the Kaihu River varied depending on the configuration of the flood protection works. It should be noted that the configuration chosen is preliminary only, and may be modified in response to more detailed consideration of economic and other influences.

On plotting the Kaihu River water levels for 6 month and 1 year ARI events, calculated by the new model version, on the longitudinal sections of the river banks supplied with this report, it was considered that it would be practical to provide protection against floods of 6 months ARI. The banks required for a 1 year ARI level of protection may be more expensive than is justifiable. This judgement is not based on an economic analysis and may be reviewed if the design proceeds beyond this preliminary stage. On the other hand the discussion of design floods in Section 7 of the Kaihu Valley Flood Management Scheme Concept (Barnett & MacMurray Ltd, June 2009) indicates that even protection against a flood of 6 months ARI would confer significant benefits. For further consideration in this report a 6 month ARI level of service is assumed.

4.2 Description of proposed flood scheme

The proposed layout is shown in the maps attached to this report, where the black lines represent stopbanks that will be managed as scheme banks, the red bank segments represent overflow spillways with defined crest levels, and major new floodgates are identified with labelled brown squares.

In general the scheme aims to modify the natural system and the status quo to a relatively small degree. The reasons for this approach have been discussed in previous reports, but are briefly restated here. Firstly, the area of land to be protected within the Kaihu valley probably does not justify major investment in flood control works. Secondly, the morphology of the floodplains and the high suspended sediment load carried by the Kaihu River indicate that a scheme based on a river channel and floodway contained by stopbanks protecting against major floods, is very probably physically unsustainable owing to sedimentation within the floodway, in addition to being economically unjustifiable as noted above.

Most of the proposed scheme banks are in effect accentuated natural levees. This requires minimum modification of the status quo (in which digger spoil has been placed more or less on top of the existing natural levees), and it mimics the natural system. Taking this approach limits the height of banks that can be built, and therefore the level of protection that can be provided.

The overflow crests are positioned to encourage flow of sediment from the river to the floodplains. The overflow crests are schematic only at this stage, and will require model simulations to set appropriate lengths and levels.

The managed banks would extend upstream of Waihue Road on the true right bank. The model indicates that upstream of that point the Kaihu River channel capacity is sufficient to carry the 1 year ARI flood without bank overtopping. It should be noted that there are only a few surveyed cross sections in this reach, and therefore there is some uncertainty in the model assessment of channel capacity. The areas of floodplain on the true left bank upstream of Waihue Road are relatively small and were assumed not to justify protection works.

On the true left floodplains at the Taita Stream confluence, the proposed bank layout uses the existing causeway across the Waiatua Stream, where a floodgate would be installed to protect the Waihue and Waiatua floodplains against flooding from the Kaihu River. This arrangement leaves an area of floodplain on the true left side at the Taita confluence unprotected. The Taita Stream valley is also open to flooding by backflow from the Kaihu River in the proposed scheme.

Upstream of the Rotu Bottleneck, it is proposed to allow flooding of the true right side of the valley, as suggested by previous flood management proposals. Also on the true right upstream of the Rotu Bottleneck, the valley at the dip in State Highway 12 is an important off channel ponding zone, and it proposed to allow free flow into it by only protecting the true right floodplain upstream of its State Highway 12 crossing. This requires construction of a bank across the floodplain, and installation of a floodgate in it.

On the true left side of the valley upstream of the Rotu Bottleneck, it is proposed to protect the floodplains upstream of the point where the river channel runs close to high ground on the true left side. This requires installation of a floodgate.

Downstream of the Rotu Bottleneck on the true right side, it is proposed to retain the existing stopbanks on the Pouto Farms property. This is one of the widest parts of the floodplain and therefore one of the most justifiable pieces of stopbank. The stopbank is already high enough to provide the proposed 6 month ARI level of service.

The relatively narrow strip of floodplain on the true left downstream of the Rotu Bottleneck also has flood protection in this proposal. This area has one floodgate at present and would need one more, near its southern (downstream) end.

The existing system of banks allows free back flow from the Kaihu River up the Rotu Stream, and it is proposed to retain this. It is proposed not to protect the narrow piece of floodplain on the true right from the Rotu Stream confluence to the Babylon Stream confluence.

