Waingaehe Stream Hydraulic Capacity Review

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Chapter 1: Introduction

The Waingaehe Stream Catchment, located in the Rotorua District, drains just over 11.2 km^2 of mainly agricultural land (forestry and livestock) and flows into Lake Rotorua approximately 5km North East of downtown Rotorua. The catchment is composed primarily of Rotomahana coarse sandy loam which, is characteristic of most Rotorua District Catchments and tends to be moderately to very well drained (See Figure 1 below). Rainfall ranges on average between 1200mm to 1300mm per annum (NIWA, 2006 median annual mean daily rainfall), falling consistently throughout the year with marginally higher rainfall experienced in autumn and winter.

Upper Kaituna stream works were first undertaken on the Waingaehe Stream in 1974. Motivation for the works was provided by extensive flooding, experienced on the 13 August 1968. The flooding encompassed a large area of residential property located adjacent to the original stream channel. The works resulted in the construction of a floodway on the Waingaehe Stream primarily aimed at diverting flood flows away from residential property (See figure 1)(Wallace, 2003).

This report undertakes a hydraulic capacity review of the Waingaehe Stream. The review aims to assess the level of flood protection offered by the 1.3km of floodway constructed in 1975, located between State Highway 30 and Lake Rotorua. The original design called for protection from the 100 year Average Recurrence Interval (ARI) or 1% Annual Exceedence Probability (AEP) event, set at $23m^3/s$ in 1999 but re-assessed in this report. Management of the Waingaehe floodway assets have been set in accordance with the Kaituna Asset Management Plan of October 2003. Environment Bay of Plenty undertakes hydrologic capacity reviews on a 15 year basis as part of the effective monitoring component set in the current Kaituna Asset Management Plan.

Chapter 2: Model Layout

2.1 Model software

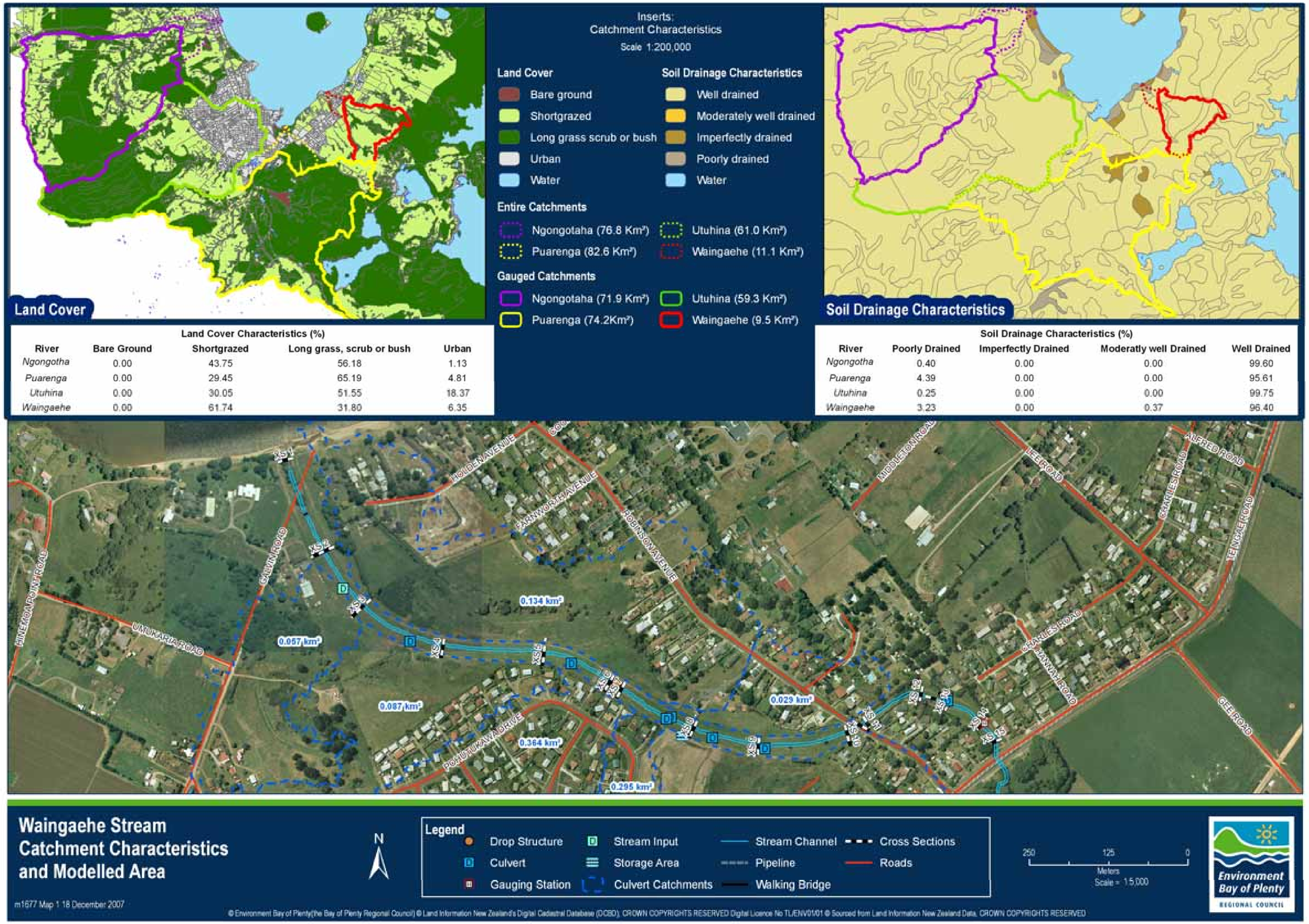
Mike 11 hydraulic modelling software (MIKE 11 Release 2007) has been used to set up a status quo model of the Waingaehe Stream flood condition from state highway 30 downstream to Lake Rotorua.

A detailed description of the Mike11 software can be found in the Reference Manual (DHI, 2007) and the User Guide (DHI, 2007).

2.2 Floodway's

A flood control scheme, including stopbank construction, was implemented in 1974 from the junction with state highway 30 (Te Ngae Road) down to Lake Rotorua (See Figure 1). The stream channel was widened from state highway 30 to a point 120m downstream. At that point a floodway channel to the lake has been built. That channel diverts water from the streams original course westward, under Robinson Avenue and to the lake some 400m to the South West of the original mouth. A number of drop structures, a weir and several culverts were constructed at that time. The original stream channel is maintained by an intake structure and culvert. This structure can accommodate the bulk of total normal flows set in the region of 280l/s but allowing a minimum of between 28l/s and 56l/s. Some rock rip rap and concrete rubble has been used to stabilise constructed banks however these features were no longer visible upon field inspection (Wallace, 2003).

A stream survey completed in 2007 (cross sections, long sections and structures were captured) indicated that the present floodway is thickly grassed, and in some places grazed. In stream or bed conditions consist of dense hydrophytes with the sandy gravel bed surface exposed in isolated locations. Stream banks exhibit slumping in some areas. The Waingaehe floodway is largely free of willows except for two isolated individuals growing in the stream channel and a scattering of trees at the confluence with Lake Rotorua. Survey cross sections were undertaken on 24 January 2007. Cross Sections have been marked on Figure 1 with cross section 1 located at Lake Rotorua and cross section 15 located just upstream of the gauging station (see Table 1).



Land Cover Characteristics (%)

River	Bare Ground	Shortgrazed	Long grass, scrub or bush	Urban
Ngongotha	0.00	43.75	56.18	1.13
Puarenga	0.00	29.45	65.19	4.81
Utuhina	0.00	30.05	51.55	18.37
Waingaehe	0.00	61.74	31.80	6.35

Soil Drainage Characteristics (%)

River	Poorly Drained	Imperfectly Drained	Moderatly well Drained	Well Drained
Ngongotha	0.40	0.00	0.00	99.60
Puarenga	4.39	0.00	0.00	95.61
Utuhina	0.25	0.00	0.00	99.75
Waingaehe	3.23	0.00	0.37	96.40

Figure 1 Waingaehe Stream Catchment Characteristics and Modelled Area. Land cover and soil drainage characteristics are indicative of the entire catchment area, but were reduced to reflect gauged area only for this review.

Table 1 Cross-sections used in the design model

Branch	Cross-Section	Chainage	Comment
Waingaehe	XS15	0	Above gauging station
Waingaehe	XS14	33	Gauging station
Waingaehe	XS13	99	
Waingaehe	XS12	145	
Waingaehe	XS11	244	Bridge upstream
Waingaehe	XS10.5	256	Bridge at Robinson Avenue (modelled as culvert, height reduced by 0.6m)
Waingaehe	XS10	279	Bridge Downstream
Waingaehe	XS9	422	
Waingaehe	XS8	528	
Waingaehe	XS7	659	Loose-rock Drop-structure upstream
Waingaehe	XS6	681	Loose-rock Drop-structure Downstream
Waingaehe	XS5	796	
Waingaehe	XS4	955	
Waingaehe	XS3	1097	
Waingaehe	XS2	1204	
Waingaehe	XS1	1359	Lake Front

2.3 Structures

One bridge, one culvert and one weir structure were included in the model. All structures have been slightly modified in order to reduce instability in the model. This involved lowering (downstream) and raising (upstream) stream cross-sections, in order to create a downstream gradient. Adjustments were in the order of a few centimetres. Levels for the weir structure (between cross-section 6 and 7) and the downstream cross section for the culvert on the historic channel (just below cross section 13) were extracted from LIDAR data (Vertical accuracy 0.15m).

Details of these structures are shown in Table 2.

Table 2 Details of the bridge and culvert used in the mike11 model

Branch	Chainage	Structure	Comment
Waingaehe Historic	2	Culvert	Modelled as culvert
Waingaehe	256	Bridge	Modelled as culvert
Waingaehe	671	Weir	Modelled as a broad crested weir. Values extracted from LIDAR data.

2.4 Computational parameters

The following conditions were set in Mike 11 and remained the same for all design event combinations.

