Hydraulic Modelling of the Rangitaiki Plains Tarawera River to Rangitaiki River

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Contents

Chapter 1: Introduction

One of the recent Drainage Flow Modelling project objectives of Environment Bay of was to ensure that *"Maps for the 1% and 2% AEP flood events are prepared for the Rangitaiki Plains"*.

This report summarises the hydraulic modelling and preparation of flood maps for that portion of the Plains between the Tarawera and Rangitaiki River. The report is an update of that prepared for Environment Bay of Plenty in August 2005.

Chapter 2: Previous Modelling

A MIKE 11 model was built of the canal and drainage network in 1997 (Environment B·O·P, 2002). This model covered the area between the Tarawera and Rangitaiki Rivers as well as the Awakaponga Canal to the west of the Tarawera River. The floodplain was not modelled, and flows up to the 10% AEP were simulated.

Since the mid-1990s a MIKE 11 model of the lower Rangitaiki River (from Te Teko to the sea) and the Rangitaiki Floodway/Reids Central Canal has also been developed and refined.

Chapter 3: Scenarios Modelled

The following 1% AEP scenarios have been modelled:

(i) A breach at Laws Bend, Rangitaiki River (Left Bank), with 1% AEP Tide and 5% AEP Rangitaiki River Flow.

(ii) A breach at Laws Bend, Rangitaiki River (Left Bank), with 5% AEP Tide and 1% AEP Rangitaiki River Flow

(iii) A breach downstream of Thornton Road, Rangitaiki River (Left Bank), with 1% AEP Tide and 5% AEP Rangitaiki River Flow

(iv) A breach at downstream of Thornton Road, Tarawera River (Right Bank), with 1% AEP Tide and 5% AEP Tarawera River Flow

(v) 1% AEP Rainfall on the Rangitaiki Plains and 5% AEP Tide

Tides include an allowance for sea level rise due to the Greenhouse Effect.

The breaches modelled are scenarios only. The breach sites do not necessarily represent the most likely positions, nor has any attempt to estimate the probability of breaching been made. In the scenarios, a breach of 100m length, down to general ground level, was assumed open for the duration of the simulation. The simulations continued after the peak flow or tide for four or 4.5 tide cycles, in recognition of the likelihood that it will be difficult to access and repair the breaches.

In addition, several 2% AEP scenarios were modelled, and flood maps prepared, in 2005. The scenarios have not been rerun in 2006. Modifications to the model have been made since 2005, so those results and the maps resulting should be treated as preliminary only.

Chapter 4: Model Software

In this exercise, MIKE FLOOD, a software program developed by DHI, has been used as the principal modelling tool. MIKEFLOOD incorporates MIKE 21 (i.e. 2-D flow equations) and MIKE 11 (1-D flow equations), allowing them to be dynamically linked during a simulation. This program is better suited to floodplain modelling than MIKE 11 alone, particularly for a floodplain as flat as the Rangitaiki Plains. It still however allows the use of MIKE 11 in well-defined flow channels such as the canals.

Chapter 5: Model Layout

5.1 MIKE 11 Component

The 1997 MIKE 11 model of the drains and canals was used as a starting point for this current model. Some modifications were made, as follows:

- The Awakaponga Canal and Wilsons Drain, to the west of the Tarawera River, were removed from the model.

- The Tarawera River branch has been truncated a little upstream of the Awaiti Canal confluence.

- The model was extended to include Murrays Drain (actually named Omeheu Canal in the model files, as it is an extension of that). No cross-sections were available for this drain, but invert levels (and in some cases, top of bank levels) surveyed in 1987 and 1993 were used together with an assumed section shape.

- The maximum dx (i.e. the distance between computational H-points) was decreased to 400m in most branches. This forces the model to interpolate additional cross-sections where the distance between sections would otherwise be greater than 500m. It then allows a better linkage of MIKE 11 H-points to MIKE 21 grid points. Ideally a smaller maximum dx would have been used (at the expense of computational speed), but the software used (in initial runs at least) was limited to 250 H-points. Because of the flat grades of the waterways, that the maximum dx is greater than ideal is not expected to affect results significantly.

- Corrections made to cross-section chainages.

- Pumps can now be modelled directly within MIKE 11 (rather than having to use defined Q-t boundary conditions as was the case in the original modelling), and these have been modelled at the major pump locations. Pump curves were not available at the time of modelling, and the pumps were set to run at maximum flow once trigger levels were reached. (In an actual flood event, as happened on the Plains to the east of the Rangitaiki River in July 2005, extra pumps would probably be brought in, and in severe cases the stopbanks deliberately breached to drain floodwaters. Thus it is not too important that the pump curves were not available).

- The previous model had inflows into the canals entering via assumed 1m diameter flapgated culverts, to ensure that the canals wouldn't be artificially receiving water when the canal levels were already high. In the current model, these have either been revised by entering known the diameter and/or invert levels of actual culverts, or removed if they did not actually exist. (With the model connection to the floodplain, water could flow into floodplain if the receiving waters were too high, and so such artificial flapgated culverts are not needed).

- Mannings n values have been increased by 0.005 above the values used in the previous modelling, to reflect expected greater turbulence etc at high flows. (A sensitivity test was performed in the previous modelling where Mannings n had been increased by 0.005. This was the setup used in this current modelling).

- The lower reaches of the Rangitaiki River were inserted into the model (taken from the MIKE 11 model of that river).

The MIKE 11 component of the model is illustrated in Figure 1.

5.2 MIKE 21 Component

Topographical data were available from a post-earthquake photogrammetry survey of the Rangitaiki Plains in 1987. Spot height data were available from a 100m grid of points and along defined features such as stopbanks or stream banks.

A MIKE 21 topographical model (dfs2 file) was built from these data, using a 50m grid layout. Several issues need to be noted however.

- The floodplain is likely to be consolidating (i.e. lowering) in some locations (Environment B·O·P, 1998). Thus it is possible that the floodplain may be lower than that modelled. Stopbanks may also have been lowered in places by, for example, consolidation, stock damage or access track crossings.

- A check of stopbank levels appears to support this (Figures 2 and 3).

- Other human-made changes in the topography may have occurred in localised areas during the 16 years since the date of the topography data.

- It is not known how well ground-truthed the photogrammetry was when it was undertaken (or how possible it was to do so, given the ongoing differential settlement of the plains, particularly immediately after the 1987 earthquake). Refer also to Section 2.1 of Environment B·O·P (2002).

- A 50m grid cell size is not fine enough to pick up features such as roads or localised high or low ground. However, as the raw data are generally only provided at 100m intervals, decreasing the grid cell size will not necessarily improve model accuracy.

Floodplain resistance has been assumed to be 0.100 in maize crops and kiwifruit orchards, 0.150 in the wetland between Grieg Road and the Awaiti Canal, and 0.045 elsewhere. The resistance has been applied as a dfs2 file.

