Environment B O P
Operations Report 2000/09
October 2000

Review of the Flood Carrying Capacity of the Rangitaiki River Below Edgecumbe

Report prepared by P L Blackwood, Manager Technical Services

Environment B O P
Quay Street
P O Box 364
Whakatane
NEW ZEALAND

ISSN 1173 - 6232

Acknowledgements

To Horace Freestone (OPUS International Consultants Limited) for a wealth of background hydrological knowledge and data. Mark Stringfellow and Graeme O'Rourke for provision of data. Philip Wallace for particular advice on the Mike 11 computer software. The peer reviewers Al McKerchar, NIWA and Graeme Campbell and Philip Hoby (A C Consulting Group Limited) for constructive advice. Kim Mills for the rapid word processing.

Cover Photo: Lower Rangitaiki River

Contents

Chapter 1: Introduction

The purpose of this report is to advise on the results of a careful analysis of the hydrology and hydraulics of the Rangitaiki River below Edgecumbe. The review is being carried out with a twofold purpose:

(a) It is appropriate to review river scheme designs at regular intervals. These reviews incorporate the increased database available (which may well have been relatively small in previous design). Frequently, such reviews are commissioned following major floods, which may well contain important data. The last comprehensive review of the Rangitaiki River was completed in 1988.

(b) At the peak flow in the Rangitaiki River during the July 1998 floods, landowners in the vicinity of the lower Rangitaiki River below Edgecumbe, particularly those just above Thornton, were somewhat concerned to note that the river level was in some cases, a matter of less than ½ metre below the top of the stopbank in some places. That concern was heightened when the frequency of the flood which had occurred in the river was subsequently noted to be equivalent to the 15 year return period flood. (The Rangitaiki/Tarawera Scheme had provided that the stopbanks in this area were capable of carrying a 100 year flood in that river, in conjunction with the Rangitaiki River floodway making use of the Reids Central Canal). This occurrence and causative mechanisms require investigation.

The scope of this report includes:

- Summary of scheme background
- Review of hydrology (that is design flood flows)
- Review of hydraulics (design flood levels)
- Outline of possible remedial options.

Chapter 2: Background

Detailed information on the background and features of the Rangitaiki Tarawera Rivers is contained in the Rangitaiki-Tarawera Rivers Major Scheme documents (Bay of Plenty Catchment Condition, 1969) and post 1987 earthquake Rangitaiki River Scheme report (Dine et al, 1988). This and other information is further summarised in the Rangitaiki-Tarawera Rivers Scheme Asset Management Plan (Wallace, 1998).

2.1 Catchment and Geology

The Rangitaiki River has a total catchment of 3005km^2. It rises 130km from the Bay of Plenty coast and 32km east of Lake Taupo at an elevation of about 800m above mean sea level, and from there flows for 64km across the Kaingaroa Plains to Murupara. The Rangitaiki has been dammed towards the northern end of the plains, diverting some flow into the Whaeo, a major tributary joining the Rangitaiki upstream of Murupara, as part of the largely run-of-the-river Whaeo power scheme.

Despite a catchment area above Murupara of over 720km^2, the river at that point remains essentially similar in character to that above due to the small variation in flow. The flat pumice covered plains are very absorbent and regulate runoff to such an extent that flood flows at this point are only two or three times normal flow.

Within 20km downstream of Murupara, two major tributaries, the Whirinaki 527km^2, and the Horomanga 218km^2, enter the river on the eastern side. These rivers rise in the steep bush–covered Ikawhenua Ranges composed of greywacke rock, where the runoff is high and consequently they contribute relatively large flood flows (and quantities of shingle) to the main channel.

The river then continues over the Galatea Plains before it enters Lake Aniwhenua where it is used for electricity generation. Lake Rerewhakaaitu drains into the Rangitaiki at this point. Downstream of the dam, the main channel passes through a gorge and then out onto the Waiohau Plains. Several small tributaries arising in the Ikawhenua Ranges cross the Waiohau, contributing substantially to the flood flows. The river travels for 13 km across the Waiohau Plains before it enters Lake Matahina where once again it is used for electricity generation.

Below Matahina the Rangitaiki passes through a well-defined valley before crossing the Rangitaiki Plains. The plains covers an area of approximately 27,000 hectares, Geologically they are described as a graben, infilled by a combination of alluvial and marine processes. In many areas substantial swamps have deposited thick peat layers. In broad terms the plains covers consist of a number of northeast-southwest ridges (old foreshore sand dunes), with peat basins between them. Between the

layers of peat are alluvial sediments deposited by the three rivers (Tarawera, Rangitaiki and Whakatane) flowing northwards.

Rangitaiki River flows across the central part of the plains. It has a very high bed load of coarse pumice sands and has undoubtedly been the major contributor of alluvial sediments to the plains. It is important to recognise that it is perched above the surrounding plains making flooding an ever-present threat.

Downstream of Te Teko the river is stopbanked to provide protection from the 1% Annual Exceedance Probability (AEP) flood. (By definition, the 1% AEP flood has a 1% probability of occurring in any one year. It is also known as the “100 year” flood). The river finally enters the sea at Thornton through a 1500m cut excavated in 1913.

No major tributaries enter the river downstream of the Matahina Dam.

The catchment and river scheme features are presented in Figures 1 and 2.