- Initial conditions : See section 3.5 Waingaehe Stream Design flows
- Discharge : 0.22 m3/s
- Water depth : 0.2m
- Radius type : Total area, Hydraulic radius
- Time Step : 5 seconds
- Simulation Mode : Unsteady

2.4.1 Channel roughness

Since no calibration data is available for the Waingaehe Stream, channel roughness (Manning's n values) was derived from the Hydrological and Hydraulic Guidelines, Everitt, 2001. This document applies Manning's Roughness Coefficients from the "Urban Drainage Design", Sutherland Shire Council, Sydney, 1992. The calculated values were cross referenced using Hicks' and Mason (1991). See Appendix 3 for photo's of channel characteristics.

Table 3 Selected Roughness coefficients for the Waingaehe Stream from the Hydrological and Hydraulic Guidelines, Everitt, 2001

Chainage	Stream	Manning's n	Description
256m	Waingaehe	0.013	Concrete bridge.
Rest of Channel	Waingaehe	0.035	Natural stream channel; b dense growth of weeds, depth of flow greater than weed height.
15m	Waingaehe Historic	0.013	Closed conduit, Concrete Pipe.

2.5 File storage

All working datasets have been stored on Environment Bay of Plenty's data storage systems. For further details contact the manager of Environment Bay of Plenty, Rivers and Drainage Technical Services Section. Environment Bay of Plenty Staff can find the relevant datasets here: *R:\MIKEZero\Upper_Kaituna\Waingaehe* for mike 11 datasets. For additional working datasets TM61 etc, see Appendix 2.

Chapter 3: Hydrology

The Waingaehe Stream catchment is located on the eastern side of the Lake Rotorua Volcanic Caldera. Apart from some reworked fine material around the lake margin, surface soil types in the caldera are typically very permeable pumice and volcanic ash. The gauged catchments in the Rotorua area show very low specific discharges compared with other Bay of Plenty Catchments. Due to high infiltration rates a large proportion of waterways are intermittent or ephemeral flow paths. The design standard for the flood protection scheme is to contain the estimated 100 year Average Return Interval (ARI) flood with 500mm freeboard.

3.1 Available stream and rainfall information

3.1.1 Historical floods

The 13 August 1968 Waingaehe Stream flood flowed over the State Highway. Rainfall had been measured at 1.7 inches (43mm) in 3 hours (Revington, E.D., 1968)

On 8 May 1970 a flood was estimated by Ministry of Works staff downstream of the state highway at about 16 m^3/s (based on a single gauging at $9.3m^3/s$) (Freestone, H.J., 1970). A total of 2.17 inches (55mm) of rain fell at Rotorua Airport (1.34 inches (34mm) in one hour). The Daily Post reported extensive flooding at Lynmore, Basley Road, Holden's Bay and Hannah's Bay. Freestone does not mention that the highway was inundated so this flood may have been smaller than that in 1968.

The 1975 scheme report mentions that about 18 hectares of land adjacent to the Waingaehe Stream downstream of the highway had been inundated on "many occasions" (BOPCC, 1975).

3.1.2 Gauge data

A stream gauge has been in place on the Waingaehe Stream downstream of State Highway 30 since June 1992. However its record is intermittent and only 6 full years of flow data had been collected up until the end of 2006. This is not a long enough period to provide reliable statistical estimates of flood event probabilities. It is notable however that the largest flow in the period was only 2.0 m^3/s; no large flow events have been recorded by the gauge.

Nearby stream gauges on the Puarenga, Utuhina, and Ngongotaha Streams have operated over a longer period. Unfortunately three of these four sites; Puarenga at FRI, Puarenga at Hemo Gorge, and Utuhina at State Highway 5 were discontinued in the late 1990's. Details of the contributing catchment areas and operating periods of each gauge are provided in Table 4 and their locations are shown in below. The Puarenga at Hemo Gorge was not analysed in this investigation.

Table 4 Catchment areas and duration of data capture for stream gauges near Rotorua.

Catchments	Area (km^2)	Stream Length (km)	Location	Recording Period
Utuhina	59.3	22.2	Lake Road	1967 - 1996
Puarenga	74.2	13.2	F.R.I.	1976 - 1998
Ngongotaha	71.9	24.5	SH 5 Bridge	1976 - 2007
Waingaehe	9.5	5.4	SH 30 Bridge	1992 - 1995 2004 - 2007

Daily rainfall data has been gathered to the south of Lake Rotorua at Whakarewarewa since 1900 however there are some gaps in the record. To some extent these have been patched with data from a nearby rain gauge on Tarawera Road.

Automatic rainfall data is available for Kaharoa to the north of Lake Rotorua, collected since September 1985.

NIWA publish a spatial summary of rain gauge statistics in the form of their HIRDS software (High Intensity Rainfall Design System). At the time of writing Environment Bay of Plenty is using version 1.5b (1995) which uses a Gaussian interpolation routine to spatially distribute rainfall depth probabilities across selected rain gauges. In order to retain comparability, the HIRDS rainfall data was used as a basis for all methods of stream design flow calculation in this investigation.

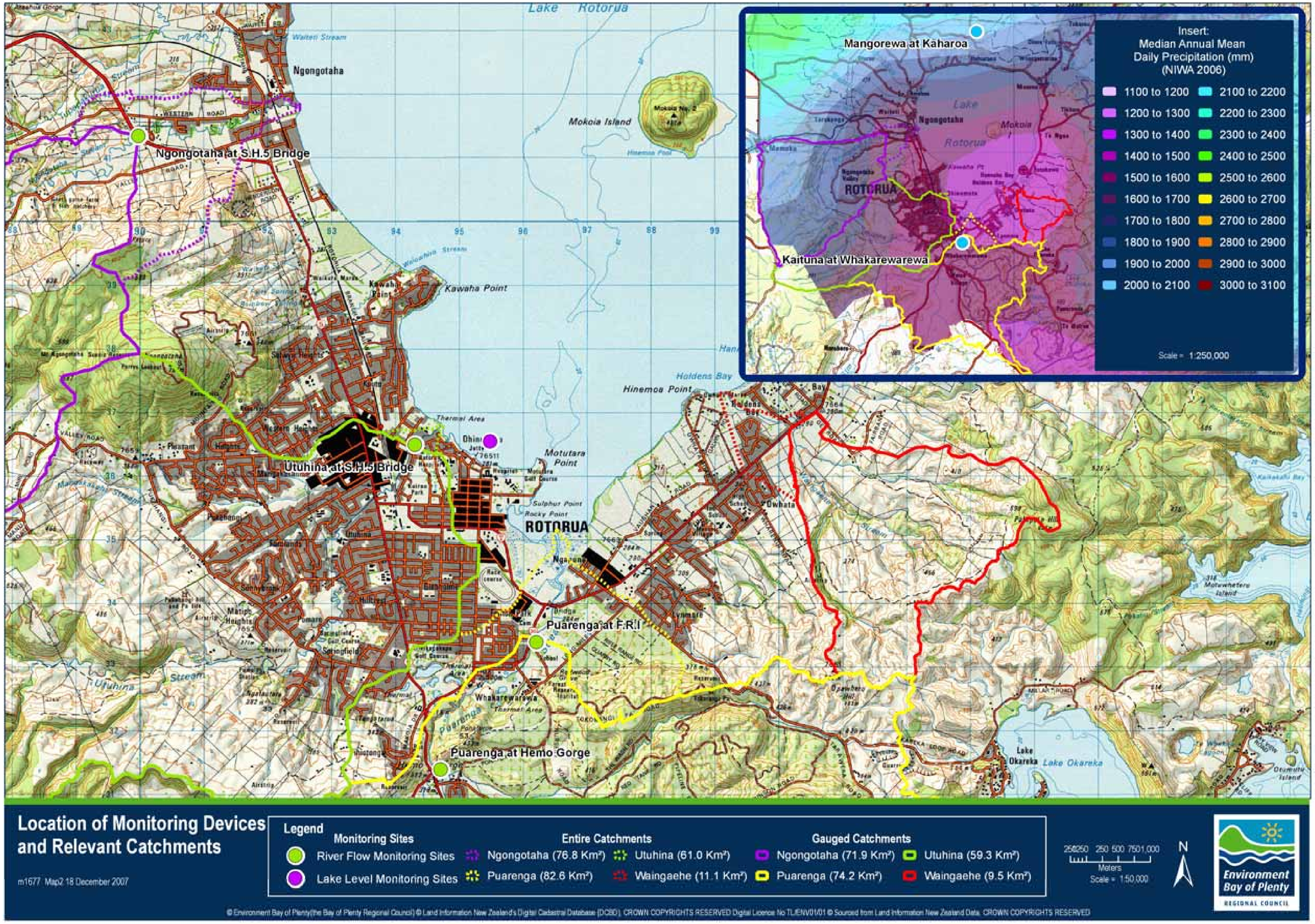


Figure 2 *Location of stream gauges for the Waingaehe, Utuhina, Puarenga, and Ngongotaha Streams; Lake Rotorua level gauge; and mean annual rainfall characteristics (Inset; NIWA, 2006)*

3.2 Previous Hydrological Estimates

Original 100 year ARI flow estimates for Waingaehe were as high as 48 m^3/s in 1968 (Environment Bay of Plenty Plan K4100) but these were revised downwards to 27.2 m^3/s following a review by the Ministry of Works in February 1969, and further analysis by Catchment Commission staff (BOPCC, 1975). These estimates were made using the TM61 and Snyder empirical parametric methods, based on catchment soils and vegetation, and with comparison to a Unit-graph type analysis of the Utuhina Stream (the Utuhina Gauge was operational from July 1967).

The 2003 Kaituna Asset Management Plan (Operations Report 2003/09) states a design 100 year ARI flow of 23 m^3/s following a review in 1999.

3.3 Design rainfall assumptions

3.3.1 Spatial patterns

The rain gauge at Kaharoa appears to get more rain than that at Whakarewarewa (inset in figure 2 above). Long duration design rainstorm depths at Kaharoa are typically about 40% greater than the corresponding event probability at Whakarewarewa (Table 5 below). Along with a component of rain-shadow, the trend probably reflects the difference in elevation between the two sites; the Kaharoa gauge is 120m higher. The trend is not apparent for the shorter duration events.