It is proposed to use the existing old railway embankment to protect the Babylon valleys, by adding a floodgate at the stream crossing. The large area of floodplain on the true left opposite the Babylon Stream confluence already has floodgates at its downstream end, and would be protected to the 6 month ARI level of service by upgrading the river banks.

It is proposed to protect the floodplain on the true right upstream of Parore Road. The configuration of banks shown is intended to ensure the free access of Kaihu River overflow to the second Parore Bridge to maintain the present flood capacity. This arrangement requires a bank to be built across the floodplain, with a floodgate in it to allow drainage.

Immediately downstream of Parore Road, it is proposed not to protect the narrow piece of floodplain on the true right between the Kaihu River and the old railway embankment. The old rail embankment would be the first piece of a bank used to protect the true right floodplains from Parore to the Mangatara Stream confluence. The existing Mangatara floodgate would be retained.

The model simulations indicate that the natural unmodified banks would give protection against a 6 month ARI flood downstream of Parore Road, on both left and right banks. The model should be reasonably reliable in this area, because it includes a good number of recently surveyed Kaihu River cross sections. It is not proposed to reduce the level of service below that which would have been provided by the natural system. However, it is clear that these banks need to be managed as part of the scheme. It is proposed that overflow spillways should be constructed in these banks to maintain the level of service that would have been provided by the natural unmodified banks.

It is proposed that the Dargaville floodplain should retain its special protection status, and the stopbanks there would provide a higher level of service.

4.3 Management of flood control scheme stopbanks

In the Kaihu Valley Flood Management Scheme Concept (Barnett & MacMurray Ltd, June 2009), the stated intention was to provide spillways which would be designed to ensure equitable levels of flooding on the various protected areas in a range of flood magnitudes greater than the scheme design flood. The levels on the spillways would be carefully managed and monitored to maintain their design levels. It was intended that the remaining reaches of stopbank would not be subject to control. This could possibly be a self managing system, because in principle there would be no benefit in building up stopbanks above the spillway crest levels, as the spillway crests would govern overtopping in a flood event.

However it is now considered that it may be necessary to maintain the levels of the non-spillway stopbanks within some limits. The reasoning is that the spillways alone are expected to produce equitable levels of flooding only up to some relatively small flood (perhaps 2 year ARI, for example). In greater floods it is intended that the whole valley should be floodable as it has been to date. The surest way to achieve this is to prevent the non-spillway stopbanks from being built up too high. For the present preliminary design stage of the scheme it has been assumed that the non-spillway stopbanks should have crest levels 0.25m above the 6 month ARI flood level in the Kaihu River.

With regard to the gradual building up of the non spillway stopbanks, although there might be no benefit in doing so in terms of delaying the onset of flooding, the reality is that as river cleaning continues, spoil is dumped which has to be managed somehow. The natural tendency is probably to put it on top of the existing banks. Thus it is expected that without monitoring and management, the banks would tend to creep upwards.

4.4 New floodgates

The proposed scheme would require construction of 6 new floodgates, as listed in Table 4.1. Calculating the necessary sizes was not included in this part of the project, but two of the proposed floodgates would be major structures because of the flow capacity required. The benefit of these two structures in relation to their cost has not been analysed at this stage. For comparison the Mangatara catchment is floodgated and has a catchment area of 252km^2. It should be noted that the proposed scheme does not protect the floodplains from flooding due to runoff from tributary catchments. Thus for example in the present system, Mangatara catchment may experience flooding, even though it is floodgated to prevent flooding from Kaihu River backflow.

Approximate NZTM coordinates		Description	Approximate catchment area upstream (km^2)
Easting (m)	Northing (m)		
1669700	6032700	Waiatua Stream causeway	294
1670400	6030500	True right floodplain upstream of dip in State Highway 12	2
1671400	6029900	True left floodplain upstream of Rotu Bottleneck	4
1673500	6028100	True left floodplain downstream of Rotu Bottleneck	1.5
1673200	6026300	Old rail embankment at Babylon Stream crossing	279
1674800	6025500	True right floodplain just upstream of Parore Road	2

Table 4.1: New floodgates required for proposed scheme

5. Stopbanks and rough order costs

The longitudinal sections of the river banks supplied with this report show the Kaihu River water levels in the 6 month and 1 year ARI flood events, with most of the floodplains protected from flooding by the proposed scheme works as described above. The lengths of stopbank requiring raising were determined by inspection of the longitudinal sections and are given in Table 5.1 to Table 5.7.