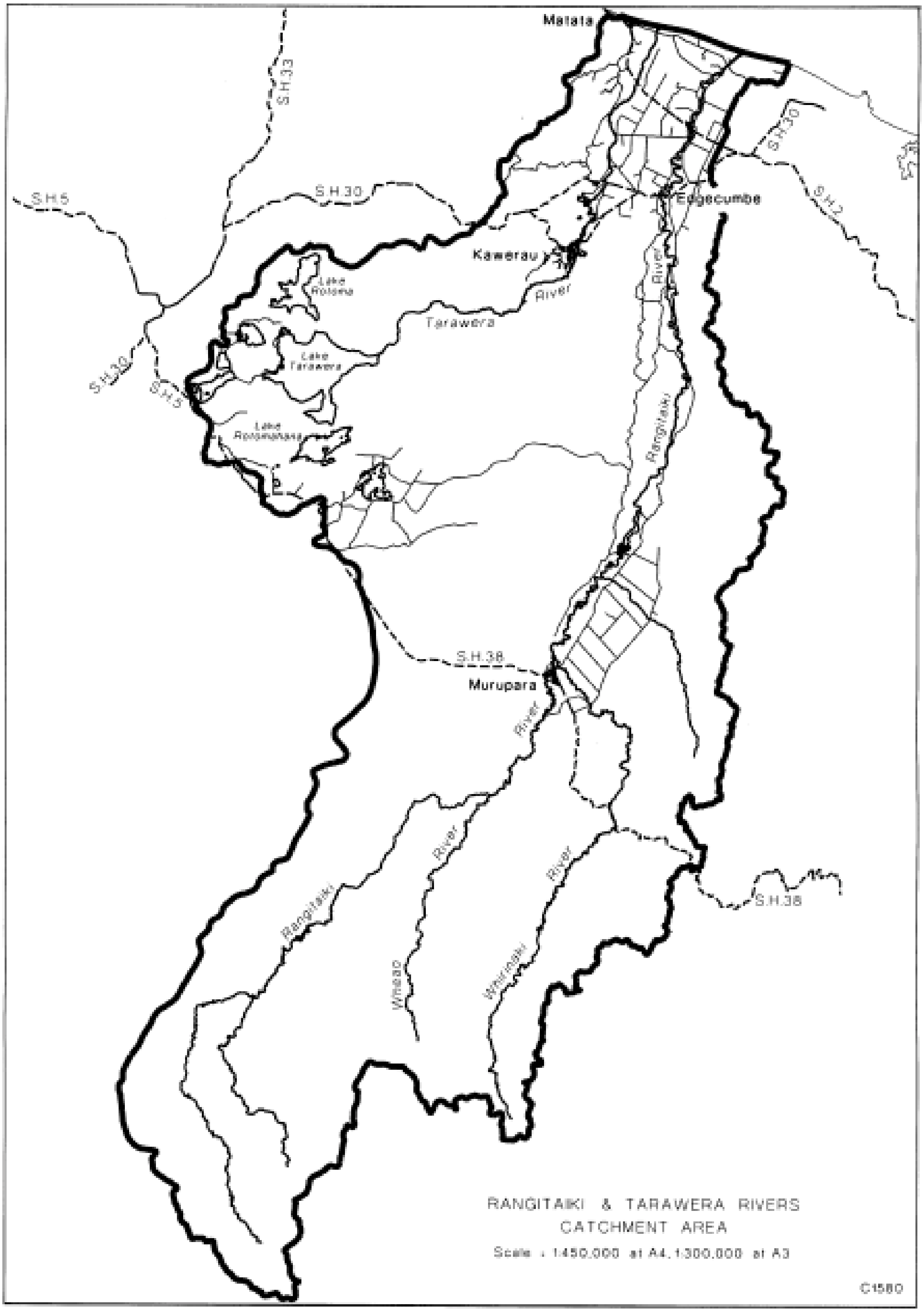

Figure 1 Rangitaiki and Tarawera Rivers Catchment Areas

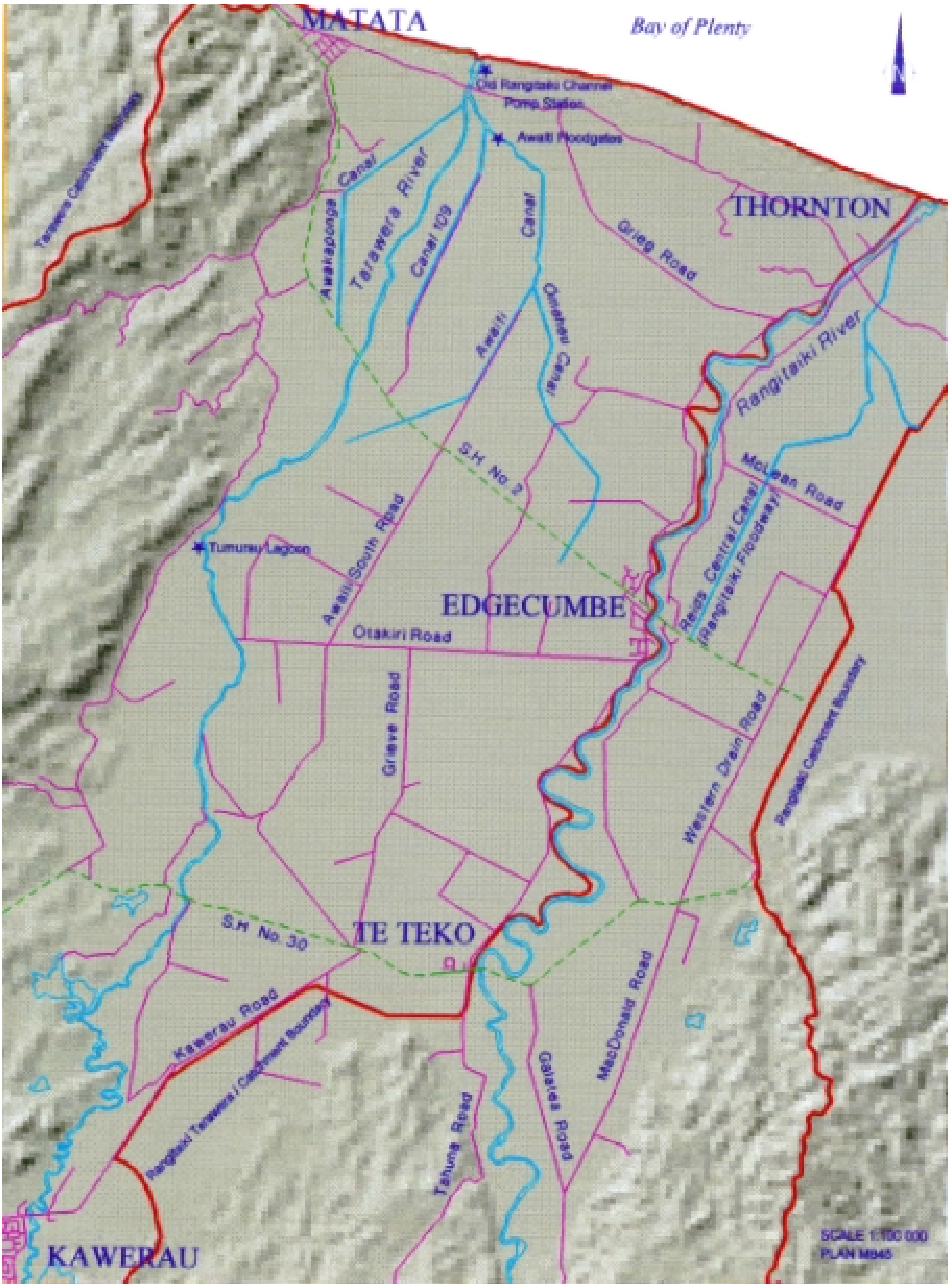

Figure 2 Rangitaiki-Tarawera Rivers Scheme: State Highway 30 to Coast

2.2 History of the Scheme

Prior to the construction of the Rangitaiki–Tarawera River Scheme, the Rangitaiki and Tarawera floodplains were subject to a risk of extensive flooding as a result of overflow from high river flows or from ad hoc stopbank failures. During the period 1944 to 1964, 15 floods occurred on the Rangitaiki River that caused extensive flooding to the surrounding areas. Ad hoc stopbanks which had been constructed along the Tarawera River breached in 1962. In addition, all riverbanks suffered from erosion, and the Galatea plains were subject to considerable gravel deposition.

The Rangitaiki–Tarawera River Major Scheme was designed by the then Bay of Plenty Catchment Commission during the 1960s to overcome these problems. The scheme objectives were principally

- to alleviate flooding on the Rangitaiki Plains from both rivers
- to alleviate flash flooding from the tributary streams and rivers of the Galatea Plains
- to control the channels of the rivers so as to reduce land loss through erosion and therefore increase the security of tenure of adjacent lands.

The works proposed were

- **Rangitaiki River from the mouth to Te Teko** — stopbank construction and channel widening and partial diversion via a floodway past Edgecumbe all with the capability of conveying the 1% AEP flood.
- **Above Te Teko to Matahina Dam** — willow clearing and bank protection works including rock rip rap.
- **Tarawera River from the sea to State Highway 30** — stopbank construction to convey the 1% AEP flood.
- **State Highway 30 to Kawerau Mill** – minor channel improvements and bank stabilisation.
- **Upper Rangitaiki River and Whirinaki River** — bank protection where erosion was occurring, channel clearing and stabilisation and establishing willow plantations to prevent debris being deposited on the river flats.
- **Galatea Streams** — Plantation establishment to stabilise the shingle fans, confine the floodway, and filter out heavy debris that causes most of the damage during flood events. Development of an adequate floodway over the lower reaches.

Most of the works were undertaken over the period from 1971 to 1980, although the Tarawera River stopbanks as designed were not completed until 1983. Stabilisation and bank erosion control has never been fully achieved however, and consequently bank protection work has continued and new bank protection assets are still being created.

In 1987, the Edgecumbe earthquake severely damaged stopbanks on the Rangitaiki River system and as a matter of urgency, a review of the scheme was undertaken with particular emphasis on the area from the fault scarp where a 1-2 metre discrepancy in top stopbank levels existed to below Edgecumbe. At that stage a review of the hydraulics and hydrology was undertaken based on the original scheme criteria. For the purposes of design, however, a 500 mm freeboard was adopted as much of the works were in the vicinity of the Edgecumbe township.

This review was carried out under some pressure in order to meet a deadline to obtain a government grant and restore levels of flood protection promptly. Little information was available to validate the computer model then used for predicting design flood levels. However, the following areas requiring remedial works to counteract the earthquake's impacts were identified on the Rangitaiki River Scheme:

- From the floodway to the fault scarp and extending downstream to Grieg Road on the left bank and Thornton on the right bank
- Immediately below Thornton both banks
- Reids Central Canal

A further study of the flood hazard from the Rangitaiki was undertaken in 1995 (Barnett Consultants, 1995). As well as investigating the effect of several breach scenarios, overtopping scenarios were considered. The study concluded that the 1% AEP flood would be conveyed without threatening to overtop the stopbanks – although there were some areas where freeboard was less than 500 mm. This model relied on the same computer model calibration as applied in the post earthquake review.

2.3 Stopbank Capacity – Asset Management Plan

Stopbanks are built to a level and grade where they will not be overtopped by the design flood (Table 1). They are also constructed to appropriate batters and top width to ensure their structural integrity. The capacity of the lower Rangitaiki River as outlined in the Asset Management Plan is presented in Table 1.

Table 1 Design Levels for Lower Rangitaiki River

Location	Design Level
Rangitaiki River – Rural	100 year plus 300 mm freeboard
Rangitaiki River – Urban (Te Teko, Edgecumbe, Thornton)	100 year plus 600 mm freeboard
Rangitaiki Floodway	100 year plus 250 mm freeboard

Chapter 3: Hydrology

This section presents the results of a detailed hydrological investigation of the magnitude and frequency of floods at the Te Teko recorder site on the Rangitaiki River (Site number 57412). For the hydraulic analysis the input flow hydrographs are applied at this location. The hydrological investigation was peer reviewed by the National Institute of Water and Atmosphere (NIWA). It is understood that hydro-meteorological processes in the Bay of Plenty are more complex than the norm, requiring application of specialist techniques to estimate design flows. A central issue has been how to incorporate the large flood recorded in 1944 into the analysis. This flood was measured at 784 cumecs as a result of up to 400 mm of rain falling in the catchment during the event.

3.1 Records of Annual Maxima

Continuous flow records since 1949 are available for the Rangitaiki River at Te Teko. Additional information is available on the size of several historical floods including the large flood events in 1925 and 1926 and the very major flood in 1944. The flow site is managed by the National Institute of Water and Atmosphere and it is assumed that the flow ratings are in order (there are regular updates, including review of the rating applying to the July 1998 flood).

The annual maxima are presented in Table 2.

Table 2 Rangitaiki River at Te Teko Annual Extremes

Year	Discharge (cumecs)	Rank	Return Period (years)	Y Variate
1925	510	5	16.47	2.771
1926	453	8	9.94	2.244
1944	784	1	134.14	4.895
1949	279	17	3.77	1.177
1950	118	54	1.01	-1.507
1951	309	15	4.42	1.361
1952	194	40	1.40	-0.228
1953	331	13	5.35	1.575
1954	198	38	1.48	-0.120
1955	128	53	1.03	-1.250
1956	220	29	2.00	0.367
1957	190	42	1.33	-0.339
1958	409	9	8.78	2.112
1959	232	23	2.61	0.729

Year	Discharge (cumecs)	Rank	Return Period (years)	Y Variate
1960	229	25	2.37	0.602
1961	131	51	1.07	-0.980
1962	384	10	7.79	1.985
1963	190	43	1.29	-0.397
1964	272	18	3.51	1.093
1965	596	3	29.34	3.362
1966	248	21	2.91	0.866
1967	567	4	21.10	3.025
1968	203	36	1.57	-0.013
1969	210	34	1.67	0.093
1970	637	2	48.15	3.864
1971	457	7	11.45	2.393
1972	227	27	2.17	0.482
1973	142	48	1.15	-0.719
1974	316	14	4.84	1.464
1975	220	30	1.92	0.311
1976	306	16	4.07	1.266
1977	182	45	1.23	-0.517
1978	228	26	2.27	0.541
1979	238	22	2.75	0.796
1980	131	52	1.05	-1.097
1981	202	37	1.52	-0.066
1982	139	49	1.12	-0.797
1983	372	11	6.76	1.832
1984	214	31	1.85	0.255
1985	266	19	3.29	1.013
1986	231	24	2.49	0.665
1987	223	28	2.08	0.423
1988	187	44	1.26	-0.456
1989	192	41	1.36	-0.283
1990	211	33	1.73	0.147
1991	198	39	1.44	-0.174
1992	148	47	1.17	-0.648
1993	154	46	1.20	-0.581
1994	213	32	1.79	0.201
1995	349	12	5.97	1.697
1996	205	35	1.62	0.040
1997	135	50	1.10	-0.882
1998	464	6	13.51	2.565
1999	263	20	3.09	0.938

3.2 Flood Analysis Methodology

At site flood frequency analyses were applied to:

(i) The continuous series of annual maxima (1949-1999)

(ii) The continuous series of annual maxima plus the three largest historical peaks (1925, 1926, 1944).

It was clearly evident that a straight line Gumbel plot underestimates the higher return period events. This is a peculiar phenomenon in the Bay of Plenty, likely related to the relatively low number of flood events in a given partition interval (usually one year). This is well documented and was even commented on in the original scheme documents (1969). Section 1.6, Volume 1 of these documents states:

"Rainfall is about average for the country but occurs on fewer rain days so it is generally fairly intense."

Another cause may be due to the porous nature of the catchment.

The annual maxima data thus conforms to a General Extreme Value (GEV) Type 2 frequency distribution. This distribution is also approximately by applying the Log Pearson Type III methodology.