Table 5 20 year Average Return Interval (ARI) rainfall depths (mm) at Kaharoa and Whakarewarewa

Rainstorm Duration	Whakarewarewa Raingauge	HIRDS at Whakarewarewa	Kaharoa Raingauge	HIRDS at Kaharoa
1 hour	60	55	48	57
12 hours	144	158	163	165
24 hours	145	204	214	214
48 hours	184	253	265	265

NIWA's HIRDS software output for both of these locations is more in line with the Kaharoa estimates, with only a slight (about 5%) lessening of design rainfall intensities across the lake to Whakarewarewa. The HIRDS values are therefore considered to be slightly conservative for the subject catchments which are closer to Whakarewarewa.

In spite of this conservative aspect the HIRDS values were adopted both as the basis of the parametric evaluations (section 3.5.2), and of the regional analysis (section 3.5.4) where comparability between catchments is more important than absolute rainfall values.

3.3.2 Interdecadal pacific oscillation (IPO)

A climate effect with a temporal pattern spanning decades has been identified that affects the majority of the Pacific region (Kaituna Asset Management Plan, 2003). It has been considered that the Rotorua stream-gauge data collected since the 1960's and 70's could unduly represent the "benign phase" that is thought to have occurred from the mid 1970's to the mid 90's. This could have follow-on implications for any hydrological conclusions that have been drawn from these gauges (such as the regional methods described in section 3.5 below).

To test this effect, comparisons were made of the rainfall data for Whakarewarewa from two periods: the period from 1968 to 2005 (the period represented by the three Rotorua stream-gauges); and the record from 1903 which should span several oscillations of the IPO. Fitted to an EV1 frequency distribution, the longer data-set demonstrated a 4.5% increase in 100 year ARI event magnitude over the 1968 - 2005 data. This effect was taken into consideration in deciding design flow values.

3.3.3 Effects of climate change

While an increase in design rainfall intensities is anticipated due to the effects of global climate change, no allowance has been made for it in this review. The Asset Management Plan recommends that structures which are difficult to retrofit be designed to incorporate the anticipated effects within their design life. The existing earth embankment structures at Waingaehe are not difficult to upgrade and the interval between capacity reviews is considerably shorter than the usual time frames relating to climate change.

3.4 Contributing side catchments

Some of the side catchments that contribute to the stream channel downstream of State Highway 30 are large enough to cause significant increases to the flood flow (See Figure 2 for side catchment areas and locations). However these were discounted after careful inspection of the surrounding ground levels. None of the contributing waterways can reach sufficient head to counter the peak water levels in the Waingaehe main channel. During the peak of the design flood it can therefore be safely assumed that all flood-gated culverts will be closed and non-flood gated culverts will not be contributing. If this coincides with high intensity rainfall in the local catchments, then some flooding is possible outside of the flood protection scheme, with a general overland flow down-gradient towards the lake (see Appendix 1 for LIDAR ground levels and possible overland flow paths).

An analysis of flooding outside of the scheme caused by stormwater potentially failing to drain into the Waingaehe Stream, would require a detailed representation of the stormwater system and floodplain storages. These stormwater systems are administered by the local territorial authority, Rotorua District Council and are beyond the scope of this investigation.

3.5 Waingaehe Stream design flows

The original design flow -100 year Average Return Interval (ARI) – adopted for the 1975 flood protection scheme was 27.2 m^3/s and reset to 23 m^3/s in 1999; however this investigation into the Waingaehe Stream catchment hydrology found that a lesser flow should be adopted.

Three methods have been used to estimate design flows for the Waingaehe Stream:

(i) Parameter-type analysis by the TM61 method and the Rational Method, based on measurable values (area, length, slope, shape); and empirical parameters from literature relating to soil and ground cover types;

(ii) Transposition of statistically derived flood characteristics of the nearby Puarenga, Utuhina, Ngongotaha Streams by the Area$^{0.8}$ Method;

(iii) A regional analysis that derives surface characteristics (TM61's W_{ic}, and Rational Method's C) by calibrating for the nearby Puarenga, Utuhina, Ngongotaha stream-gauge catchments.

A direct statistical analysis of the Waingaehe stream-gauge was discounted due to its very short record;

3.5.1 Statistical methods

In analysing historical stream gauge data as discussed in this report, the following standard methods were used. Calendar year annual maxima from the historic record for each gauge were assigned event probability plotting positions by the Gringorten formula:

$$F(Q_i) = (i - 0.44) / (n + 0.12)$$

Where *i* is the rank of each flood in the order of flow magnitude, and *n* is the total number of floods in the record. Both EV1 and GEV probability distributions were then fitted to plotted points by the method of L-Moments (Hosking, 1990) using NIWA's Tideda software. GEV distributions were only considered for the Ngongotaha and Utuhina sites which have operational lives approaching 30 years; for records shorter than this the uncertainty of the GEV distribution is considered unworkable. The choice between EV1 and GEV distributions was made by visual comparison of the extreme probability tail on Gumbel plots. These plots are included in Appendix 2 along with the statistical calculation spreadsheets.

The resulting 100 year flows are shown in the second column of table 6 below.

3.5.2 Method of transposition by area$^{0.8}$

Estimates for Waingaehe Stream design flows were also made by transposing flood characteristics from Puarenga, Utuhina, and Ngongotaha. The method assumed that the corresponding peak flow is proportional to the catchment area raised to the power of 0.8:

$$Q_{p1} = Q_{p2} \times (A_1 / A_2)^{0.8}$$

The neighbouring streams' flood flow magnitudes were statistically derived from their historic stream data. Resulting estimates for Waingaehe Q_{100} are shown in Table 6 below. The area of Waingaehe Stream catchment to State Highway 30 is 9.5 km^2.

Table 6 Estimates for Waingaehe Q100 using the Area$^{0.8}$ Method of Transposition

Catchment	Q_{100} (m^3/s)	Area; A_2 (km^2)	Resulting estimate for Waingaehe Q_{100} (m3/s)
Utuhina	51.2	59.6	**11.8**
Puarenga	56.2	74.8	**10.8**
Ngongotaha	58.3	73.3	**11.4**

3.5.3 Design flows by parametric methods

Parametric estimates of design flows were made using both the TM61 method and the Rational Method. Measurable parameters were determined from the NZMS 260 series 1:50,000 scale Topographical Series maps. The TM61 ground surface parameter W_{ic} and the Rational Method runoff coefficient C were selected from literature while referring to soil maps, land cover maps and aerial photographs. Rainfall intensities were from HIRDS version 1.5b output based on the catchment centroid location.

The catchment time of concentration was selected based on estimates by three recognised methods: the Ramser Kirpich; Bransby Williams; and the U.S. Soil Conservation Service Method.

100 year ARI flow estimates for Waingaehe by these two methods are shown in Table 8 below. Copies of the calculation spreadsheets can be found in Appendix 2.

3.5.4 Regional analysis using TM61 and Rational Method

Surface characteristics for the neighbouring Utuhina, Puarenga, and Ngongotaha stream-gauge catchments were evaluated by calibrating their TM61 and Rational Method estimate flows against their statistically derived flood characteristics. This was done by adjusting the TM61 surface characteristic (W_{ic}) and Rational Method runoff coefficient (C). Catchment times-of-concentration were also compared with recorded flood hydrographs. The surface characteristics were then compared and values were selected to apply to the Waingaehe catchment.

This method relies on adequate representation of the historic rainfall characteristic over the period of stream record. In this case it was assumed that the 4 catchments were adequately represented by the HIRDS output; comparatively if not absolutely.

Table 7 below shows the resulting surface parameters for each catchment. When reviewing these parameters to select values for Waingaehe (in bold type Table 6), consideration was given to the relative proportions of each catchment under various land uses.

Table 7 Catchment characteristic parameters; showing also the selected values for Waingaehe Catchment (bold type)

Catchment	% Pasture	% Forest	% Urban	W_{ic}	C
Utuhina	30	52	18	0.54	0.12
Puarenga	29	65	6	0.52	0.12
Ngongotaha	44	56	0	0.44	0.11
Waingaehe	66	34	0	**0.56**	**0.14**

TM61 and Rational Method parameter analyses were then carried out for Waingaehe Stream based on these selected surface parameter values. The results for Waingaehe are shown in Table 8 below along with those from the literature-based parameter selection (section 3.5.3).

Table 8 100 year ARI flow estimates for Waingaehe Stream by various parameter-based methods

Method:	W_{ic}	C	Resulting Q_{100} (m^3/s)
TM61 Literature Based	0.50		**14.0**
TM61with regional parameter selection	0.56		**17.6**
Rational Method Literature Based		0.18	**31.2**
Rational Method with regional parameter selection		0.14	**24.3**

3.5.5 Design flows; summary and discussion

A range of methods have been used to give adequate background for final selection of design flows for the Waingaehe Stream. The methods are outlined in the body of Section 3.4 above. Table 9 below summarises the results by method.

Table 9 Summary of design flow estimates for Waingaehe Stream by method

Method	Confidence weighting	Q_{100}	Q_{50}	Q_{20}	Q_{10}
TM61 (Lit based)	Moderate	14.0	12.6	10.9	9.6
TM61 (Regional)	Moderate/high	17.6	15.8	13.7	12.0
Rational Method (Lit based)	Low	31.2	28.2	24.3	21.3
Rational Method (Regional)	Low/moderate	24.3	21.9	18.9	16.6
Transposed Utuhina	Moderate/high	11.8	10.0	8.1	6.7
Transposed Puarenga	Moderate/high	10.8	9.7	8.2	7.1
Transposed Ngongotaha	Moderate/high	11.4	10.2	8.6	7.4

The second column in Table 9 shows a confidence weighting value for the perceived relative certainty with each method. Methods of transposition are widely considered to be reliable, especially when they involve careful checks on the similarity between catchments. That all three nearby catchments give similar results enhances the confidence in these values. The TM61 method with regionally derived surface parameters is also considered highly; it combines an established catchment response model with the added certainty of calibrated values nearby.

The Rational Method by comparison, even when used with a regionally derived runoff coefficient is weighted Low Confidence because it is known to typically over-estimate flows from catchments any larger than 1-2 square kilometres.

When selecting design flows for the Waingaehe Stream a degree of conservatism was applied. The values in Table 10 below are an amalgamation of the various hydrological inputs, including reviews of previous engineers' reports and newspaper coverage of historical flood events. These values were adopted for the hydraulic modelling capacity review of the Flood Protection Scheme as described in Chapter 2 above.

