Waioeka-Otara Rivers: Provisional Flood Frequency Analyses

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E mahi ngatahi e pai ake ai te taiao

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Executive Summary

This document describes the process of undertaking flood frequency analyses on the Waioeka-Otara and the Tutaetoko Rivers. In this review the Waioeka (at Cableway (Site No 15901) and Otara (at Browns Bridge, Site No 16002) rivers were assessed by means of statistical analyses of annual and biennial annual maxima from 40 to 50 years of gauged data.

The Tutaetoko catchment is part of the greater Otara catchment however it joins the Otara downstream of the Browns Bridge gauge. The relatively large size of the Tutaetoko catchment area in relation to that of the Otara's suggested that its flows were considered significant enough to warrant an independent investigation. Due to the absence of gauge data on the Tutaetoko River, the Regional and Transposition Area methods, based on the combination of estimates from other sites in the region, were adopted. The influence of Climate Change and the Interdecadal Pacific Oscillation (IPO) has also been taken into account in this assessment. The nature of the Waioeka-Otara catchments is such that Opotiki Township, located at the confluence of the Waioeka and Otara Rivers approximately 1km upstream of the coast, is exposed to the influence of both coastal and river hydrological dynamics. These independent yet highly variable natural systems make flood mitigation an interesting challenge in the region.

"…hazard management' has been no cure-all. The illusion of safety encourages further development, but is exposed as false security whenever particularly vigorous natural events occur. The cost of flood losses for example has continued to rise despite- or because of- investment in flood control schemes." (Mckinnon, 1997)

March 1964 – Flood in the Opotiki Township

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Chapter 1: Catchment Characteristics

1.1 Flood History

The Waioeka and Otara River Catchments and hence the Opotiki township, have been subject to several significant flood events since the towns inception 140 years ago. The Soil Conservation and Rivers Control Council Document: Floods in New Zealand (1957), gives a detailed account of many of these flood events. The most notable of which were the 1904, 1918 and1925 flood events. Significant events were also experienced in 1930, 1942, 1948, 1957 and 1958 (Wallace, 1999). The 1918 event was regarded as the largest in recorded History, prior to 1964 and the arrival of regular monitoring stations (Wallace, 1999). The 1964 flood event was the largest on recorded for both the Waioeka (1494 m³/s) and the Otara (984 m³/s) rivers at that time. To this day the 1964 event has not been surpassed on the Otara, however, a flood with a magnitude of 1521 m³/s, was experienced in July of 1998 on the Waioeka.

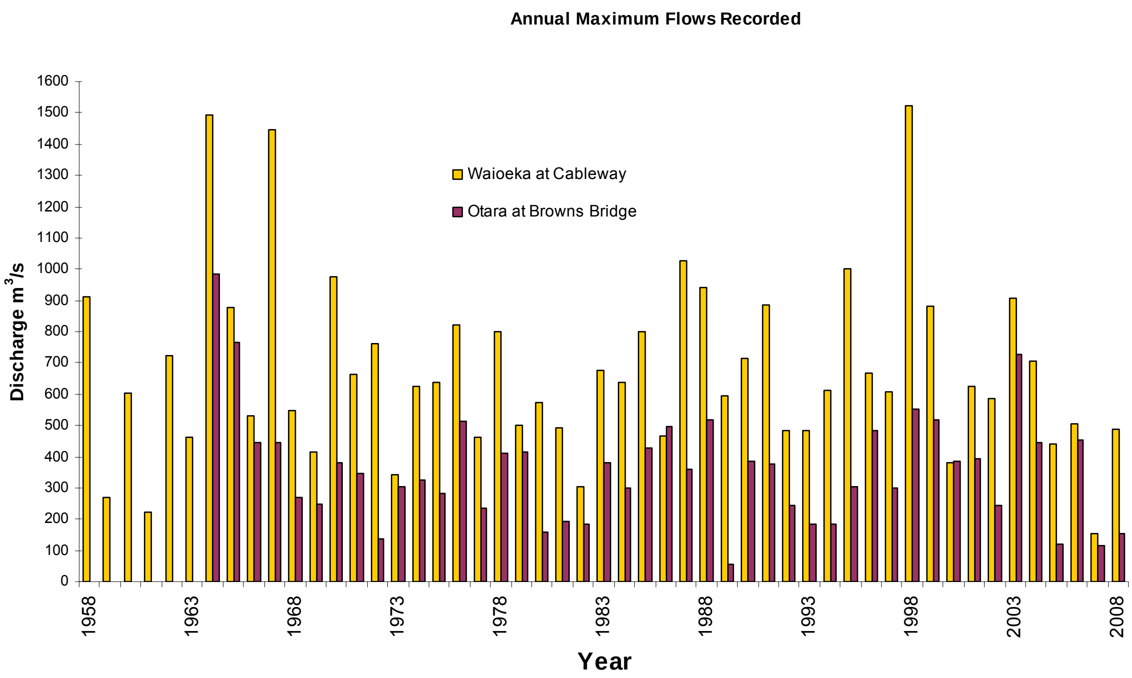

Figure 1 Gauged flood annual maxima in the Waioeka and Otara systems. Data from Tideda.

1.2 Catchment description

The Waioeka-Otara catchment covers an area of approximately 1,130km^2, much of which is steep and well forested. Roughly 70% of the catchment is covered by conservation estate in the form of the Urewera National Park and Waioeka Gorge Scenic Reserve. The Geology is characterised by alluvium on the floodplain, Kaharoa ash on the lower foothills and greywacke in the upper catchment.

Although there is some exotic pasture, both the Waioeka (upstream of the cableway recorder) and the Otara (upstream of the Browns Bridge recorder) are predominantly covered by scrub and indigenous forest. It is due to this forest cover that the catchments have a high capacity to absorb rainfall, delaying runoff and potential flooding downstream. The steep, rugged character of the catchments has the effect of increasing erosion, as is typified by the large number of slips evident on aerial photographs. Such slips have been associated with the development of temporary dams which, when breached, send a flood wave downstream. A 2008 field survey and a review of more recent aerial photography (2003 photography) confirmed that slips are still a frequent occurrence in both catchments.

