# Review of the Flood Carrying Capacity of the Tarawera River below State Highway 30

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# Chapter 1: Introduction

The purpose of this report is to advise on the results of analysis of the hydrology and hydraulics of the Tarawera River below SH30. It is appropriate to review river scheme designs at regular intervals to incorporate changes to catchment conditions and the increased database available. The last comprehensive review of the Tarawera River was completed in 1986 (Pemberton 1986).

The scope of this report includes:

- Summary of scheme background
- Review of the hydrology (design flood flows)
- Review of the hydraulics (design flood levels)
- Results of the calibration, verification and the Q100 design events
- Attempt at calibration of an historical (1962) flood

# Chapter 2: Background

Detailed information on the background and features of the Tarawera River Scheme is contained in the Rangitaiki-Tarawera Rivers Major Scheme Asset Management Plan (Wallace 1998) and the Proposed Regional Plan for the Tarawera River Catchment.

## 2.1 Catchment and Geology

The Tarawera River catchment has an area of approximately 984 $km^2$ and is shown in Figure 1. The headwaters of the Tarawera River include Lakes Okataina, Okareka, Tikitapu (Blue Lake), Rotokakahi (Green Lake) and Rotomahana. These drain into Lake Tarawera (elevation just under 300 m), from where the Tarawera River begins. Within 6.5 km of leaving the lake the river has fallen to a level of 150 m. From this point to Kawerau the river falls steadily and moderately steeply through deep pumice country. The river valley is well developed and the bed lined with large boulders and aquatic plants. Below Kawerau the grade is gentler and the bed is perched above the general level of the Rangitaiki Plains on a ridge formed from flood deposits. (The lower Rangitaiki River is similarly perched). Below the State Highway 30 bridge the river is stopbanked to provide protection from the 1% AEP flood.

Significant tributaries are the Mangawhio, the Waiwhakapa and the Mangate upstream of Kawerau, the Mangaone and the Ruruanga near Otakiri, and Awakaponga Stream near Matata. The deep pumice of the Tarawera catchment together with the ponding effect of the lake regulates the runoff from heavy storms so the maximum recorded floods are only two to three times the normal flow.

The Tarawera catchment also includes much of the drainage network on the Rangitaiki Plains. This network has been configured from the network of streams and river channels that existed last century. Major canals in the current network, constructed earlier this century, include the Awaiti, Omeheu, Awakaponga and the 109. The old Rangitaiki Channel, the path of the Rangitaiki before it was diverted into the cut at Thornton, also forms part of the Tarawera catchment.

The Tarawera River below SH30 is shown in Figure 2, through this reach the river is relatively straight with few major bends. The river has three tributaries through this reach they are the Mangaone and Waikamihi streams and Awaiti canal.

## 2.2 History of the Scheme

The Rangitaiki-Tarawera Rivers Scheme was designed by the then Bay of Plenty Catchment Board during the 1960's to overcome problems of flooding during periods of high flow in the river. Stopbank construction to convey the 1% AEP flood was proposed for the Tarawera River from the sea to SH30, these banks were completed in 1983.