Table 10 Design flows for Waingaehe Stream at State Highway 30

ARI Event	Discharge (m^3/s)
Q_{100}	17.2
Q_{50}	15.5
Q_{20}	13.3
Q_{10}	11.7

3.6 Lake Rotorua design levels

The downstream boundary condition for the model is the water level in Lake Rotorua. This level is controlled by a stop-log weir on the Ohau Channel at Mourea and fluctuates due to rainfall and wind effects. Design water levels are from Environment Bay of Plenty's data summaries (Environment Bay of Plenty, 2005) and are based on a statistical analysis of lake level data since 1953. Design water levels are shown in Table 11 below.

Table 11 Design water levels for Lake Rotorua

Event	Elevation (metres above Moturiki Datum)
L_{100}	280.79
L_{50}	280.68
L_{20}	280.54

The conditions leading to a large magnitude flow event in the Waingaehe Stream are also likely to cause high levels in Lake Rotorua. Event combinations were used as prescribed in Environment Bay of Plenty's *Hydrological and Hydraulic Guidelines* (Guidelines 2001/01) to manage the combination of event probabilities for these associated waterways. The event combinations are shown in Table 12 below.

Sensitivity analyses showed that the model is virtually insensitive to changes to lake levels within the range shown here.

Table 12 Design standard combinations for floods and downstream water levels

	Stream Flow		Lake Level	
Case 1	Q_{100}	17.2 m^3/s	L_{20}	280.54 m
Case 2	Q_{20}	13.3 m^3/s	L_{100}	280.79 m

Chapter 4: Results

Table 12 below and Figure 3 below; illustrate the maximum 100 year event for the Waingaehe Stream. The results are a combination of the design flow events described in point 3.5. Results have been selected based on the highest modelled values for the various events and combined into table 12 below. Further details for the individual model runs can be found in Appendix 4 and 8.

The Kaituna Asset Management Plan (Wallace, 2003) stipulates that stopbank design freeboard levels of less than half their required freeboard require topping up. The AMP sets an initial design freeboard levels for the Waingaehe Stream of 600mm, reduced to 500mm in 1975. A minimum of 250 mm freeboard is the required trigger for top ups.

Table 12 Maximum 100 year water levels Waingaehe Stream.

Mike 11 Cross Section	Mike 11 Chainage (m)	Left Bank Height	Right Bank Height	Maximum Water Level			Maximum Design Event
				Calculated	Water level + 250mm	Water level + 500mm	
XS 15	0	288.53	288.91	287.607	287.857	288.107	Q100 L20
XS 14	33	288.52	288.35	287.337	287.587	287.837	Q100 L20
XS 13	99	288.09	288.23	286.78	287.03	287.28	Q100 L20
XS 12	145	287.1	286.7	285.835	286.085	286.335	Q100 L20
XS 11	244	286.36	287.37	284.964	285.214	285.464	Q100 L20
XS 10.5 upstream	250	286.74	286.74	284.983	285.233	285.483	Q100 L20
XS 10.5 downstream	260	286.66	286.66	284.941	285.191	285.441	Q100 L20
XS 10	279	286.67	286.91	284.836	285.086	285.336	Q100 L20
XS 9	422	285.19	285.3	284.077	284.327	284.577	Q100 L20
XS 8	528	284.7	284.57	283.696	283.946	284.196	Q100 L20
XS 7	659	284.34	284.61	283.486	283.736	283.986	Q100 L20
XS 6	681	284.18	283.759	282.319	282.569	282.819	Q100 L20
XS 5	796	282.747	282.738	282.117	282.367	282.617	Q100 L20
XS 4	955	282.711	282.494	281.787	282.037	282.287	Q100 L20
XS 3	1097	282.239	282.269	281.39	281.64	281.89	Q100 L20
XS 2	1204	281.774	282.011	280.949	281.199	281.449	Q100 L20
XS 1	1359	280.764	280.68	280.788	281.038	281.288	H100 Q20

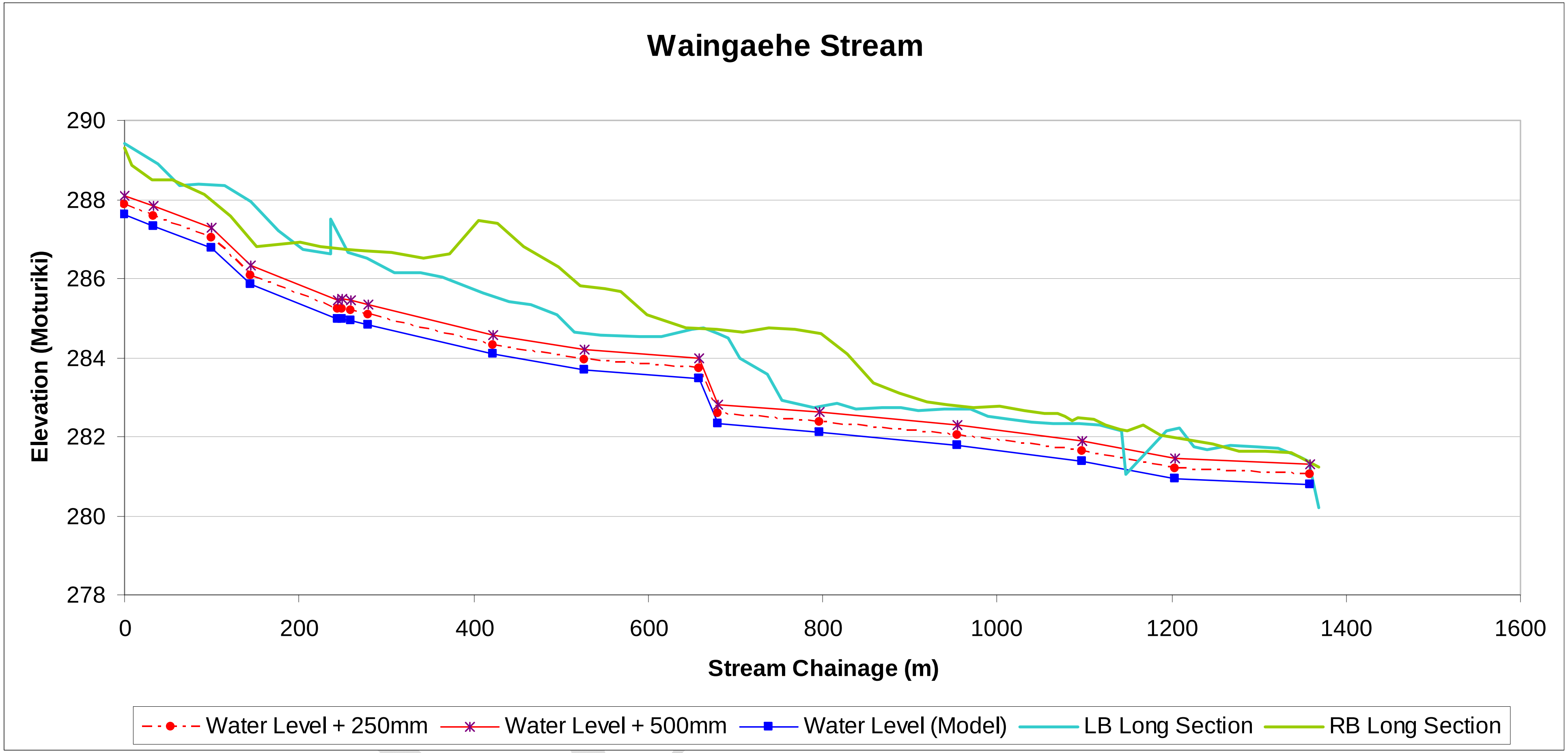


Figure 3 Maximum 100 year water level Waingaehe Stream.

Chapter 4: Conclusions and Recommendations

4.1 Conclusions

The capacity review of the Waingaehe Stream has indicated that existing stop banks are well above the 100 year ARI (1%AEP) design levels. A review of the Waingaehe Stream hydrology suggests that a somewhat lesser design flow should be applied to the Waingaehe Stream and the Upper Kaituna Asset Management Plan. A reduction in ARI values for the Waingaehe is to some degree supported by a general reduction in statistical flows for those gauges with longer periods of data capture in the Rotorua catchments.

Cross-section 1 does indicate a standard below the 100 year ARI; however this cross section is located outside of the stopbank area. The exceedence experienced at cross-section 1 was as a result of a localised drop down to lake levels and does not compromise stopbank protection.

It may be necessary to consider installing a culvert on the left bank just below cross section 3. This is illustrated by the obvious dip in the stopbank long section in Figure 3 above. The results indicate that ponding (approximately 500m^2) in this area will be confined within the side channel because it is bounded by high ground on all sides. The local catchment is very small in size.

4.2 Recommendations

- No top ups required at this time. The model results show adequate freeboard along the full length of the protected reach.

- Further analyses might be considered by Rotorua District Council, to assess local stormwater effects on adjacent residential properties, during Waingaehe Stream flood events. This is discussed in section 3.4.

References

Bay of Plenty Catchment Commission (1975); *Upper Kaituna Catchment Control Scheme*.

Bay of Plenty Catchment Commission report (1968); *Waingaehe Stream*; prepared by Revington, E.D. September 1968, File 146/08/3

Environment Bay of Plenty (2005); *Environmental Data Summaries, Air Quality, Meteorology, Rainfall, Hydrology and Water Temperature Report to 31 December 2005*, prepared by Environment Bay of Plenty Environmental Data Services.

Environment Bay of Plenty (2003); *Kaituna Asset Management Plan" Environment Bay of Plenty Operations Report 2003/09*, October 2003, prepared by Philip Wallace.

Environment Bay of Plenty (2001); *Hydrological and Hydraulic Guidelines*, prepared by Steve Everitt 16 July 2001.

Ministry of Works, letter of 17 July 1970, Freestone, H.J., details on BOPCC drawing K4169.

National Institute of Water and Atmospheric Research Ltd (NIWA) (1998); *Roughness Characteristics of New Zealand Rivers*, September 1998, prepared by DM Hicks and PD Mason.