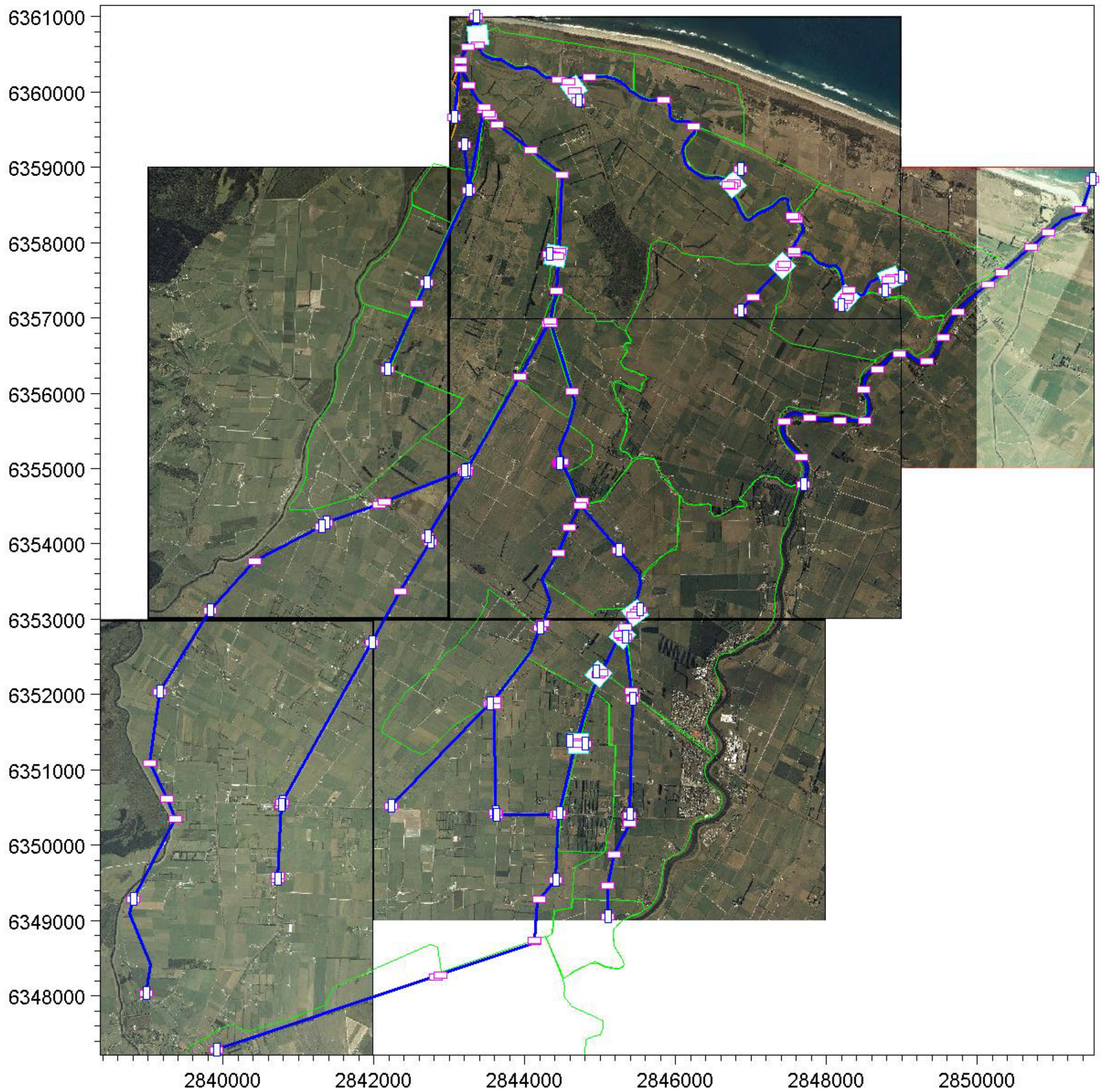

Figure 1 MIKE 11 Model Layout

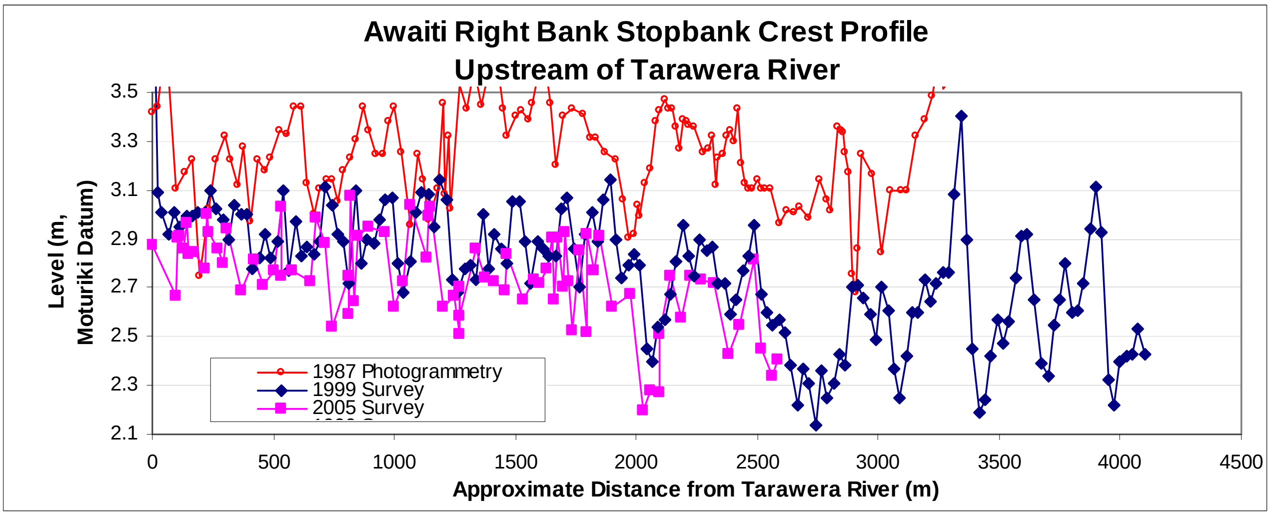

Figure 2 Stopbank Crest Profile, Awaiti Canal

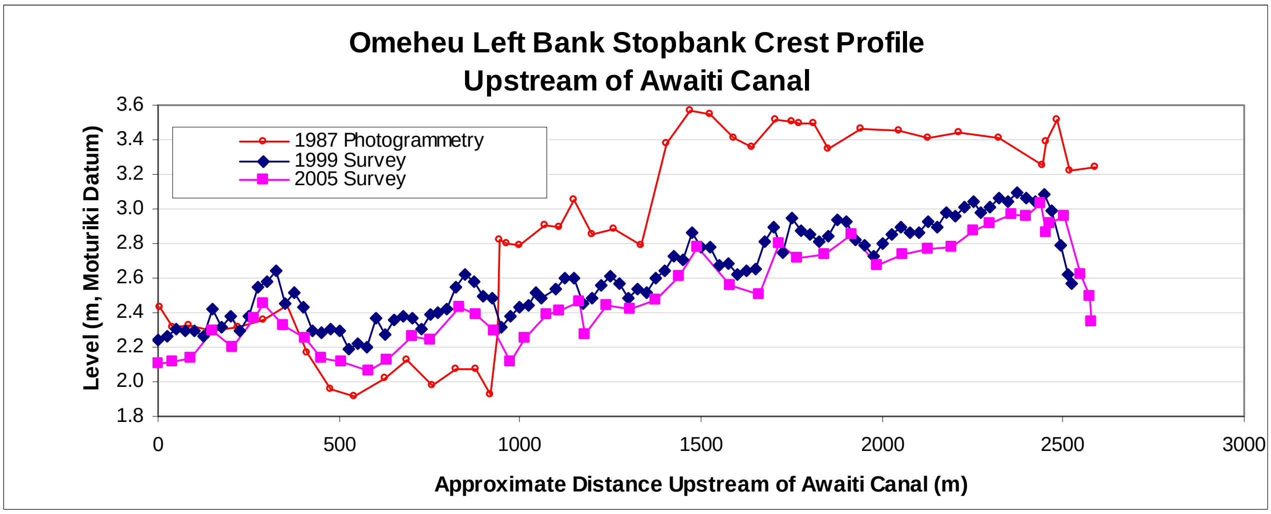

Figure 3 Stopbank Crest Profile, Omeheu Canal

(Note: The distances of different profiles within each plot are not aligned exactly. Also, the photogrammetry profile points may not be on the crest in some locations).

5.3 MIKE11 – MIKE 21 Links

MIKE FLOOD dynamically links MIKE 11 and MIKE 21 models and requires links to be entered by the modeller. In this model, the links represent lengths of stopbank or, where there are no stopbanks, channel edges. Several lengths of canal stopbank were surveyed between 1999 and 2005, and the most recent profiles were used to define the links over those reaches (Table 1). Elsewhere, the link profile was defined by the higher of the relevant MIKE 11 cross-section edge of channel marker (via interpolation of the upstream and downstream sections) and the MIKE 21 topography cell to which the channel is linked.