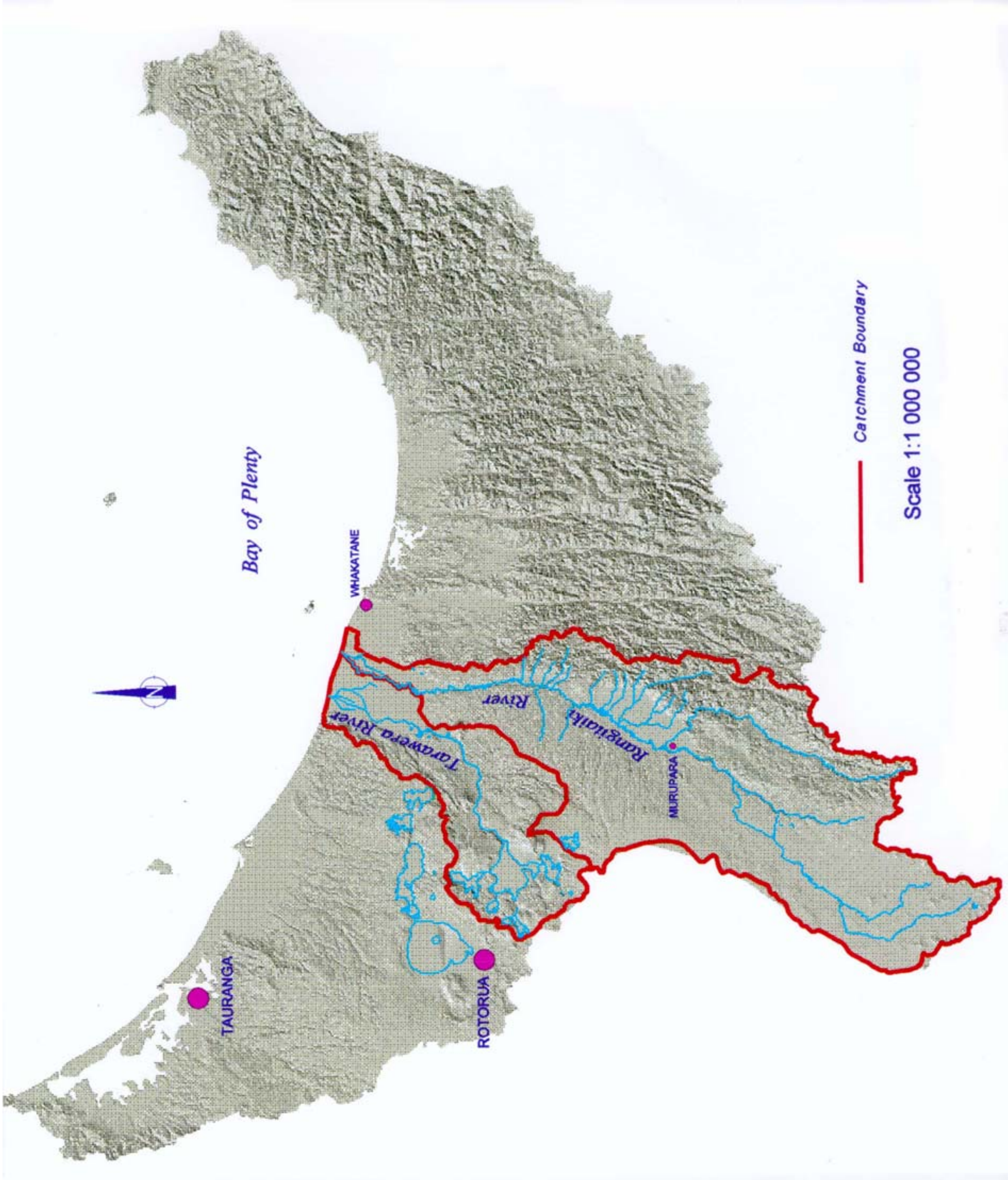


*Figure 1 Location of Rangitaiki and Tarawera River Catchments*

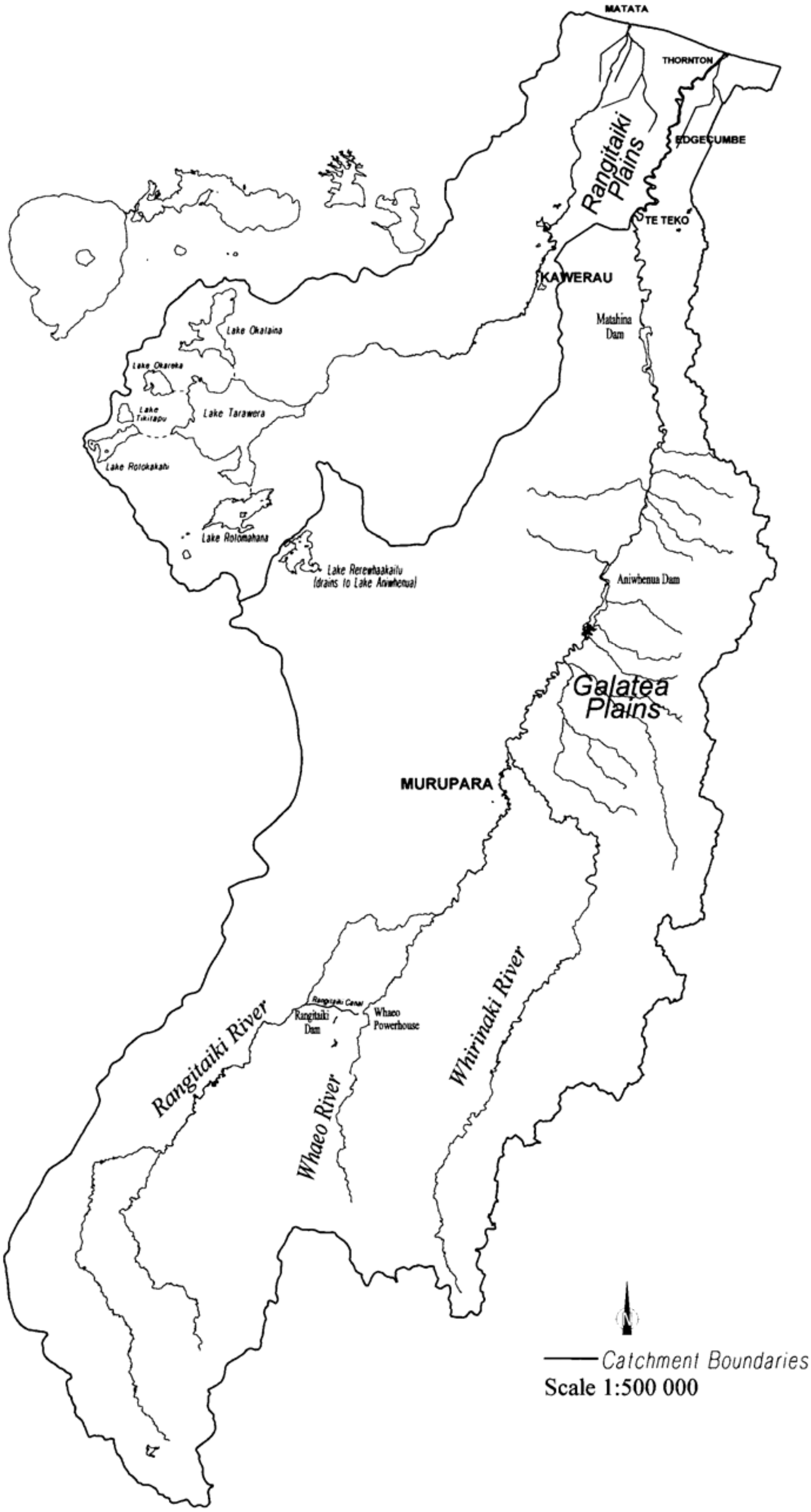


*Figure 2 Rangitaiki Tarawera Catchment Major Features*

## 2.3 Stopbank Capacity and Condition – 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.

*Table 1 Design levels for Rangitaiki-Tarawera River Scheme Stopbanks - source (Rangitaiki-Tarawera AMP 2000)*

| Location | Design Level |
|---|---|
| Tarawera Right Bank | 100 year plus 300mm freeboard |
| Tarawera Left Bank Below Taneatua Branch Railway | 100 year plus 300mm freeboard |
| Tarawera Left Bank Above Taneatua Branch Railway | 100 year plus 150mm freeboard |
| Awakaponga | 10 year plus 300mm freeboard |

The Tarawera River stopbanks were the subject of a review between 1983 and 1986, in response to seepage problems caused by underlying coarse pumice material. Material was dredged from the Tarawera River bed to toe-load the banks and reduce the problem.

An updated estimate of the 100 year flood peak was made in 1986. The revised value of 99.3 $m^3/s$ was lower than the value of 113.3 $m^3/s$ used in the original design in the 1960's. Even after allowing for some settlement after construction, the available freeboard was still greater than design freeboard. (Pemberton, 1986). Subsequent estimates of design flood flows (Blackwood (1998), McLarin and Stringfellow (1997), and Stringfellow and Bowis (1994)) have shown the value is close to 100 $m^3/s$. The hydrology will be discussed further in Chapter 3.

The 1987 Edgecumbe earthquake caused some cracking of the stopbanks, which was repaired with emergency works. However, the earthquake did not result in any subsidence of the stopbanks.

Current stopbank levels are variable, and it may be that in some areas freeboard is locally below design criteria outlined in the Rangitaiki-Tarawera AMP 2000.

# Chapter 3: Hydrology

This section presents the results of a detailed hydrological investigation of the magnitude and frequency of floods at the Awakaponga recorder site on the Tarawera River (site 15302).

## 3.1 Records of Annual Maxima

Continuous flow records since 1955 are available for the Tarawera River at Awakaponga, the annual maxima are presented in Table 2.

*Table 2 Tarawera River at Awakaponga Annual Extremes - source: (Environmental Data Summaries 1994/14 & 2001/01)*

| Year | Max Annual Flow (cumecs) | Rank | Return Period (Years) | Y variate |
|---|---|---|---|---|
| 1962 | 92.4 | 1 | 80.57 | 4.40 |
| 1971 | 83.5 | 2 | 28.92 | 3.37 |
| 1970 | 79.6 | 3 | 17.63 | 2.86 |
| 1972 | 77.1 | 4 | 12.67 | 2.52 |
| 1995 | 74.2 | 5 | 9.89 | 2.26 |
| 1998 | 68.9 | 6 | 8.12 | 2.05 |
| 1956 | 63.6 | 7 | 6.88 | 1.87 |
| 1979 | 63.3 | 8 | 5.97 | 1.72 |
| 1966 | 62.7 | 9 | 5.27 | 1.58 |
| 1969 | 61.2 | 10 | 4.72 | 1.46 |
| 1967 | 61.2 | 11 | 4.27 | 1.35 |
| 1965 | 60.8 | 12 | 3.90 | 1.24 |
| 1999 | 60.0 | 13 | 3.59 | 1.15 |
| 1963 | 59.0 | 14 | 3.33 | 1.06 |
| 1960 | 57.2 | 15 | 3.10 | 0.97 |
| 1959 | 56.8 | 16 | 2.90 | 0.89 |
| 1958 | 56.6 | 17 | 2.72 | 0.81 |
| 1983 | 56.3 | 18 | 2.57 | 0.74 |
| 1975 | 56.3 | 19 | 2.43 | 0.66 |
| 1961 | 54.2 | 20 | 2.31 | 0.59 |
| 2000 | 52.7 | 21 | 2.19 | 0.53 |
| 1996 | 52.2 | 22 | 2.09 | 0.46 |
| 1988 | 51.7 | 23 | 2.00 | 0.40 |
| 1974 | 50.2 | 24 | 1.92 | 0.34 |
| 1968 | 49.6 | 25 | 1.84 | 0.27 |
| 1964 | 49.5 | 26 | 1.77 | 0.21 |
| 1973 | 49.0 | 27 | 1.70 | 0.15 |
| 1985 | 49.0 | 28 | 1.64 | 0.09 |
| 1994 | 48.4 | 29 | 1.58 | 0.03 |
| 1990 | 48.1 | 30 | 1.53 | -0.02 |
| 1986 | 47.4 | 31 | 1.48 | -0.08 |