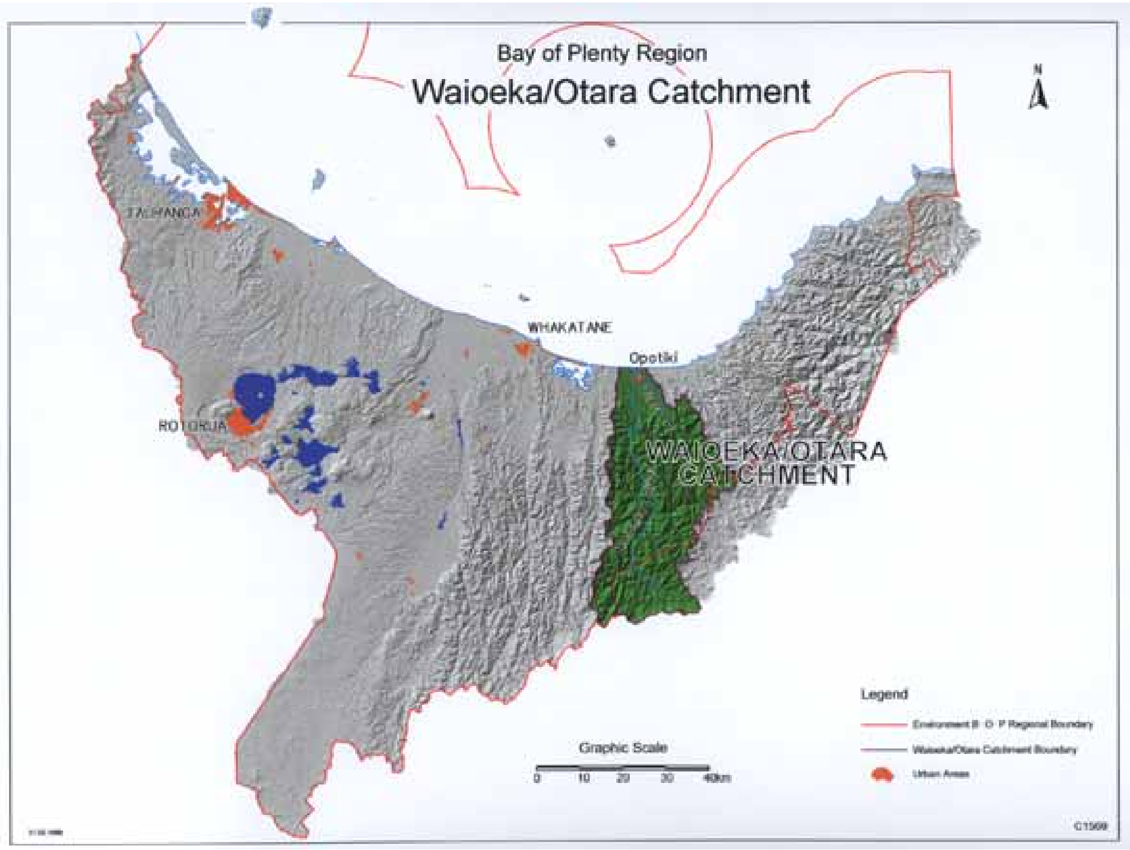

Figure 2 Waioeka-Otara Catchment location

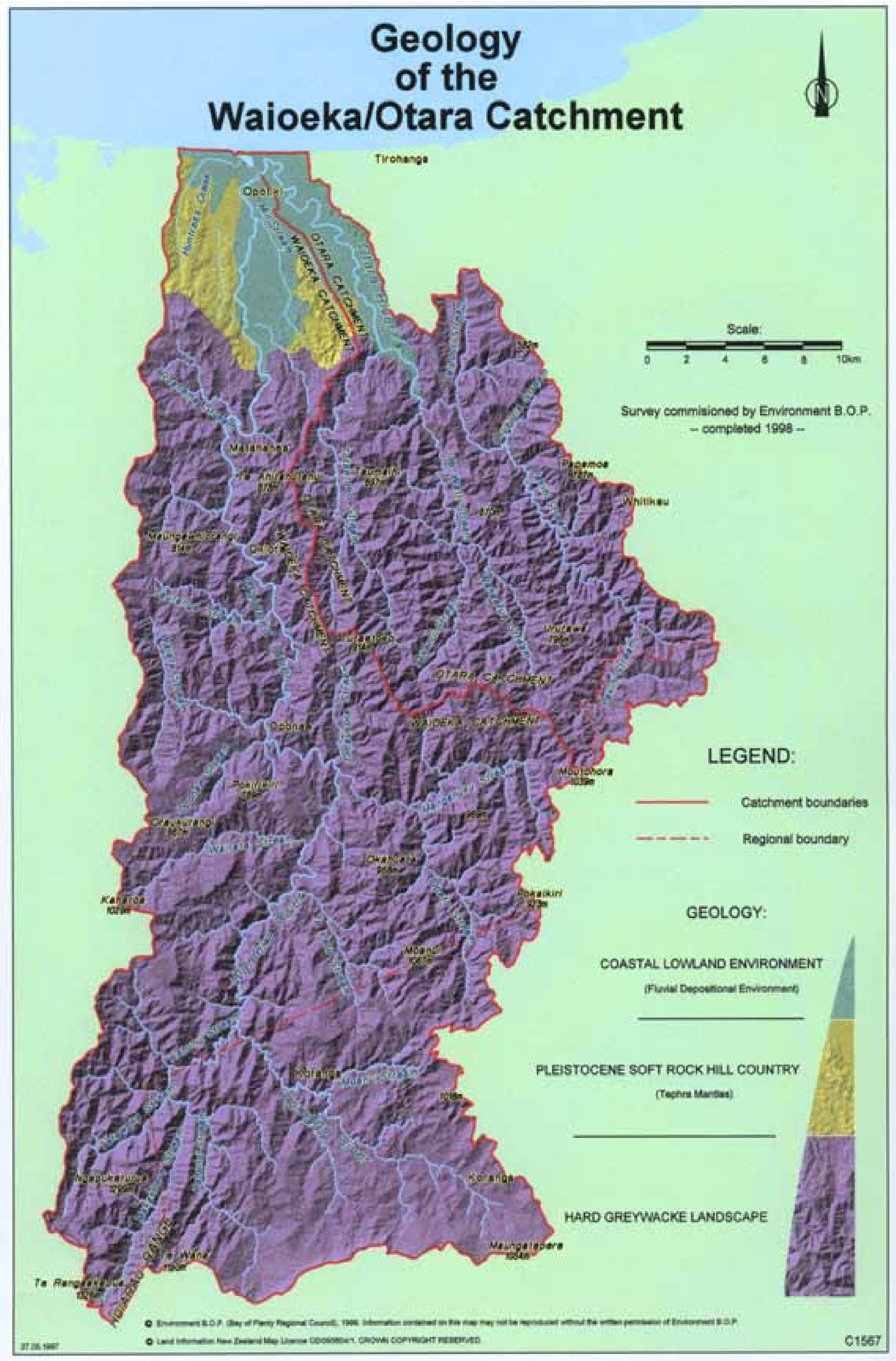

Figure 3 *Geology of the Waioeka-Otara Catchment*.

Figure 4 Waioeka-Otara Catchment land use.

Figure 4 *Flow Monitoring Sites in the Bay of Plenty. Waioeka at Cableway and Otara at Browns Bridge are numbers 31 and 32 respectively.*

Chapter 2: Historical Analyses

2.1 1966 Flood Frequency Analyses

Actual details of the design could not be found, however the ECCB (1989) report suggests that 1966 flows were generally 10% lower than the revised 1989 design flows discussed in section 2.2 below. These original design flows were based on only five years of recorded data and 20 to 30 years of rainfall records. The Otara design flows were based purely on the 20 to 30 year rainfall record. The original 1966 design curves were based on very limited hydrological data and therefore the results could be viewed as questionable. The short record also affected the annual exceedence methods used and caused difficulty in deriving reliable flood frequency curves, with often conflicting results.

Table 1 Results of the 1966 flood frequency analyses as per the ECCB (1988) report. Otara at scheme is assumed to be 117% of the flows at Browns Bridge.

Results of the 1966 flood frequency analyses		
Discharge (m³/s)		**Return Period**
Waioeka at Cableway	Otara at Scheme	(Years)
1300	675	30
1974	965	250

2.2 1989 Flood Frequency Analyses

The 1989 review, undertaken by Tichmarsh on behalf of the East Cape Catchment Board (see Tichmarsh B R, 1990) was based on 20 years of river stage records on the Waioeka River (Cableway) and 8 years staged records plus15 years annual maximum gauged flood data, on the Otara (Browns Bridge). Tichmarsh's analyses was two fold and focused on annual maxima and a full duration or event probability analyses including all floods. A regional Flood Frequency Estimation technique was used as a further check. The 1988 design curve was developed based on the comparison of the above methods, taking into account their relative strengths and weaknesses. The resulting design curve, used for extrapolating flood frequency, can be seen in Appendix 3. The details of the East Cape Catchment Board study can be found in Appendix 3.

Table 2 Results of the 1989 flood frequency analyses, based on the ECCB (1988) report and the probability graphs in Appendix 3.

Results of the 1989 flood frequency analyses		
Discharge (m³/s)		**Return Period**
Waioeka at Cableway	**Otara at Browns Bridge**	**(Years)**
3000	1550	1000
2680	1350	500
2370	1180	250
2280	1125	200
2000	980	100
1730	840	50
1540	750	30
1400	680	20
1200	560	10
990	460	5
730	340	2
560	260	1

2.3 1998 Flood Frequency Analyses

The 1998 review, was undertaken by Peter Blackwood on behalf of Environment Bay of Plenty. While the results of Peter's analyses are available unfortunately the methodology has not been formally documented. The methods described here have been put together on the basis of personal communications with Peter Blackwood and some results as documented by Phil Wallace. This method undertook statistical analyses of both annual and biennial maxima using the software package "FORTRAN". The results of which can be viewed in Appendix 4.

A "censored analyses" was undertaken on the highest flood flows in order to accurately represent these flows over the period for which they are know to be the highest (i.e. approx 30 years of recorded data but the highest flows are know to be the largest since 1918 or over approx 80 years.) It was suggested that the "censored analyses" would drop the 1998 curve, on the Otara, by about 20 cumecs. Further comment was made on the severity of the 1964 flood event, the confidence of its estimated discharge and its effect on frequency estimates, primarily on the Otara. This event is by far the largest event experienced on the Otara and has been included in Peter's frequency estimates on the basis that the period from 1980 – 1997 was considered a very benign period and removing it would provide unrealistically low results.

The results from the Biennial analysis were eventually adopted for the Waioeka but not on the Otara. Biennial analyses was justified on the basis that in certain hydrological situations the number of flood events per year is small and we are dealing with the distribution of extreme events from and array of floods (Blackwood Pers. com 2008). Analyses of the data suggested that this effect was more prevalent on the Waioeka and had little influence on the Otara. Consideration was also given to a strongly quiescent IPO (25 years inactive and 10 years active) which tends to underestimate less frequent flood flows. The results for both the Waioeka and the Otara Rivers can be seen in Table 3 below.

Table 3 Adopted results of the 1998 flood frequency analyses.

Results of the 1998 flood frequency analyses		
Discharge (m^3/s)		Return Period
Waioeka at Cableway	Otara at Browns Bridge	(Years)
2600	1300	500
2140	1062	200
1845	932	100
1583	812	50
1279	666	20
1075	562	10
904	463	5
656	327	2
494	197	1

Chapter 3: Data Capture

3.1 Available datasets

The following hydrological datasets are available for the Waioeka and Otara catchments:

- River level and river flow records for the Waioeka River at the Cableway site (Site No. 15901, located in the Gorge, just downstream of Oponae), from 1959 to present.
- River level records for the Waioeka River at the Mouth of Gorge (Site No 15912), from 1987 to present.
- River level and river flow records for the Otara River at Browns Bridge (site No.16002) from 1964 to present. Reliable stage records were only available from 1979 to present. The data record from 1964 to1979 was obtained from diary records of annual maximum peak discharge.
- River level records for the Otara River (Site No16007) at the wharf (near the Waioeka confluence) from 1991 to present.
- Rain gauge data is available for the Waioeka at Cableway (Site No 872301, 1990 to present), Otara at Browns Bridge (Site No 781410, 1990 to present) and Pakihi (Site No 872507, 1976 to present).

The most recent gauged dataset have been recorded at 15 minute intervals. Only the Waioeka at Cableway (Site No.15901) and the Otara at Browns Bridge (Site No.16002) records were used for flood frequency analysis.

3.2 Data integrity

3.2.1 Otara at Browns Bridge

As a consequence of an extremely active bedload, the Browns Bridge site (Site No. 16002) has been re-rated on several occasions. A consequence of this is an increase in potential error. A comparison between the 1998 hydrology and this analysis indicates some difference in the Tideda results for annual maxima. These differences are to be within 5% – 10% and do not appear to have a bearing on the less frequent more sever events.

Of some concern is the data from 1964 to 1979 as it is of unknown integrity and was based on the 1978 rating curve. The ECCB (1989) states the following: "Although the gaugings that are used to make up the ratings show a considerable degree of variance, it is felt that the current curves will not prove too much in error *as more data becomes available in the future. The ratings should be seen as best averages of loop curves, about which actual events will apply."* Considering the extreme nature of the events between 1964 and 1979 and a reliable 29 year stage record since 1979, it was considered prudent to include these earlier results in this analysis. In doing so consistency is maintained between the 1989, 1998 and 2008 hydrology reviews.

Analyses of the Tideda record (1979 to present) brought to light a number of incomplete annual series. Incomplete years were removed for this analysis if there was not sufficient evidence to suggest that a calendar year contained the annual maxima. 1981, 1987 and 1989 were removed on this basis. The presence or absence of the annual maxima flow was based on comparison with flows in the Motu (Site No 16501, 1957 - 2008), Waioeka (Site No, 15901, 1958 - 2008) and recorded rainfall at the Pakihi (Site No 871410, 1970 – 2008) station.

3.2.2 Waioeka at cableway

The accuracy of the Waioeka stage-time data at Cableway (Site No. 15901) is expected to be good. These records extend from 1958 to 2008, approximately 50 years' worth of data. This site is operated by NIWA and has undergone several confidence tests. 1968 and 1969 have been removed from the record due to a lack of confidence in annual maxima representation. This is based on a similar comparative process to that used on the Otara River, except comparison was made to the Motu River flow (Site No 16501, 1957 - 2008) and Pakihi rainfall (Site No 871410, 1970 – 2008) records only. It is worth mentioning that the events of 1964 and 1967 did exceed the recorder range, but in both cases accurate levels were obtained from marks left inside the recorder housing.

Chapter 4: Flood Frequency Analyses

4.1 Method

The methodology applied for this review has adopted a slightly different approach to that of the historical reviews described in chapter 2. This was mainly due to a change in the availability of previously used statistical software (FORTRAN), which is now out of date and could no longer operate on modern windows operating systems. In an attempt to maintain some consistency, and therefore comparability, similar techniques to those applied in the 1998 review have been applied as follows:

4.1.1 Waioeka and Otara design flows by Statistical analyses of gauged data using L-moments.

4.1.2 Tutaetoko design flow by Regional Analyses.

4.1.3 Tutaetoko design flow by transposition using the Area Method.

4.1.1 Statistical Analyses

Analyses of statistical gauged data involved plotting both calendar year annual maxima and bi-annual maxima from the historic record for each gauge. Event probability positions were plotted based on Gringoten formula as follows:

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

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. It must be made apparent that GEV distributions should only be considered suitable for flow records approaching 30 years or more. Exceedance probabilities for biennial (BEP) and annual (AEP) probabilities were plotted using the Gumbel reduced variate, described by Mckerchar and Pearson (1989) as follows:

- Annual Probability: $y(Q_i) = -\ln[-\ln(1-1/T)]$
- Biennial Probability: $y(Q_i) = -\ln[-2\ln(1-1/T)$

Where T is the desired recurrence interval.