New Zealand Hydrological Society (1992); *Waters of New Zealand*, edited by Paul Mosley.

Appendices

Appendix 1 – Surface topography for the Waingaehe Stream in the vicinity of the excluded culvert catchments.

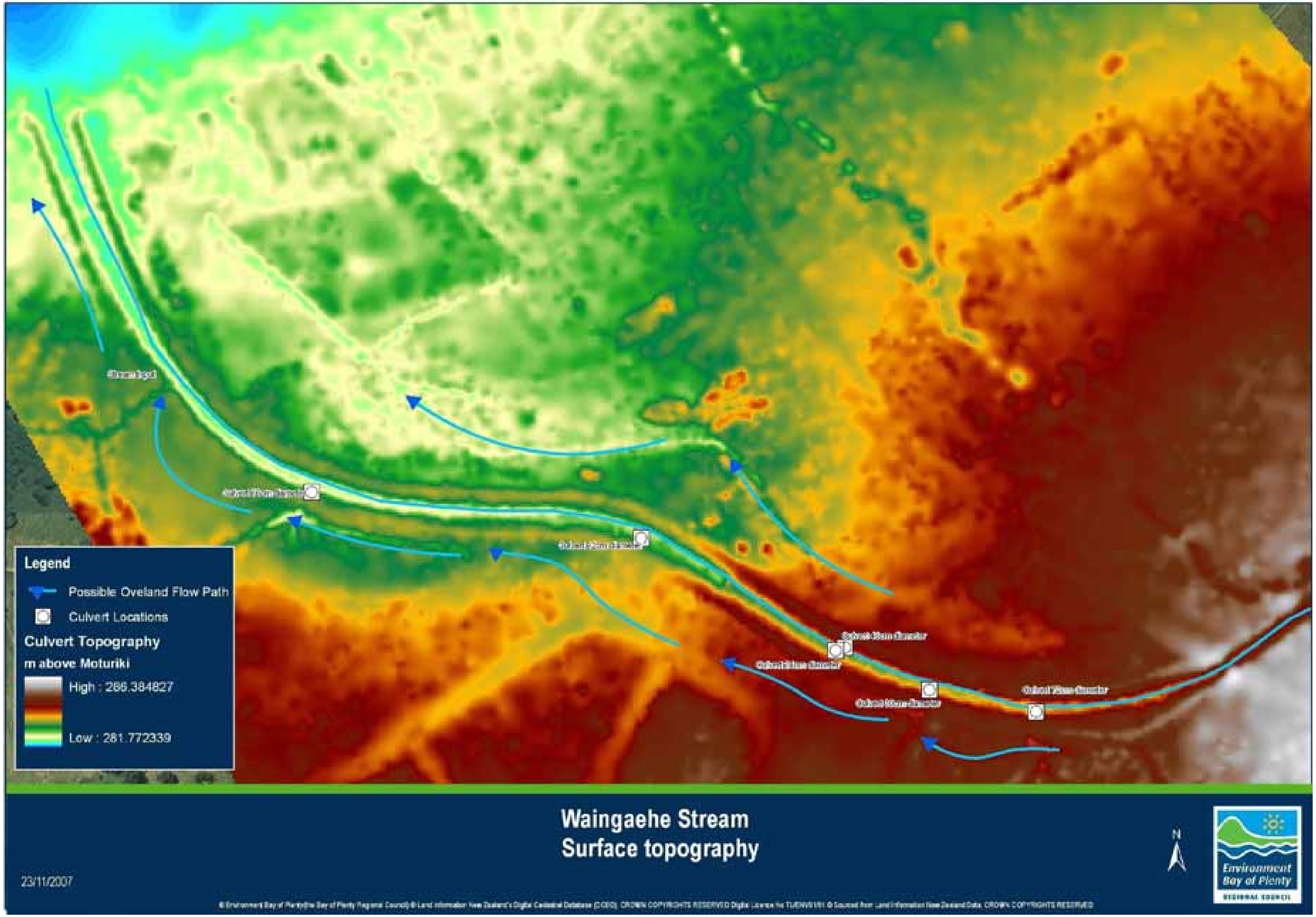
Appendix 2 – Calculation spreadsheets used for design flows by various methods.

Appendix 3 – Cross section / channel characteristics for the Waingaehe Stream

Appendix 4 – Model Results for Q100 and L20 event combinations

Appendix 5 – Model results for Q20 and L100 event combinations

Appendix 1: Surface topography for the Waingaehe Stream in the vicinity of the excluded culvert catchments


Legend
Possible Oveland Flow Path
Culvert Locations
Culvert Topography
m above Moturiki
High : 286.384827
Low : 281.772339
Waingaehe Stream
Surface topography
N
23/11/2007
Environment Bay of Plenty
REGIONAL COUNCIL

Appendix 2 – Calculation spreadsheets used for design flows by various methods

Hydrological Catchment Analysis

Environment Bay of Plenty
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Stream Name:	**Waingaehe (based on literature values)**		
Reference:	Excel File R:\Calculations\TM61-calcs\Waingaehe.TM61.071114		
Location:	MAP GRID - NZMG 2800810 E 6335832 N	**Date:**	28-Feb-08
Calculated:			

Technical Memorandum 61

Area	A	km^2	9.5
Direct length	L_d	km	4.27
Stream length	L	km	5.37
Average slope	S_a	m/m	0.0168
Height difference	H	m	189
Time of Concentration			
Ramser-Kirpich Eq.	Tc_1	min	70.0
Bransby Williams Eq.	Tc_2	min	120.3
U.S. Soil Con. Service.	Tc_3	min	52.7
	Ave.	min	81.0
	Use	min	81
Surface Characteristic	W_{ic}		0.5
Slope Factor	W_s		41.2
W_s x W_{ic}	W		20.6
Discharge coefficient	C		176
Standard rainfal depth		mm	85.7
A/L_d^2	K		0.521
Shape factor	S		1.02

Very absorbent/absorbent soils. 34% long grass, scub and bush, 66% shortgrazed

Calculation of Average Slope
(Modified Taylor-Schwarz Method)

elevation (m)	distance (m)	slope (m/m)	li/Si^0.5
287	0		
290	351.8481	0.009	3810
301	2125.02	0.006	22513
339	3404.946	0.030	7428
371	4002.658	0.054	2583
398	4532.798	0.051	2349
476	5373.872	0.093	2762

Average slope (S_a)	0.0168
S_a as %	1.68

Hirds data

Interpolation of Hirds — Version: 1995

Duration	100y	50y	20y	10y	5y	2y
	Design Rainfall Depth (mm)					
60	80	72	62	54	46	33
81	89	80	69	61	52	38
120	105	95	82	73	62	47

			100y	50y	20y	10y	5y	2y
Rainfall Factor	R		1.03	0.93	0.80	0.71	0.60	0.44
Peak Flow Rate	Q_p	m^3/s	**14.0**	**12.6**	**10.9**	**9.6**	**8.2**	**6.0**

Rational Method

Area	A	km^2	9.5
Runoff Coefficient	C		0.18

			100y	50y	20y	10y	5y	2y
Rainfall Intensity	I	mm/h	66	59	51	45	38	28
Flow rate	Q_p	m^3/s	**31.2**	**28.2**	**24.3**	**21.3**	**18.2**	**13.3**

Hydrological Catchment Analysis

Environment Bay of Plenty
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Stream Name: **Utuhina (compared with historic data summary)**

Reference: Excel File R:\Calculations\TM61-calcs\

Location: MAP GRID - Easting:2794230 Northing:6336602 **Date:** 28-Feb-07

Calculated:

Technical Memorandum 61

Area	A	km^2	59.6
Direct length	L_d	km	14.3
Stream length	L	km	22.19
Average slope	S_a	m/m	0.0062
Height difference	H	m	385

Q100 by statistical analysis of Utuhina Streamgauge is 51.2m3/s (attached sheet).

Time of Concentration

Ramser-Kirpich Eq.	Tc_1	min	306.3
Bransby Williams Eq.	Tc_2	min	476.4
U.S. Soil Con. Service.	Tc_3	min	206.2
	Ave.	min	329.7
	Use	min	330

Calculation of Average Slope

(Modified Taylor-Schwarz Method)

elevation (m)	distance (m)	slope (m/m)	li/Si^0.5
280	0		
287	3476	0.002	77459
308	9025	0.004	90201
337	13883	0.006	62876
444	15124	0.086	4226
544	16434	0.076	4741
590	18100	0.028	10026
616	20306	0.012	20320
665	22187	0.026	11654
	Average slope (S_a)		0.0062
	S_a as %		0.62

Surface Characteristic	W_{lc}		0.54
Slope Factor	W_s		50.3
W_s x W_{lc}	W		27.1
Discharge coefficient	C		305
Standard rainfal depth		mm	193.8
A/L_d^2	K		0.291
Shape factor	S		0.76

Hirds data

Interpolation of Hirds — Version: 1995

Duration	100y	50y	20y	10y	5y	2y
	Design Rainfall Depth (mm)					
180	110	100	86	76	65	49
330	143	130	112	99	85	64
360	149	136	117	103	89	67

			100y	50y	20y	10y	5y	2y
Rainfall Factor	R		0.74	0.67	0.58	0.51	0.44	0.33
Peak Flow Rate	Q_p	m^3/s	**50.8**	**46.4**	**39.9**	**35.1**	**30.3**	**22.8**

Rational Method

Area	A	km^2	59.6
Runoff Coefficient	C		0.12

			100y	50y	20y	10y	5y	2y
Rainfall Intensity	I	mm/h	26	24	20	18	15	12
Flow rate	Q_p	m^3/s	**51.5**	**47.0**	**40.4**	**35.6**	**30.7**	**23.1**

Hydrological Catchment Analysis

Environment Bay of Plenty REGIONAL COUNCIL

Stream Name: **Utuhina (compared with historic data summary)**
Reference: Excel File R:\Calculations\TM61-calcs\
Location: MAP GRID - Easting:2794230 Northing:6336502 **Date:** 28-Feb-07
Calculated:

Technical Memorandum 61

Q100 by statistical analysis of Utuhina Streamgauge is 51.2m3/s (attached sheet).