A second feature of the annual maxima data was that the annual maxima possibly conforms to two separate flood frequency distributions. That is, there is a separate distribution for more extreme floods. This is known as the Two Component Extreme Value (TCEV) distribution. This TCEV characteristic was evident in the 1969 scheme flood frequency analysis and is a feature of the flood frequency analysis presented on BOPCC (Bay of Plenty Catchment Commission) Plan No. R400, sheet 55. The flood frequency line abruptly steepens at around a five year return period. Analysis based on the increased database now available confirms this. As the Matahina Dam was commissioned in 1967 and the TCEV characteristic was evident in preceding data, it can be concluded this is not due to the influence of the dam. It is likely due to hydro-meteorological phenomenon.

Application of a GEV frequency distribution can reasonably approximate the TCEV characteristics in the range of floods up to (and including) 100 years for the Rangitaiki River.

3.3 Effect of Matahina Dam

Detailed discussion on the effects of Matahina Dam is presented in section 3.4.5 of the post earthquake report (Dine, 1988). This report noted that the dam has the capability of providing some reduction to minor flood peaks. However, because of the very high volume of discharge relative to the dam impoundment size, potential attenuation is minor. This is especially the case for the larger floods. For example, the dam is capable of storing only six percent of the volume of the 100 year flood hydrograph. The report concluded:

"Matahina dam has a small effect on reduction of flood peaks such that the Te Teko peak flows record can be considered statistically homogeneous."

The minimal impact of dam storage on flood flows was reconfirmed by Opus International Consultants Limited in investigations carried out for ECNZ preparatory to the reconstruction of the Matahina dam.

3.4 Previous Estimates for the 100 Year Discharge

The principal estimates produced for the 100 year return period flow of Te Teko have been:

- 1969 Scheme Report: 793 cumecs based on identical estimates using annual maxima (1951-1967 plus 6 historical peaks) and unit hydrographs.
- 1988 Review: 755 cumecs based on application of the Log Pearson Type III methodology to the annual maxima (1949-1986). It appears that an unadjusted skew coefficient was applied.
- 1992 Environment B·O·P estimate: 740 cumecs based on application of Log Pearson Type III methodology to the annual maxima (1949-1983). The results suggested that the fitting method of Bobee (Journal of Water Resources Research, 1975) may have been applied.

3.5 February 1944 Flood

Inclusion of the historical floods in a censored flood frequency analysis has a small influence on the 100 year flood estimate (around five percent increase on continuous series analysis). The main component of this small increase is the influence of the February 1944 flood. This flood is the largest recorded and it is therefore appropriate that it be subject to some scrutiny.

The February 1944 flood was gauged at Te Teko and overflows included in the flow estimate (Horace Freestone, previous BOPCC hydrologist, pers comm). The flow was measured at 784 cumecs. The 4 day catchment rainfall above Te Teko (BOPCC Plan No R400, Sheet 49) varied from 9 to 15 inches and averaged approximately 12 inches (305 mm).

A quick verification of the 1944 flow is presented based on the following assumptions:

(i) Recorded flows are adjusted to provide a flow estimate based on the 1944 rainfall, according to the 2 day mean catchment rainfall total above Te Teko in this and other floods recorded at Te Teko recorder.

(ii) The rainfall maps for the 1944 and other floods are either 3 or 4 day. These are converted to 2 day rainfalls by applying a ratio of (Rainfall Duration/Two Days) to the power of 0.45. All things being equal this should balance out over the various storms. Thus the 1944 2 day rainfall is assessed at 223 mm.

(iii) The original scheme design 100 year discharge was derived identically from a unit hydrograph and flood frequency analysis. The Rangitaiki River Major Scheme design 48 hour isohyet map (BOPCC Plan No.R400, Sheet 49) applied to the unit hydrograph is also used to provide a 1944 flow estimate.

(iv) The western part of the catchment is much more porous (volcanic ash) than the eastern side (largely greywacke in the Ikawhenua Range). However, the asymmetry of storm isohyets with respect to catchment infiltration characteristics is not large (effect less than 10%).

The average derived 1944 flow from Table Three is 807 cumecs. Excluding upper and lower estimate it is 797. Obviously a formal unit hydrograph or other rainfall-runoff model could further improve the validation accuracy (albeit with little temporal information). However, as inclusion of the 1944 flood has little impact on final value, the work is not warranted.

Therefore it can be concluded that the 1944 flood estimate is reasonable, based on the fact it was gauged and correlates well with the rainfall-runoff relationships in other storms.

Table 3 Validation of 1944 Flow Measurement

Date	Rainfall Duration (Days)	Catchment Mean (mm)	Catchment 2-Day Mean (mm)	Recorded Discharge (Cumecs)	Calculated 1944 Discharge (Cumecs)
December 1958	4	127	93	409	981
June 1962	3	152	127	384	674
February 1965	4	203	149	596	892
February 1967	3	203	169	567	748
11 July 1998	2	136	136	460	754
Design Unit Hydrograph	2	254	254	906	795

3.6 Continuous Series Flood Frequency Estimates

Flood frequency estimates were produced for the GEV distribution using the computer programme FRAN. NIWA also calculated estimates for the GEV and TCEV distributions. The results are presented in Table 4.

Table 4 Flood Frequency Estimates: Annual Maxima 1949-1999 (cumecs)

Return Period (years)	Environment B·O·P Estimate	NIWA Estimates	
	GEV	GEV	TCEV
2	224	224	-
5	320	317	310
10	399	398	410
20	490	487	509
50	634	627	636
100	765	755	735
200	919	904	826

3.7 Censored Flood Frequency Estimates

Flood frequency estimates were produced for the GEV distribution using the computer programme FRANCES. There is currently no known computer software for calculating TCEV estimates for a censored sample. However, a distribution was

manually fitted to the data and appeared very reasonable. NIWA also calculated estimates for the GEV and Log Pearson III (LP3) distributions. The results are presented in Table 5.

Table 5 Flood Frequency Estimates: Annual Maxima 1949-1999 Plus 1925, 1926 and 1944 Historical Floods (cumecs)

Return Period (years)	Environment B·O·P Estimates		NIWA Estimates	
	GEV	TCEV	GEV	LP3
2	225	220	226	226
5	325	325	326	330
10	410	410	411	417
20	508	530	509	541
50	666	660	667	666
100	813	760	813	801
200	988	850	987	958

3.8 Design Flood Frequency Estimates

It is clear that the historical flood peaks should be included in the analysis (with appropriate plotting position), particularly the 1944 flood. Based on this an equal weighting was applied to the four NIWA estimates, yielding a 100 year return period design estimate of 780 cumecs (application of the averaged Environment B·O·P estimates yields an identical figure). NIWA have confirmed this figure is appropriate in their peer review (dated September 2000) which states:

"Your choice of a 1 in 100 AEP design flood estimate for the Rangitaiki River at Te Teko of 780 m^3/s seems reasonable. As noted, the standard error of estimate is about ± 120 m^3/s. Also, it is very close to the maximum for the 1944 flood of 784 m^3/s."

The full design estimates are presented in Table 6 and Figure 3. The longitudinal axis in Figure 3 is presented in terms of the "y variate". Values for the y variate corresponding to return periods are presented in Table 6.

Prior to completing a floodplain management strategy for the Rangitaiki Plains the estimates for over-design floods should be reviewed. Currently these range from around 1200 cumecs for the 1000 year flood to 2300 cumecs for the Probable Maximum Flood (PMF).

Figure 3 Rangitaiki River: Flood Frequency 1949-99 Plus 1925, 1926 and 1944 Historical Peaks

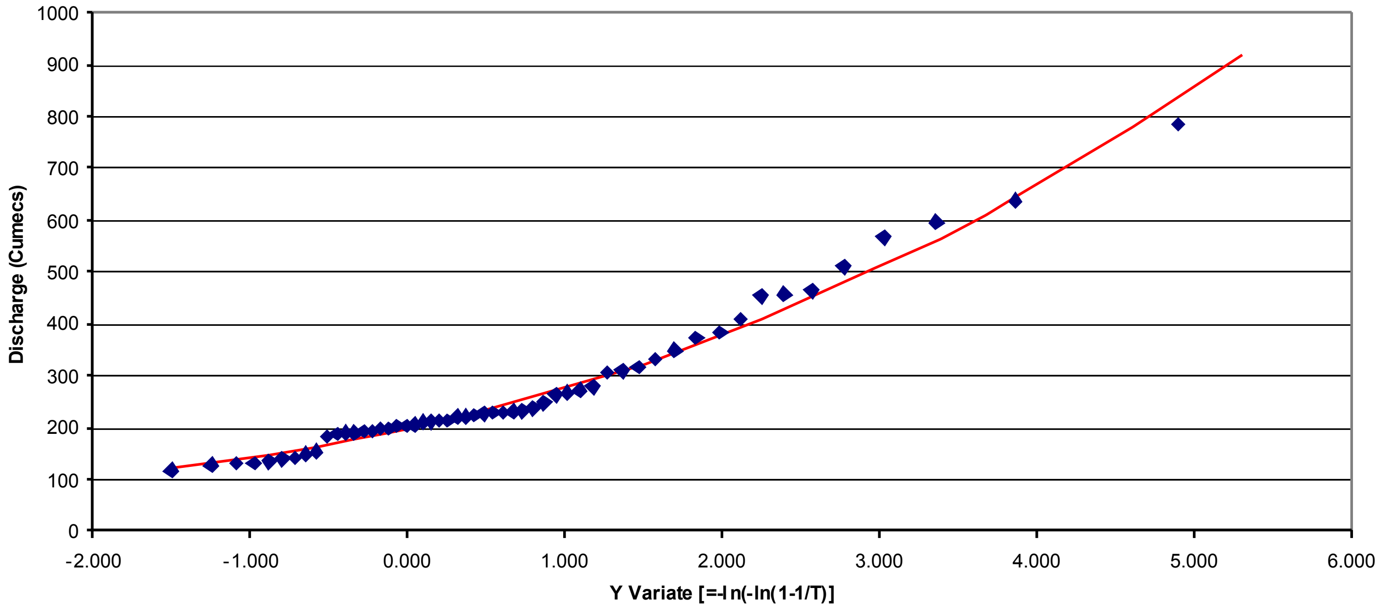

Table 6 Design Flood Frequency Estimates

Return Period (years)	Discharge (cumecs)	Approximate Standard Error (cumecs)	Y Variate
2	225	14	0.678
5	320	26	1.500
10	410	38	2.250
20	505	55	2.970
30	565	65	3.384
40	610	75	3.676
50	650	86	3.902
100	780	120	4.600
200	920	166	5.296

3.9 Design Flood Hydrographs

The design hydrograph is a directly scaled version of the actual July 1998 flood hydrograph recorded at Te Teko. This hydrograph shape is consistent with that recorded in the August 1970 flood (at 637 cumecs the second largest recorded) and those hydrographs derived from unit hydrographs as presented in the 1969 and 1988 reports. Both the July 1998 and design hydrographs are shown in Figure 4.