Using this method we are able to relate both AEP and BEP by plotting them on the same distribution. The resulting Gumbel plots were assed by means of visual comparison of the extreme probability tail. These plots have been included in Appendix 1.

4.1.2 Regional analyses

The Tutaetoko Catchment, located on the Otara River, is the only ungauged catchment that offers a considerable ungauged discharge to the Otara River. This catchment covers an area 25% that of the Otara, upstream of Browns Bridge, and therefore contributes quite significantly to discharge estimates downstream of Browns Bridge. Previous analyses have estimated the discharge from this catchment at 17% of the discharge recorded on the Otara at Browns Bridge (Tichmarsh, 1990). The regional method, described by Mckerchar and Pearson (1989), has been applied to the Tutaetoko, Waioeka (at Cableway) and Otara (at Browns Bridge) Rivers, for comparative purposes.

The regional method estimates flow quantiles by combining estimates of annual flood peak from other sites in the region. This method is based on maps of specific discharge and flood frequency factors, generated from 343 estimates of annual maxima through out New Zealand. An estimation of discharge for an ungauged catchment is obtained as follows:

$$Qt = (\tilde{Q} * A^{0.808}) \times q_{100}$$

Where Qt is the estimated discharge for the 100 year recurrence interval, $\tilde{Q}$ and q_{100} are the specific discharge and flood frequency factors respectively, estimated from the maps provided in McKerchar and Pearson (1989). This method can be applied to floods other than the 100 year flood in the following manner:

$$\text{Discharge } (Q_t) = \tilde{\tilde{Q}}[xt + (1 - xt)q100]$$

Where xt is expressed by the following formula:

$$xt = 1.1435 - 0.2486yt$$

Where yt is the Gumbel reduced variant expressed by the following formula:

$$yt = -\ln[-\ln(1-1/T)]$$

T is the desired recurrence interval.

This method was considered appropriate as it is applicable to catchments in which the following factors are not a contributing factor; snow melt, glaciers, lake storage, ponding or urban development. Both the Waioeka and the Otara catchments do not express any of these characteristics. Detailed calculation spreadsheets can be viewed in Appendix 2.

4.1.3 Method of transposition by area

Estimates for the Tutaetoko Stream design flows were also made by transposing flood characteristics from the Waioeka and Otara flood magnitudes derived from statistical methods. This method assumes that the corresponding peak flow is proportional to the catchment area raised to the power of 0.808:

$$Q_{p1} = Q_{p2} \times (A1 / A2)^{0.808}$$

Where Qp1 is the estimated discharge of the desired catchment, Q_p2 is the known discharge of the transposed catchment. A_1 is the area of Q_p1 and A_2 is the area of Q_p2.

Chapter 5: Climatic Variability

5.1 Inter-decadal Pacific Oscillation (IPO)

The IPO can be described as a climatic effect with a temporal pattern spanning decades. Its influence is felt in the majority of the Pacific region. The IPO has the effect of accentuating or curtailing the impacts of ENSO (El Nino Southern Oscillation) events. This is dependent on whether it is in a positive or a negative phase which, the IPO oscillates between approximately every 20-30 years. Three phases have been identified since the 1920's: 1922-1940 (positive), 1946-1977 (negative) and 1978 to 1998 (positive). In Bay of Plenty a positive phase is generally associated with a reduced occurrence of flood events while the negative phase tends to encourage extreme events. This is consistent with events experienced on the Waioeka and the Otara Rivers. The 1960's and early 2000's have experienced several flood events while the period from 1977 to 1998 was relatively benign. (MFE, 2008)These effects have been taken into account in this review.

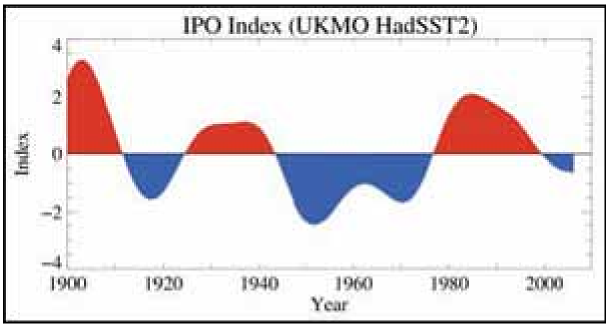

Figure 5 The IPO index based on sea-surface temperatures (SST). The IPO is estimated to change phase every 20 – 30 years (MFE, 2008).

5.2 The influence of Climate Change

Under the Local Government Act 2002 and the Resource Management Act, regional councils are responsible for the management of regional water, air and land resources. As a result regional councils have a duty to avoid, remedy or mitigate adverse effects associate with climate change. Climate change impacts can and should be taken into account when contemplating new activities and development. The Ministry for the Environment (MFE, 2008) states that the risks associated with climate change are not new, however they may change the frequency and intensity of existing climatic events. Considering the potential impact of climate change on regional council functions and services it is prudent to include its effects in this review.

The best estimate of climate change in the Bay of Plenty according to the MFE (2008) document is as follows:

Table 4 Projected changes in seasonal and annual mean temperature (in ^{o}C) relative to 1990, for the Bay of Plenty (MFE, 2008). The first number is the mid range estimate of what the change will be. The figures in the brackets provide the modal range within which change could lie.

Decade	**Mean annual temperature change ^{o}C relative to 1990**				
	Summer	Autumn	Winter	Spring	Annual
2040	1.0 [0.3, 2.5]	1.0 [0.3, 2.7]	0.9 [0.1, 2.2]	0.8 [0.0, 2.1]	0.9 [0.2, 2.4]
2090	2.2 [0.8, 6.2]	2.2 [0.6, 5.6]	2.0 [0.5, 5.2]	1.8 [0.3, 5.1]	2.1 [0.6, 5.5]

Table 5 Factors for use in deriving extreme rainfall information for preliminary assessment scenarios from MFE (2008). (ARI = Average Recurrence Interval).

ARI (years)	**Factors used in deriving extreme rainfall (% change)**										
		Duration									
	<10 min	10 min	30 min	1 hr	2 hrs	3 hrs	6 hrs	12 hrs	24 hrs	48 hrs	72 hrs
2	8	8	7.2	6.7	6.2	5.9	5.3	4.8	4.3	3.8	3.5
5	8	8	7.4	7.1	6.7	6.5	6.1	5.8	5.4	5	4.8
10	8	8	7.6	7.4	7.2	7	6.8	6.5	6.3	6.1	5.9
20	8	8	7.8	7.7	7.6	7.5	7.4	7.3	7.2	7.1	7
30	8	8	8	8	8	8	8	8	8	7.8	7.7
50	8	8	8	8	8	8	8	8	8	8	8
100	8	8	8	8	8	8	8	8	8	8	8

Tables 4 and 5 above, recommend percentage change adjustments to extreme rainfall per degree Celsius of warning e.g. for the 100 year ARI to 2090, a 2.2^{o}C temperature change in summer would result in a 17.6% change in extreme rainfall intensity. The climate change impacts and assessments presented here are subject to a large amount of uncertainty. The Ministry for the Environment recommend using these figures as a guide only. This has been taken into account with regard to estimating ARI on the Waioeka and Otara Rivers. For further details of this assessment see the MFE (2008) document *"Preparing for Climate Change: A Guide for Local Government".*

Chapter 6: Results

6.1 Statistical Analyses

The results of the statistical analyses provide a range of estimates for ARI in both the Waioeka and the Otara Rivers. The methods used reflect a variation in the region of 5% - 10% and have been assessed on their relative strengths and weaknesses. The resulting design estimates are provided below.

6.1.1 Waioeka Design Flows

The Waioeka Design flows illustrate a slight deviation from results recommended by the 1998 review. Table 6 provides a summary of these results for all methods.

Table 6 Summary of design flows for the Waioeka River at Cableway.

	Annual Analyses		Biennial Analyses	
ARI Event	**EV1 (m^3/s)**	**GEV (m^3/s)**	**EV1 (m^3/s)**	**GEV (m^3/s)**
2.33	700	706	686	678
5	910	914	894	868
10	1080	1080	1062	1040
20	1244	1236	1224	1221
50	1457	1431	1434	1480
100	1616	1574	1591	1695
200	1774	1714	1747	1928
500	1983	1893	1954	2268

The recommended design flows illustrated below have been based on the GEV (General Extreme Value) Distribution from a biennial data series. A detailed description of these and other results, for the Waioeka Rivers can be found in Appendix 1. A biennial GEV distribution was adopted through analyses of the goodness of fit, for the fitted extreme values distribution curves, for both the annual and the biennial distribution curves. The Waioeka distribution contains three extremes which are documented to be the largest flood since 1918. Therefore it was considered statistically correct to plot these three highest flood flows over the time period in which they are known to be the highest i.e. 89 years instead of 50 years. This resulted in a better fit with flows ranked two and three but not rank one. Consequentially the “censored analyses” was disregarded and the results from the GEV Distribution were considered most suitable.

Recommended design flows for the Waioeka River at Cableway are as follows:

Table 7 Design Flow Estimates for the Waioeka River at Cableway.

ARI Event	Discharge m³/s
2.33	678
5	868
10	1040
20	1221
50	1480
100	1695
200	1928
500	2268

A degree of conservatism was applied in this selection due to a considerable reduction in results between the 1998 flood frequency analyses and these results. An analysis of the 1998 results suggests that the large difference could be as a consequence of incorrectly plotting biennial data using an annual plotting position. Personal communication with Peter Blackwood have suggested that this is not so. Replication of the biennial 1998 review, using the latest Tideda data up to 1998, indicated a 100 year ARI event discharge of 1808m³/s (GEV, 2008) in comparison to 1845m³/s (Pearson III, 1998). GEV and Pearson III distributions do tend to produce similar results hence direct comparison of the two results was considered suitable and the resulting 2% difference is not considered significant. The reduction in discharge between 1998 and 2008 for the 100 year ARI event seems to illustrate a flattening of the distribution curve as a consequence of a longer gauged record and the effects of a censored analysis.

6.1.2 Otara design flows

Similar criteria to that used in the Waioeka were subsequently applied to Otara River at Browns Bridge. Table 8 provides a summary of these results.

Table 8 Summary of design flows for the Otara River at Browns Bridge.

	Annual Analyses		Biennial Analyses	
ARI Event	EV1 (m³/s)	GEV (m³/s)	EV1 (m³/s)	GEV (m³/s)
2.33	370	373	384	376
5	498	501	500	472
10	602	602	595	568
20	702	696	686	677
50	831	816	803	848
100	928	903	891	1003
200	1024	988	979	1184
500	1152	1097	1095	1473

The recommended design flows illustrated below have been based on the EV1 (Extreme Value 1) Distribution from an annual data series. A detailed description of these and other results, for the Waioeka Rivers can be found in appendix 1. An annual EV1 distribution was adopted through analyses of the "goodness of fit", for the fitted extreme values distribution curves, for both the annual and the biennial distribution curves. The Otara distribution is known to contain one record, the most extreme event, which is document as being the largest since 1918.