Area	A	km^2	59.6
Direct length	L_d	km	14.3
Stream length	L	km	22.19
Average slope	S_a	m/m	0.0062
Height difference	H	m	385
Time of Concentration			
Ramser-Kirpich Eq.	Tc_1	min	306.3
Bransby Williams Eq.	Tc_2	min	476.4
U.S. Soil Con. Service.	Tc_3	min	206.2
	Ave.	min	329.7
	Use	min	330
Surface Characteristic	W_{lc}		0.54
Slope Factor	W_s		50.3
$W_s \times W_{lc}$	W		27.1
Discharge coefficient	C		305
Standard rainfal depth		mm	193.8
A/L_d^2	K		0.291
Shape factor	S		0.76

Calculation of Average Slope
(Modified Taylor-Schwarz Method)

elevation (m)	distance (m)	slope (m/m)	li/Si^0.5
280	0		
287	3476	0.002	77459
308	9025	0.004	90201
337	13883	0.006	62876
444	15124	0.086	4226
544	16434	0.076	4741
590	18100	0.028	10026
616	20306	0.012	20320
665	22187	0.026	11654
	Average slope (S_a)		0.0062
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Hirds data

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Duration	100y	50y	20y	10y	5y	2y
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330	143	130	112	99	85	64
360	149	136	117	103	89	67

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Rainfall Factor	R		0.74	0.67	0.58	0.51	0.44	0.33
Peak Flow Rate	Q_p	m^3/s	**50.8**	**46.4**	**39.9**	**35.1**	**30.3**	**22.8**

Rational Method

Area	A	km^2	59.6
Runoff Coefficient	C		0.12

			100y	50y	20y	10y	5y	2y
Rainfall Intensity	I	mm/h	26	24	20	18	15	12
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Hydrological Catchment Analysis

Environment Bay of Plenty
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Stream Name: **Utuhina (compared with historic data summary)**
Reference: Excel File R:\Calculations\TM61-calcs\
Location: MAP GRID - Easting:2794230 Northing:6336602 **Date:** 28-Feb-07
Calculated:

Technical Memorandum 61

Q100 by statistical analysis of Utuhina Streamgauge is 51.2m3/s (attached sheet).

Area	A	km^2	59.6
Direct length	L_d	km	14.3
Stream length	L	km	22.19
Average slope	S_a	m/m	0.0062
Height difference	H	m	385
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Bransby Williams Eq.	Tc_2	min	476.4
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	Ave.	min	329.7
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Rational Method

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Hydrological Catchment Analysis

Environment Bay of Plenty REGIONAL COUNCIL

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Reference: Excel File R:\Calculations\TM61-calcs\

Location: MAP GRID - Easting:2794230 Northing:6336502

Date: 28-Feb-07

Calculated:

Technical Memorandum 61

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Rainfall Factor	R		0.74	0.67	0.58	0.51	0.44	0.33
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Rational Method

			100y	50y	20y	10y	5y	2y
Area	A	km^2	59.6					
Runoff Coefficient	C		0.12					
Rainfall Intensity	I	mm/h	26	24	20	18	15	12
Flow rate	Q_p	m^3/s	**51.5**	**47.0**	**40.4**	**35.6**	**30.7**	**23.1**

L Moments from Tideda
Formula from Hosking 1990

For Site: **Puarenga at FRI; From 1976 to 1998 inclusive**

Mean	Std. Dev.	Skew	Kurtosis
L1	L2	T3	T4
22885	5745	0.271	0.161

For EV1

$\alpha = \frac{l_2}{\ln 2}$ = 8288

$\xi = l_1 - 0.5772\ \alpha$ = 18101

$y_T = -\ln(-\ln(1 - \frac{1}{T}))$

$Q_{T1} = \xi + \alpha y_T$

For GEV

$z = 2/(3 + t_3) - \log 2/\log 3$ = -0.019

$k = 7.8590z + 2.9554z^2$ = -0.152

$\alpha = l_2 k / \{(1 - 2^{-k})\Gamma(1 + k)\}$ = 7052

$\xi = l_1 + \alpha\{\Gamma(1 + k) - 1\}/k$ = 17577

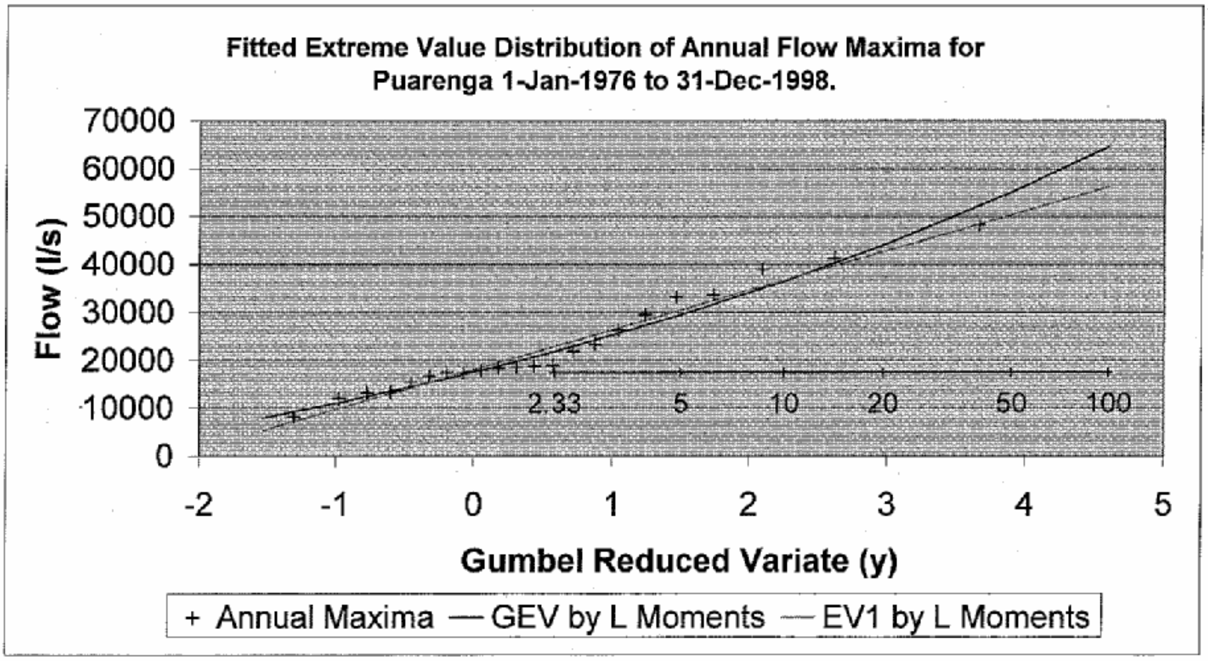
$y_T = -\ln(-\ln(1 - \frac{1}{T}))$

$$Q_{T2} = u + \frac{\alpha}{k} \times \left(1 - e^{(-ky_T)}\right)$$

Return Period		EV1	GEV
T	Y_T	Q_{T1}	Q_{T2}
1.01	-1.5293	5425	7955
1.1	-0.8746	10852	11802
1.5	-0.0940	17322	16919
2	0.3665	21139	20235
2.33	0.5786	22897	21842
5	1.4999	30533	29458
10	2.2504	36753	36500
20	2.9702	42719	44054
50	3.9019	50441	55144
100	4.6001	56228	64547
200			
500			

n= 22

i	Sample Values	F(Qi)	y(Qi)
1	8263	0.025	-1.302
2	12005	0.071	-0.975
3	13291	0.116	-0.768
4	13306	0.161	-0.603
5	15482	0.206	-0.457
6	16739	0.251	-0.323
7	17454	0.297	-0.195
8	17511	0.342	-0.071
9	17949	0.387	0.052
10	18455	0.432	0.176
11	18582	0.477	0.302
12	18846	0.523	0.432
13	18995	0.568	0.569
14	21861	0.613	0.715
15	23339	0.658	0.872
16	26326	0.703	1.045
17	29519	0.749	1.240
18	33235	0.794	1.466
19	33868	0.839	1.740
20	38984	0.884	2.096
21	41356	0.929	2.615
22	48307	0.975	3.664