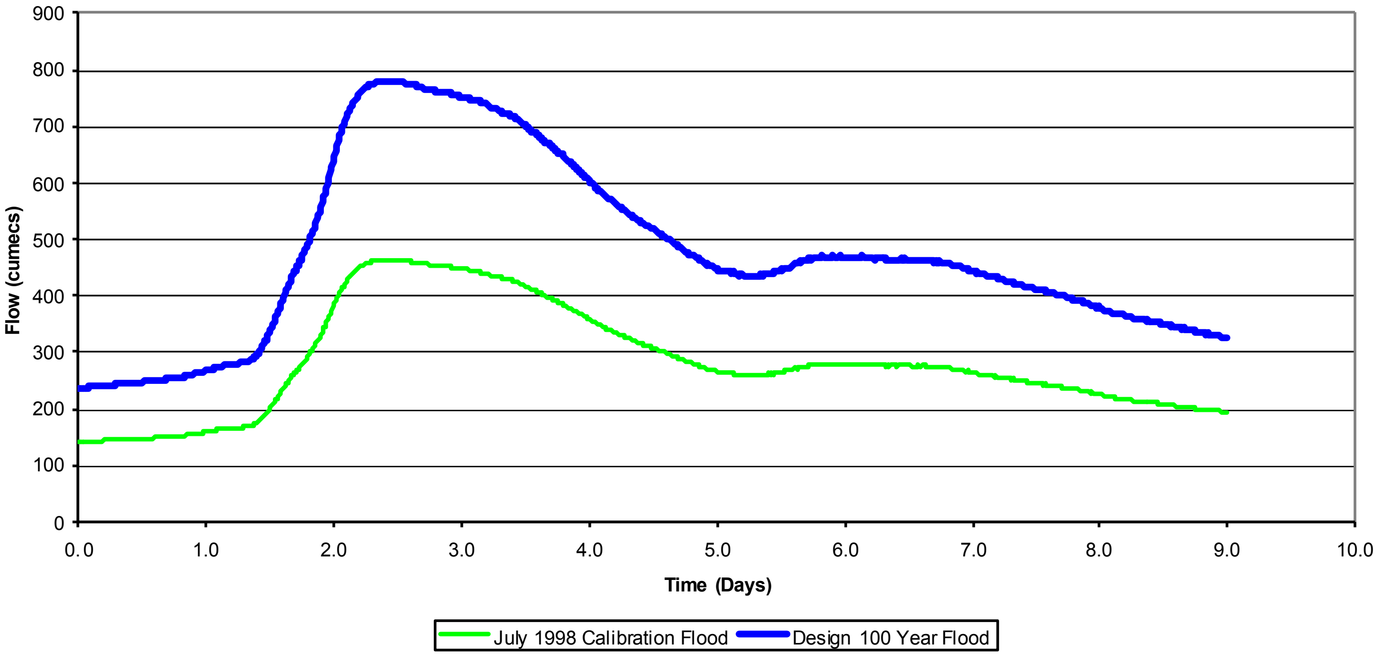

Figure 4 Rangitaiki River at Te Teko Design Hydrographs

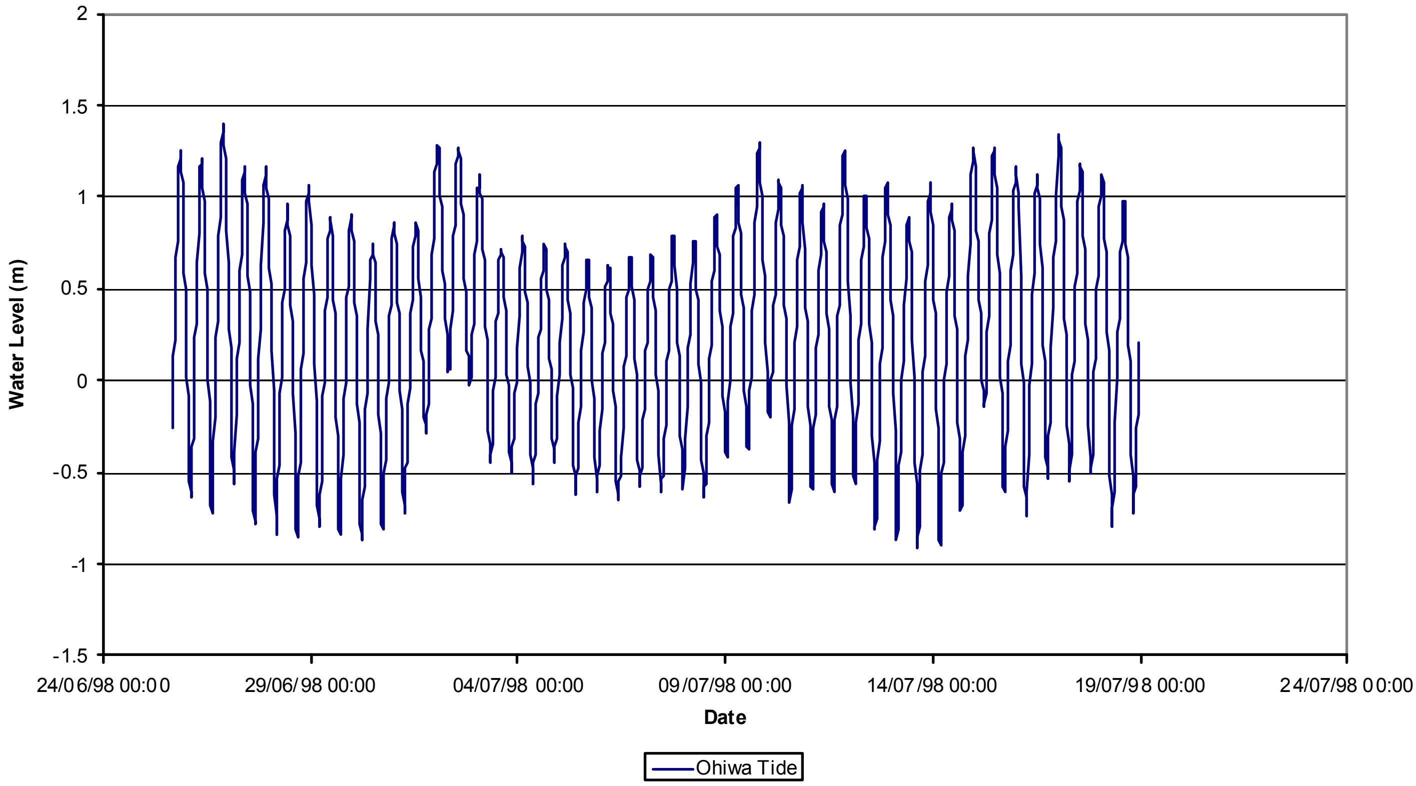

Figure 5 Calibration Tide Hydrograph

Chapter 4: Hydraulic Analysis

The hydraulic analysis and computer modelling have been peer reviewed by A C Consulting Group Limited. This consulting engineering firm have offices in major New Zealand cities and are well experienced in hydraulic analyses.

4.1 Computer Software

The Mike 11 (Version 2000) hydraulic modelling software package was used to simulate different flood level scenarios on the lower Rangitaiki River and Reids Central Canal. Mike 11 uses an implicit finite difference scheme for the computation of unsteady flows in rivers. It also incorporates advanced computational models for the description of flow over hydraulic structures. The non-steady flow handling properties of the programme (Mike 11) were used to effectively apply the varying stage versus time relationship at the Te Teko flow recorder and downstream tidal boundary. It is relevant that Mike 11 or similar sophisticated hydraulic modelling software were not available in 1988.

4.2 Model Configuration

The model consists of 68 cross sections (surveyed in 1999) on the Rangitaiki River connected to the Reids Central Canal at cross-sections number 2 and 36 to 37 (spillway). There are 34 cross-sections (surveyed in 1994) in the Reids Central Canal branch. The upstream spillway connection is by way of two simple weirs to reflect varying flood levels over the outflow reach.

The hydraulics of the spillway are complex and important as they ultimately determine the flow split-up at this point. In the timeframe available for this report it has not been possible to confirm the adequacy of this control and it is recommended that it be further fine-tuned (to confirm adequacy and effect of bend super-elevation on outflow characteristics). However, as the spillway did not activate in the 1998 calibration flood (this was only a 15 year flood, somewhat below the 40 year flow at which the spillway is meant to commence operation) its performance in the model does not affect the model calibration. In any case, as it is a simple matter to reconfigure the spillway control any inherent error is of very little consequence at this stage for design levels in the Rangitaiki River.

The models boundary conditions are:

(i) Flow at Te Teko recorder at Mike 11 river distance 0.0 km;

(ii) Tidal level at the coast at Mike 11 river distance 25.0 km. (Note Mike 11 uses a reverse river distance). This cross-section was added to more accurately replicate the tidal boundary. It is located 450 m outside the river mouth (cross-section 1A) and is a rectangular section some 300 m wide at invert level of –2 m (Moturiki Datum).

(iii) Initial flow of 0.1 cumecs in Reids Central Canal.

Cross-section number 1A under normal conditions is very narrow. Under flood flow conditions it naturally enlarges by scouring on the left bank (as was observed during the July 1998 flood). The cross-section has been artificially enlarged for both the calibration and predictive scenarios using the method of "critical velocity" and comparisons with upstream velocities.

4.3 Calibration

Calibration is the process by which the computer model results are compared to a well known recorded flood to test the models ability to reproduce actual flood events. This model has been calibrated on the 12 July 1998 flood events (peak discharge 465 cumecs). Ideally the model should be validated by application to other recorded flood events.

In this case there is no other data available for a suitably large flood. It may well be misleading to use data from a smaller flood as the lower Rangitaiki River appears to exhibit totally different hydraulic characteristics in the calibration flood to those previously observed in smaller floods. This characteristic has been discussed in some detail with the peer reviewers who concur with this comment.

4.3.1 Flood Levels

A set of 32 peak flood levels in the lower Rangitaiki River were surveyed immediately following the 12 July 1998 flood. These were principally based on observed debris marks, with the quality of these appearing to be reasonably representative of peak river levels. However, the recorded water level at cross-section 1A was disregarded as it was clearly generated by coastal wave runup. Because of the absence of recorded levels on the left bank downstream of Edgecumbe a further peak level recorded on the property of Mr and Mrs R Jones was surveyed in September 2000. This mark concurred very well with the flood level recorded on the opposite right bank. The calibration level data is presented in Table 7.