Therefore it was considered statistically correct to plot this discharge over the time period in which it is know to be the highest i.e.89 years rather then 44 years. This resulted in better fit from the annual EV1 distribution.

Table 9 Design Flows for the Otara River at Browns Bridge

ARI Event	Discharge m^3/s
2.33	370
5	498
10	602
20	702
50	831
100	928
200	1024
500	1152

These results illustrate a marginal drop ($4m^3/s$) in flood frequency when compared to the 1998 analyses.

6.2 Regional Method and Method of Transposition by Area

Previous analyses have estimated the Tutaetoko catchment at 17% of Otara at Browns Bridge. In order to test this assumption, the regional method (Mckerchar and Pearson, 1989) and the method of transposition by area has been applied. Further details of these methods can be found in 4.1.3 and 4.1.2. Table 10 illustrates the results from these methods.

Table 10 Summary of Design Flow estimates for the Tutaetoko River.

River	Method	Area	ARI event and Resulting discharge (m^3/s)					
			100	**50**	**20**	**10**	**5**	**2.33**
Tutaetoko	Regional	60	227	204	174	150	125	98
Tutaetoko (Waioeka at Cableway)	$Area^{0.8}$	60	244	213	176	150	125	98
Tutaetoko (Otara at Browns Bridge)	$Area^{0.8}$	60	303	271	229	196	162	121
Tutaetoko	17% Browns Bridge	60	158	141	119	102	85	63
Mean of Area methods			273	242	202	173	144	109

The results from Table 10 above suggest that previous estimates of discharge for the Tutaetoko catchment (17% of Browns Bridge) may have been underestimations. The Tutaetoko catchment is located directly between the Waioeka and the Otara catchments. As a result you would expect the area method to provide a reasonable estimate of discharge. Interestingly, when the Regional method is applied to the Otara catchment, discharge in the Tutaetoko is proportional to area and is in the region of 30% that of Browns Bridge. Appendix 2 provides further details of these methods and results. Recommended discharge for the Tutaetoko catchment has been based on the mean of the results from the Area and regional methods.

Table 11 Estimated Design Flows for the Tutaetoko River at the intersection with the Otara River.

ARI Event	Discharge m³/s
2.33	109
5	144
10	173
20	202
50	242
100	273

6.3 IPO and Climate Change Results

The data available for the Waioeka and the Otara spans a 40 to 50 year period from 1958 to 2008. During this period the Bay of Plenty has experienced a similar length of positive and negative IPO expression. It is therefore expected that the effects of the IPO, on flood frequency estimates, have been more than likely been minimised for the overall duration of this study i.e. the results represent neither an overestimate or underestimate.

While the influence of climate change is anticipated to increase design rainfall intensity and possibly discharge, the interval between hydrologic reviews was considered far shorter than those time-frames relating to climate change. The results from this analysis are included for completeness sake and have not had a bearing on the final design flows.

Table 12 Estimates of climate change and its affects on the Waioeka and Otara Rivers for 2040 and 2090. Scenarios are based on methods described in section 5.2.

				AEP and Resulting discharge (m³/s)						
Scenario	**Decade**	**% Increase**	**Method**	**River**	**1%**	**2%**	**5%**	**10%**	**20%**	**50%**
Excluding	2008	0.000	L Moments	Waioeka	1695	1480	1221	1040	868	678
Including + 6.4%	2040	0.064		Waioeka	1803	1575	1299	1107	924	721
Including + 8%	2040	0.080		Waioeka	1831	1598	1319	1123	937	732
Including + 14.4%	2090	0.144		Waioeka	1939	1693	1397	1190	993	776
Including + 17.6%	2090	0.176		Waioeka	1993	1740	1436	1223	1021	797
Excluding	2008	0.000	L Moments	Otara	928	831	702	602	498	370
Including + 6.4%	2040	0.064		Otara	987	884	747	640	529	393
Including + 8%	2040	0.080		Otara	1002	897	758	650	537	399
Including + 14.4%	2090	0.144		Otara	1062	951	803	688	569	423
Including + 17.6%	2090	0.176		Otara	1091	977	825	708	585	435

The results in table 12 above illustrate the maximum and minimum amount of change expected to both 2040 and 2090 as per the MFE, (2008) document, *"Preparing for Climate Change, A guide for Local Government".* The purpose of running both a minimum and maximum influence is in order to determine the sensitivity of these rivers to climate change scenarios. It is recommended that the same scenarios be applied to the Tutaetoko estimates should climate change scenarios be required for this river.

Chapter 7: Discussion

A variety of methods have been applied in the process of estimating flood frequency or ARI for the Waioeka, Otara and Tutaetoko Catchments. The discussion presented here has attempted to account for the relevant strengths and weaknesses of all methods applied. Design recommendations have been based on those results most suitable to the individual catchments. Considering the nature of the urban environment i.e. Opotiki Township, that Environment Bay of Plenty protects, it is considered prudent to adopt a conservative approach that is guided by the results presented in Chapter 6.

The recommended biennial GEV Waioeka design flows at Cableway reflect approximately a 10% reduction in flow predictions since 1998. Applying a “goodness of fit” methodology to the relative results (see appendix 1) of the four methods, suggests that there is no clear distinction between them i.e. all distributions appear to provide a suitable fit when compared to the plotted recorder data. The suitability of one distribution over another was based purely on the “goodness of fit” for the floods at the upper extreme. This approach brings the relevance of the top three floods into discussion. A “non-censored” approach suggests a subtle flattening of the distribution curves when compared to the 1998 results and would tend to support the selection of a biennial GEV distribution. This effect was somewhat predicted by Tichmarsh in his 1989 report. These three highest floods are known to be the largest since 1918 and it was felt that a “censored analyses” would provide more realistic results. This involved plotting of the top three floods over 89 years instead of 50 years and resulted in the recorded data resembling an EV3 distribution. The censored analysis does provide a better fit for the second and third highest flows in the record, although the highest flow appears as an outlier and is responsible for the apparent EV3 distribution. This result is considered highly unlikely for the Bay of Plenty, in which EV2 distributions appear to be more commonplace (P. Blackwood 2008, pers.com). Consequentially the “censored analyses” was disregarded and the biennial GEV distribution was selected on the basis that it provided a better fit, a conservative reduction from the 1998 review and applied comparable methodology to that of the 1998 review. The 10% reduction in discharge for the 1% ARI appears to be consistent with the analyses of a longer data series, when compared to 1998.

Biennial analysis is generally considered appropriate in catchments which exhibit infrequency of flood events i.e. we are examining the frequency of extreme events and biennial analyses allows for the exclusion of non-extreme events. Analyses of the Waioeka and Otara Rivers rainfall records suggest that this can be the case on the East Cape of the Bay of Plenty. Consequently biennial analysis encourages a better fitting EV1 distribution due to the presence of enough independent flood events. This method also encourages a better fit at the extreme end of the distribution curve, as was apparent in the Waioeka. Results from the Waioeka analyses suggested a marginally better EV1 distribution fit using biennial analyses; however results for the Otara showed little improvement, especially at the extreme end of the distribution curve. Biennial analysis was therefore not adopted in the Otara River and the results from the annual EV1 distribution are recommended. Plotting the highest recorded flow on the Otara over the number of years for which it is known to be the highest i.e. a “censored analyses” also, had the effect of improving the fit of the EV1 distribution.

Although this “censored analyses” was disregarded on the Waioeka, plotting extreme flows over the time period for which they are known have occurred provides improved statistical accuracy and improves accuracy of estimates at the upper extremes of the probability curve.

Results from a regional analysis of the Tutaetoko Catchment indicate that the original estimate of 17% the Otara discharge at Browns Bridge was probably an underestimation. This was indicated by both the regional method and the method of transposition by area. The method of transposition by area is based on the statistical discharge estimates in both the Waioeka and the Otara Rivers. Due to its geographic location, between the Waioeka and the Otara River, relative catchment uniformity between all three catchments and the considerable length of statistical record, the area method is expected to provide reasonable results. The mean of both methods was eventually adopted as the best estimate and represents a 43% increase from previous estimates.

The influence of climatic variability i.e. IPO and Climate Change effects have been considered for both the Waioeka and the Otara Rivers. Recommendations provided by the Ministry for the Environment, suggest that the Waioeka and Otara Catchments have experienced approximately equivalent benign and active phases of the IPO to 2008. The resulting data series is therefore not expected to bias the statistical results from this analysis in any particular direction. Ironically the 1998 review was expected to reflect a bias toward a benign IPO and a slight reduction in expected discharge. The results of this analysis may suggest that the IPO may not be as influential as previously expected. In order to estimate the sensitivity of flood frequency to IPO influences in the Bay of Plenty, it may be useful to undertake and make comparison between flood frequency analyses during positive and negative phases of the IPO respectively. Time constraints have not allowed for such detailed analyses at this time.

Dr Andy Resinger from MFE has advised Environment Bay of Plenty that the frequency of floods of a particular size could increase between zero and four-fold by the year 2070. The implications are that the 400-year flood could become the 100 year flood (Waugh, 2008). Appendix 1 provides linearly extrapolated estimates of possible 200 and 500 year flood events for the Waioeka and Otara Rivers. When compared to the results in section 6.3 these estimates appear consistent with a mean approximation of Dr Resinger’s comments i.e. the100 year to 2090 including climate change was 1993 m^3/s compared to the existing 200 year of 1926 m^3/s. The 200 year event has becomes a 100 year event by 2090. The length of time for which Environment Bay of Plenty has recorded data on the Waioeka-Otara Catchments (50 years of data) and the relative uncertainty of climate change predictions make confident estimates of such extreme events difficult to achieve. Consequentially the application of the climate change estimates is situation dependent but is recommended for longer term construction projects. Climate change has not been included in the recommended ARI events from this analysis although estimates of its effect on these recommended flows can be viewed in section 6.3.

Chapter 8: Recommendations

The following recommendations apply to estimates of flood frequency or ARI for the Waioeka, Otara and Tutaetoko Catchments.

- Section 6.1.1, Table 7 should be applied as the design estimates of ARI for the Waioeka River at Cableway (Site No 15901).
- Section 6.1.2 Table 9 should be applied as the design estimates of ARI for the Otara River at Browns Bridge (Site No 16002).
- Section 6.2 Table 11 should be applied as design estimates of ARI for the Tutaetoko River at the confluence with the Otara located approximately 500m downstream of Browns Bridge (Site No 16002).
- The effects of climate change have not been applied to the above estimates. It is recommended that climate change estimates are applied in situations where the nature of the proposed works are not easily retrofitted or are not reviewed within the MFE recommended 2040 or 2090 climate change time frames e.g. road bridges.