Table 1 Surveyed Stopbank Crest Profiles

Canal	Bank	Reach	Canal distance [1]	Survey Dates
Awaiti	Left	Tarawera Western Drain to Omeheu Canal		1999
Awaiti	Left	Omeheu Canal to Awaiti Floodgates		1999
Awaiti	Right	Omeheu Canal to Awaiti Floodgates		1999
Awaiti	Right	Awaiti Floodgates to Tarawera River		1999, 2005
Omeheu Adjunct	Left	SH2 to Omeheu Canal		1999, 2001
Omeheu Canal	Left	SH2 to Omeheu Adjunct		1999
Omeheu Canal	Left	Omeheu Adjunct to Gow Road		1999
Omeheu Canal	Left	Gow Road to Awaiti Canal		1999, 2005
Omeheu Canal	Right	SH2 to Omeheu Adjunct		1999, 2001
Omeheu Canal	Right	Omeheu Adjunct to Gow Road		1999
Omeheu Canal	Right	Gow Road to Awaiti Canal		1999
Partial surveys of isolated low sections of banks				
109 Canal	Left		2.175-2.35	2005
109 Canal	Right		0.4-0.9	2005
Awaiti Canal	Left		5.7-5.97	2005
Awaiti Canal	Right		4.69-4.89	2005
Awaiti Canal	Right		5.675-5.9	2005
Old Rangitaiki Channel	Left		3.46-8.54	2005
Old Rangitaiki Channel	Right	Upstream of Thornton Road	2.00-8.54	2005
Omeheu Adjunct	Left		0.1-0.4	2005
Omeheu Adjunct	Left		2.9-3.1	2005
Omeheu Adjunct	Right		0.1-0.4	2005
Omeheu Adjunct	Right		2.9-3.1	2005
Omeheu Canal	Left		5.66-6.17	2005
Omeheu Canal	Left		6.57-6.7	2005
Omeheu Canal	Left	Upstream of Otakiri Road	7.18-7.5	2005
Omeheu Canal	Right	Upstream of Awaiti Canal	0-1.21	2005
Omeheu Canal	Right	Upstream of Edgecumbe Soldiers Road	4.225-4.49	2005
Omeheu Canal	Right	Upstream of Otakiri Road	7.185-7.425	2005
Omeheu Canal	Right		7.95-8.25	2005
Omeheu Canal	Right		8.69-8.95	2005
Omeheu Drain	Left		2.975-3.16	2005
Omeheu Drain	Right		2.98-3.1	2005

Note (1) km upstream of downstream end, refer Environment Bay of Plenty Plans M1261

5.4 Model Boundary Conditions

5.4.1 Tarawera River

(a) Scenarios 1, 2, 4, 5

The inflow hydrograph used in the previous modelling for the Tarawera River at SH30 has been used in this exercise. It is based on the shape of the December 1995 flood and scaled to provide a peak flow of 10% AEP at SH30. In this current exercise it is applied at the upstream end of the Tarawera River in the model, i.e. just upstream of the Awaiti Canal, where it equates to a 20% AEP flow.

(b) Scenario 2

The hydrograph used in the other scenarios has been scaled up to provide a 5% AEP peak flow at the model inflow point.

5.4.2 Rangitaiki River

The inflow hydrographs used have been taken from the earlier Rangitaiki River modelling. In scenario 2, the 1% AEP hydrograph has been used. The 5% AEP hydrograph has been used in the other scenarios.

5.4.3 Sea

Design sea levels are as recommended by the Environment Bay of Plenty Hydrological and Hydraulic Guidelines (Environment B·O·P, 2001). In scenarios 1 to 4, a tidal cycle has been used. The design peak sea levels are assumed to be reached at two high tides, as shown in Figure 4.

In scenario 5, the sea level is assumed to be constant over the duration of the simulation at a level of 2.3m (5% AEP).

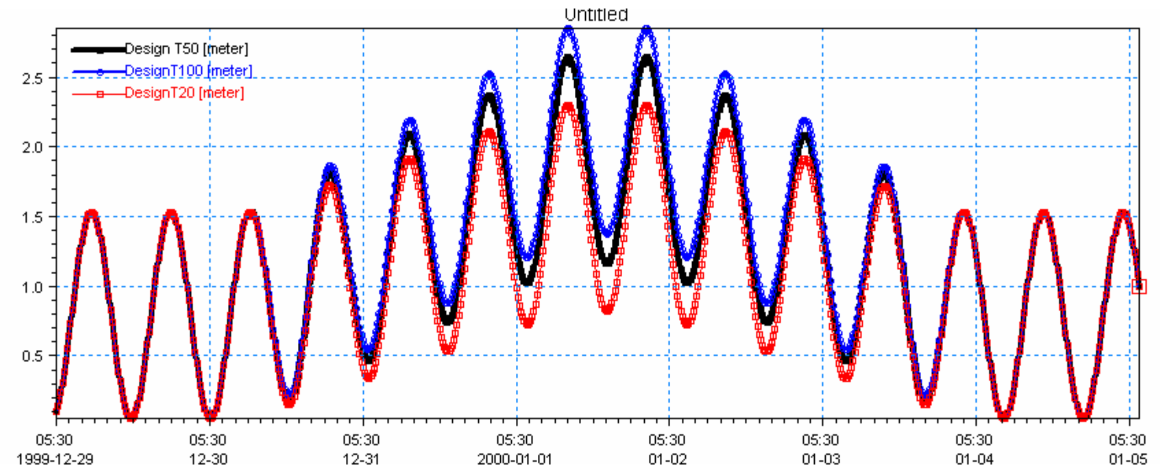

Figure 4 Design Tidal Cycles

5.4.4 Rangitaiki Plains (Scenarios 1 to 4)

For scenarios 1 to 4, nominal constant low flows have been applied to the canals and drains.

5.4.5 Rangitaiki Plains (Scenario 5)

The study area has been divided up into subcatchments, as in the 1997 study. Some minor modifications to the boundaries and area estimates for those subcatchments have been made here. In the initial modelling carried out in this project, two approaches to dealing with the flow contribution from each subcatchment were used.

The first was a combination of the “Direct Inflows” and “Rainfall” techniques described below. It was assumed that the runoff from all gravity-drained subcatchments reaches the main drains and canals, other than from the large “catchment 71” that drains into Murrays Drain. The same was also assumed for the Omeheu Adjunct Pump Scheme and the Poplar Lane subcatchments. Thus inflow hydrographs were applied as point inflows at appropriate locations on the canals and drains, using MIKE 11 boundary condition files (see “Direct Inflows” below). In these cases, water finds its way from the canals onto the floodplain via the MIKE 11 - MIKE 21 links in the model. For all other pump scheme subcatchments and catchment 71, rainfall excess was applied directly to the subcatchment area (using dfs2 files) and routed across the floodplain (see “Rainfall” below). Such floodplain flow may also find its way into the canals via the MIKE 11 – MIKE 21 links.

Figure 5 shows the subcatchments for which rainfall excess has been applied directly onto the floodplain.

The second approach applied rainfall excess over the entire model area and, other than for the Rangitaiki and Tarawera River branches, has no defined MIKE 11 boundary inflows (Figure 6).

Both approaches have their merits and their limitations. The first allows for the effect of unmodelled smaller drains carrying water from the floodplain into the canals. The second recognises that rainfall has to fall on the floodplain before it can reach the main canals and drains. Maximum flood depths and extents for each approach were mapped in 2005, for both the 1% AEP and 2% AEP flood scenarios. In almost all areas the second approach gave the higher level, the exception being a small area within the Poplar Lane Pump Scheme. In the final modelling, a conservative stance has been taken and only the second approach was used.

(a) Direct Inflows

For those subcatchments where inflows were applied as MIKE 11 boundary conditions (in the initial modelling), a unit hydrograph approach was used to convert the design rainfall excess hyetograph (see below) into a hydrograph. Each unit of rainfall excess is assumed to produce a triangular unit hydrograph with a 1:2 ratio of time to peak to time of recession.

The hydrographs so produced were converted to a base hydrograph shape per hectare and scaled up by the subcatchment area for those subcatchments concerned. As a check on the results, a comparison was made with an earlier 1% AEP estimate by Peter Blackwood for the 563 ha Grieves Road drain culvert. That estimate, adopted after considering results from a regional flood frequency analysis and from the Rational Method, was 7 m^3/s. Using the approach outlined above, a 1% estimate of 7.55 m^3/s was arrived at – an acceptably close figure.

As noted above, however, in the final model runs this technique was not used.

(b) Rainfall

A nested storm approach has been assumed. For each of the 2% AEP and 1% AEP scenarios, rainfall depths for given durations have been obtained from HIRDS. (Values near each of the four corners of the study area have been averaged, to provide an estimate of a representative rainfall depth for each duration). Taking the 2% AEP case as an example, the design storm has been built up as follows:

- 30 minute duration, depth = 34mm. Assume this falls at a constant rate over a 30 minute period

- 60 minute duration, depth = 49mm, so assume that 15mm (i.e. 49mm - 34mm) falls in the 30 minutes following the most intense 30 minute period.

- 2 hour duration, depth = 67mm. Assume that 9mm falls in each of the 30 minute periods either side of the most intense 60 minute period. (As 67mm - 49mm = 18mm, 18mm/2 = 9mm)

- 6 hour duration, depth = 108mm. Assume that 20.5mm falls in each of the 2 hour periods either side of the most intense 2 hour period. (As 108mm – 67mm = 41mm, 41mm/2 = 20.5mm).

Etc, up to a 24 hour period.