| Year | Max Annual Flow (cumecs) | Rank | Return Period (Years) | Y variate |
|---|---|---|---|---|
| 1989 | 47.3 | 32 | 1.43 | -0.14 |
| 1978 | 45.5 | 33 | 1.39 | -0.20 |
| 1955 | 45.2 | 34 | 1.34 | -0.26 |
| 1976 | 45.0 | 35 | 1.31 | -0.32 |
| 1981 | 44.2 | 36 | 1.27 | -0.39 |
| 1957 | 44.2 | 37 | 1.23 | -0.45 |
| 1984 | 43.5 | 38 | 1.20 | -0.52 |
| 1991 | 43.0 | 39 | 1.17 | -0.59 |
| 1997 | 43.0 | 40 | 1.14 | -0.67 |
| 1982 | 40.2 | 41 | 1.11 | -0.75 |
| 1977 | 40.0 | 42 | 1.09 | -0.84 |
| 1992 | 39.7 | 43 | 1.06 | -0.94 |
| 1980 | 39.1 | 44 | 1.04 | -1.06 |
| 1993 | 37.2 | 45 | 1.01 | -1.22 |
| 1987 | 34.2 | 46 | 0.99 | -1.48 |

## 3.2 Flood Analysis Methodology

At site flood frequency analysis was applied to the continuous series of Annual Maxima. The results are shown in Figure 3.

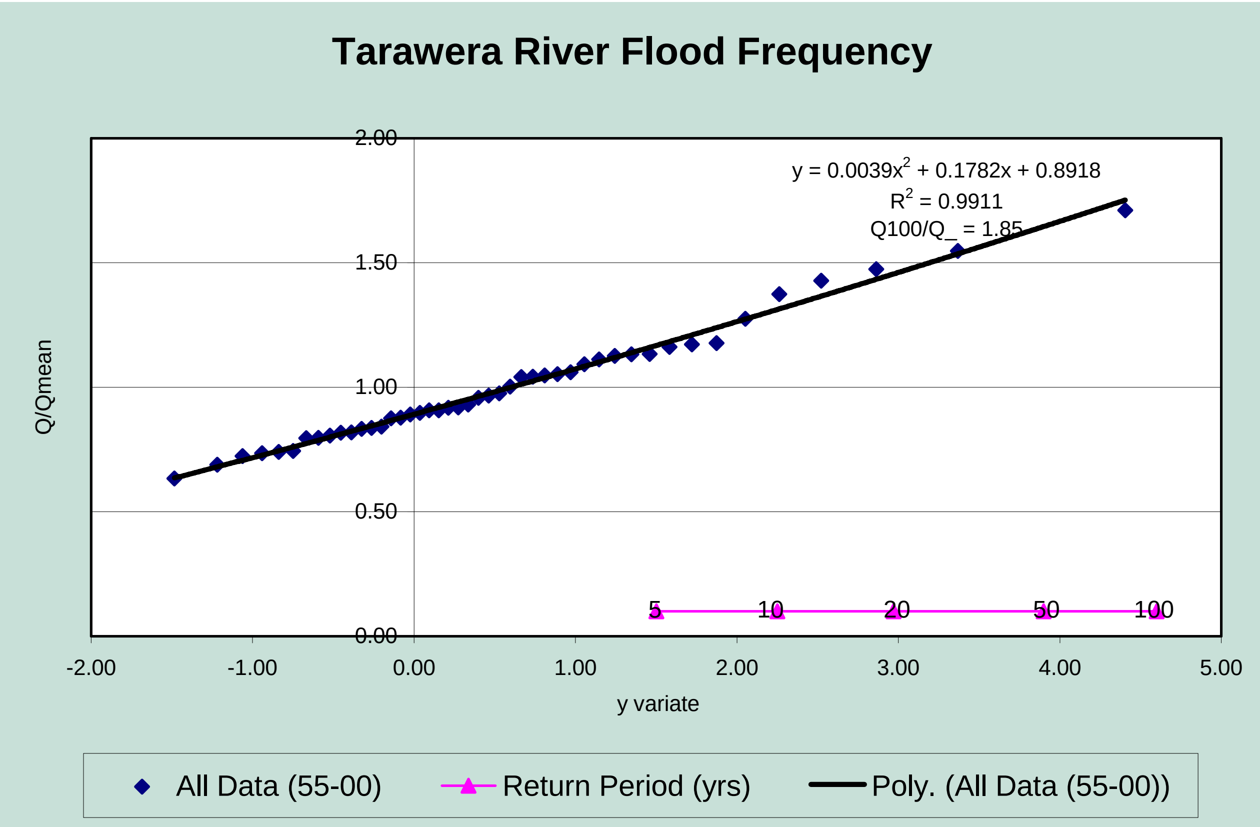


*Figure 3 Tarawera River Flood Frequency*

## 3.3 Design Flood Hydrographs and Tides

Figure 4 below shows the three hydrographs used in this study; the 16-21 February 2001 event was used to calibrate the model. Calibration is the process by which a computer models results are compared to a known event to test the models ability to reproduce actual flood events. The February event had a peak discharge of 40.25 cumecs giving it a return period of 1.2 years. Ideally after a model has been calibrated it should be verified using other recorded events. This was possible for this model using another high flow event in the same year. The event was from 11-16 April 2001, it had a peak discharge of 58.42 cumecs and a return period of 4 years. The verification results are presented in section 5 below. The $Q_{100}$ design hydrograph is a directly scaled version of the event between 16-21 February 2001 recorded at Awakaponga.

Figure 5 shows the tides that were used for the modelling. Unfortunately due to the loss of the Thornton recorder (site 15401) at 11.15 a.m. on 19 February the February tide is a combination of the values recorded at Thornton and at Whakatane Town Wharf (site 15509). The mean tidal level for the February event was 0.365 m. The April tide was totally recorded at Thornton the mean tidal level for this event was .156 m. Two other tides combining a spring tide with $Q_{20}$ and $Q_{100}$ storm surge levels were also used.

| Return Period (years) | Discharge (cumecs) | Approximate Standard Error (cumecs) | Y Variate |
|---|---|---|---|
| 5 | 61.0 | 8 | 1.5 |
| 10 | 70.15 | 9 | 2.25 |
| 20 | 78.00 | 10 | 2.97 |
| 50 | 90.20 | 11 | 3.90 |
| 100 | 100.22 | 13 | 4.60 |