The assumptions made to calculate the earthwork volumes for stopbank construction are as follows:

- The heights of existing banks are given by the difference between the surveyed bank crest levels and the estimated natural unmodified bank levels.
- Where stopbanks need to be raised, the new bank would be built on more or less on top of the existing bank.
- The existing banks have a 1m top width, and the new banks would have a 2m top width.
- The side slopes of both existing and raised banks are 1V:3H. Therefore raised banks will necessarily have a wider base than the existing banks.
- The banks will have a freeboard of 0.25m above calculated flood level.
- The volume can be calculated with adequate accuracy by assuming a uniform variation of bank height along each reach – this approximate calculation is not based on actual heights at each chainage.

Geotechnical considerations are beyond the scope of this preliminary design. It is assumed here that because the raised stopbanks are low enough to be considered accentuated natural levees, easily available material including river dredging spoil will be good enough for the bank raising.

In addition to the main earthworks identified in the following tables, additional earthworks will be required to level discrete heaps of river cleaning spoil to form stopbanks, and to cut spillways to the correct levels. These works are not expected to require large volumes of earthworks. They have not been included in the cost estimate.

Chainages for bank raising (m)	Height of bank raising (m)	Indicative existing bank height (m)
0 to 1400	0.1 to 0.9	0.5

Table 5.1: Bank raising required on true left bank from Waihue Road to high ground upstream of Taita Stream confluence (survey profile LB9)

Chainages for bank raising (m)	Height of bank raising (m)	Indicative existing bank height (m)
1400 to 1900	0.0 to 0.3	1.0
1900 to 2850	0.0 to 0.3	0.8
3050 to 4350	0.5 to 0.75	0.5

Table 5.2: Bank raising required on true left bank from high ground upstream of Taita Stream confluence to high ground upstream of Rotu Bottleneck (survey profile LB6)

Chainages for bank raising (m)	Height of bank raising (m)	Indicative existing bank height (m)
1350 to 2200	0.0 to 0.5	1.0
2300 to 2400	0.0 to 0.75	0.0

Table 5.3: Bank raising required on true left bank from Rotu Bottleneck to high ground upstream of Rotu Stream confluence (survey profile LB4)

Chainages for bank raising (m)	Height of bank raising (m)	Indicative existing bank height (m)
3850 to 4050	0.0 to 0.8	0.0

Table 5.4: Bank raising required on true left bank from high ground upstream of Rotu Stream confluence to high ground upstream of Parore Road (survey profile LB3)

Chainages for bank raising (m)	Height of bank raising (m)	Indicative existing bank height (m)
0 to 1000	0.1 to 0.9	0.7
1000 to 2000	0.4 to 0.9	0.7
2000 to 2900	0.5 to 0.9	0.7

Table 5.5: Bank raising required on true right bank from Waihue Road to Taita Stream confluence, and Taita Stream true left bank from confluence to State Highway 12 (survey profile RB5)

Chainages for bank raising (m)	Height of bank raising (m)	Indicative existing bank height (m)
2100 to 2700	0.0 to 0.5	1.3
2700 to 3500	0.3 to 0.7	0.4
3500 to 4200	0.3 to 0.5	0.8
4200 to SH12	0.0 to 1.3	0.0

Table 5.6: Taita Stream and Kaihu River true right bank from State Highway 12 at Maitahi to Rotu Bottleneck (survey profile RB4)

Chainages for bank raising (m)	Height of bank raising (m)	Indicative existing bank height (m)
7730 to high ground near Brown Rd	0.5 to 0.6	0.0

Table 5.7: True right bank from Rotu Bottleneck to Parore Road (survey profile RB2)

The volumes of earthworks required for the main bank works are given in Table 5.8. The total volume of the main earthworks amounts to 71560m^3. At an estimated rate of \$13/m^3, the indicative cost of the main earthworks would be \$930,000.