Chapter 9: References

Wallace P L, October 1999, Towards a Waioeka-Otara Floodplain Management Strategy, Review of the Current Flood Hazard and Existing Responses. Environment Bay of Plenty Operations Report 99/05.

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Mckerchar A.I. and Pearson C.P. 1989, Flood Frequency in New Zealand, Publication of the Hydrological Centre, Christchurch.

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Hall . G.W., April 1988, Engineering Technical Report 1988/2 : "Waioeka-Otara Catchment Management Study : Part 1 Hydrological and Statistical Aspects" Environment Bay of Plenty library No. 09780

Waugh. R. April 2008, Waioeka-Otara Asset Management Plan, Environment Bay of Plenty Operations Publication 2008/04.

Appendices

Appendix 1 – Calculation Spreadsheet used for design flows by various methods for the Waioeka and Otara Rivers

L Moments from Tideda
Formula from Hosking 1990

For Site:
Waioeka at Cableway (15901)

Mean	Std. Dev.	Skew	Kurtosis	
L1	L2	T3	T4	Method
699.375	157.867	0.1499	0.2121	Annual
841.88	155.91	0.2517	0.1729	Biennial

For EV1

		Annual	Biennial
$\alpha = \dfrac{l_2}{\ln 2}$	=	227.754	224.931
$\xi = l_1 - 0.5772\,\alpha$	=	567.915	712.050

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

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

For GEV

$z = 2/(3+t_3) - \log 2/\log 3$	=	0.004011038	-0.015866863
$k = 7.8590z + 2.9554z^2$	=	0.031570297	-0.123953636
$\alpha = l_2 k/\{(1-2^{-k})\Gamma(1+k)\}$	=	234.3001022	197.8495139
$\xi = l_1 + \alpha\{\Gamma(1+k)-1\}/k$	=	571.2443811	700.2414307

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

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

Statistical Flood Frequency Analyses on the Waioeka and Otara Rivers

Return Period	Plotting Position		Annual analyses		Biennial analyses	
			EV1	GEV	EV1	GEV
T	YT(Annual)	YT (Biennial)	QT1	QT2	QT1	QT2
1.01	-1.53	-2.22	220	204	212	316
1.1	-0.87	-1.57	369	363	359	418
1.5	-0.09	-0.79	546	549	535	552
2	0.37	-0.33	651	657	639	637
2.33	0.58	-0.11	700	706	686	678
5	1.50	0.81	910	914	894	868
10	2.25	1.56	1080	1080	1062	1040
20	2.97	2.28	1244	1236	1224	1221
50	3.90	3.21	1457	1431	1434	1480
100	4.60	3.91	1616	1574	1591	1695
200	5.30	4.60	1774	1714	1747	1928
500	6.21	5.52	1983	1893	1954	2268

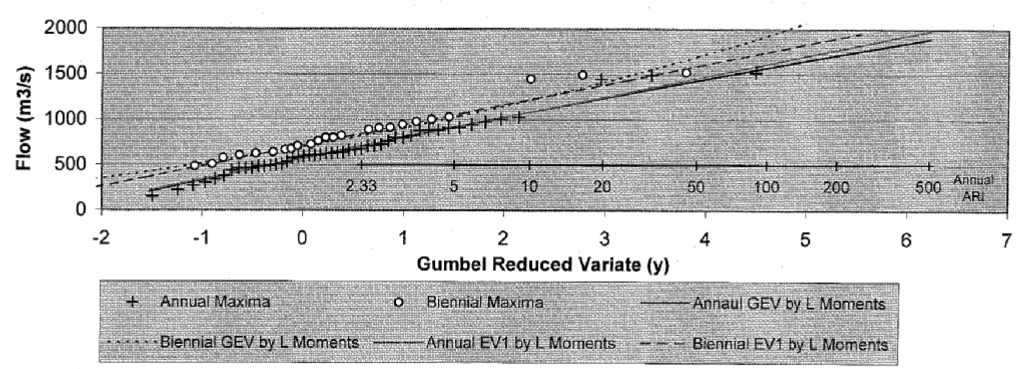
Recommended Design Flows for the Waioeka at Cableway

ARI Event	Discharge m^3/s
2.33	678
5	868
10	1040
20	1221
50	1480
100	1695
200	1928
500	2268

Plotting Position

n= 50			Annual	n= 24	Biennial
i	Sample Values	F(Qi)	y(Qi)	Sample Values	y(Qi)
1	154	0.0111732	-1.502796	484	-1.07627756
2	221	0.0311263	-1.244078	506	-0.90514249
3	269	0.0510774	-1.090047	575	-0.795189179
4	304	0.0710295	-0.972542	605	-0.632264332
5	342	0.0909816	-0.874259	623	-0.46780334
6	379	0.1109338	-0.787922	639	-0.297718786
7	440	0.1308859	-0.709724	667	-0.179074096
8	463	0.150838	-0.637396	675	-0.117628344
9	464	0.1707901	-0.569464	706	-0.054484462
10	465	0.1907422	-0.504908	725	0.07789747
11	483	0.2106943	-0.442984	762	0.147721683
12	484	0.2306464	-0.38313	799	0.220434104
13	492	0.2505986	-0.324908	800	0.296458645
14	500	0.2705507	-0.267961	821	0.376298462
15	506	0.2905028	-0.211995	887	0.645482753
16	532	0.3104549	-0.156762	907	0.748230286
17	575	0.330407	-0.102042	911	0.859731914
18	586	0.3503591	-0.047643	943	0.981992789
19	594	0.3703113	0.0066104	976	1.117762784
20	605	0.3902634	0.0608828	1002	1.270961742
21	606	0.4102155	0.1153293	1026	1.447459086
22	610	0.4301676	0.1701001	1447	2.255166188
23	623	0.4501197	0.2253438	1494	2.760818796
24	625	0.4700718	0.28121	1521	3.795478668
25	636	0.4900239	0.3378518	Historic Plotting Position n=89	
26	639	0.5099761	0.3954284	1447	2.842
27	662	0.5299282	0.4541085	1494	3.343
28	667	0.5498803	0.5140731	1521	4.374
29	675	0.5698324	0.5755188		
30	706	0.5897845	0.6386627		
31	714	0.6097366	0.7037468		
32	725	0.6296887	0.7710447		
33	762	0.6496409	0.8408689		
34	799	0.669593	0.9135813		
35	800	0.6895451	0.9896058		
36	821	0.7094972	1.0694456		
37	876	0.7294493	1.1537063		
38	882	0.7494014	1.2431279		
39	887	0.7693536	1.3386299		
40	907	0.7893057	1.4413775		
41	911	0.8092578	1.5528791		
42	943	0.8292099	1.67514		
43	976	0.849162	1.81091		
44	1002	0.8691141	1.9641089		
45	1026	0.8890662	2.1406063		
48	1447	0.9489226	2.9483134		
49	1494	0.9688747	3.453966		
50	1521	0.9888268	4.4886258		
Historic Plotting Position			n=89		
87	1447	0.9712747	3.5354389		
88	1494	0.9824955	4.0364812		
89	1521	0.9937163	5.0666522		

Fitted Extreme Value Distribution of Annual Flow Maxima for Waioeka at Cableway (1958-2007)



L Moments from Tideda
Formula from Hosking 1990

For Site: Waioeka at Cableway (15901)

Mean	Std. Dev.	Skew	Kurtosis	
L1	L2	T3	T4	Method
699.375	157.867	0.1499	0.2121	Annual
841.88	155.91	0.2517	0.1729	Biennial

For EV1

		Annual	Biennial
$\alpha = \frac{l_2}{\ln 2}$	=	227.754	224.931
$\xi = l_1 - 0.5772\,\alpha$	=	567.915	712.050

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

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

For GEV

$z = 2/(3+t_3) - \log 2/\log 3$	=	0.004011038	-0.015866863
$k = 7.8590z + 2.9554z^2$	=	0.031570297	-0.123953636
$\alpha = l_2 k / \{(1-2^{-k})\Gamma(1+k)\}$	=	234.3001022	197.8495139
$\xi = l_1 + \alpha\{\Gamma(1+k)-1\}/k$	=	571.2443811	700.2414307

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

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

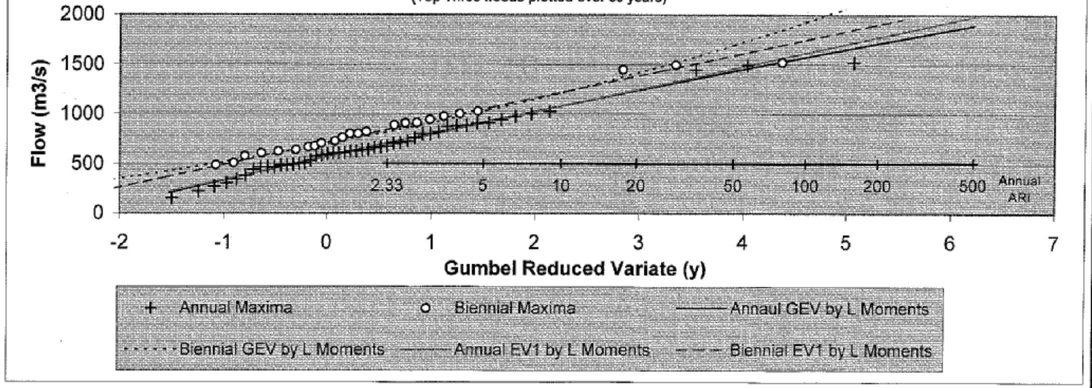
Statistical Flood Frequency Analyses on the Waioeka and Otara Rivers

Return Period	Plotting Position		Annual analyses		Biennial analyses	
			EV1	GEV	EV1	GEV
T	YT(Annual)	YT (Biennial)	QT1	QT2	QT1	QT2
1.01	-1.53	-2.22	220	204	212	316
1.1	-0.87	-1.57	369	363	359	418
1.5	-0.09	-0.79	546	549	535	552
2	0.37	-0.33	651	657	639	637
2.33	0.58	-0.11	700	706	686	678
5	1.50	0.81	910	914	894	868
10	2.25	1.56	1080	1080	1062	1040
20	2.97	2.28	1244	1236	1224	1221
50	3.90	3.21	1457	1431	1434	1480
100	4.60	3.91	1616	1574	1591	1695
200	5.30	4.60	1774	1714	1747	1928
500	6.21	5.52	1983	1893	1954	2268

Recommended Design Flows for the Waioeka at Cableway

ARI Event	Discharge m^3/s
2.33	678
5	868
10	1040
20	1221
50	1480
100	1695
200	1928
500	2268

Fitted Extreme Value Distribution of Annual Flow Maxima, for Waioeka at Cableway (1958-2007).
(Top Three floods plotted over 89 years)