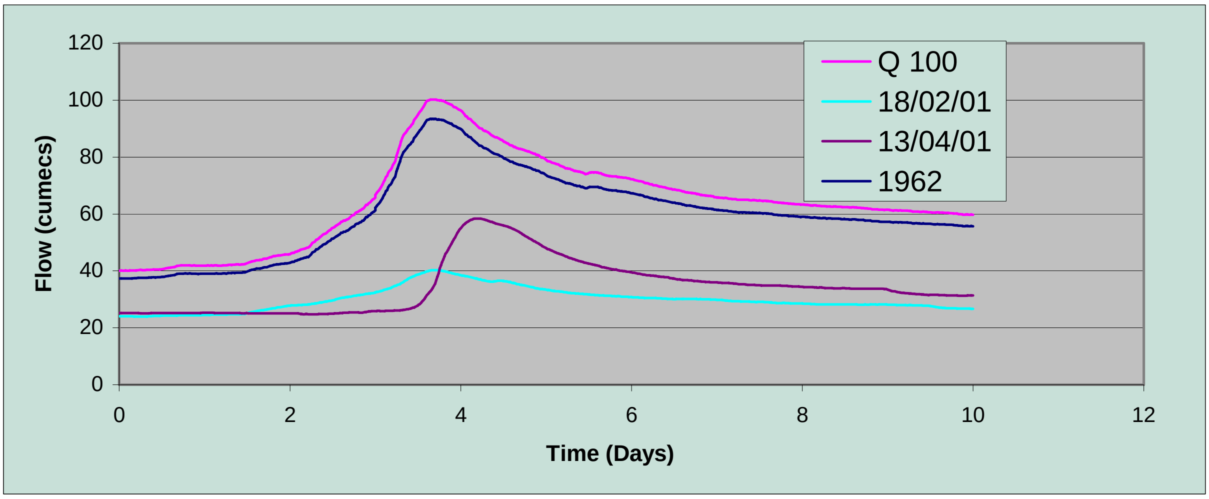


*Figure 4 Hydrographs Used in the Tarawera River Capacity Model*

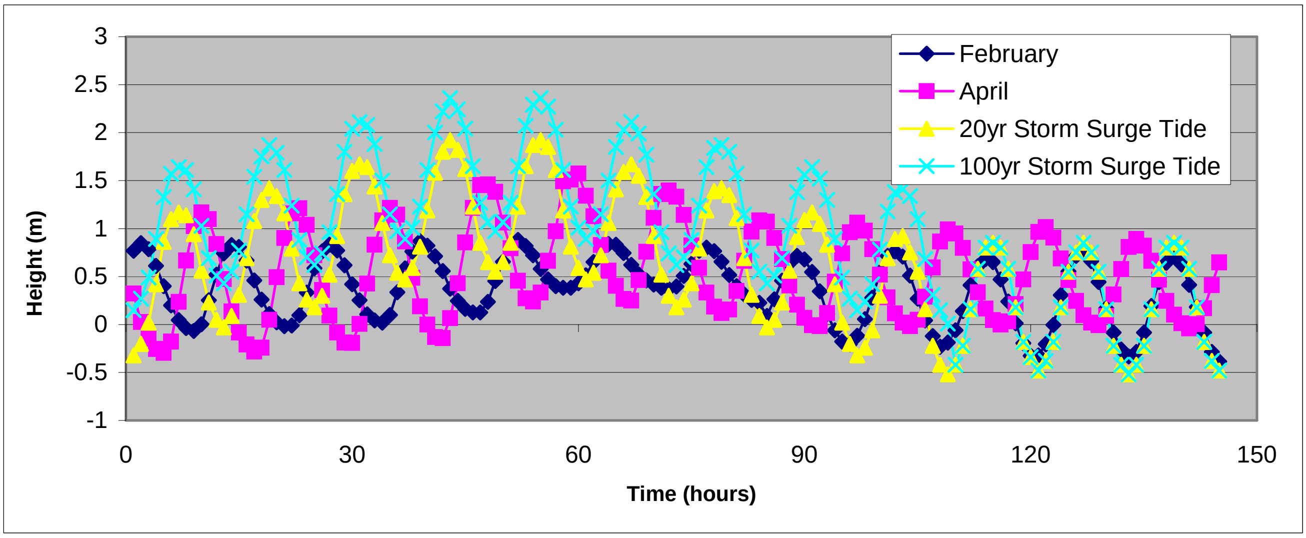


*Figure 5 Tides Used in the Tarawera River Capacity Model*

# Chapter 4: Hydraulic Analysis

## 4.1 Computer Software

The Mike 11 (Version 2002) hydraulic modelling software package was used to simulate different flood level scenarios on the lower Tarawera River. Mike 11 uses the 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.

## 4.2 Model Configuration

The files used for the modelling of each of the scenarios discussed in this report are below in Table 4. More detailed discussion of the content of these files follows.

*Table 4 Files Used for Modelling Each Scenario*

| Scenario | Sim | Nwk | Xns | bnd | Hd | Res |
|---|---|---|---|---|---|---|
| Calibration | Tarawera2000 | Tarawera | 2000_1962 | Feb | 2000Calibration | Feb |
| Validation | Tarawera2000 | Tarawera | 2000_1962 | Apr | 2000Calibration | Apr |
| $Q_{100}$ with 20 yr tide | Tarawera2000 | Tarawera | 2000_1962 | $Q_{100}$ | 1962_calib | $Q_{100}$ |
| $Q_{20}$ with 100 yr tide | Tarawera2000 | Tarawera | 2000_1962 | $T_{20}$-$Q_{100}$ | 1962_calib | $Q_{20}$_$T_{100}$ |

### 4.2.1 Model Set Up

The model consists of 19 cross sections on the Tarawera River (surveyed in 2000) and sections taken at the bridges in December 2002.

The model boundary conditions are:

- Flow at Awakaponga recorder at cross section 7 (river distance 6.3 km).
- Recorded tide at Thornton for the calibration and validation events.
- Tide level at Thornton with 20 year 100 year storm surge added for $Q_{100}$ scenarios.
- $Q_{20}$ and $Q_{100}$ inflows have also been assessed for Mangaone Stream, Waikamihi Stream, Awaiti Canal.

### 4.2.2 Resistance Values

The resistance values have been adjusted 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 sections 17, 10, 5) and a super elevation of around 100 mm should be added to the final design levels. Similarly they ignore bridge effects. In general the resistance value used was 0.03, however there were some exceptions and these are given in Table 5 below. The text "Open Channel Flow" (Henderson 1966) gives 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 |

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).

*Table 5 Roughness coefficients used for the calibration of modelled water levels to those recorded during the February 2001 event.*

| Cross Section | River Distance (km) | Model Chainage | Mannings n |
|---|---|---|---|
| 17 | 17.75 | 100000 | .028 |
| 15 | 15.45 | 102210 | .028 |
| 14 | 14.40 | 103370 | .032 |
| 10 | 10.10 | 107650 | .036 |
| 9 | 8.70 | 108910 | .033 |
| 7 | 6.30 | 111370 | .024 |
| 3 | 2.10 | 115510 | .022 |
| 2A | 1.60 | 116200 | .020 |

### 4.2.3 Bridges

Road bridges cross the lower Tarawera River in two places; At Awakaponga 50 metres downstream of cross section 7 and near the mouth between sections 2 and 1B. At Awakaponga there is also a TranzRail bridge over the river. Each of these has been modelled as a combination of a culvert and weir.

# Chapter 5: Results

## 5.1 Calibration

### 5.1.1 Calibration Flood Levels

Figure 6 and Table 6 show the comparison between peak water levels recorded on the 18 February 2001 and the peak depths calculated by the model. It can be seen that for all sections above the extent of the tidal effects at the river mouth there is less than a 10 cm difference between the recorded and modelled values.