Description	Survey profile name	Volume of earthworks to raise stopbanks (m^3)
True left bank from Waihue Rd to high ground upstream of Taita confluence	LB9	8540
True left bank from high ground upstream of Taita confluence to high ground upstream of Rotu Bottleneck	LB6	16030
True left bank from Rotu Bottleneck to high ground upstream of Rotu Stream confluence	LB4	4390
True left bank from high ground upstream of Rotu Stream confluence to high ground upstream of Parore Rd	LB3	550
True right bank from Waihue Rd to Taita confluence, and Taita true left bank from confluence to SH12	RB5	24380

Description	Survey profile name	Volume of earthworks to raise stopbanks (m^3)
Taita and Kaihu true right bank from SH12 at Maitahi to Rotu Bottleneck	RB4	15910
True right bank from Rotu Bottleneck to Parore Rd	RB2	1760

Table 5.8: Summary of earthworks volumes for main bank works

Additional capital costs to be considered are for the required new floodgates, for the lesser earthworks on the banks such as levelling off spoil heaps to form continuous banks, and for detailed design of the earthworks. Maintenance costs would include regular monitoring of the condition and level of the stopbanks and spillways.

Based on asset valuations for large floodgates in the Waikato area, with sizes similar to the Mangatara floodgate, it is estimated that the two large new floodgates proposed in the preliminary design would cost approximately $200,000 each to build. Based on the same data source, it is expected that the four smaller floodgates could be built for $50,000 each.

An indicative rough order capital cost for the proposed works would therefore be approximately $2 million, including $1.4 million for earthworks (allowing $0.5 million for the lesser works where bank adjustments are minor) and $0.6 million for floodgates.

6. Benchmark simulations

6.1 Purpose

The purpose of the benchmark simulations was to establish the proportions of flooding between the different drainage areas under “natural” conditions, so that these proportions could be maintained in the design of any future flood management works. By natural conditions, we mean conditions before construction of stopbanks and floodgates. However the State Highway 12 embankments were considered a non-negotiable feature of the landscape, and were therefore taken as part of the natural conditions. The railway embankment is now disused, and could in principle be removed. From experience in other parts of the country, there may be significant difficulties with obtaining consent to remove the old rail embankment. Therefore two versions of the benchmark simulations were carried out, with and without the old rail embankment, so that the further scheme design can proceed regardless of any difficulty with adjusting the rail embankment crest.

As discussed earlier in this report, it is proposed to provide protection against the design flood event of 6 months ARI (the “design flood” means the flood event established by flood frequency analysis and hydrological modelling of the ungauged catchment area, as discussed earlier in this report). A design flood of 1 year ARI would therefore overtop the scheme stopbanks, and for the scheme to function equitably, the natural proportions of flooding should be maintained in such an event. It is proposed to achieve this by suitable design and management of overflow spillways from the Kaihu River to the various drainage areas.

It is expected that in a large event, of say 5 years ARI or larger, the scheme stopbanks would have little effect on the distribution of flooding, because in such an event stopbanks providing a 6 month ARI level of service would be overtopped in many places. Therefore no particular design should be required to achieve equitable flooding proportions in large events.

The benchmark simulations were carried out for a flood of 1 year ARI. The next stage of the overall feasibility investigation into the proposed Kaihu flood management scheme would be to design the overflow spillways to maintain equitable flooding proportions. This would be done at least for the design 1 year ARI flood event, and probably for one or two other design floods of less than 5 year ARI.

6.2 Benchmark model

Flooding of the drainage areas under natural conditions would have occurred by overflow of the river banks, backflow of Kaihu River floodwater up the tributary streams, and from tributary stream floodwater. The natural unmodified bank levels were estimated as described in Section 3.4, and were adjusted to compensate for water surface gradient using the procedure described in Section 3.2. The adjusted bank levels were installed in the model as link branches between the Kaihu River and the floodplain branches of the model. The locations of the link branches are the same as in other versions of the model, but the shapes and levels of the overflow crests are particular to this version of the model.

There is no accurate way to estimate the cross sections of the tributary streams in pre-settlement times, which are of interest because of their potential influence on flooding of the floodplains by backflow. Digging drains was probably one of the first modifications made by settlers. Drains would have followed natural drainage paths in some cases, but it is common for drains to be cut through areas of high ground, for example to drain lower ground behind natural levees. Therefore there are probably more connections between the river and the floodplains in the model than there would have been before human modification of the floodplains.