Plotting Position

n= 50			Annual	n= 24	Biennial
i	Sample Values	F(Qi)	y(Qi)	Sample Values	y(Qi)
1	154	0.0111732	-1.502796	484	-1.07627756
2	221	0.0311253	-1.244078	506	-0.90514249
3	269	0.0510774	-1.090047	575	-0.795189179
4	304	0.0710295	-0.972542	605	-0.632264332
5	342	0.0909816	-0.874259	623	-0.46780334
6	379	0.1109338	-0.787922	639	-0.297718786
7	440	0.1308859	-0.709724	667	-0.179074096
8	463	0.150838	-0.637396	675	-0.117628344
9	464	0.1707901	-0.569464	706	-0.054484462
10	465	0.1907422	-0.504908	725	0.07789747
11	483	0.2106943	-0.442984	762	0.147721683
12	484	0.2306464	-0.38313	799	0.220434104
13	492	0.2505986	-0.324908	800	0.296458645
14	500	0.2705507	-0.267961	821	0.376298462
15	506	0.2905028	-0.211995	887	0.645482753
16	532	0.3104549	-0.156762	907	0.748230286
17	575	0.330407	-0.102042	911	0.859731914
18	586	0.3503591	-0.047643	943	0.981992789
19	594	0.3703113	0.0066104	976	1.117762784
20	605	0.3902634	0.0608828	1002	1.270961742
21	606	0.4102155	0.1153293	1026	1.447459086
22	610	0.4301676	0.1701001	1447	2.255166188
23	623	0.4501197	0.2253438	1494	2.760818796
24	625	0.4700718	0.28121	1521	3.795478668
25	636	0.4900239	0.3378518	Historic Plotting Position n=89	
26	639	0.5099761	0.3954284	1447	2.842
27	662	0.5299282	0.4541085	1494	3.343
28	667	0.5498803	0.5140731	1521	4.374
29	675	0.5698324	0.5755188		
30	706	0.5897845	0.6386627		
31	714	0.6097366	0.7037468		
32	725	0.6296887	0.7710447		
33	762	0.6496409	0.8408689		
34	799	0.669593	0.9135813		
35	800	0.6895451	0.9896058		
36	821	0.7094972	1.0694456		
37	876	0.7294493	1.1537063		
38	882	0.7494014	1.2431279		
39	887	0.7693536	1.3386299		
40	907	0.7893057	1.4413775		
41	911	0.8092578	1.5528791		
42	943	0.8292099	1.67514		
43	976	0.849162	1.81091		
44	1002	0.8691141	1.9641089		
45	1026	0.8890662	2.1406063		
48	1447	0.9489226	2.9483134		
49	1494	0.9688747	3.453966		
50	1521	0.9888268	4.4886258		
Historic Plotting Position		n=89			
87	1447	0.9712747	3.5354389		
88	1494	0.9824955	4.0364812		
89	1521	0.9937163	5.0666522		

L Moments from Tideda
Formula from Hosking 1990

For Site: Otara at Browns Bridge (16002)

Mean	Std. Dev.	Skew	Kurtosis	
L1	L2	T3	T4	Method
369.5876	96.1937	0.1499	0.1815	Annual
434.9329	98.3675	0.1072	0.2636	Biennial

For EV1

	Annual	Biennial
$\alpha = \frac{l_2}{\ln 2}$	= 138.778	141.914
$\xi = l_1 - 0.5772\,\alpha$	= 289.485	353.020

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

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

For GEV

	Annual	Biennial
$z = 2/(3+t_3) - \log 2/\log 3$	= 0.004011038	0.01273657
$k = 7.8590z + 2.9554z^2$	= 0.031570297	0.100576128
$\alpha = l_2 k/\{(1-2^{-k})\Gamma(1+k)\}$	= 142.7669731	154.4691286
$\xi = l_1 + \alpha\{\Gamma(1+k)-1\}/k$	= 291.5132823	359.8590029

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

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

Statistical Flood Frequency Analyses on the Waioeka and Otara Rivers

Return Period	Plotting Position		Annual analyses		Biennial analyses	
			EV1	GEV	EV1	GEV
T	YT(Annual)	YT (Biennial)	QT1	QT2	QT1	QT2
1.01	-1.53	-2.22	77	68	118	219
1.1	-0.87	-1.57	168	165	200	260
1.5	-0.09	-0.79	276	278	299	317
2	0.37	-0.33	340	344	357	356
2.33	0.58	-0.11	370	373	384	376
5	1.50	0.81	498	501	500	472
10	2.25	1.56	602	602	595	568
20	2.97	2.28	702	696	686	677
50	3.90	3.21	831	816	803	848
100	4.60	3.91	928	903	891	1003
200	5.30	4.60	1024	988	979	1184
500	6.21	5.52	1152	1097	1095	1473

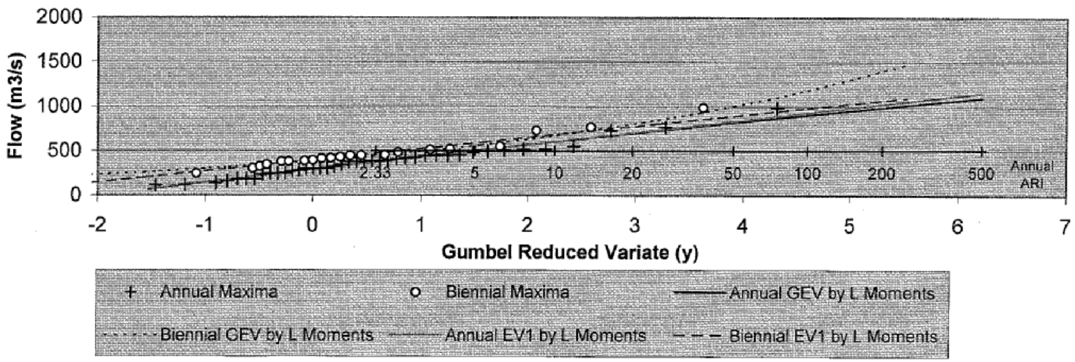
Recommended Design Flows for the Otara at Browns Bridge

ARI Event	Discharge m³/s
2.33	370
5	498
10	602
20	702
50	831
100	928
200	1024
500	1152

Plotting Position

n= 42			Annual	n= 22	Biennial
i	Sample Values	F(Qi)	y(Qi)	Sample Values	y(Qi)
1	114	0.0132953	-1.463334	243	-1.087170386
2	119	0.037037	-1.19266	303	-0.559500466
4	137	0.0845204	-0.904527	325	-0.494103334
5	158	0.1082621	-0.798948	346	-0.427925667
6	182	0.1320038	-0.705532	379	-0.29219628
7	183	0.1557455	-0.620325	379	-0.222093818
8	183	0.1794872	-0.540958	386	-0.075855957
9	235	0.2032289	-0.465884	392	-0.62436772
10	243	0.2269706	-0.394023	411	0.080957674
11	244	0.2507123	-0.32458	417	0.16450386
12	248	0.2744539	-0.256943	427	0.252274904
13	271	0.2981956	-0.190625	445	0.34502707
14	282	0.3219373	-0.125221	447	0.443694457
15	298	0.345679	-0.060384	452	0.663828616
16	298	0.3694207	0.0041896	483	0.788831065
17	302	0.3931624	0.0687795	512	1.082928269
18	303	0.4169041	0.1336467	517	1.26182835
19	325	0.4406458	0.1990438	550	1.733786255
20	346	0.4643875	0.2652215	728	2.076179978
21	378	0.4881292	0.3324351	765	2.583878868
22	379	0.5118708	0.4009509	984	3.620509206
23	379	0.5356125	0.4710534	Historic Plotting Position n=89	
24	385	0.5593542	0.5430522	984	4.373505003
25	386	0.5830959	0.6172912		
26	392	0.6068376	0.6941596		
27	411	0.6305793	0.7741049		
28	417	0.654321	0.857651		
29	427	0.6780627	0.9454221		
30	445	0.7018044	1.0381743		
31	447	0.7255461	1.1368416		
32	447	0.7492877	1.2426021		
33	452	0.7730294	1.3569758		
34	483	0.7967711	1.4819783		
35	496	0.8205128	1.6203687		
36	512	0.8442545	1.7760754		
37	517	0.8679962	1.9549755		
38	519	0.8917379	2.1664555		
39	550	0.9154796	2.4269334		
40	728	0.9392213	2.7693272		
41	765	0.962963	3.277026		
42	984	0.9867047	4.3136564		
Historic Plotting Position			n=89		
89	984	0.9937163	5.0665522		

Fitted Extreme Value Distribution of Annual Flow Maxima for Otara at Browns Bridge (1964-2007)



L Moments from Tideda
Formula from Hosking 1990

For Site: Otara at Browns Bridge (16002)

Mean	Std. Dev.	Skew	Kurtosis	
L1	L2	T3	T4	Method
369.5876	96.1937	0.1499	0.1815	Annual
434.9329	98.3675	0.1072	0.2636	Biennial

For EV1

		Annual	Biennial
$\alpha = \frac{l_2}{\ln 2}$	=	138.778	141.914
$\xi = l_1 - 0.5772\ \alpha$	=	289.485	353.020

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

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

For GEV

$z = 2/(3+t_3) - \log 2/\log 3$	=	0.004011038	0.01273657
$k = 7.8590z + 2.9554z^2$	=	0.031570297	0.100576128
$\alpha = l_2 k/\{(1-2^{-k})\Gamma(1+k)\}$	=	142.7669731	154.4691286
$\xi = l_1 + \alpha\{\Gamma(1+k)-1\}/k$	=	291.5132823	359.8590029

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

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

Statistical Flood Frequency Analyses on the Waioeka and Otara Rivers

Return Period	Plotting Position		Annual analyses		Biennial analyses	
			EV1	GEV	EV1	GEV
T	YT(Annual)	YT (Biennial)	QT1	QT2	QT1	QT2
1.01	-1.53	-2.22	77	68	118	219
1.1	-0.87	-1.57	168	165	200	260
1.5	-0.09	-0.79	276	278	299	317
2	0.37	-0.33	340	344	357	356
2.33	0.58	-0.11	370	373	384	376
5	1.50	0.81	498	501	500	472
10	2.25	1.56	602	602	595	568
20	2.97	2.28	702	696	686	677
50	3.90	3.21	831	816	803	848
100	4.60	3.91	928	903	891	1003
200	5.30	4.60	1024	988	979	1184
500	6.21	5.52	1152	1097	1095	1473

Recommended Design Flows for the Otara at Browns Bridge

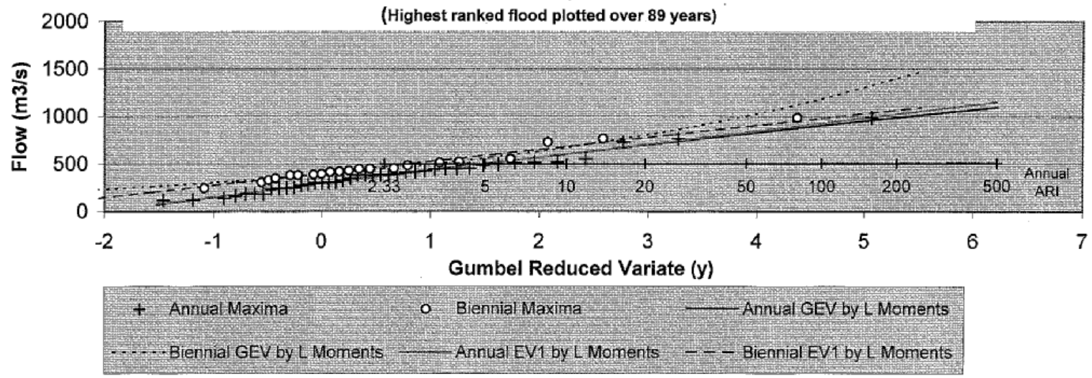
ARI Event	Discharge m^3/s
2.33	370
5	498
10	602
20	702
50	831
100	928
200	1024
500	1152