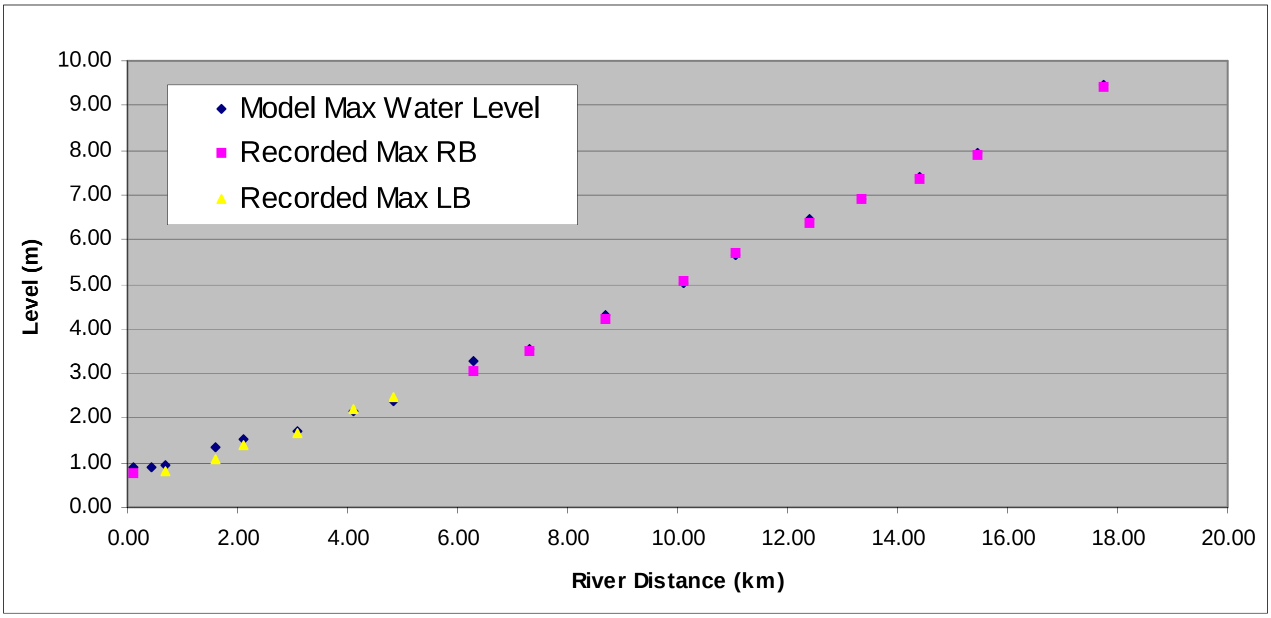


*Figure 6 Graph Showing Calibration of Recorded and Modelled Flood Levels*

*Table 6 Table Showing Difference Between Recorded and Modelled Flood Levels*

| MIKE 11 Chainage | Cross Section | River Distance | Model Max Water Level | Recorded Max LB | Recorded Max RB | Difference (Model - Recorded) |
|---|---|---|---|---|---|---|
| 117380 | 1 | 0.10 | 0.89 | | 0.75 | 0.140 |
| 117180 | 1B | 0.45 | 0.92 | | | |
| 116980 | 2 | 0.70 | 0.96 | 0.83 | | 0.134 |
| 116200 | 2A | 1.60 | 1.34 | 1.08 | | 0.258 |
| 115510 | 3 | 2.10 | 1.51 | 1.37 | | 0.136 |
| 114810 | 4 | 3.10 | 1.71 | 1.67 | | 0.037 |
| 113610 | 5 | 4.10 | 2.16 | 2.18 | | -0.023 |
| 112820 | 6 | 4.85 | 2.37 | 2.46 | | -0.092 |
| 111370 | 7 | 6.30 | 3.25 | | 3.06 | 0.191 |
| 110340 | 8 | 7.30 | 3.56 | | 3.52 | 0.039 |
| 108910 | 9 | 8.70 | 4.32 | | 4.23 | 0.088 |
| 107650 | 10 | 10.10 | 5.03 | | 5.09 | -0.055 |
| 106570 | 11 | 11.05 | 5.67 | | 5.70 | -0.027 |
| 105245 | 12 | 12.40 | 6.44 | | 6.35 | 0.086 |
| 104300 | 13 | 13.35 | 6.92 | | 6.91 | 0.011 |
| 103370 | 14 | 14.40 | 7.42 | | 7.34 | 0.085 |
| 102210 | 15 | 15.45 | 7.93 | | 7.90 | 0.037 |
| 100000 | 17 | 17.75 | 9.46 | | 9.43 | 0.034 |

## 5.2 Verification

A second event on 11-16 April 2001 was used to verify the results of the calibration. As with the calibration the event lasted five days with a peak discharge of 58.42 cumecs. The verification event had a four year return period. Figure 7 and Table 7 show the comparison between water levels recorded on 13 April and modelled water levels for the verification event. Differences between modelled water levels and recorded levels may be due to dune formation on the riverbed. This phenomenon is explored in the following section 5.3. Recorded flood levels below cross section 4 are questionable as they are less than the peak tide level, however this may be due to the dynamic storage available.

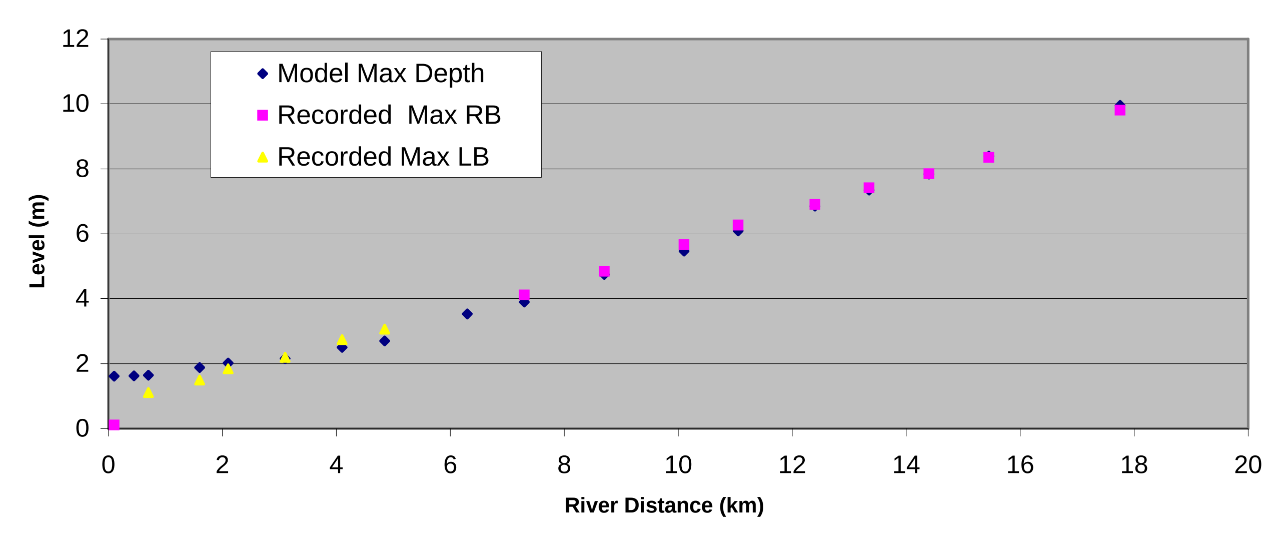


*Figure 7 Graph showing verification of recorded and modelled flood levels*

*Table 7 Table Showing Difference Between Recorded and Modelled Flood Levels for Verification Event*