On the advice of Peter Blackwood, an initial loss of 20mm and a continuing loss of 1.2mm/hour have been assumed. These have been subtracted from the design storm rainfalls to produce a design storm rainfall excess hyetograph, with values expressed in mm/hour. This has been stored in a dfs2 file, with 30 minute timesteps for the catchments for which the rainfall excess is applied directly. Figures 4 and 5 show examples.

Full workings for both the “Direct Inflows” and “Rainfall” techniques can be found in the worksheets *24hr RF Q50* and *24hr RF Q100* of the spreadsheet *working.xls.*

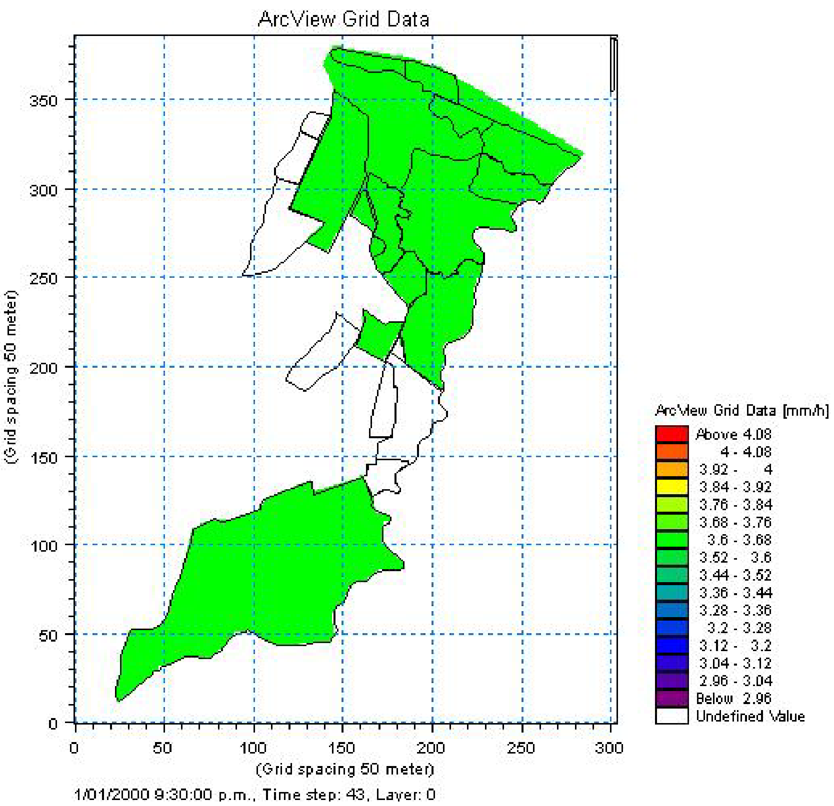

Figure 5 Subcatchments on which rainfall excess is applied directly, under the first modelling approach (sample timestep)

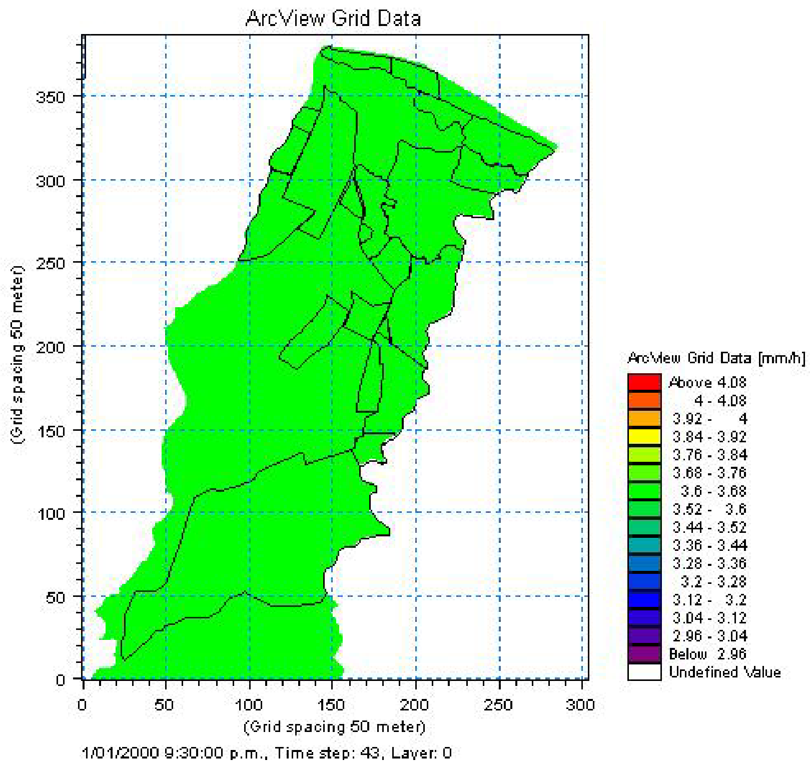

Figure 6 Rainfall excess applied directly to all subcatchments, under the second modelling approach (sample timestep)

(Note: Only some subcatchments are shown (black outline); not all have been digitised)

Chapter 6: Calibration

Only limited calibration information was available to calibrate the original MIKE 11 model, as described in Environment B·O·P (2002).

No flows or water levels were recorded in the May 2005 event. Any attempt to calibrate is unlikely to be successful, because of the great variation in rainfall depths and intensities recorded across the Plains. Furthermore, the Te Teko automatic raingauge failed during the event and there are some discrepancies between manual daily rain gauge totals (Figure 7). Nonetheless, records were taken of the canal overflow locations and some photographs of the flood event are available, and a calibration attempt could be made at a later date.

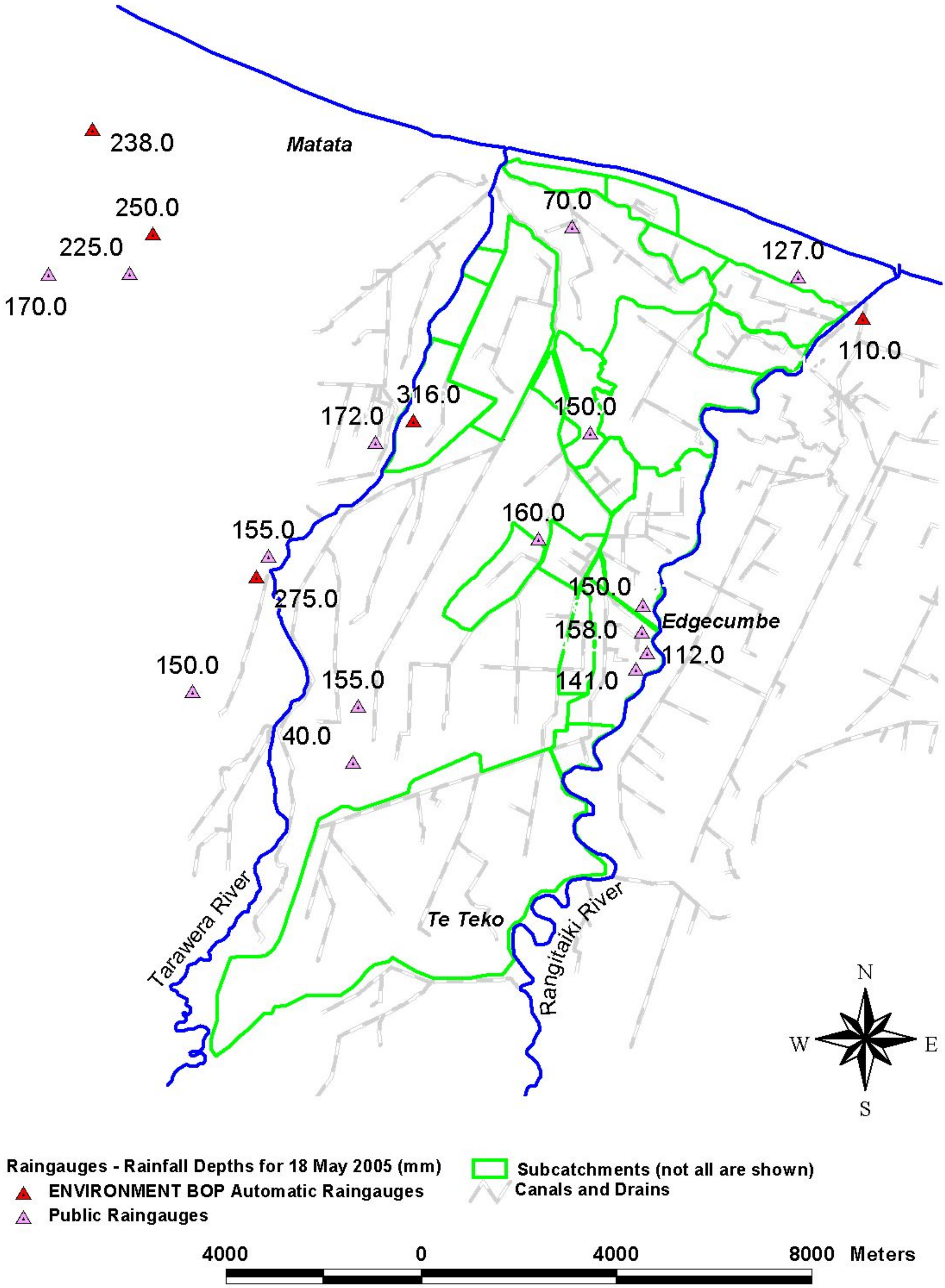

Figure 6 Rainfall Gauges and 24 Hour Rainfall Depths for 18 May 2005

Chapter 7: Floodmaps

Flood maps for each of the five scenarios, showing the maximum inundation depth, are presented in Figures 8 to 12. These maps do not include any allowance for freeboard.