Plotting Position

n= 42			Annual	n= 22	Biennial
i	Sample Values	F(Qi)	y(Qi)	Sample Values	y(Qi)
1	114	0.0132953	-1.463334	243	-1.087170386
2	119	0.037037	-1.19266	303	-0.559500466
4	137	0.0845204	-0.904527	325	-0.494103334
5	158	0.1082621	-0.798948	346	-0.427925667
6	182	0.1320038	-0.705532	379	-0.29219628
7	183	0.1557455	-0.620325	379	-0.222093818
8	183	0.1794872	-0.540958	386	-0.075855957
9	235	0.2032289	-0.465884	392	-0.62436772
10	243	0.2269706	-0.394023	411	0.080957674
11	244	0.2507123	-0.32458	417	0.16450386
12	248	0.2744539	-0.256943	427	0.252274904
13	271	0.2981956	-0.190625	445	0.34502707
14	282	0.3219373	-0.125221	447	0.443694457
15	298	0.345679	-0.060384	452	0.663828616
16	298	0.3694207	0.0041896	483	0.788831085
17	302	0.3931624	0.0687795	512	1.082928269
18	303	0.4169041	0.1336467	517	1.26182835
19	325	0.4406458	0.1990438	550	1.733786255
20	346	0.4643875	0.2652215	728	2.076179978
21	378	0.4881292	0.3324351	765	2.583878868
22	379	0.5118708	0.4009509	984	3.620509206
23	379	0.5356125	0.4710534	Historic Plotting Position n=89	
24	385	0.5593542	0.5430522	984	4.373505003
25	386	0.5830959	0.6172912		
26	392	0.6068376	0.6941596		
27	411	0.6305793	0.7741049		
28	417	0.654321	0.857651		
29	427	0.6780627	0.9454221		
30	445	0.7018044	1.0381743		
31	447	0.7255461	1.1368416		
32	447	0.7492877	1.2426021		
33	452	0.7730294	1.3569758		
34	483	0.7967711	1.4819783		
35	496	0.8205128	1.6203687		
36	512	0.8442545	1.7760754		
37	517	0.8679962	1.9549755		
38	519	0.8917379	2.1664555		
39	550	0.9154796	2.4269334		
40	728	0.9392213	2.7693272		
41	765	0.962963	3.277026		
42	984	0.9867047	4.3136564		
Historic Plotting Position			n=89		
89	984	0.9937163	5.0666522		

Fitted Extreme Value Distribution of Annual Flow Maxima for Otara at Browns Bridge (1964-2007)

(Highest ranked flood plotted over 89 years)



Appendix 2 – Calculation Spreadsheet used for the Regional Method

Regional Method: McKercher and Person (1989) Flood Estimation - A Revised Design Procedure

	Specific Discharge Factor		Flood Frequency Factor		Discharge (Q_t) $Qt = \bar{Q}[xt + (1 - xt)q100$					Area
	$\bar{Q}$ from maps	$\bar{Q} = \bar{Q}$ from maps$*A^{0.808}$	q_{100}	$Q100 = \bar{Q}*q_{100}$	Q_t 50 (2%)	Q_t 20 (5%)	Q_t10 (10%)	Q_t5 (20%)	Q_t2.33 (50%	km²
Tutaetoko	3.50	94.76	2.40	227.43	204.42	173.69	149.95	125.20	97.88	59.29
Waioeka at Cableway	3.00	576.53	2.50	1441.33	1291.31	1090.99	936.24	774.91	596.85	670.45
Otara at Browns Bridge	3.70	309.55	2.40	742.91	667.73	567.35	489.80	408.95	319.73	239.53

$\bar{Q}$ from maps = regional mean value for $(\bar{Q}/A^{0.808})$

$Qt = (Q^{-}*A^{0.808}) \times q_{100}$

Discharge $(Q_t) = Q^{-}[xt + (1 - xt)q100]$

% difference between Tutaetoko and Otara (Browns Bridge) and Waioeka (Cableway)						River
31	31	31	31	31	31	Waioeka
16	16	16	16	16	16	Otara

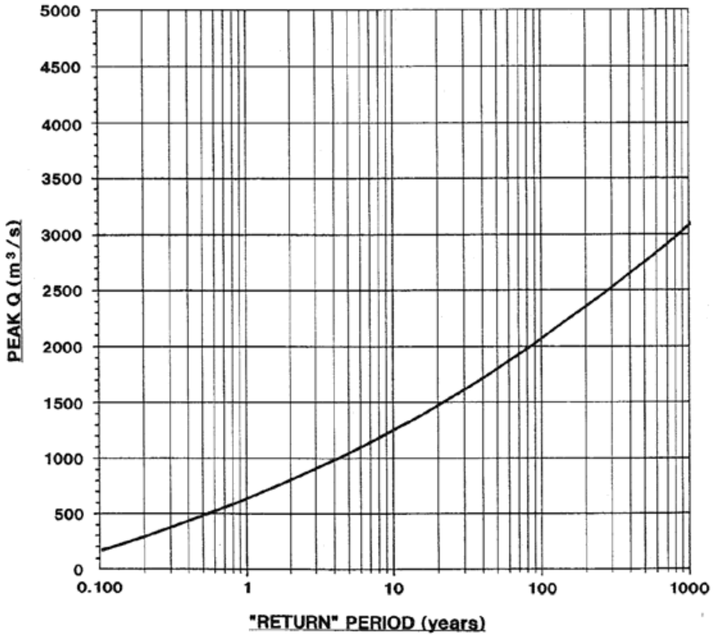
$y_t = -\ln[-\ln(1-1/T)]$	3.90194	2.9702	2.25037	1.49994	0.67173
$x_t = 1.1435 - 0.2486yt$	0.17348	0.40511	0.58406	0.77061	0.97651

Estmates of AEP for the Tutaetoko Catchment

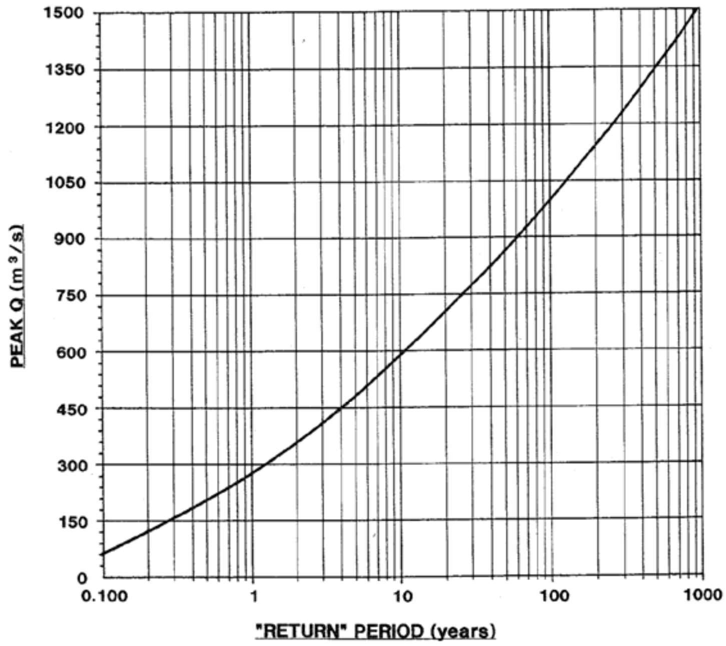
River	Method	Area	AEP and Resulting discharge (m³/s)					
			1%	2%	5%	10%	20%	50%
Waioeka at Cableway	L Moments	660	1695	1480	1221	1040	868	678
Otara at Browns Bridge	L Moments	240	928	831	702	602	498	370
Tutaetoko	Regional	60	227	204	174	150	125	98
Tutaetoko (Waioeka at Cableway)	$Area^{0.8}$	60	244	213	176	150	125	98
Tutaetoko (Otara at Browns Bridge)	$Area^{0.8}$	60	303	271	229	196	162	121
Tutaetoko	17% Browns Bridge	60	158	141	119	102	85	63
		Mean	233	208	174	150	124	95

Appendix 3 –1989 Flood Frequency Analyses

From Titchmarsh (1990) on behalf of Environment B.O.P and Hall (1988) on behalf of the East Cape Catchment Board.



"RETURN" FREQUENCY CURVE
Waioeka River at Cableway
SITE No. 15901



"RETURN" FREQUENCY CURVE
Otara at Browns Bridge
SITE No. 16002

ENGINEERING TECHNICAL REPORT 1988/2

WAIOEKA-OTARA CATCHMENT MANAGEMENT STUDY: PART I

HYDROLOGICAL AND STATISTICAL ASPECTS

1.0 INTRODUCTION AND OBJECTIVES:

1.1 The original (1966) Design curves for the Waioeka Otara Flood Control scheme were based on hydrological data limited to only a short time span, which affected the accuracy of the rating curves. The short record also affected the annual exceedence methods used causing difficulty in deriving reliable flood frequency curves from the confusing and sometimes even conflicting results.

This report summarises work requested to re-derive the frequency curves for the Waioeka and Otara based on the longer records of stage-time and gaugings now available, and using more recent flood-frequency estimation techniques derived by ECCB staff.