In the benchmark model, all existing drainage links between the Kaihu River and the floodplains were retained, and all the floodgates on those drainage links were removed to allow free backflow. After the first benchmark simulations, some of the drainage links appeared to be more restrictive than would have been the case under natural conditions, by inspection of the animated water surface profiles. For example in the Pouto West, Antibrown, Parore LB, and Valley drainage areas (for locations see the sketch plans attached to this report), relatively small culverts restricted the exchange of flow between the river and floodplains, affecting both flooding and drainage. These culverts were increased in size so that there was no significant water level difference across the culverts during the simulation. Thus although the culverts remained in the model, they were adjusted so that they represent reasonably realistically the flow through a natural stream. Culverts in the railway embankment were left unchanged, as they represent part of the existing state of the rail embankment, but any floodgates on such culverts were removed.

To simulate the flooding from tributaries or local rainfall in each drainage area with improved accuracy, the arrangement of boundary conditions was slightly modified from previous model versions. The new arrangement was based on areas of floodplain pockets determined by planimeter from the topomap. For example, in previous versions of the model, Pouto floodplain branch had tributary inflow of 0.3 of Parore subcatchment hydrograph, which was based on inspection of the topomap and the subcatchment boundaries. Following measurement of the areas, that was adjusted to 0.21 of Parore subcatchment hydrograph for the benchmark model. The full list of inflow boundary conditions is given in Appendix D, and details of the differences from the previous model versions may be seen by comparing that table with Appendix F of the report on Stage 1 and 2 of the Kaihu flood management investigation.

Section 4.1.5 of the report on Stages 1 and 2 of the Kaihu investigation describes the baseflows that were applied to the floodplain branches to help ensure a smooth transition from low flow conditions to flooding. After calculating the flooding volumes, as described below, it appeared that in some branches the baseflows were large enough to affect the storage available in the drainage system. In a relatively small flood such as the 1 year ARI design flood, the storage available within the drains is significant in some branches. Therefore the baseflows were reduced in the following branches (which are named on the scheme sketch plans in Appendix E): Mangatara, Parore RB, Parore LB, Valley, Antibrown, Spillway, Pouto West, Waihue.

The Frith stopbank (the ring bank which encloses a section of floodplain near Frith Road) was included in the benchmark model, because it is understood to have been constructed with the consent of the relevant authorities at the time, and because it is an internal embankment within a floodplain area, and therefore does not prevent overflow of the Kaihu River banks. Functionally it is analogous to the railway embankment in the Pouto Farms property.

6.3 Benchmark model with rail embankment removed

The sections of the old railway embankment that could affect flooding of the areas proposed to be protected by the flood management scheme, and the changes made to the model to represent their removal, are given in Table 6.1 (for locations of the drainage areas refer to Appendix E).

Flow between	Natural ground level from inspection of Lidar contour maps (m)	Treatment in model
Pouto and Pouto West	2.25 to 2.5	Lowered weir to ground level
Korariwhero branch (Babylon valleys) and the main valley	3 to 3.5	No change – the rail embankment waterway provides free exchange of flow
upstream end of Parore RB and the main valley	2.5 to 3.0	Lowered weir to ground level

Flow between	Natural ground level from inspection of Lidar contour maps (m)	Treatment in model
downstream end of Parore LB and the main valley	2.1 to 2.25	Lowered weir to ground level
downstream end of Valley and the main valley	2.1 to 2.9	Lowered weir to ground level

Table 6.1: Changes to the model to represent removal of the old rail embankment

6.4 Benchmark flooding volumes and durations

The variation during the 1 year ARI design flood event of the water volume in each drainage area proposed to be protected by the scheme was calculated by summing the inflows from the tributaries, the flows over the river banks, and the backflows up the drains, and subtracting from that sum all the outflows. In some branches nearly all floodwater comes from the Kaihu River, as for example in Pouto branch. No tributrary streams enter that part of the floodplain, and the local rainfall makes only a very small contribution to the flooding. In other branches most of the flooding is due to the tributary streams, although high water levels in the Kaihu River play a role by reducing the outflow. This occurs for example in Mangatara branch, where the tributary flood volume is much greater than the maximum volume of water stored in the branch.

Flooding durations were calculated from the time series of water levels at the reference cross sections. The reference cross sections are mostly the same as those used for Stage 3a of the Kaihu flood management investigation (see Appendix B and Appendix C of that report), but there are some additions and deletions to suit the present purpose. The reference cross sections and flooding durations for the benchmark simulations are given in Table 6.2. Maropiu, Mamaranui, and Settlement drainage areas are not proposed to be protected by the flood management scheme, but have been included in the flooding volume and duration calculations so that any effect that the proposed scheme may have on them can be detected.