| MIKE 11 Chainage | Cross Section | River Distance | Model Max Water Level | Recorded max LB | Recorded max RB | Difference (Model - Recorded) |
|---|---|---|---|---|---|---|
| 117380 | 1 | 0.1 | 1.607 | | | |
| 117180 | 1B | 0.45 | 1.621 | | | |
| 116980 | 2 | 0.7 | 1.642 | 1.107 | | 0.535 |
| 116200 | 2A | 1.6 | 1.879 | 1.492 | | 0.387 |
| 115510 | 3 | 2.1 | 2.014 | 1.843 | | 0.171 |
| 114810 | 4 | 3.1 | 2.151 | 2.192 | | -0.041 |
| 113610 | 5 | 4.1 | 2.502 | 2.737 | | -0.235 |
| 112820 | 6 | 4.85 | 2.699 | 3.064 | | -0.365 |
| 111370 | 7 | 6.3 | 3.529 | | | |
| 110340 | 8 | 7.3 | 3.895 | | 4.116 | -0.221 |
| 108910 | 9 | 8.7 | 4.747 | | 4.846 | -0.099 |
| 107650 | 10 | 10.1 | 5.468 | | 5.664 | -0.196 |
| 106570 | 11 | 11.05 | 6.087 | | 6.265 | -0.178 |
| 105245 | 12 | 12.4 | 6.848 | | 6.895 | -0.047 |
| 104300 | 13 | 13.35 | 7.344 | | 7.416 | -0.072 |
| 103370 | 14 | 14.4 | 7.842 | | 7.844 | -0.002 |
| 102210 | 15 | 15.45 | 8.393 | | 8.357 | 0.036 |
| 100000 | 17 | 17.75 | 9.956 | | 9.808 | 0.148 |

## 5.3 Dune Formation and Calibration to 1962 Flood

During the July 1998 floods that occurred in the Eastern Bay of Plenty, higher than expected water levels were observed and recorded in the Rangitaiki River. Modelling after the event found that in order to replicate these water levels high resistance values (between 0.03 to 0.058) were required (Blackwood 2000). The increased roughness is apparently caused by dune formations in the river channel that form under certain flow conditions. Dunes are sand waves that appear as the most common feature in sandy-bed rivers. They are responsible for an important percentage of sediment transport as they move downstream, and they are responsible for the total resistance exerted on the flow.

“Water or air flowing over a bed of loose sediment will form bed forms, regular topographic patterns with internal structure. The formation of bed forms reflects feedback between fluid flow and sediment. The patterns that form depend upon flow velocity, the fluid itself and the sediment supply. In water, ripples (few centimetres wavelength) and dunes (longer wavelengths) form at slow to moderate flow velocities. These migrate and grow as grains roll or saltate up the upstream face. Turbulence built up around the ripple crest preferentially transports grains to the top of the lee face. This causes the lee side of the crest to aggrade until an avalanche returns the slope to the angle of repose, depositing a foreset bed or strata. The build up of lee-side layers creates cross bedding within the sand, silt or gravel layer. These dip in the direction of fluid flow. At higher flow velocities, very fast currents plane off ripples, producing flat beds. Grains are kept in suspension and no bed forms develop

(http://www.es.ucsc.edu/~jsr/EART10/Lectures/HTML/Lecture.09.html).”

Dune formations have been recorded in measurements made in the Tarawera River at Awakaponga by Opus Consultants Ltd. Opus found roughness values ranging from 0.033 to 0.042 for different flow velocities, Figure 8 below shows a plot of the values recorded by Opus and those that have been calibrated in this report (Also calibration to 1962 flood in section 5.4 below).

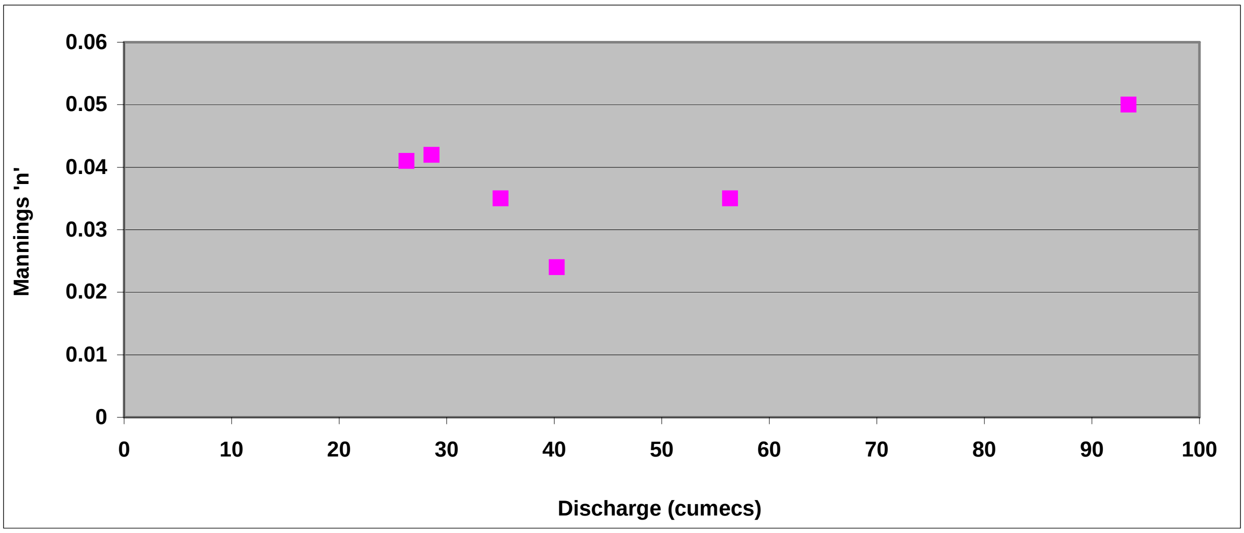


*Figure 8 Comparison of Discharge and Mannings n Values for the Tarawera River at Awakaponga (XS 07, 6.3km)*

In a further attempt to verify the apparently higher roughness during higher flow events a calibration was attempted using water levels from the highest recorded flood event in the river. This event occurred in 1962 and had a peak discharge of 93.4 cumecs. A plot of debris levels recorded at points along the channel was found along with bank heights and channel invert. Unfortunately the 1962 cross sections could not be located so these were derived from a comparison of the 1962 and 2000 invert levels. Appendix A shows a comparison of the 1962 derived cross section with the surveyed 1985 and 2000 sections and shows that the sections derived for 1962 seem reasonable. Figure 9 shows the results of the calibration and the Manning's 'n' values used are given in Table 8.