L Moments from Tideda
Formula from Hosking 1990

For Site: **Utuhina at SH5 Lake Road; From 1968 to 1996 inclusive**

Mean	Std. Dev.	Skew	Kurtosis
L1	L2	T3	T4
19492	4170	0.294	0.213

For EV1

$$\alpha = \frac{l_2}{\ln 2} = 6016$$

$$\xi = l_1 - 0.5772\,\alpha = 16020$$

$$y_T = -\ln(-\ln(1-\frac{1}{T}))$$

$$Q_{T1} = \xi + \alpha y_T$$

For GEV

$$z = 2/(3+t_3) - \log 2/\log 3 = -0.024$$

$$k = 7.8590z + 2.9554z^2 = -0.185$$

$$\alpha = l_2 k / \{(1-2^{-k})\Gamma(1+k)\} = 4912$$

$$\xi = l_1 + \alpha\{\Gamma(1+k) - 1\}/k = 15567$$

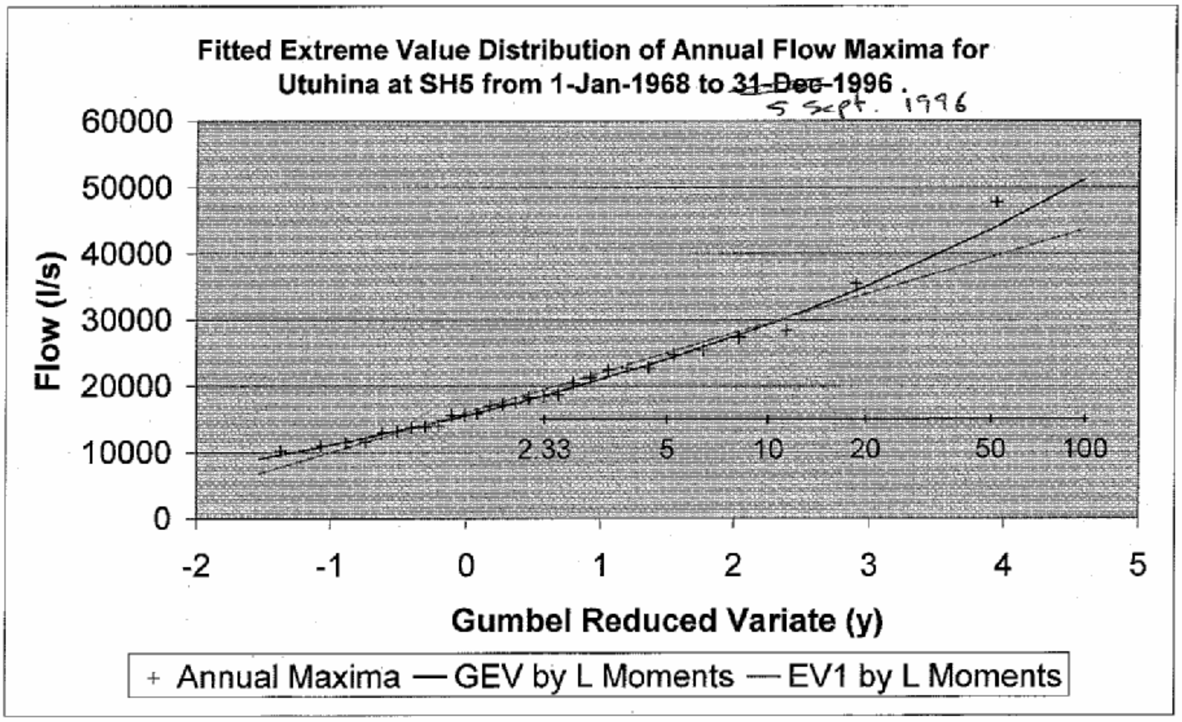
$$y_T = -\ln(-\ln(1-\frac{1}{T}))$$

$$Q_{T2} = u + \frac{\alpha}{k} \times \left(1 - e^{(-ky_T)}\right)$$

Return Period		EV1	GEV
T	Y_T	Q_{T1}	Q_{T2}
1.01	-1.529	6819	9025
1.1	-0.875	10758	11601
1.5	-0.094	15454	15109
2	0.367	18224	17430
2.33	0.579	19500	18567
5	1.500	25043	24059
10	2.250	29558	29279
20	2.970	33888	35015
50	3.902	39494	43671
100	4.600	43694	51209
200			
500			

n= 29

i	Sample Values	F(Qi)	y(Qi)
1	10196	0.019	-1.374
2	11100	0.054	-1.074
3	11498	0.088	-0.888
4	11615	0.122	-0.743
5	12993	0.157	-0.617
6	13251	0.191	-0.504
7	13733	0.225	-0.399
8	13881	0.260	-0.299
9	13982	0.294	-0.202
10	15569	0.328	-0.108
11	15600	0.363	-0.014
12	15874	0.397	0.079
13	17017	0.431	0.173
14	17149	0.466	0.269
15	17493	0.500	0.367
16	18241	0.534	0.467
17	18408	0.569	0.572
18	18702	0.603	0.682
19	20454	0.637	0.798
20	21243	0.672	0.921
21	22488	0.706	1.055
22	22759	0.740	1.202
23	22790	0.775	1.366
24	24785	0.809	1.552
25	25418	0.843	1.770
26	27383	0.878	2.037
27	28403	0.912	2.366
28	35509	0.946	2.899
29	47746	0.981	3.942



L Moments from Tideda
Formula from Hosking 1990

For Site: **Ngongotaha at SH5 from 1976 to 2006 inclusive**

Mean	Std. Dev.	Skew	Kurtosis
L1	L2	T3	T4
23256	6045	0.087	0.029

For EV1

$$\alpha = \frac{l_2}{\ln 2} = 8721$$

$$\xi = l_1 - 0.5772\,\alpha = 18222$$

$$y_T = -\ln(-\ln(1 - \frac{1}{T}))$$

$$Q_{T1} = \xi + \alpha y_T$$

For GEV

$$z = 2/(3+t_3) - \log 2/\log 3 = 0.017$$

$$k = 7.8590z + 2.9554z^2 = 0.134$$

$$\alpha = l_2 k/\{(1-2^{-k})\Gamma(1+k)\} = 9731$$

$$\xi = l_1 + \alpha\{\Gamma(1+k)-1\}/k = 18791$$

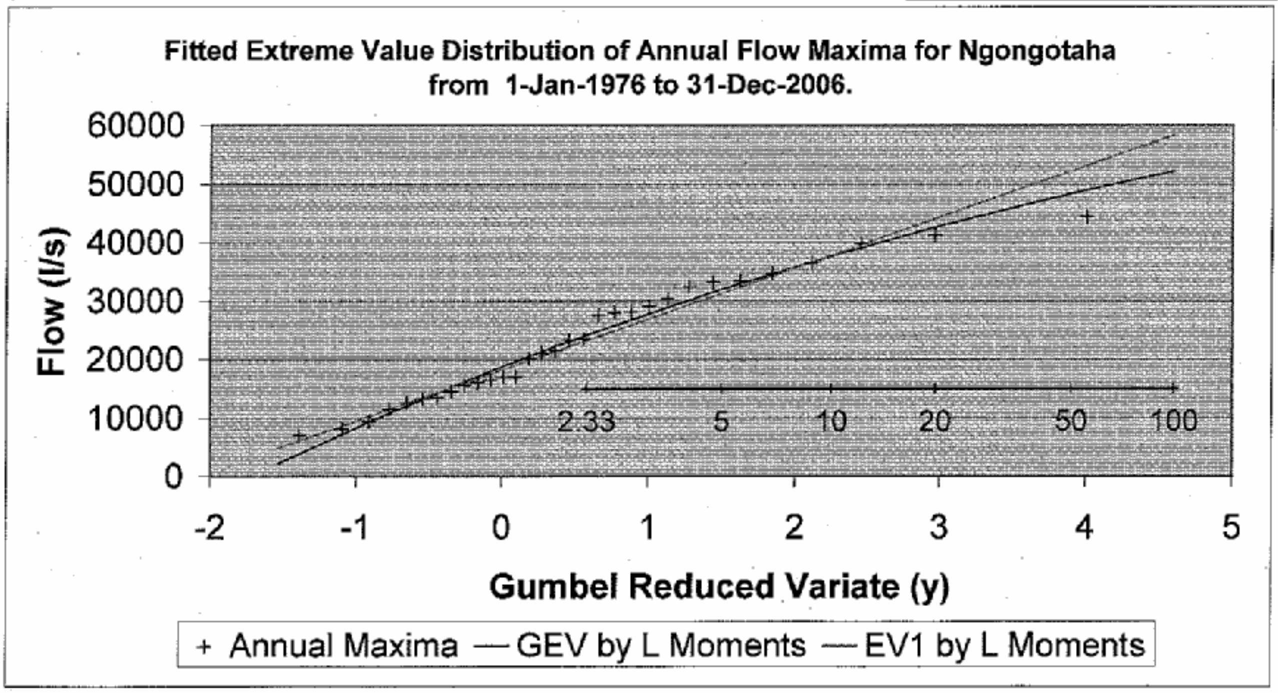
$$y_T = -\ln(-\ln(1 - \frac{1}{T}))$$

$$Q_{T2} = u + \frac{\alpha}{k} \times \left(1 - e^{(-ky_T)}\right)$$

Return Period		EV1	GEV
T	Y_T	Q_{T1}	Q_{T2}
1.01	-1.529	4885	2274
1.1	-0.875	10595	9761
1.5	-0.094	17402	17870
2	0.367	21419	22271
2.33	0.579	23268	24208
5	1.500	31303	32013
10	2.250	37848	37695
20	2.970	44126	42633
50	3.902	52251	48358
100	4.600	58341	52202
200			
500			

n= 31

i	Sample Values	F(Qi)	y(Qi)
1	6991	0.018	-1.391
2	8181	0.050	-1.096
3	9422	0.082	-0.915
4	11500	0.114	-0.774
5	12632	0.147	-0.653
6	13359	0.179	-0.544
7	13429	0.211	-0.443
8	14535	0.243	-0.347
9	15531	0.275	-0.255
10	16041	0.307	-0.166
11	16490	0.339	-0.078
12	16991	0.371	0.010
13	17049	0.404	0.097
14	20220	0.436	0.185
15	21084	0.468	0.275
16	21535	0.500	0.367
17	23307	0.532	0.461
18	23463	0.564	0.558
19	27623	0.596	0.660
20	28049	0.629	0.767
21	28130	0.661	0.881
22	29101	0.693	1.002
23	30388	0.725	1.134
24	32427	0.757	1.279
25	33245	0.789	1.441
26	33450	0.821	1.625
27	34690	0.853	1.842
28	36447	0.886	2.108
29	39848	0.918	2.455
30	41344	0.950	2.968
31	44419	0.982	4.009



Appendix 3 — Cross section / channel characteristics for the Waingaehe Stream

See R:\SURVEY DATA\EBOP River Cross Section Data\Waingaehe Stream\070125 Waingaehe Stream\Photos.