A combined map, showing the envelope of maximum flood levels for each of five flood scenarios, i.e. the maximum depth at each model cell from all the scenarios, has also been produced (Figure 13). In this case the depths include a freeboard allowance of 500mm, applied as described below. This map has also been prepared at A1 scale, and is stored as Environment Bay of Plenty Map G127. An A1 map of flood elevation contours, relative to Moturiki Datum has likewise been prepared, and is also stored under this plan number. Again the contours include a freeboard allowance.

Map G127 also includes an insert with results from a breach scenario at Kokohinau Bend on the Rangitaiki River. That was the subject of a separate modelling exercise (Environment Bay of Plenty, 2006). It has not been shown in Figure 13.

In all maps, flooding to depths less than 50mm is not shown.

7.1 Freeboard

Before the raw results from the model can be used for design advice, a freeboard allowance should be added to levels to account for uncertainties and waves. In a confined river channel this is relatively straightforward and a constant amount is usually added to levels. Adding a constant amount to levels on floodplains however, particularly to such a flat area as the Plains, could mean that the mapped flood extent would carry on for a long way laterally.

One option, used in preliminary maps of 2005, design levels at any point could be set to the maximum of (flood level + freeboard) and (ground level + freeboard). This could have some value if there was a general requirement (regardless of flood issues) to have floor levels elevated e.g. to avoid damp problems.

Another is to run simulations with either river inflows or rainfall increased by for instance 10-20% and plot results. That would mean that there is no fixed freeboard amount, although it does avoid problems of flooding extents going on for a long way.

A further technique is to increase river resistance until river levels are raised by the freeboard amount. However in that case, water may simply spill water out further upstream. The technique is also not so appropriate when considering the internal flooding (generated by rainfall directly on catchment).

Lowering the stopbank levels by the freeboard amount (as was done for recent Whakatane modelling) works when considering at flooding from river or tide but again not from internal flooding.

The approach eventually used in this current study was a multi-step process, that uses a nominal 500mm freeboard but hydraulically tapers this down to zero at the flood extent edges, as follows:

1 For each scenario, the maximum of the flood depths produced by the model at each cell was established. Fifty millimetres was defined here as the “threshold” flood depth of flooding, and thus any of these maximum depths that were less than 50mm were effectively reset to a negative value, by redefining the maximum flood level (relative to Moturiki Datum) to be some arbitrary negative value (e.g. -100m). (This large negative value meant that there would be no confusion in Step 3 with ground levels that are below sea level – the case in some parts of the plains.)

2 A single maximum flood level at each cell, taken from all the scenarios, was identified. Then a base freeboard of 500mm was added to the flood level at all cells where the maximum depth was greater than 50mm.

3 The higher of the maximum flood levels from Step 2 and the ground level was identified at each cell.

4 A simulation was then started with the result of Step 3 as an initial condition, and run for a tidal cycle without any additional flow or rainfall entering. Thus the base freeboard amount was able to distribute itself and remove any discontinuities at the edges of the raised flood surface.

5 The final step was to extract the maximum flood level at each cell from this simulation, to provide the final design levels. The base freeboard of 500mm thus becomes the maximum effective freeboard applied. Five hundred millimetres is considered appropriate given the model uncertainties (particularly regarding the floodplain topographical data as discussed earlier).

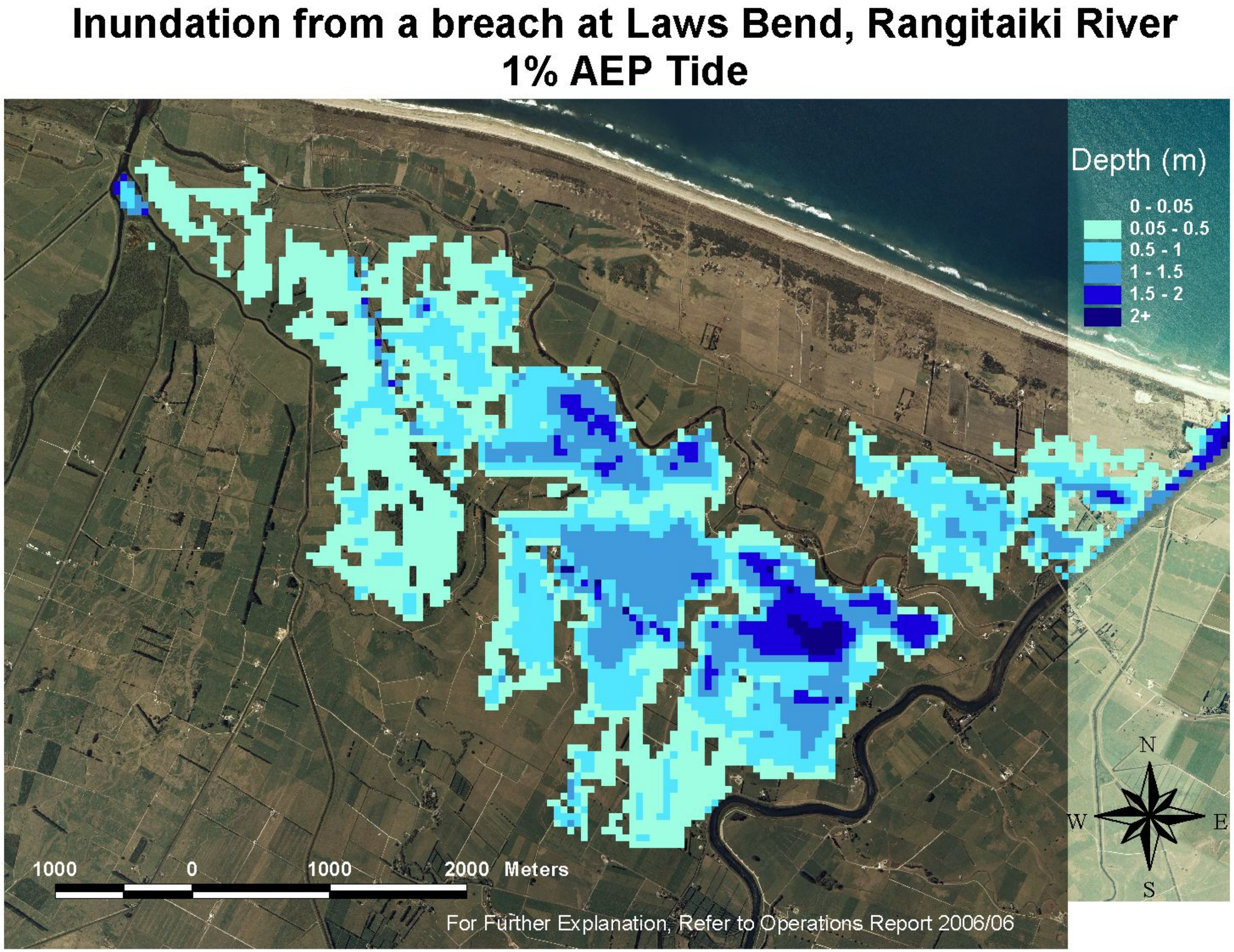

Figure 8 Maximum Flood Depths and Extent, Laws Bend Breach with 1% AEP tide (Scenario 1) No Freeboard