4.3.2 Calibration Boundary Conditions

The flow hydrograph applied is that recorded at Te Teko throughout the event (refer to Figure 4). The tide hydrograph applied is that recorded at the Ohiwa Harbour recorder throughout the event (refer to Figure 5).

Table 7 *Rangitaiki River Calibration Data for 12 July 1998 Flood Event*

Ref: LB 629 p1-11,14,15,17,18 and LB 648 p 66

Section	Mike 11 Distance (km)	Distance (km)	WL Right bank	WL Left bank	Comments	DWL Dine 87	PLB Calib Adj Lower Riv	Offsets Corrected	Diff With Offsets Corr
Sea	25						1.268	1.268	0
1a	24.55	0	2.25			2	1.273	1.273	0
1	23.97	0.58				2.13	1.345	1.35	0.005
2	23.66	0.89	1.47			2.21	1.497	1.498	0.001
3	23.12	1.43				2.34	1.749	1.756	0.007
4	22.87	1.68	1.9			2.4	1.931	1.912	-0.019
5	22.32	2.23				2.54	2.196	2.224	0.028
6	21.92	2.63	2.64			2.63	2.4	2.426	0.026
7	21.51	3.04				2.73	2.579	2.603	0.024
8	21.12	3.43	2.67			2.89	2.74	2.759	0.019
9	20.74	3.81	2.91			3.04	2.929	2.942	0.013
10	20.38	4.17				3.19	3.068	3.079	0.011
11	19.92	4.63	3.38			3.38	3.231	3.24	0.009
12	19.59	4.96	3.43			3.51	3.352	3.354	0.002
13	19.18	5.37				3.68	3.488	3.487	-0.001
14	18.78	5.77	3.63			3.84	3.621	3.62	-0.001
15	18.21	6.34				4.07	3.79	3.787	-0.003
16	17.81	6.74	3.91	3.89	Jones gate 20 m dstm (LB)	4.3	3.94	3.942	0.002
17	17.44	7.11				4.51	4.078	4.05	-0.028
18	17.12	7.43	4.18			4.69	4.174	4.139	-0.035
19	16.61	7.94				4.98	4.329	4.289	-0.04
20	16.35	8.2	4.4			5.13	4.412	4.372	-0.04
21	15.8	8.75				5.44	4.544	4.503	-0.041
22	15.4	9.15	4.62			5.67	4.687	4.65	-0.037
23	15.04	9.51				5.87	4.801	4.744	-0.057
24	14.66	9.89	5.1			6.02	4.951	4.883	-0.068
25	14.16	10.39				6.21	5.141	5.085	-0.056
26	13.81	10.74	5.25			6.35	5.265	5.22	-0.045
27	13.36	11.19				6.52	5.344	5.353	0.009
28a	13.21	11.34	5.53			6.58	5.408	5.412	0.004
28b	13.01	11.54				6.66	5.458	5.457	-0.001
29	12.7	11.85				6.78	5.521	5.513	-0.008
30	12.41	12.14	5.78			6.89	5.621	5.591	-0.03
31	12.04	12.51				6.94	5.707	5.7	-0.007
32	11.69	12.86	6			6.99	5.771	5.768	-0.003
33	11.32	13.23				7.04	5.801	5.797	-0.004
34	10.87	13.68	6.04	6.03		7.1	5.872	5.865	-0.007
35	10.47	14.08				7.15	5.969	5.963	-0.006
36	10.06	14.49		6.16		7.32	6.089	6.058	-0.031
37	9.69	14.86				7.47	6.192	6.127	-0.065
38	9.44	15.11	5.98	6.16		7.57	6.241	6.21	-0.031
39	9.09	15.46				7.71	6.318	6.282	-0.036
40	8.78	15.77		6.38		7.84	6.415	6.371	-0.044
41	8.59	15.96		6.66			6.507	6.458	-0.049
41a	8.39	16.16				8.04	6.586	6.537	-0.049
42	8.01	16.54				10.14	6.601	6.558	-0.043
43a	7.58	16.97				10.5	6.739	6.681	-0.058
43b	7.41	17.14				10.52	6.819	6.765	-0.054

Section	Mike 11 Distance (km)	Distance (km)	WL Right bank	WL Left bank	Comments	DWL Dine 87	PLB Calib Adj Lower Riv	Offsets Corrected	Diff With Offsets Corr
43c	7.09	17.46				10.57	6.934	6.881	-0.053
44	6.7	17.85		7.3		10.62	7.268	7.222	-0.046
45	6.3	18.25				10.78	7.561	7.499	-0.062
46	5.94	18.61		7.78		10.92	7.808	7.681	-0.127
47a	5.43	19.12	8.2			11.11	8.15	8.005	-0.145
47b	5.21	19.34				11.2	8.281	8.155	-0.126
48	4.99	19.56				11.29	8.361	8.239	-0.122
49	4.47	20.08		8.955		11.49	8.7	8.604	-0.096
50	4.19	20.36				11.6	8.915	8.827	-0.088
51a	3.94	20.61				11.69	9.126	9.02	-0.106
51b	3.59	20.96				11.83	9.38	9.29	-0.09
51c	3.24	21.31				12.07	9.619	9.543	-0.076
52	2.94	21.61		10.01		12.28	9.849	9.774	-0.075
53	2.52	22.03				12.57	10.094	10.027	-0.067
54	2.31	22.24				12.72	10.186	10.123	-0.063
55	1.597	22.953	10.97			13.18	10.646	10.587	-0.059
56	1.094	23.456				13.46	11.012	10.96	-0.052
57	0.793	23.757				13.76	11.136	11.087	-0.049
58	0.39	24.16				13.97	11.281	11.241	-0.04
59	0	24.55		11.41			11.398	11.355	-0.043

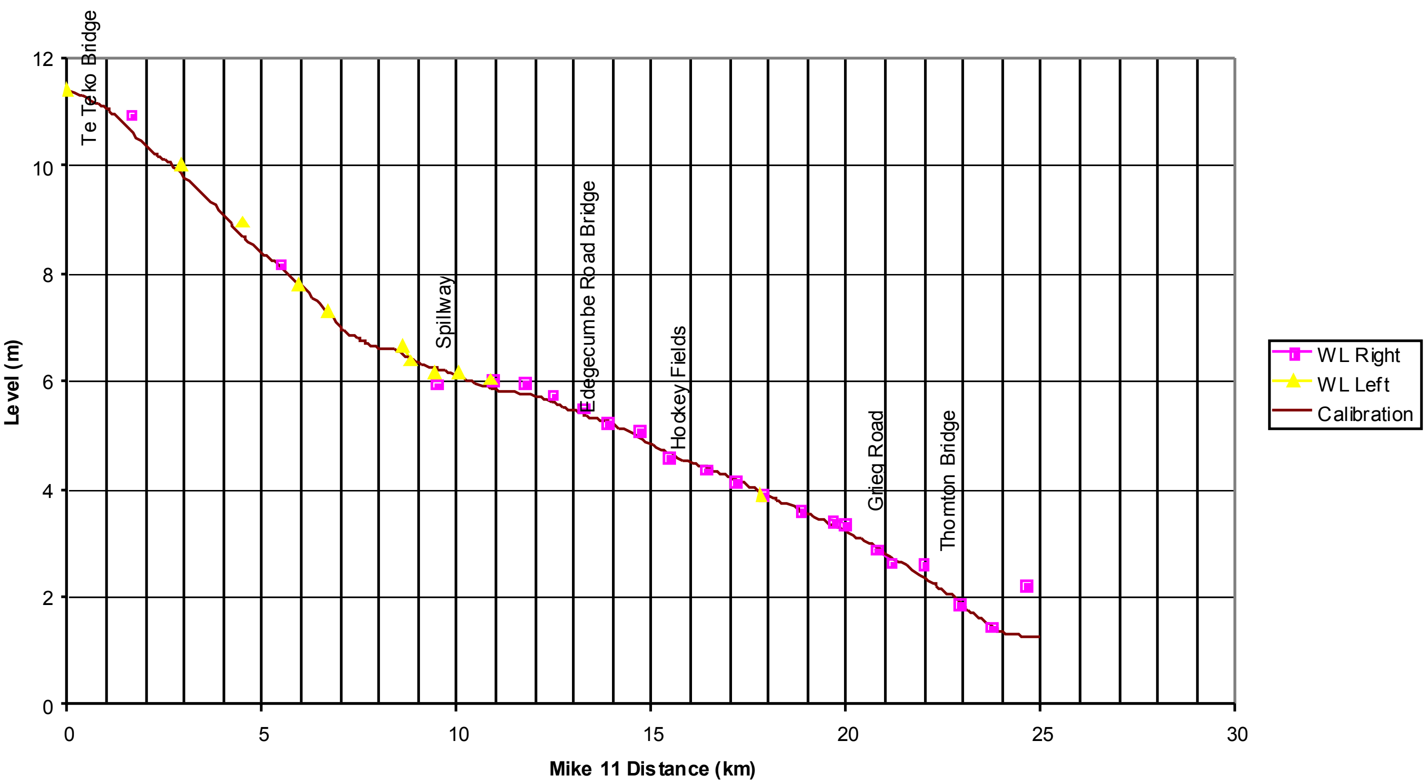

Figure 6 *Rangitaiki River: Calibration 464 Cumecs with Mouth Cross-section Adjusted*

4.3.3 Resistance Value

The resistance values used in the final calibration are presented in Table 8. Details of the calibration are shown in Figure 6 and Table 7.

Table 8 Mannings N Coefficient

Mike 11 Chainage (km)	River Distance (km)	Mannings N
24.550	0.0	0.030
24.30 to 23.6	0.25 to 0.95	0.040
23.30 to 15.11	1.25 to 9.44	0.0425
14.77 to 14.38	9.78 to 10.17	0.051
13.91 to 11.49	10.64 to 13.06	0.053
11.11 to 8.94	13.44 to 15.61	0.045
8.65 to 4.96	15.90 to 19.59	0.048
4.44 to 1.22	20.11 to 23.33	0.058
0.82 to 0.0	23.73 to 24.55	0.048

The resistance values have been varied in accordance with recorded flood levels and physical river characteristics. In order to estimate river "centre-line" values they will slightly under estimate levels recorded on the outside of bends (cross-section numbers 11, 32, 52 and 55) and a super-elevation allowance of around 100 mm should be added to the final design levels. Similarly they exclude bridge effects (refer to section 4.3.5). These values are all significantly more than those applied in the 1988 model and are very high for a river of this nature (very flat gradient of around 0.04 percent below Edgecumbe and little turbulence).