Model branch / drainage area	Model chainage (m)	Nominal flooding level (m)	Flooding duration (h)
Maropiu	1870	18.0	8
Settlement	1805	12.3	0
Settlement	2320	11.1	3
Mamaranui	1030	11.0	10
Mamaranui	1860	10.3	13
Maitahi	1925	7.4	9
Maitahi	3875	5.3	31
Waihue	1725	6.3	21
Waiatua	1665	5.7	31
Cemetry	940	4.1	35
Frith	350	5.0	20
Frith	2530	3.7	33
Bush	975	3.7	29
Bush	1555	3.6	25

Model branch / drainage area	Model chainage (m)	Nominal flooding level (m)	Flooding duration (h)
Pouto	1250	2.0	63
Pouto West	560	2.2	69
Pouto East	1380	2.5	52
Spillway	1180	2.3	43
Brown	1425	2.9	11
Korariwhero	2415	2.1	121
Antibrown	425	1.9	40
Parore LB	590	2.0	36
Valley	450	2.2	9
Parore RB	365	2.0	27
Parore RB	2080	2.0	12
Mangatara	2305	2.1	29

Table 6.2: Flooding durations at reference locations in 1 year ARI design flood under benchmark conditions with existing rail embankment

Flooding volumes for the drainage areas proposed to be protected by the scheme are plotted in Figure 29, where the branches are listed in the legend in order of decreasing maximum flooding volume. Results for the drainage areas Bush and Mangatara are also plotted. Bush branch includes the bush reserve immediately upstream of the Rotu Bottleneck, which has been regarded as a ponding zone in previous flood management proposals, and so is not protected against flooding in the proposed scheme. The Mangatara valley would not be further protected by the proposed flood management scheme, but these benchmark results allow the benefits of the existing floodgate to be compared with those provided to other drainage areas by the proposed flood management works.

Referring to Figure 29, the benchmark simulation shows that under "natural" conditions, Maitahi drainage area would reach maximum flooding earliest, with a relatively high flooding volume compared with other areas, but would drain quickly. Pouto branch has the highest flooding volume and a relatively long drainage time. Korariwhero branch (the Babylon valley) has a high flooding volume and the longest drainage time. The flooding volume in Parore RB branch shows some influence of the tide, even near the peak of the flood.

It should be noted that the flooding volumes plotted are absolute volumes, not volumes per hectare. It is to be expected that larger pockets will have larger volumes, other factors being equal.

The progression of flooding down the valley can be seen in Figure 29, but it does not necessarily follow that drainage areas nearer the river mouth reach the peak of the flood later. The reason is that in some cases the local tributary flood is the main influence, for example in Mangatara valley, where the maximum flood volume is reached at about the same time as in Cemetry drainage area. However Korariwhero drainage area reaches its maximum flood volume more than one day later than Maitahi drainage area. Korariwhero and Mangatara both have large tributary flood volumes, but a difference of approximately one day in the flood peak time. This difference is probably mainly a result of the greater backwater effect from the Kaihu River on Korariwhero. The Mangatara stream joins the

Kaihu River near the mouth, where the rise of the flood is smaller than further up the valley.

The volumes plotted in Figure 29 include the volumes stored in the drains and tributary streams, which can be substantial in comparison with the total flooding volume in this relatively small flood event (the 1 year ARI design flood). Some branches have significant lengths of large old Kaihu River channel, for example Parore RB, Antibrown, Parore LB, and Spillway. Some branches have long stream channels, such as Mangatara and Korariwhero. Old river channels are well resolved by the LiDAR survey and subsequent 2m gridding process. The smaller drains are generally not well resolved by LiDAR survey, because of the reflection from the water surface and the non-vertical viewing angle over most of the survey area (both of which tend to result in the drain invert level in the LiDAR survey being higher than the actual drain invert). Smaller drains were often adjusted in the model development process, to improve the computational stability. The adjustment was based on general observation of drains in the field, but not on actual data for each drain.