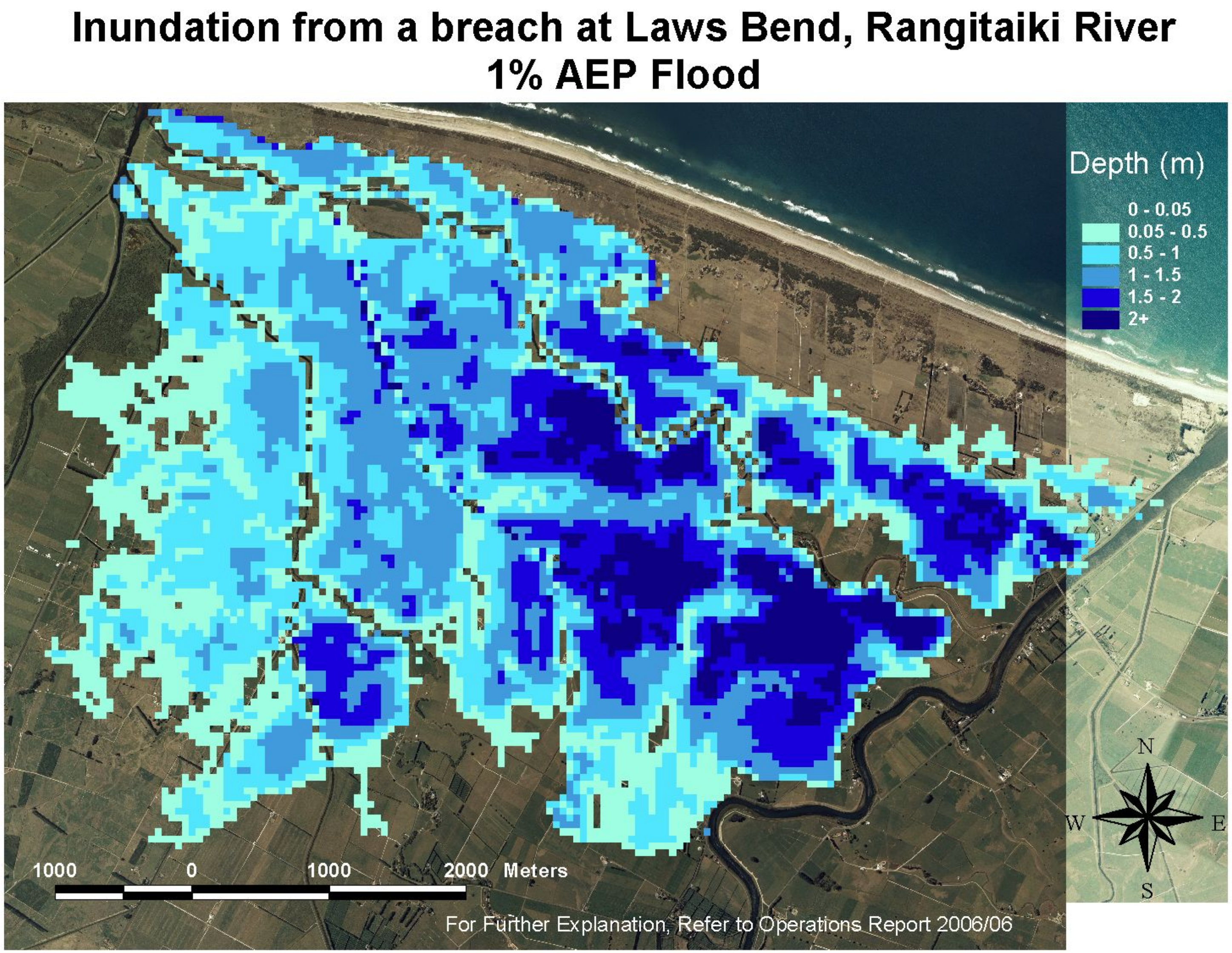

Figure 9 Maximum Flood Depths and Extent, Laws Bend Breach with 1% AEP River Flow (Scenario 2) No Freeboard

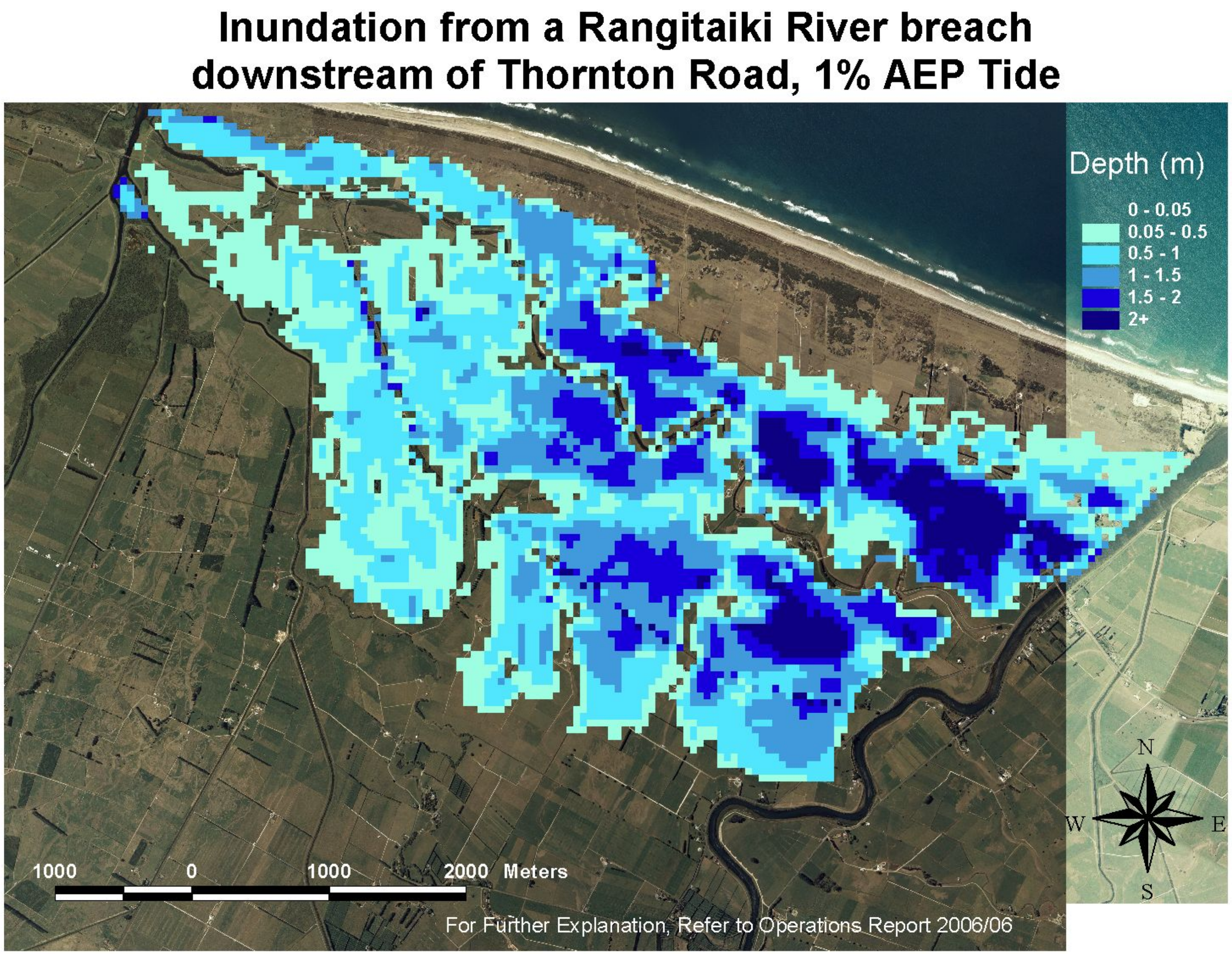

Figure 10 Maximum Flood Depths and Extent, Rangitaiki Breach Below Thornton Road with 1% AEP Tide (Scenario 3) No Freeboard