The text "Open Channel Flow" (Henderson, 1966) provides some guidance on the range of values to be expected in natural river channels as follows:

Clean and straight	0.025 – 0.030
Winding with pools and shoals	0.033 – 0.040

Whilst higher values than those advised by Henderson are common in the steeper New Zealand rivers, the Rangitaiki should conform to the lower end of these values. The Henderson values are valid for many New Zealand rivers near the coast, where calibration values generally range from 0.025 (at the mouth) to 0.035 (at a slope of around 0.3 percent).

The Mannings N Values used in the post-earthquake computer model are presented in the 1987 technical report. These were slightly adjusted for the 1988 model (but appear to be not recorded). In the reach of river below Edgecumbe they were set at 0.022 from the mouth to cross-section number 7 (then river distance 3.117 km) and rose reasonably consistently to reach 0.034 at cross-section number 16 (then river distance 6.8 km). Above this point they generally increased further with isolated peak values of 0.047 and 0.075. Whilst the post-earthquake values in the lower river are much below those of the calibration flood they are only slightly below those recommended by Henderson. This model was calibrated on three separate events:

(i) A low flow event at high tide on 16 June 1987

(ii) A low flow event at low tide on 26 June 1987

(iii) A peak flow of 223 cumecs when Electricorp quickly lowered Lake Matahina (after concern about earthquake damage to the dam).

During the course of this peer review it has become evident that during a transformation of the input cross-section data the section markers have been reset to default values. Returning them to their correct positions slightly improves the calibration and in initial testing shows the predicted 100 year flood levels below Edgecumbe to slightly decrease (by up to 40 mm). However, this may be due in part to an anomalously large amount of water travelling down Reids Central Canal. Prior to final design the model should be adjusted to account for the marker repositioning.

4.3.4 Likely Cause of High Resistance Values

The calibrated resistance values do not concur with any of the normally observable river characteristics including slope, sediment size, sinuosity, changes in section and structures. The only rational explanation is that dunes form in the riverbed at high flows.

Dunes were previously observed on the Tarawera River at the Awakaponga site by the hydrological field parties in the 1950's and 1960's. Their effect was to generate vertical currents (boils) which adversely affected river gauging necessitating a change in flow measuring equipment (Horace Freestone, pers comm.). Mannings N values recorded during this period at this site varied from 0.035 to 0.042 – again much higher than those for normally observably river characteristics.

Gauging data at the Awakaponga site since the 1987 earthquake has been further examined and the relationship between derived Mannings N and hydraulic radius is shown on Figure 7. Based on this data it appears that resistance at this site does indeed increase at high flows.

The effect of dunes is central to the calibration of the hydraulic model. It would seem unlikely that the Mannings N values would increase much further than those evident in the July 1998 flood. However, further investigations into the dunes and hydraulic conditions for their formation should be conducted (by inspecting research reports and international literature). Procedures are in place to record and examine this phenomenon should a large flood occur.

4.3.5 Bridges

The observed flood level of 2.64 m at cross-section number 6 cannot be matched unless a very high resistance value (far outside the range of those in that reach of river) is applied. Possibly the value is erroneous, or it is due to the hydraulic restriction imposed by the Thornton Bridge. This restriction is reasonably minor (refer to Figure 8) and no significant efflux was observed during the 12 July 1998 flood.

In the interim an amount of 100 mm has been added to the computed levels for cross-sections 5 and 6 in the predictive scenarios.

The Thornton Bridge should be explicitly modelled to account for the hydraulic impacts of the abutments and piers. Similarly, at some stages the bridge at

Edgecumbe and Te Teko should be explicitly modelled although they have no real impact on levels in lower reaches of the river.

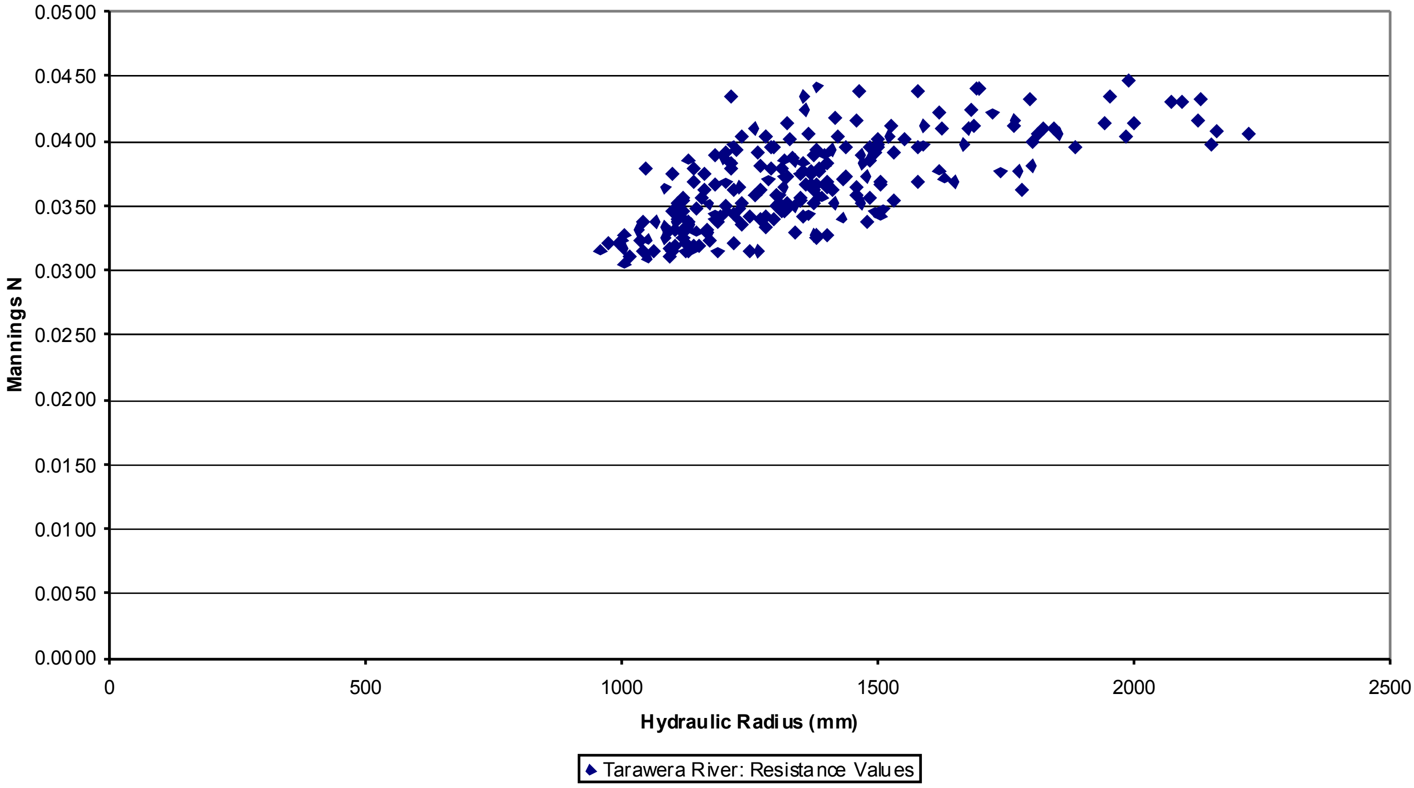

Figure 7 Tarawera River: Resistance Values Post 2 March 1987 Earthquake

Figure 8 Thornton Bridge

4.3.6 Reids Central Canal and Spillway

The calibration flood did not enter Reids Central Canal and therefore this part of the model remains uncalibrated. In the absence of calibration information an appropriate Mannings N value of 0.05 was applied. This will account for minor obstructions such as hedges. However, it is recommended that specific obstructions such as bridges receive explicit modelling.

The reach of river at the spillway outlet was carefully calibrated to ensure an accurate division of flows during design events. The problem with markers in the model will alter the calibration here and impact on the flow split (refer to section 4.3.3). However, once recalibrated it is likely a similar amount to that predicted in the initial computer runs (of around 90 cumecs in the 100 year event) will result. If this is not the case, then it is a small task to adjust the spillway level to obtain the modelled flow.

4.3.7 Recommendations for Further Work on the Computer Model

The following recommendations should be carried out prior to final detailed design of remedial works. However, they are unlikely to significantly affect the principal conclusions of this report.

(i) Recalibration to allow for marker repositioning;

(ii) Explicit modelling of Thornton Road Bridge and structures in Reids Central Canal;

(iii) Particular review of the spillway hydraulics;

(iv) Further confirmation of the likelihood of dune formation and their likely impact.

4.4 Scenario Combinations

Water levels in the 100 year design flood event are influenced by both the design flow and downstream tide level. The same meteorological conditions that caused flooding in the Rangitaiki catchment are also likely to generate a storm surge. However, it is unlikely that a 100 year storm surge will combine with 100 year flood flow because the unique conditions producing these separate events will rarely occur simultaneously.

The exact design combinations of events are to a degree subjective. However, a reasonable combination of events is the 100 year flood combining with a 20 year storm surge tide. Similarly, the converse scenario is a reasonable combination and will determine 100 year levels in the vicinity of the mouth.

The design peak storm surge values are:

20 year: 1.92 m
100 year: 2.36 m

These are based on estimates at Moturiki increased by 0.3 m to allow for increased surge in the eastern Bay of Plenty (due to the interaction of the concave shoreline with prevailing cyclone wind directions). For further details refer Goring et al (1997) and Blackwood (1997). The peak storm surge is assumed to increase to peak levels at two successive tide cycles and then decrease.

As the time of concentration of flood flows to Te Teko is in the range 36 to 48 hours it is most unlikely that the peak surge will coincide with peak flood flows. Assuming a centrally located hyetograph the peak surge values are applied one and two tide cycles before the peak flow reaches the river mouth.

The design storm surge hydrographs are presented in Figure 9.