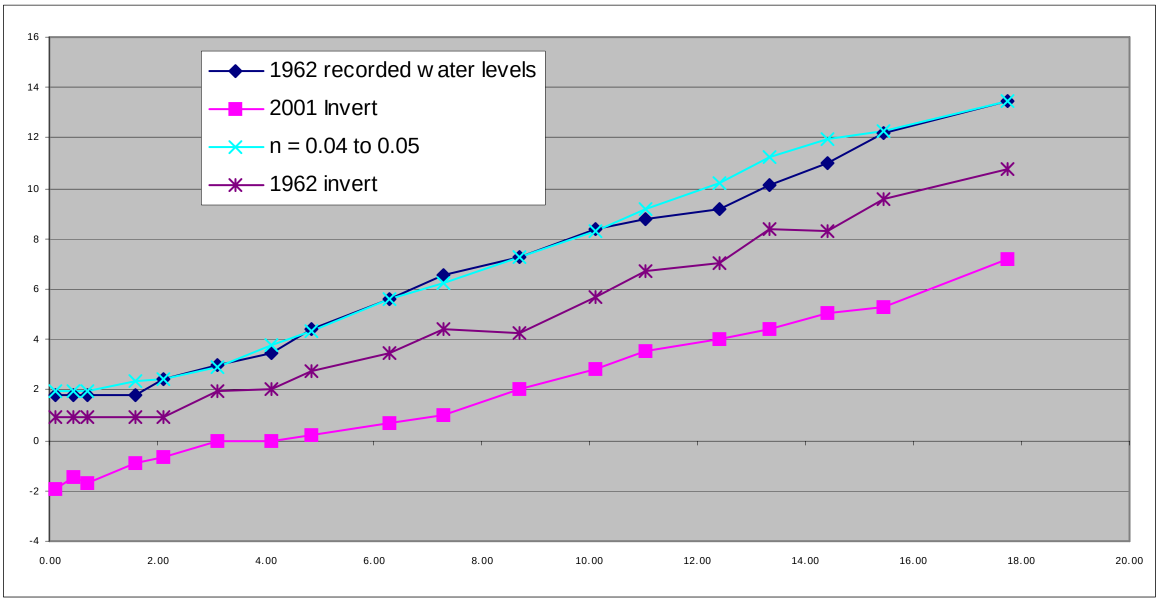


*Figure 9 Plot Showing Calibration to the 1962 Recorded Flood Levels*

*Table 8 Manning's 'n' Values for the 1962 Calibration*

| MIKE 11 Chainage | Cross Section | River Distance | Mannings 'n' |
|---|---|---|---|
| 117380 | 1 | 0.1 | 0.04 |
| 117180 | 1B | 0.45 | 0.04 |
| 116980 | 2 | 0.7 | 0.04 |
| 116200 | 2A | 1.6 | 0.04 |
| 115510 | 3 | 2.1 | 0.04 |
| 114810 | 4 | 3.1 | 0.04 |
| 113610 | 5 | 4.1 | 0.04 |
| 112820 | 6 | 4.85 | 0.05 |
| 111370 | 7 | 6.3 | 0.05 |
| 110340 | 8 | 7.3 | 0.05 |
| 108910 | 9 | 8.7 | 0.0425 |
| 107650 | 10 | 10.1 | 0.0425 |
| 106570 | 11 | 11.05 | 0.035 |
| 105245 | 12 | 12.4 | 0.035 |
| 104300 | 13 | 13.35 | 0.035 |
| 103370 | 14 | 14.4 | 0.035 |
| 102210 | 15 | 15.45 | 0.04 |
| 100000 | 17 | 17.75 | 0.04 |

It can be seen from Table 8 that in the upper sections of the river in order to calibrate to the recorded water levels some of the roughness values are unrealistically low. Possible reasons for this include, inaccuracies in the derived cross sections, inaccuracies in recording of the water levels in 1962 or the effects of a stopbank breach at cross section 14 during the 1962 event. Any of these or a combination of all three could account for the difference between modelled and recorded water levels. After careful consideration of these factors and a review of the roughness values recorded by Opus from those calibrated in this report and in the work done on the Rangitaiki River it was decided to use the combination of roughness values in Table 9 below.

*Table 9 Roughness Values Used to Simulate Dune Forms in Greater than $Q_{10}$ Flow.*

| MIKE 11 Chainage | Cross Section | River Distance | Manning's 'n' |
|---|---|---|---|
| 117380 | 1 | 0.10 | 0.04 |
| 117180 | 1B | 0.45 | 0.04 |
| 116980 | 2 | 0.70 | 0.04 |
| 116200 | 2A | 1.60 | 0.04 |
| 115510 | 3 | 2.10 | 0.04 |
| 114810 | 4 | 3.10 | 0.04 |
| 113610 | 5 | 4.10 | 0.04 |
| 112820 | 6 | 4.85 | 0.05 |
| 111370 | 7 | 6.30 | 0.05 |
| 110340 | 8 | 7.30 | 0.05 |
| 108910 | 9 | 8.70 | 0.0425 |
| 107650 | 10 | 10.10 | 0.0425 |
| 106570 | 11 | 11.05 | 0.035 |
| 105245 | 12 | 12.40 | 0.035 |
| 104300 | 13 | 13.35 | 0.035 |
| 103370 | 14 | 14.40 | 0.035 |
| 102210 | 15 | 15.45 | 0.04 |
| 100000 | 17 | 17.75 | 0.04 |

## 5.4 Model Scenarios

Water levels in the 100 year design flood event are influenced by both design discharge and downstream tide level. It is unlikely that a 100 year storm surge will combine with a 100 year flood flow because the unique conditions producing these separate events will rarely occur simultaneously. The exact design combinations 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 is a reasonable combination and will determine the 100 year levels in the vicinity of the river 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 increase surge in the eastern Bay of Plenty (due to the interaction of the concave shoreline with prevailing cyclone wind directions).

The results of the two model scenarios are shown below, it was found in the modelling of the Rangitaiki River that dunes formed in flows with a return period of greater than 15 years, therefore the 'dune' roughness values (Table 9 above) have been used for modelling each of the 1% AEP combination events.

Figure 9 shows the comparison of combined peak water levels with bank heights for the Tarawera River below SH30 using the roughness values calibrated above. Table 8 shows the comparison between maximum flow and maximum bank height and gives a calculation of the freeboard at each cross section. From Table 8 we can see that in several places the freeboard drops below the 300 mm required in the Rangitaiki/Tarawera Asset management Plan (areas below 300 mm are in bold type).