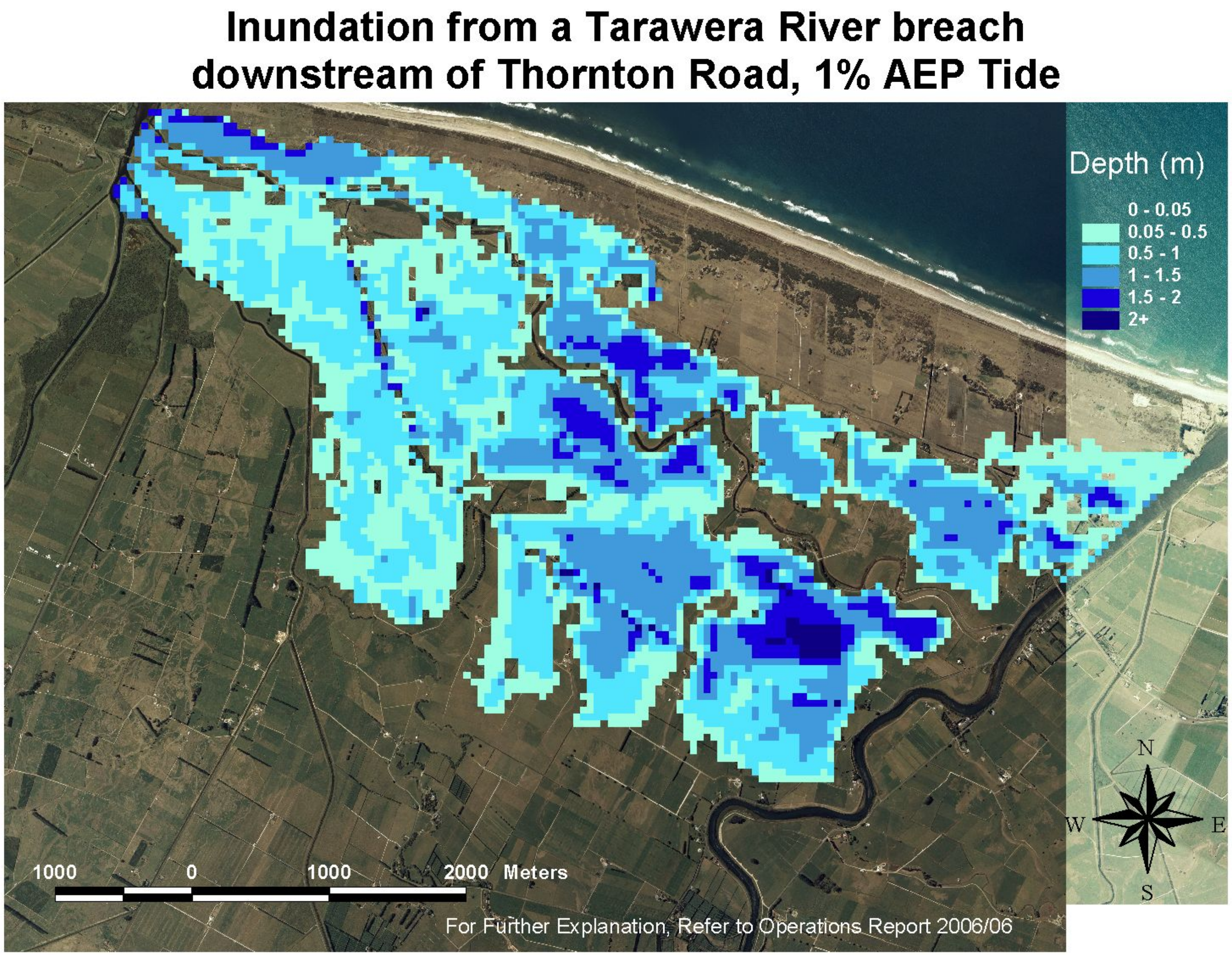

Figure 11 Maximum Flood Depths and Extent, Tarawera Breach Below Thornton Road with 1% AEP Tide (Scenario 4), No Freeboard

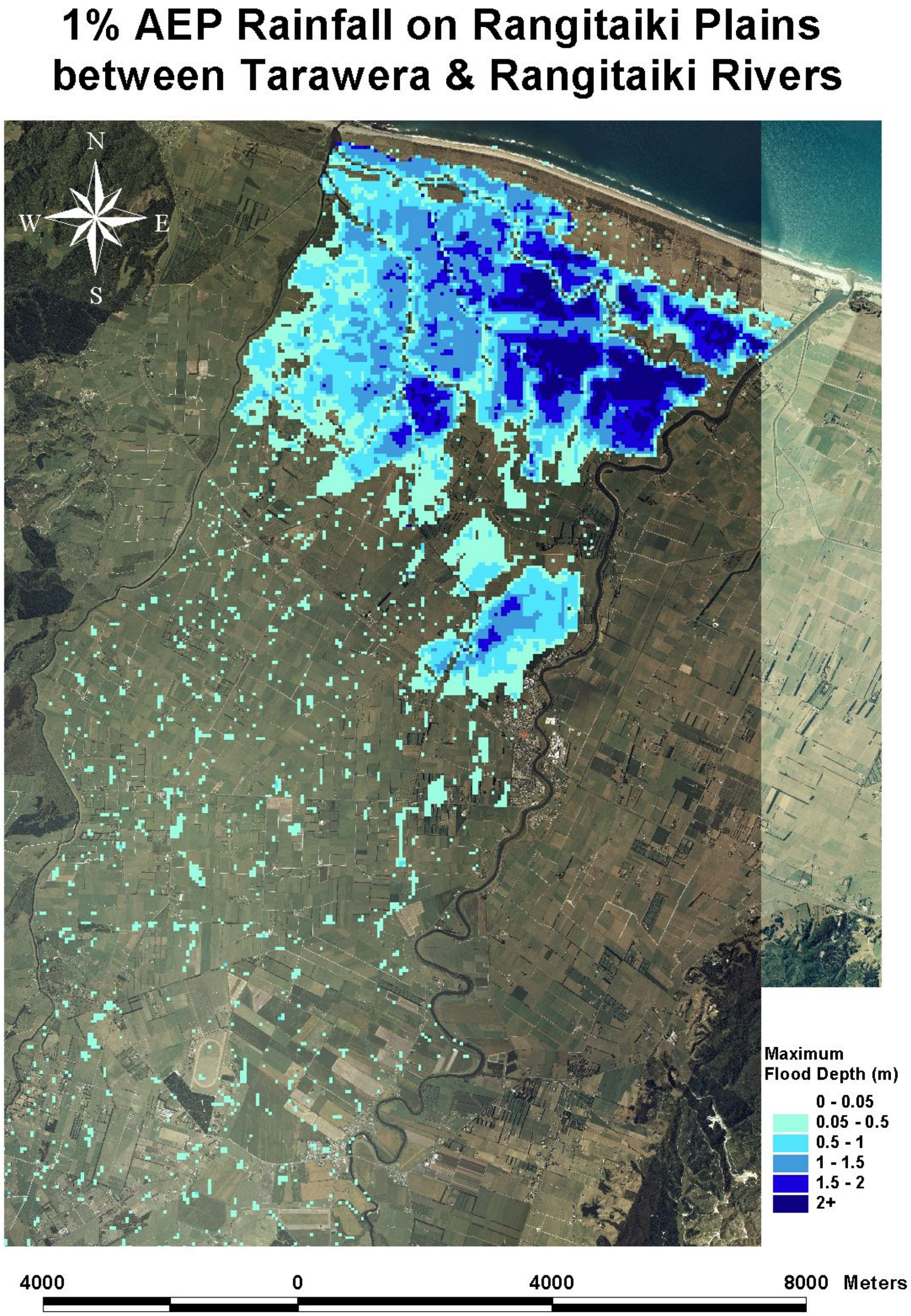

Figure 12 *Maximum Flood Depths and Extent, 1% AEP Rainfall on Plains (Scenario 5), No Freeboard*

1% AEP Floodmap
Maximum of All Scenarios
(Freeboard Applied)