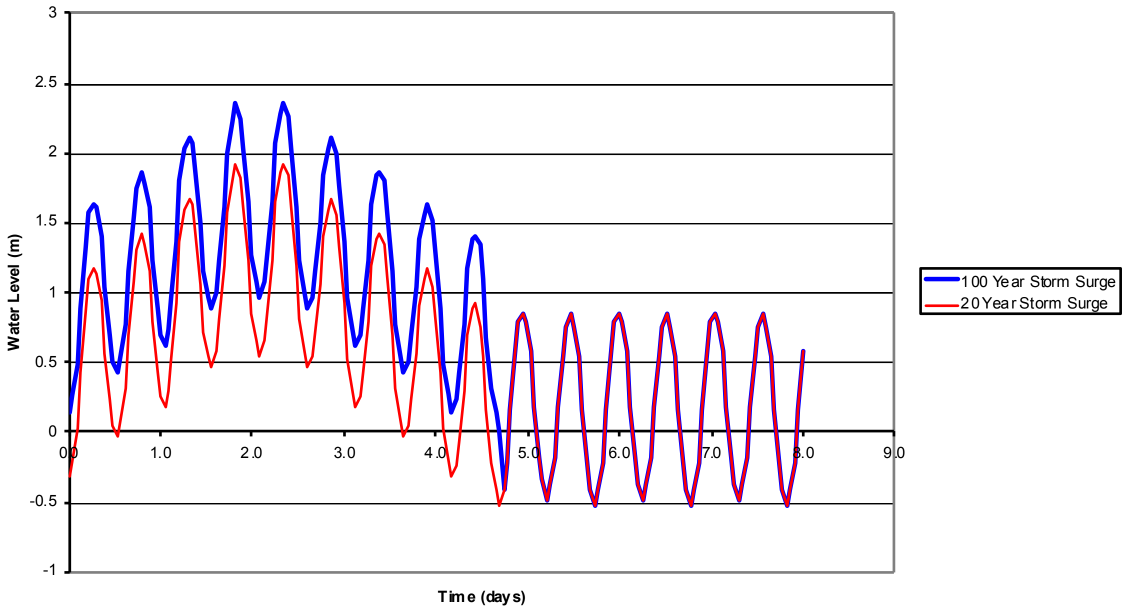

Figure 9 Rangitaiki River Mouth Design Storm Surge Scenarios

4.5 Results

4.5.1 Calibration Flood

The flood levels during the calibration flood can be directly compared with stopbank heights. With the data available, a comparison is only possible for the right-hand bank up to Edgecumbe (BM27); there is only one water level recorded for the left bank. The results of this exercise show that the minimum freeboard for the right-hand bank is approximately 300 mm between the mouth and Thornton Bridge and approximately 370 mm between Thornton Bridge and Edgecumbe under the calibration event. These are isolated points, however more significantly there is a total of some 600 m along the right bank where the freeboard is calculated to be less than 500 mm. Assuming the same recorded water levels (right-hand bank) for the left-hand bank shows that there is approximately 400 m where the freeboard is less than 500 mm.

This appears to correspond reasonably well with observations made during the flood by staff and local residents.

4.5.2 100 Year Flood

Predicted flood levels in the Rangitaiki River during the 100 year flood are presented in Table 9 and Figure 10. The values given are exclusive of freeboard requirements. In the table the last six columns are as follows:

DWL Dine 87

Design line (exclusive of freeboard) as modelled in the post-earthquake investigations. These values are derived from BOPCC Plan R595 Sheets 6 to 9.

PLB Calib

The calibration flood.

780 L20YR

100 Year flood combined with 20 year storm surge.

505 L100YR

20 Year flood combined with 100 year storm surge.

Combined Scenarios

The upper envelope of 780 L20YR and 505 L100YR. As can be seen in the table the latter scenario dominates only as far up the river as cross-section 3.

Combined Minus 87

The increase in flood levels of this review above that of 1987. The peak difference is an increase of 0.865 m at cross-section number 9 at Grieg Road. This increase is due entirely to the increase in Mannings N resistance coefficient.

Figure 10 shows that once the freeboard is added, apart from a few isolated low points the stopbank is between 0.3 and 0.7 metres too low over a length of some 6600 metres on the left bank and to a slightly lesser degree on the right bank.

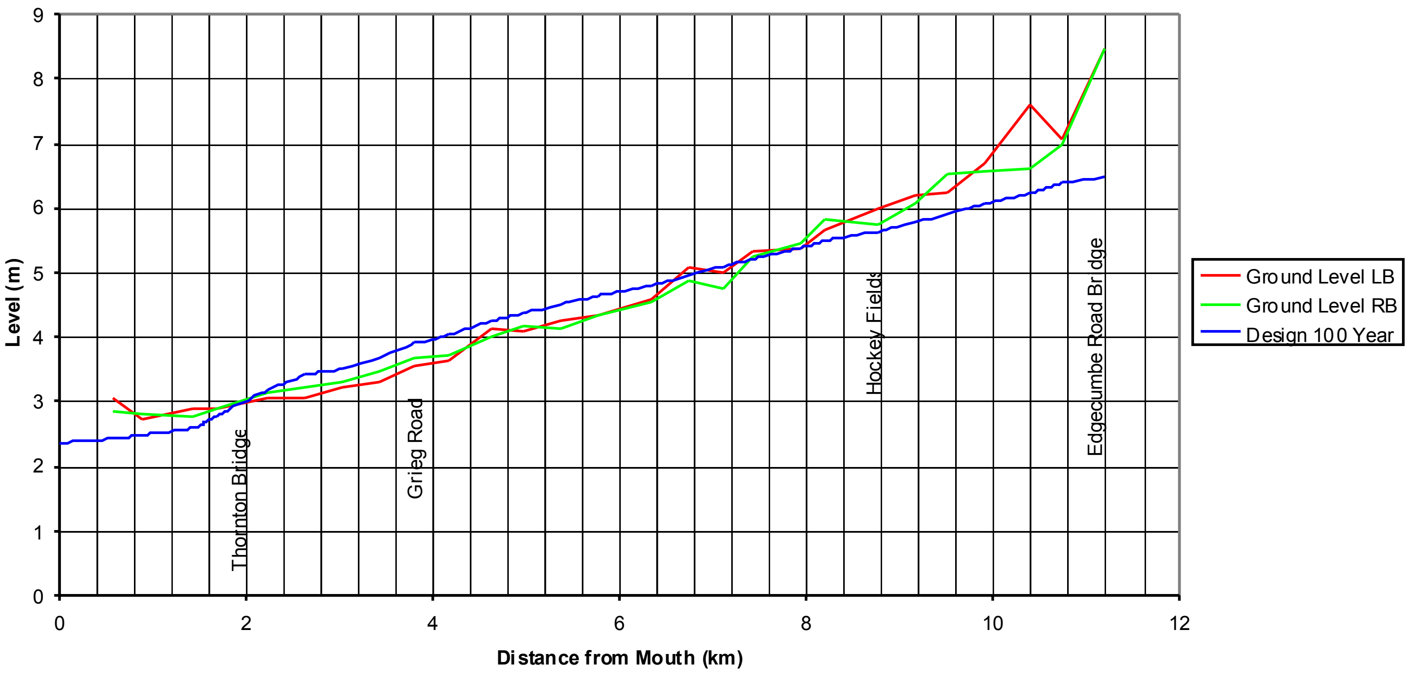

Figure 10 Rangitaiki River: Mouth to Edgecumbe - 100 Year Flood Levels (Moturiki Datum)

Table 9 Rangitaiki River - 100 Year Design Flood Levels (Moturiki Datum)

Section	Mike 11 Distance (km)	Distance (km)	Ground Level LB (m)	Ground Level RB (m)	DWL Dine 87	PLB Calib	780 L20YR	505 L100YR	Combined 100 Scenarios	Combined Minus 87
Sea	25					1.268	1.92	2.36	2.36	
1a	24.55	0			2	1.273	1.937	2.363	2.363	0.363
1	23.97	0.58	3.048	2.850	2.13	1.345	2.166	2.428	2.428	0.298
2	23.66	0.89	2.720	2.800	2.21	1.497	2.34	2.485	2.485	0.275
3	23.12	1.43	2.880	2.770	2.34	1.749	2.597	2.595	2.597	0.257
4	22.87	1.68	2.900	2.880	2.4	1.931	2.779	2.684	2.779	0.379
5	22.32	2.23	3.070	3.130	2.54	2.196	**3.172**	**2.938**	**3.172**	0.632
6	21.92	2.63	3.040	3.220	2.63	2.4	**3.412**	**3.076**	**3.412**	0.782
7	21.51	3.04	3.220	3.286	2.73	2.579	3.514	3.108	3.514	0.784
8	21.12	3.43	3.290	3.490	2.89	2.74	3.696	3.236	3.696	0.806
9	20.74	3.81	3.550	3.680	3.04	2.929	3.905	3.392	3.905	0.865
10	20.38	4.17	3.650	3.730	3.19	3.068	4.035	3.498	4.035	0.845
11	19.92	4.63	4.140	3.990	3.38	3.231	4.248	3.649	4.248	0.868
12	19.59	4.96	4.090	4.160	3.51	3.352	4.368	3.749	4.368	0.858
13	19.18	5.37	4.250	4.120	3.68	3.488	4.509	3.877	4.509	0.829
14	18.78	5.77	4.330	4.350	3.84	3.621	4.642	3.994	4.642	0.802
15	18.21	6.34	4.600	4.550	4.07	3.79	4.81	4.144	4.81	0.74
16	17.81	6.74	5.096	4.880	4.3	3.94	4.961	4.276	4.961	0.661
17	17.44	7.11	5.010	4.760	4.51	4.078	5.096	4.399	5.096	0.586
18	17.12	7.43	5.320	5.240	4.69	4.174	5.21	4.489	5.21	0.52
19	16.61	7.94	5.370	5.470	4.98	4.329	5.385	4.638	5.385	0.405
20	16.35	8.2	5.639	5.810	5.13	4.412	5.487	4.719	5.487	0.357
21	15.8	8.75	5.970	5.740	5.44	4.544	5.635	4.848	5.635	0.195
22	15.4	9.15	6.210	6.090	5.67	4.687	5.789	4.988	5.789	0.119
23	15.04	9.51	6.230	6.510	5.87	4.801	5.894	5.092	5.894	0.024
24	14.66	9.89	6.692	6.560	6.02	4.951	6.057	5.236	6.057	0.037
25	14.16	10.39	7.580	6.610	6.21	5.141	6.233	5.412	6.233	0.023

26	13.81	10.74	7.059	6.970	6.35	5.265	6.39	5.541	6.39	0.04
27	13.36	11.19	8.489	8.485	6.52	5.344	6.472	5.62	6.472	-0.048

Note: 100 mm has been added to computed 100 year levels at Sections 5 and 6 to allow for efflux upstream of Thornton Bridge.

4.5.3 Current Capacity

The predicted flood levels for both 20 and 30 year floods combined with a 20 year storm surge are presented in Table 10 and Figure 11. From these it can be deduced that:

(i) Apart from isolated low spots the scheme will convey the 20 year flood with 300 mm freeboard;

(ii) In a 30 year flood freeboard on the left bank reduces to 24 and 31 mm respectively at cross-section numbers 6 and 8. Some overtopping of stopbanks (due to waves, super-elevation and other variables) is likely in this event – although it is unlikely to be major.