As noted elsewhere in this report (Section 3.9), initially the simulations were undertaken with the default settings for the cross section processed data tables, which specify automatic selection of 20 processed data levels. In evaluating the flooding volume results, some floodplain volumes seemed unrealistic, and more detailed checks were made. It was found that there were major mass conservation errors in some floodplain branches, meaning that the inflow into a floodplain area was not consistent with the volume calculated from water level and ground levels. Investigation by Danish Hydraulic Institute (the developers of Mike 11) discovered that the error was due to linear interpolation of poorly resolved processed data tables. In particular the automatic selection of processed data levels gave poor results, even when the maximum number of levels was set to 200, because the low flow channels (drains) were poorly resolved. Further experiments showed that the error could be reduced to acceptable levels in the test branches by specifying 100 equidistant processed data levels. The simulations were repeated with all the floodplain branches changed accordingly. The mass conservation has not been checked for every branch, and Danish Hydraulic Institute continues to investigate the problem. When a full explanation is available, the results of this study should be reviewed, as it is possible that some branches that have not been checked in detail may be susceptible to error.

As a result of storage in low areas and drains, only a part of the flooding volume plotted in Figure 29 causes flooding at the reference cross sections. In Figure 30 to Figure 49, the flooding volume for each drainage area is plotted together with the water level at the reference cross section. It should be noted that the reference cross sections are typical but not necessarily the lowest or most floodable, so the absence of flooding at a reference cross section does not necessarily imply no flooding at all.

Figure 35 shows the results for Waiatua branch, which was one of those checked for mass conservation errors. Volume calculated from discharge relies on the principle that volume in the branch must equal the inflow minus the outflow. The dashed curve shows the volume calculated from the cross sectional area at each time step saved in the additional result file, using the average end area method. It may be seen that the two calculations agree very well until after the peak of the flood, when the calculation from discharge gradually diverges. For the purpose of this investigation, the agreement is good enough.

The other branches checked following the change to 100 equidistant processed data levels were Mangatara, which gave a very similar general pattern of agreement followed by gradual divergence, and Valley, where there was good agreement except for an error of roughly 10% at the peak of the flood and at the peak of each tidal cycle. In the initial investigation of the problem those same branches plus Parore RB were tested and all were found to have major mass conservation errors.

In the second version of the benchmark model, the rail embankment was removed, in addition to the removal of all constructed stopbanks. The drainage areas Pouto, Pouto West, Korariwhero, Parore LB, Valley, and Parore RB would be directly affected by removal of the rail embankment. For those branches the results with and without the embankment are plotted. Figure 39 shows that Pouto drainage area would be only slightly affected by removal of the rail embankment. The effect in that case would be to allow Pouto floodwaters to flow unimpeded into Pouto West drainage area. Figure 40 shows that Pouto West drainage area would suffer an increase of maximum flooding volume of approximately 20% if the rail embankment were removed in addition to the removal of the constructed stopbanks. However the duration of flooding would be practically unaffected.

Figure 44 shows that the benchmark flooding volume in the Babylon valleys (Korariwhero branch in the model) would be practically unaffected by removal of the railway embankment. The reason for this is that the existing rail embankment waterway already allows free exchange of flow between the main valley and the Babylon valleys.

Figure 46 shows that the peak benchmark flooding volume in Parore LB drainage area would be increased by approximately 30% by removal of the rail embankment. The corresponding increase in maximum water level is approximately 0.15m. However the duration of flooding would be practically unchanged.

Figure 47 and Figure 48 show that the benchmark flooding in Parore RB and Valley drainage areas would be practically unaffected by removal of the rail embankment.

The maximum flooding volumes and the percentage shares of the total flooding volume for the benchmark system including the rail embankment are shown in Table 6.3. The benchmark flooding proportions are calculated using the sum of the maximum values as denominator. It would also be possible to calculate a time series of flooding volume, and use the maximum value of that time series to calculate the proportions. At this stage such a level of detail is probably not justified, because it remains to be seen how well the overflow crests can be made to preserve the benchmark flooding volume proportions.

Similarly, with regard to whether or not the rail embankment should be included in the benchmark model, the simulations to date have shown that the effect of the rail embankment is significant at only two branches. It may be possible to reproduce the flooding proportions of either version of the benchmark model, but this remains to be confirmed by the overflow crest design process. Accordingly there is probably no urgent need to decide whether or not to include the rail embankment in the benchmark system at this stage.