XS1 Upstream


XS1 Downstream


XS2 Upstream


XS2 Downstream


XS3 Upstream


XS3 Downstream

XS4 Upstream

XS4 Downstream

XS5 Upstream

XS5 Downstream

XS6 Upstream

XS6 Downstream


XS7 Upstream


XS7 Downstream


XS8 Upstream


XS8 Downstream


XS9 Upstream


XS9 Downstream


XS10 Upstream


XS10 Downstream


XS11 Upstream


XS11 Downstream


XS12 Upstream


XS12 Downstream


XS13 Upstream


XS13 Downstream


XS14 Upstream

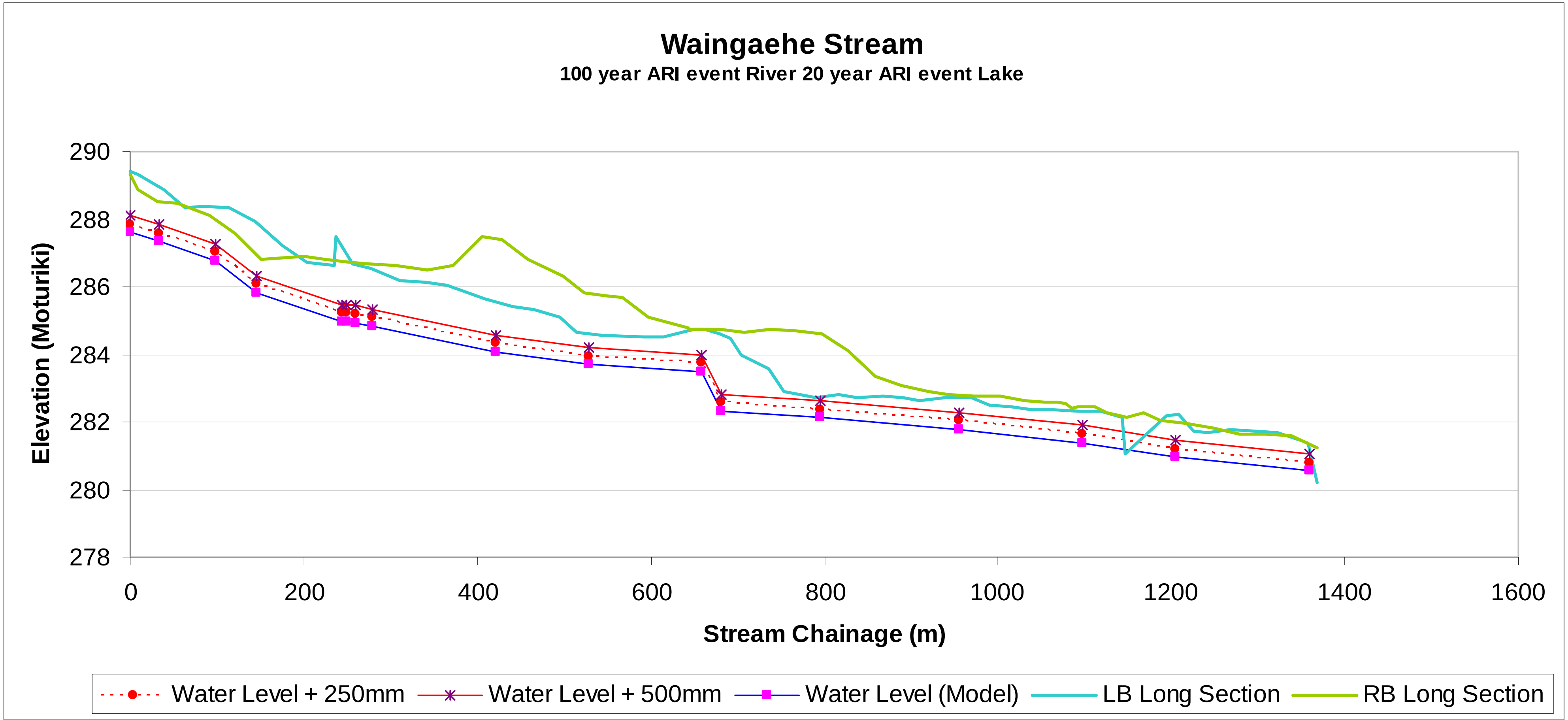

XS14 Downstream


XS15 Upstream


XS15 Downstream

Appendix 4 — Model Results for Q100 and L20 event combinations.

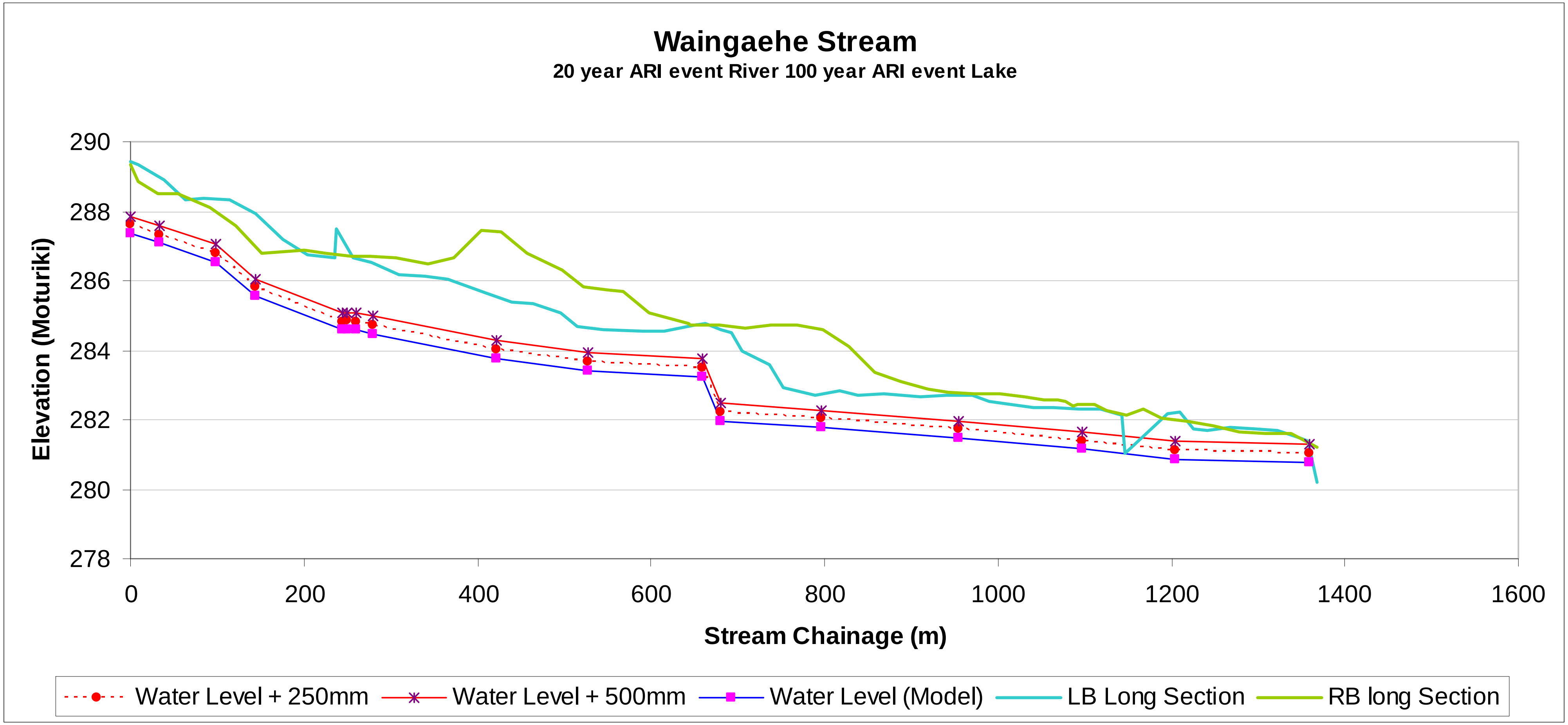
Mike 11 Cross Section	Mike 11 Chainage (m)	Left Bank Height	Right Bank Height	Maximum Water Level		
				Calculated	Water level + 250mm	Water level + 500mm
XS 15	0	288.53	288.91	287.607	287.857	288.107
XS 14	33	288.52	288.35	287.337	287.587	287.837
XS 13	99	288.09	288.23	286.78	287.03	287.28
XS 12	145	287.1	286.7	285.835	286.085	286.335
XS 11	244	286.36	287.37	284.964	285.214	285.464
XS 10.5 upstream	250	286.74	286.74	284.983	285.233	285.483
XS 10.5 downstream	260	286.66	286.66	284.941	285.191	285.441
XS 10	279	286.67	286.91	284.836	285.086	285.336
XS 9	422	285.19	285.3	284.077	284.327	284.577
XS 8	528	284.7	284.57	283.696	283.946	284.196
XS 7	659	284.34	284.61	283.486	283.736	283.986
XS 6	681	284.18	283.759	282.319	282.569	282.819
XS 5	796	282.747	282.738	282.117	282.367	282.617
XS 4	955	282.711	282.494	281.787	282.037	282.287
XS 3	1097	282.239	282.269	281.39	281.64	281.89
XS 2	1204	281.774	282.011	280.949	281.199	281.449
XS 1	1359	280.764	280.68	280.545	280.795	281.045



Waingaehe Stream
100 year ARI event River 20 year ARI event Lake
Elevation (Moturiki)
290
288
286
284
282
280
278
0
200
400
600
800
1000
1200
1400
1600
Stream Chainage (m)
Water Level + 250mm
Water Level + 500mm
Water Level (Model)
LB Long Section
RB Long Section

Appendix 5 — Model Results For Q20 and L100 Event Combinations

Mike 11 Cross Section	Mike 11 Chainage (m)	Left Bank Height	Right Bank Height	Maximum Water Level		
				Calculated	Water level + 250mm	Water level + 500mm
XS 15	0	288.53	288.91	287.361	287.611	287.861
XS 14	33	288.52	288.35	287.086	287.336	287.586
XS 13	99	288.09	288.23	286.549	286.799	287.049
XS 12	145	287.1	286.7	285.556	285.806	286.056
XS 11	244	286.36	287.37	284.576	284.826	285.076
XS 10.5 upstream	250	286.74	286.74	284.588	284.838	285.088
XS 10.5 downstream	260	286.66	286.66	284.572	284.822	285.072
XS 10	279	286.67	286.91	284.474	284.724	284.974
XS 9	422	285.19	285.3	283.777	284.027	284.277
XS 8	528	284.7	284.57	283.42	283.67	283.92
XS 7	659	284.34	284.61	283.249	283.499	283.749
XS 6	681	284.18	283.759	281.976	282.226	282.476
XS 5	796	282.747	282.738	281.775	282.025	282.275
XS 4	955	282.711	282.494	281.469	281.719	281.969
XS 3	1097	282.239	282.269	281.146	281.396	281.646
XS 2	1204	281.774	282.011	280.866	281.116	281.366
XS 1	1359	280.764	280.68	280.788	281.038	281.288


Waingaehe Stream
20 year ARI event River 100 year ARI event Lake
Elevation (Moturiki)
290
288
286
284
282
280
278
0
200
400
600
800
1000
1200
1400
1600
Stream Chainage (m)
Water Level + 250mm
Water Level + 500mm
Water Level (Model)
LB Long Section
RB long Section