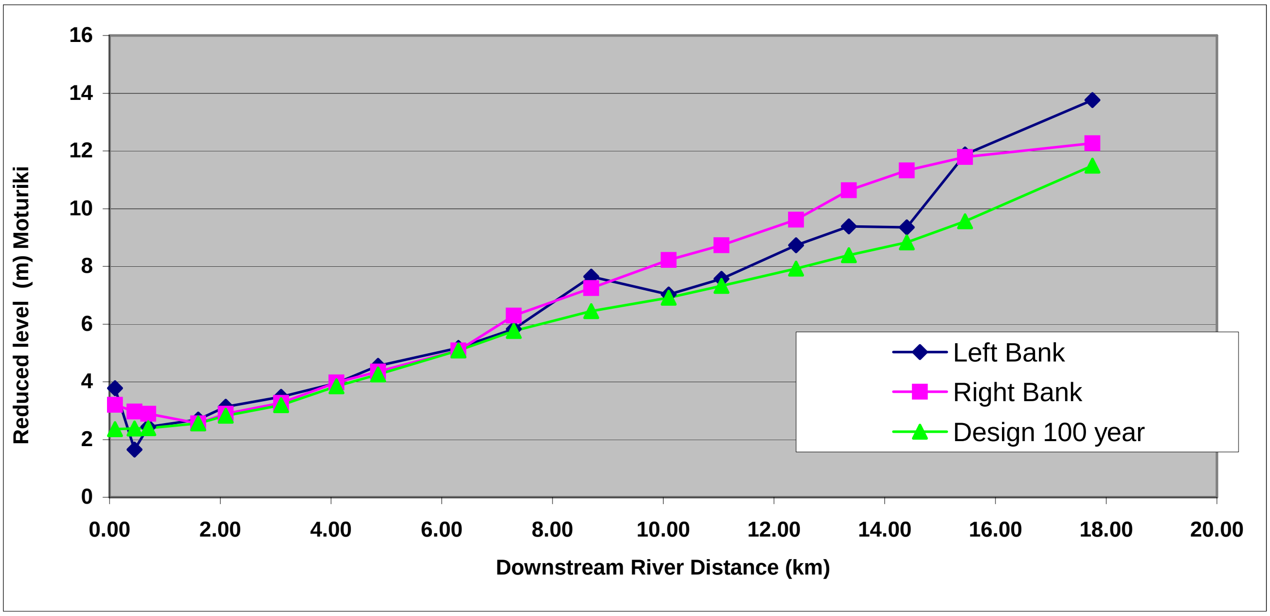


*Figure 10 Tarawera River Stopbanks and Combined 1% AEP Water Levels*

*Table 10 Tarawera River stopbanks heights and combined 1% AEP water levels*

| MIKE 11 Chainage | Cross Section | River Distance | Left Bank | Right Bank | Max WL 1%AEP 20yr tide | Max WL 5% AEP 100yr tide | Combined 100yr scenarios | Left Freeboard | Right Freeboard |
|---|---|---|---|---|---|---|---|---|---|
| 117380 | 1 | 0.10 | 3.78 | 3.198 | 1.934 | 2.36 | 2.36 | 1.42 | 0.838 |
| 117180 | 1B | 0.45 | 1.65 | 2.972 | 2.003 | 2.377 | 2.377 | **-0.727** | 0.595 |
| 116980 | 2A | 0.70 | 2.436 | 2.897 | 2.074 | 2.388 | 2.388 | **0.048** | 0.509 |
| 116200 | 2 | 1.60 | 2.69 | 2.558 | 2.559 | 2.489 | 2.559 | **0.131** | **-0.001** |
| 115510 | 3 | 2.10 | 3.131 | 2.889 | 2.828 | 2.555 | 2.828 | 0.303 | **0.061** |
| 114810 | 4 | 3.10 | 3.477 | 3.265 | 3.188 | 2.689 | 3.188 | **0.289** | **0.077** |
| 113610 | 5 | 4.10 | 3.955 | 3.983 | 3.849 | 3.026 | 3.849 | **0.106** | **0.134** |
| 112820 | 6 | 4.85 | 4.552 | 4.351 | 4.25 | 3.213 | 4.25 | 0.302 | **0.101** |
| 111370 | 7 | 6.30 | 5.173 | 5.085 | 5.091 | 3.605 | 5.091 | **0.082** | **-0.006** |
| 110340 | 8 | 7.30 | 5.826 | 6.297 | 5.758 | 4.095 | 5.758 | **0.068** | 0.539 |
| 108910 | 9 | 8.70 | 7.643 | 7.246 | 6.449 | 5.024 | 6.449 | 1.194 | 0.797 |
| 107650 | 10 | 10.10 | 7.029 | 8.225 | 6.917 | 5.82 | 6.917 | **0.112** | 1.308 |
| 106570 | 11 | 11.05 | 7.569 | 8.729 | 7.326 | 6.462 | 7.326 | 0.243 | 1.403 |
| 104300 | 12 | 12.40 | 8.732 | 9.616 | 7.922 | 7.249 | 7.922 | **0.81** | 1.694 |
| 105245 | 13 | 13.35 | 9.386 | 10.637 | 8.387 | 7.763 | 8.387 | 0.999 | 2.25 |
| 103370 | 14 | 14.40 | 9.351 | 11.324 | 8.83 | 8.234 | 8.83 | 0.521 | 2.494 |
| 102210 | 15 | 15.45 | 11.878 | 11.79 | 9.567 | 8.797 | 9.567 | 2.311 | 2.223 |
| 100000 | 17 | 17.75 | 13.76 | 12.27 | 11.492 | 10.408 | 11.492 | 2.268 | 0.778 |

Q50 With 20 Year Tide

*Table 11 Tarawera River stopbank heights and combined 2% AEP water levels*

| MIKE 11 Chainage | Cross Section | River Distance | Left Bank | Right Bank | Max Water Level | Left Freeboard | Right Freeboard |
|---|---|---|---|---|---|---|---|
| 117380 | 1 | 0.10 | 3.78 | 3.198 | 1.931 | 1.849 | 1.267 |
| 117180 | 1B | 0.45 | 1.65 | 2.972 | 1.987 | -0.337 | 0.985 |
| 116980 | 2A | 0.70 | 2.436 | 2.897 | 2.045 | 0.391 | 0.852 |
| 116200 | 2 | 1.60 | 2.69 | 2.558 | 2.47 | **0.22** | 0.088 |
| 115510 | 3 | 2.10 | 3.131 | 2.889 | 2.719 | 0.412 | **0.17** |
| 114810 | 4 | 3.10 | 3.477 | 3.265 | 3.066 | 0.411 | **0.199** |
| 113610 | 5 | 4.10 | 3.955 | 3.983 | 3.706 | **0.249** | **0.277** |
| 112820 | 6 | 4.85 | 4.552 | 4.351 | 4.1 | 0.452 | **0.251** |
| 111370 | 7 | 6.30 | 5.173 | 5.085 | 4.895 | **0.278** | **0.19** |
| 110340 | 8 | 7.30 | 5.826 | 6.297 | 5.552 | **0.274** | 0.745 |
| 108910 | 9 | 8.70 | 7.643 | 7.246 | 6.25 | 1.393 | 0.996 |
| 107650 | 10 | 10.10 | 7.029 | 8.225 | 6.715 | 0.314 | 1.51 |
| 104300 | 11 | 11.05 | 7.569 | 8.729 | 7.116 | 0.453 | 1.613 |
| 106570 | 12 | 12.40 | 8.732 | 9.616 | 7.701 | 1.031 | 1.915 |
| 105245 | 13 | 13.35 | 9.386 | 10.637 | 8.166 | 1.22 | 2.471 |
| 103370 | 14 | 14.40 | 9.351 | 11.324 | 8.618 | 0.733 | 2.706 |
| 102210 | 15 | 15.45 | 11.878 | 11.79 | 9.363 | 2.515 | 2.427 |
| 100000 | 17 | 17.75 | 13.76 | 12.27 | 11.306 | 2.454 | 0.964 |

# Conclusions

This report outlines the most recent hydrologic and hydraulic modelling of the Tarawera River. It makes use of data recorded from past events and hydraulic studies and recent relevant findings from a study of the Rangitaiki River. The conclusion is that in its lower reaches the bed of this river is subject to dune formation during high flows. These dune formations cause high water levels that in some cases remove any existing freeboard from the stopbank design. It is apparent that remedial works will need to be undertaken to restore the scheme to the $Q_{100}$ design standard.

# Recommendations

That tabulated water levels be adopted as design water levels for the given cross sections and interpolated as appropriate.

That where the freeboard on the stopbanks of the Tarawera River is below 300 mm that the stopbanks be topped up to their design level.

# References

Blackwood P L, 2000, Review of the flood carrying capacity of the Rangitaiki River below Edgecumbe Operations report 2000/09, October 2000.

Pemberton G, 1986, Lower Tarawera River Stopbank Seepage, Whakatane BOPCC, April 1986.

Wallace P, 1998, Rangitaiki-Tarawera Rivers Major Scheme Asset Management Plan, Environment BOP Operations Report 2000/09, October 2000.