Drainage area	Maximum flooding volume (m^3)	Percentage of total
Pouto	1245200	16.0
Korariwhero	1143600	14.7
Maitahi	959800	12.3
Cemetry	838200	10.8
Frith	740700	9.5
Spillway	631900	8.1
Waihue	436000	5.6
Bush	362400	4.7
Parore RB	264000	3.4
Mangatara	227500	2.9
Waiatua	218600	2.8
Pouto West	205600	2.6
Pouto East	183600	2.4
Brown	87700	1.1
Parore LB	84200	1.1
Maropiu	72400	0.9
Mamaranui	41400	0.5
Antibrown	26700	0.3
Valley	11600	0.1
Settlement	8900	0.1
Totals	7790000	100.0

Table 6.3: Benchmark flooding volumes and percentages with existing rail embankment

7. Conclusions

1. The Kaihu valley hydrology has been refined by creating a hydrologic model for 2 upper catchments and 21 lower catchments. The HEC-HMS hydrologic model applies the SCS curve number method for loss and the kinematic wave method for routing of overland and channel flow.
2. The upper catchment hydrologic model was calibrated to three observed events at the Kaihu Gorge. A reasonable calibration was achieved for two of the events, with peak flows within +/- 23% and runoff volume within +/- 27% of observed. The last event was a poorer match, with both peak flow and volume about 46% less than observed.
3. Calibration parameters were transferred to the valley catchments hydrology model and historic floods simulated using the updated hydrologic and hydraulic Kaihu models.
4. The June 2002 historic flood simulation demonstrated improved peak water level and flood duration estimates over the original model.
5. The June 2000 historic flood simulation gave a flood duration at cross section 20, 1.5km upstream of the Rotu Bottleneck, significantly less than the original model, and 51 hours less than observed. Possible reasons for this are:
    a) Assuming that the storm occurs at the same time across the valley may have affected the timing of peak flows such that flood durations were reduced.

b) Applying the Dargaville rainfall temporal pattern may have missed more intense rainfall further up the valley which caused higher runoff and longer flooding.
c) Saturated antecedent conditions in June 2000 may have generated more runoff than the more general model can predict.

6. Based on these historic flood results the refined hydrologic model was used to generate the design hydrographs for the 0.5 year to the 100 year ARI events.
7. The original scaled hydrographs and the refined hydrology total runoff volumes for the whole Kaihu valley agree within +/- 10%. However, the volumes, peak runoff and hydrograph forms for individual catchments are quite different with the refined hydrology as each reflects more closely the local catchment topography and vegetation.
8. The Kaihu valley hydraulic model has been updated to include new bank survey information and new drain outfall data, and the calibration of the new model version has been checked against historical flood data.
9. The new hydraulic model version has been used to estimate the height of stopbanks required to protect most of the floodplains against floods of 6 months ARI.
10. The rough order capital cost of works for the proposed flood management scheme has been estimated to be $2 million.
11. A "benchmark" hydraulic model of the Kaihu valley has been created which represents the system before the erection of any stopbanks. The benchmark includes the embankments of State Highway 12, because the highway is considered a fixed and necessary topographical feature. However versions with and without the old rail embankment were created.
12. The benchmark model was used to determine the maximum flooding volumes in 20 drainage areas in the 1 year ARI design flood. These volumes and volume proportions are to be used as the basis for design of overflow crests from the Kaihu River to the various drainage areas, in such a way as to preserve the benchmark flooding volume proportions in a range of floods greater than the scheme design flood.

8. Recommendations

It is recommended that:

1. The updated hydrologic model for the Kaihu valley be adopted and used in subsequent investigations.
2. A flood level monitoring system be installed in the valley to collect more detailed flood level data in flood events.
3. When more detailed flood level and discharge data is collected for a significant flood event, it be used to refine the calibration of the hydrologic and hydraulic models of the Kaihu catchment.
4. The project should proceed to design of the overflow crests, which would complete the feasibility investigation for the flood management scheme.

9. References

Auckland Regional Council, *Guidelines for stormwater runoff in the Auckland region: TP108*, April 1999.

Barnett and MacMurray Ltd, *Kaihu Flood Control Scheme Investigation, Report on Stages 1 and 2*, produced for Northland Regional Council, November 2008

Cathcart, B. Northland Regional Council, *Soil types of the Kaihu River catchment*, Dec 2009. Notes prepared for this report.

Wilson G. 2007: Pers. Comm.