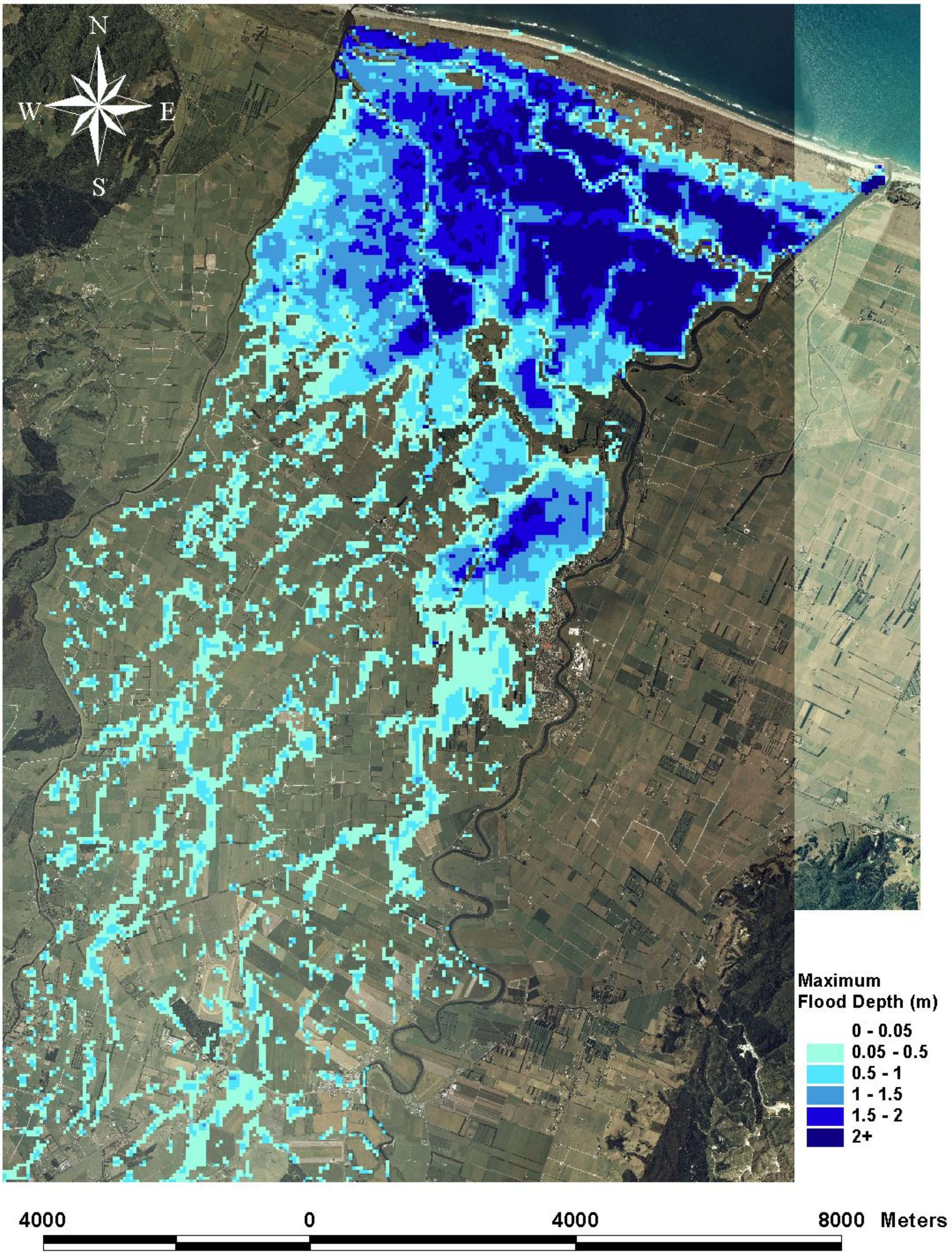

Figure 13 1% AEP Flood Depths and Extent, Maximum of all Scenarios, Freeboard Applied

Chapter 8: Conclusions and Recommendations

Flood depth maps for 1% AEP flood scenarios have been produced. Depths less than 50mm are not shown. In addition, electronic GIS files of maximum flood depths and of flood levels relative to Moturiki Datum have been produced.

Greater certainty in flood advice applying at a small individual property level may require that the ground elevation be checked by site survey however, as it is clear from evidence along the stopbanks at least that the photogrammetric levels should not be relied upon.

Furthermore, Figures 2 and 3 suggest that there is significant variation in stopbank crest height along canal stopbanks, and that the stopbanks may also be lowering over time. The previous MIKE 11 modelling took crest profiles only at cross-section sites, often a few hundred metres apart, and as surveyed in 1997 (i.e. nine years ago). The conclusions of that study, that the canals generally had capacity to carry 20% AEP flows under spring tide conditions and 10% AEP floods, may therefore need reviewing.

Several steps can be taken to refine the hydraulic model and the floodmaps, although these could be phased over time. They include

- Obtaining more flood event data - installing water level recorders, undertaking flow gauging (particularly during high flow events), recording flood debris mark levels.

- Refining procedures for operating manual raingauges to improve the reliability of data.

- Recording water levels upstream and downstream of pumps (the Old Rangitaiki Channel Pump Station would be an obvious site, as data on the pump operation are already collected).

- Surveying additional cross-sections from the canal network, and confirming the dimensions (diameter, invert levels) of relevant culverts.

- Undertaking a LIDAR survey of the floodplain topography. (This is planned for 2007).

- Reducing the MIKE 21 grid cell size and the MIKE 11 H-point spacing, once LIIDAR data and additional surveyed cross-section data are available.

Chapter 9: References

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

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Environment B·O·P (2002); Awaiti Omeheu Canal Modelling Study. Prepared by Philip Wallace. Operations Report 2002/03, May 2002.

Environment Bay of Plenty (2006); Rangitaiki River - Kokohinau Breach Scenario. Memorandum from Ingrid Pak to Peter Blackwood, 26 June 2006. File 5680 05

Wallace, Philip (2004); Hydraulic Modelling of Lower Whakatane River And Floodplain. Report prepared for Environment Bay of Plenty, July 2004.

Appendices

Appendix I – Files Used

Appendix I – Files Used

MIKE FLOOD Files

Used for all simulations:

mf9.nwk11
mf-revised.xns11
plus005.HD11
rangitaikires.dfs2 (resistance map)

Otherwise files are as listed below.

Scenario	1	2	3	4	5	With freeboard (Refer Section7.1)
.couple	Q20T100-LawsBendBreach	Q100T20-LawsBendBreach	Q20T100-RangitaikiBreach	Q20T100-TaraweraBreach	Q100-2-v2	100-freeboard
.sim11	Q20T100-Lawsbreach	Q20T100-Lawsbreach	Q20T100-Rbreach	Q20T100-Tbreach	Q100-2	100-freeboard
.bnd11	Q20T100-lawsbreach	Q100T20-lawsbreach	Q20T100	Q20T100	mf5-T20-2-v2	Q20T100
MIKE 11 Hot-start file	MF-HS.RES11	MF-HS.RES11	MF-HS.RES11	MF-HS.RES11	MF-HS.RES11	Q20T100-RangiBreach
.RES11	Q20T100-LawsBreach	Q100T20-LawsBreach	Q20T100-RangiBreach	Q20T100-Tarawera_Breach	Q100-Q10-T20-2	100-freeboard
.m21	Q20T100-LawBendBreach	Q100T20-LawBendBreach	Q20T100-Rbreach	Q20T100-Tbreach	rangitaikiQ100-2	100-freeboard
Topography .dfs2 input file[1]	Topo-LawBreach	Topo-LawBreach	TaraBreach-Topo	TaraBreach-Topo	rangi5ModelTopo	TaraBreach-Topo
Initial condition .dfs2 input file	Topo-LawBreach	Topo-LawBreach	TaraBreach-Topo	TaraBreach-Topo	rangi5ModelTopo	maxfbic
Rainfall .dfs2 input file					rainfall-Q100-2	
.dfs2 (full results)	Q20T100-LawBendBreach	Q100T20-LawBendBreach	Q20T100-Rbreach	Q20T100-Tbreach	Q100-2	100-freeboard
.dfs2 (maximum results)	Q20T100-LawBendBreach -maxHy	Q100T20-LawBendBreach-maxHy	Q20T100-Rbreach-maxHy	Q20T100-Tbreach-maxHy	Q100-2-maxHy	100-freeboard-maxHy-v2

Note 1: Differences in the topography files used are minor, and are only due to the expansion of the model over the course of the investigations as new scenarios added.

Text Files (used in all simulations to define stopbank crest profiles)

109-RB2.txt
109-LB2.txt
AwaitiLB-FGtofork.txt
AwaitiLB-TWDtoOmeheu.txt
AwaitiRB_OmeheutoFG.txt
AwaitiRBFGtoTarawera.txt
AwaitiRBFGtoThortonRd.txt
AwaitiRB-TWDtoOmeheu.txt

OmeheuCanal-LB-OAtoGow.txt
OmeheuCanal-LB-SH2toOA.txt
OmeheuCanal-RB-GowtoAC.txt
OmeheuCanalRB-OAto GowRd.txt
OmeheuCanal-RB-SH2toOA.txt
OmeheuLB-ACtoGow.txt
OmeheuCanal-RB-SH2toOA.txt
OmeheuAdjun-LB-SH2toOC.txt
ORC-LB-17230-19950.txt
ORC-RB-17230-19950.txt
taraweraBreach.txt for that breach scenario, otherwise *taraweraRB.txt*
rangiLB-Breach-Thornton_to_BM1.txt for that breach scenario, otherwise *rangiLB-Thornton_to_BM1.txt*
LawsBendBreach.txt (for that breach scenario only)

Spreadsheets

working.xls	Working calculations and assumptions
longsect.xls	Stopbank long-sections

Arcview Grid File Folders

100fbh	Flood levels for 1% AEP (including freeboard)
100fby	Flood depths for 1% AEP (including freeboard)