Table 10 Rangitaiki River - 20 and 30 Year Design Flood Levels (Moturiki Datum)

Section	Mike 11 Distance (km)	Distance (km)	Ground Level LB (m)	Ground Level RB (m)	DWL Dine 87	PLB Calib	20 Year Scenario	20 Year Freeboard	30 Year Scenario	30 Year Freeboard
Sea	25					1.268	1.92		1.92	
1a	24.55	0			2	1.273	1.926		1.928	
1	23.97	0.58	3.048	2.850	2.13	1.345	2.03	0.820	2.06	0.790
2	23.66	0.89	2.720	2.800	2.21	1.497	2.112	0.608	2.161	0.559
3	23.12	1.43	2.880	2.770	2.34	1.749	2.265	0.505	2.345	0.425
4	22.87	1.68	2.900	2.880	2.4	1.931	2.387	0.493	2.487	0.393
5	22.32	2.23	3.070	3.130	2.54	2.196	2.687	0.383	2.82	0.250
6	21.92	2.63	3.040	3.220	2.63	2.4	2.859	0.181	3.016	0.024
7	21.51	3.04	3.220	3.286	2.73	2.579	2.919	0.301	3.095	0.125
8	21.12	3.43	3.290	3.490	2.89	2.74	3.064	0.226	3.259	0.031
9	20.74	3.81	3.550	3.680	3.04	2.929	3.242	0.308	3.451	0.099
10	20.38	4.17	3.650	3.730	3.19	3.068	3.364	0.286	3.58	0.070
11	19.92	4.63	4.140	3.990	3.38	3.231	3.527	0.463	3.764	0.226
12	19.59	4.96	4.090	4.160	3.51	3.352	3.64	0.450	3.881	0.209
13	19.18	5.37	4.250	4.120	3.68	3.488	3.777	0.343	4.028	0.092
14	18.78	5.77	4.330	4.350	3.84	3.621	3.905	0.425	4.161	0.169
15	18.21	6.34	4.600	4.550	4.07	3.79	4.068	0.482	4.327	0.223
16	17.81	6.74	5.096	4.880	4.3	3.94	4.209	0.671	4.479	0.401
17	17.44	7.11	5.010	4.760	4.51	4.078	4.34	0.420	4.594	0.166
18	17.12	7.43	5.320	5.240	4.69	4.174	4.434	0.806	4.691	0.549
19	16.61	7.94	5.370	5.470	4.98	4.329	4.589	0.781	4.851	0.519
20	16.35	8.2	5.639	5.810	5.13	4.412	4.673	0.966	4.944	0.695
21	15.8	8.75	5.970	5.740	5.44	4.544	4.807	0.933	5.084	0.656
22	15.4	9.15	6.210	6.090	5.67	4.687	4.951	1.139	5.231	0.859
23	15.04	9.51	6.230	6.510	5.87	4.801	5.059	1.171	5.341	0.889
24	14.66	9.89	6.692	6.560	6.02	4.951	5.207	1.353	5.481	1.079
25	14.16	10.39	7.580	6.610	6.21	5.141	5.388	1.222	5.657	0.953
26	13.81	10.74	7.059	6.970	6.35	5.265	5.518	1.452	5.803	1.167
27	13.36	11.19	8.489	8.485	6.52	5.344	5.598	2.887	5.948	2.537

Note: 100 mm has been added to computed levels at Sections 5 and 6 to allow for efflux upstream of Thornton Bridge.

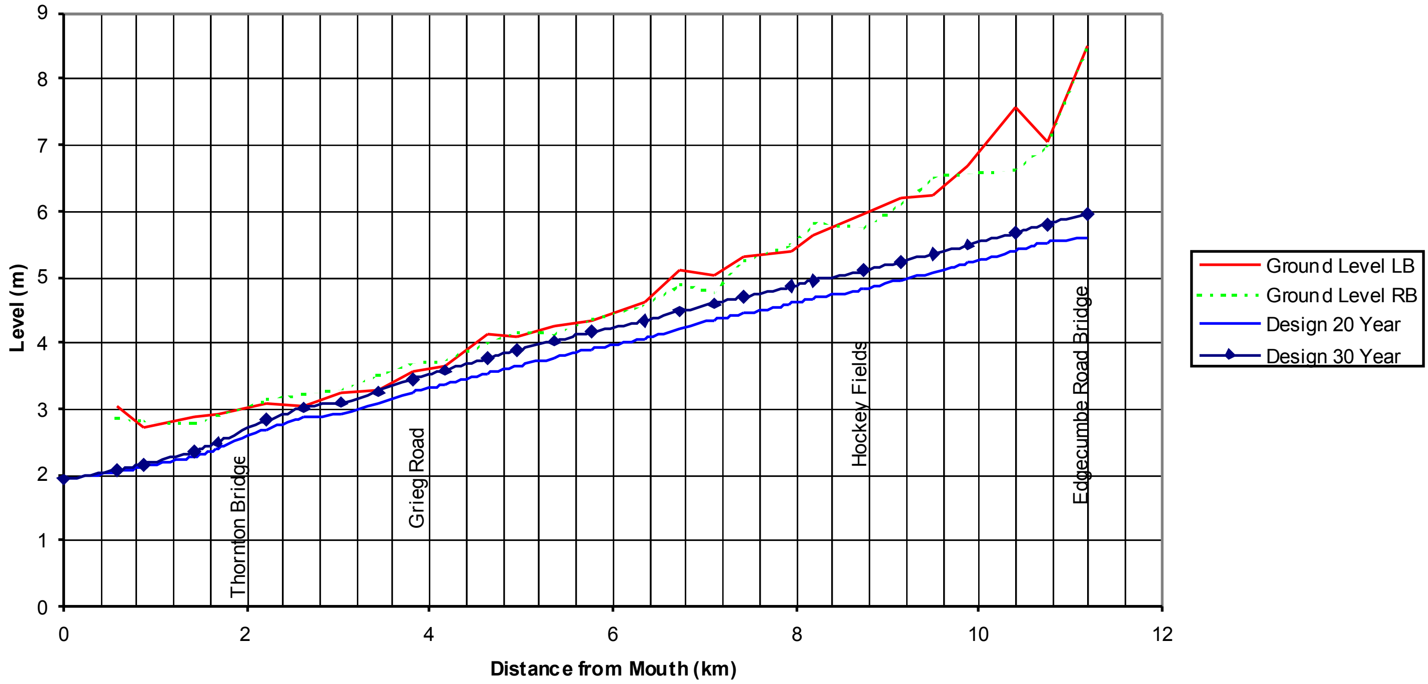

Figure 11 Rangitaiki River: Mouth to Edgecumbe – 20 and 30 Year Flood Levels (Moturiki Datum)

Chapter 5: Conclusion

The concerns of staff and local residents following observations during the July 1998 flood, that the stopbanks in lower reaches of the Rangitaiki River, that is between Edgecumbe and Thornton may not be up to scheme standard, have been confirmed.

It is apparent that remedial works will need to be undertaken. Stopbanks will need to be raised by the order of up to 700 mm over a reach of about 6.6 km in order to maintain the scheme standard as set out in the Rangitaiki-Tarawera River Asset Management Plan, the "contract" between Environment B·O·P as the river management authority and its scheme community.

This report not only outlines the results of the latest hydrologic and hydraulic modelling making use of modern high speed computers and software developed to exploit their availability, but also attempts to address the reasons why earlier assessments of design flow levels were different from what has resulted from this latest analysis.

It is crucial to the understanding of this aspect of the analysis to acknowledge that when the initial design flood levels were calculated back in the 1960's, the hydraulic part of the modelling was undertaken with the use of slide rules and small calculators. A manual iterative process was used which was extremely time consuming without the use of modern computers. The review that was undertaken under some urgency following the 1987 earthquake did have available to it a rather crude hydraulic modelling software package known as "Rivers" but it had many shortcomings. In any case that modelling did not have the advantage of the wealth of data provided by the 1998 flood, the first significant flood in the current flood series which was measured both in terms of flow and levels since the scheme was designed.

One of the most relevant findings of this latest study relates to the behaviour of the riverbed over the studied reach. The findings are that the bed of this river in its lower reaches at least, must behave in a similar manner to what has already been observed in the Tarawera River. At this stage it is the only satisfactory explanation for the apparently elevated roughness factors which have such an impact on the design water level.

Environment B·O·P as the river management authority must adopt these new much more reliable design flood levels and in consultation with its scheme community decide what it wants to do in terms of meeting the current scheme standard or whether it wishes to pursue other alternatives. These alternatives will be the subject of a further report which will also set out estimates of costs to raise stopbanks and/or excavate sections of the channel to "restore" the scheme to the scheme standards.

The word “restore” is used somewhat loosely here, as this report indicates that based on the knowledge that we now have, the scheme was never at that standard, even when it was first constructed.

References

Barnett Consultants Ltd. (1995); Rangitaiki River Flood Hazard Study. Draft Report. June 1995.

Bay of Plenty Catchment Commission (n.d.); Rangitaiki River Major Scheme Report. 4 volumes.

Bay of Plenty Catchment Commission (n.d.); Rangitaiki/Tarawera Rivers Major Scheme.

Bay of Plenty Regional Council (1993); Asset Register – Environment B·O·P file 0360-04

Blackwood, P.L. (1997). Cyclone Fergus and Drena Storm Surge: Report on the Magnitude of Storm Surges Recorded and Implications for Design Maximum Sea Levels. Environment B·O·P Operations Report 97/01. March 1997.

Bobee (1975); The Log Pearson Type 3 Distribution and its Application in Hydrology. Journal of Water Resources Research, Volume 11, No. 5.

Dine, P. D. and Pemberton, D. G. (1987); Post Earthquake Evaluation of the Lower Rangitaiki River Interim Report. July 1987.

Dine, P. D. and Journeaux, P. (1988); Rangitaiki River Scheme Post 1987 Earthquake. Bay of Plenty Catchment Board and Regional Water Board. October 1988.

Environment B·O·P files.

Goring, D., Pearson, C. and Kingsland, S. (1997). Extreme Sea Levels on the Mount Maunganui Shoreline (Moturiki island). NIWA Client Report No. 97/32. July 1997.

Henderson, F. M. (1966); Open Channel Flow.

Jones, J. A. (1987); Report on the Earthquake of 2 March, 1987 and its Effect on Bay of Plenty Catchment Commission and Rangitaiki Drainage Board River and Drainage Schemes, Functions and Responsibilities. Bay of Plenty Catchment Commission and Rangitaiki Drainage Board. 13 March 1987.

Surman, M. (1999); Natural Environment Regional Monitoring Network River and Stream Channel Monitoring Programme 1998/99. Environment B·O·P Environmental Report 99/23. September 1999.

Titchmarsh, R. (1996); Rangitaiki-Tarawera River Scheme Maintenance Review. Environment B·O·P. March 1996.

Wallace, P. (1998); Rangitaiki-Tarawera Rivers Scheme Asset Management Plan. Environment B·O·P Operations Report 98/03. September 1998.