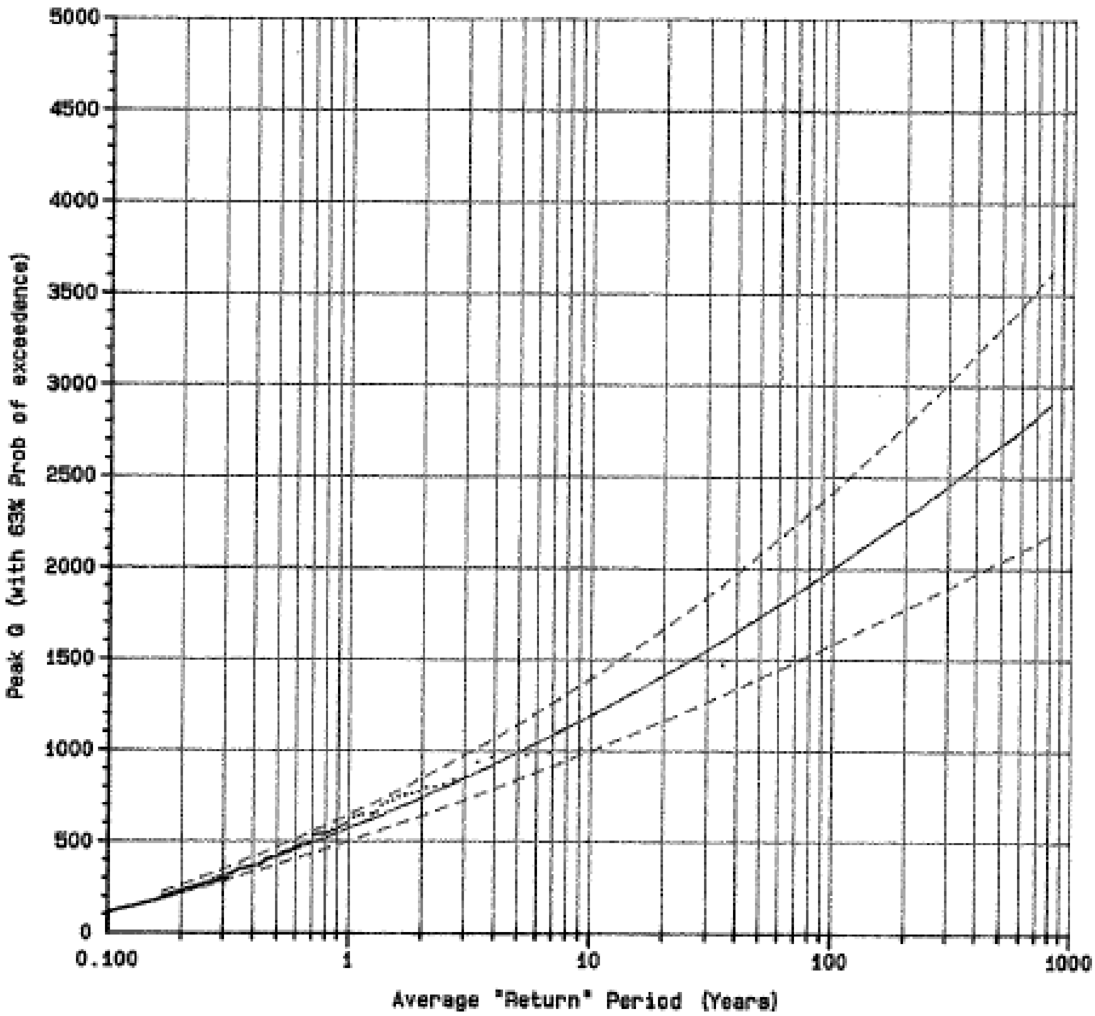
1.2 Although this report is primarily concerned only with the frequency aspect, much work was required on developing reliable ratings for both sites neither of which was entirely satisfactory. However both ratings at present generally consist of a single curve applying to the upper stages where the larger events occur, and so the absolute frequency of the larger events can be regarded as fixed, with any future adjustment in discharge rating causing only a re-scaling of the Q values of the appropriate frequency range.

1.3 This report is presented in two parts, covering the Waioeka and the Otara rivers seperately, to facilitate any future references to either site.

2.0 WAIOEKA FLOOD FREQUENCY:

2.1 Summary: The Peak Discharge which is equalled or exceeded for various design periods is as folllows:

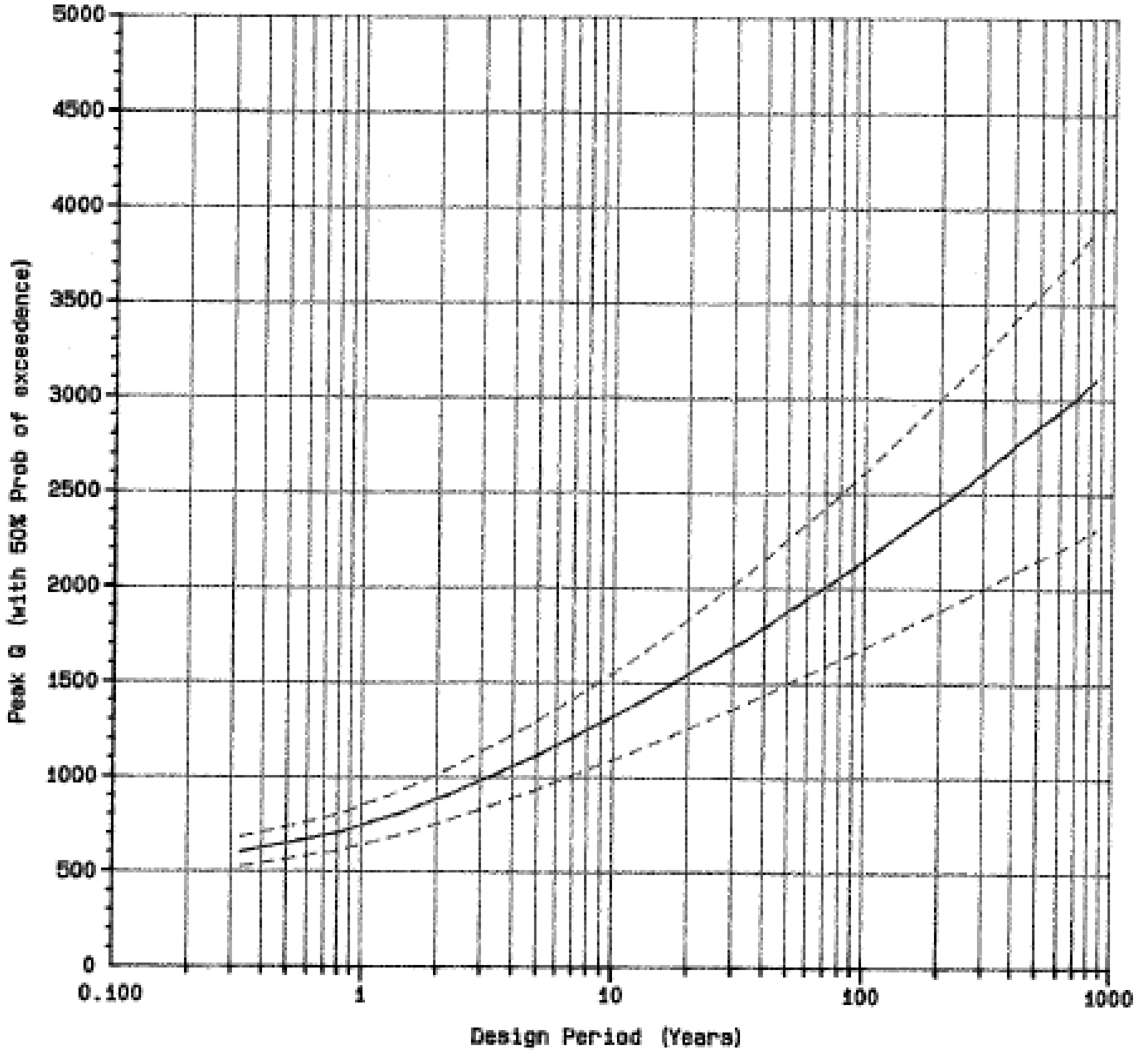
Design Period (Years)	63% Prob	C.L. (95%)	50% Prob	C.L. (95%)
1	560	+/- 60	750	100
2	730	100	890	120
5	990	150	1,100	180
10	1,200	200	1,300	230
20	1,400	250	1,540	300
30	1,540	290	1,690	320
50	1,730	330	1,890	380
100	2,000	400	2,150	460
200	2,280	500	2,430	550
250	2,370	520	2,550	600
500	2,680	650	2,850	700
1,000	3,000	750	3,150	800



"RETURN" FREQUENCY CURVE
Waioeka River at Cableway
SITE NO.: 15901

Years.....58-85 dDays..... 3
dPeriod...12 min SG....0.0
#Trials... 1 min rise..10%

Plot: MON, 22 FEB 1988 15:15:51
Sheet 1 of 3



50% PROBABILITY FREQUENCY CURVE
Waioeka River at Cableway
SITE NO.: 15901

Plot: MON, 22 FEB 1988 15:15:55
Sheet 2 of 3

These tables are derived from graphs 2.1.1. and 2.1.2. A full probability graph may be found in Appendix 2.4.12.

Comments on the sources of data and the derivation of results follow:

2.2 Stage-Time Data:

The stages of peak events used in this analysis were taken from a copy of the MWD records for the Gorge Cableway (site No. 15901) which were produced by a Float-Counterweight recorder in a standard concrete recorder housing. As such the accuracy of the records is expected to be good. The records extended from March 1958 to May 1985. More recent records would now be available but should not make a significant difference to the results. The copy of the records is stored on the ECCB computer in the normal Hydro-suite archives (Pathname<ECCB>Water-Res>Flow-Data>Stage-Time>BinaryFiles>ST15901xx). Few gaps appear in the record and it is believed no significant event is excluded. Two large events occured in 1964 and 1967 which exceeded the recorder range, but in each case accurate peak levels were obtained from marks left inside the recorder housing itself.

A list of peak events was derived using the ECCB program EVENTS.F77 which processes each and every stage time point using a filter to obtain the true maximum peak stage of each significant event. Only events which were seperated by a minimum of 3 days and consisted of at least a 10% rise in stage were used. This produced a "Full Distribution" list of events. This list was used in all analysis such as Gumbel Analysis, for consistent comparison of methodologies.

The list of peaks is provided in Appendix 2.1.1 - 2.1.7

2.3 Gauging Data:

For this exercise, only MWD discharge ratings were used to avoid introducing yet another variable. The actual gaugings making up the Ratings are unavailable to us at this time; as TIDEDA ratings consist of a table of several stage-Q points, these were used as pseudo-gaugings to construct equivalent rating curves in our own format. Some 46 ratings apply to the 27.17 year period, but these differ mainly in the lower (below 6m) range, the upper portions tending to merge into one curve. As a print-out of all 46 curves would be large, a copy of only the Ratings which apply to the 1964 and 1967 events is detailed (Curve #3 and #6, see App 2.2.1 - 2.2.9). In addition the curve originally derived from the ECCB gaugings is appended (2.2.12, 15). These Ratings are stored in the ECCB computer Hydro Archives.

Fig. 2.2.10 shows that the fitting of ECCB format curves to the MWD table points works well. However, a comparison of the upper curve with an ECCB rating derived from our own gaugings does show marked discrepency at the upper end (e.g. at 10m MWD=1350, ECCB=1900 cumecs). As mentioned in sect. 1.2 above, this does not affect the absolute frequency of events, only the relative discharge. Nevertheless, this difference should be resolved before results are actually applied to scheme design work etc. To do this, it is recommended that an updated list of MWD Gaugings for the site be obtained and these used together with our gaugings and observed slope-data to extrapolate the rating curve to high stages using back-water profile and possibly LATIS as was done for the Otara at Browns.

It should be noted that there has been contention with MWD over the Waioeka Cableway ratings since the 1964 and 1967 events occurred.

For this report, only the MWD ratings have been used.

2.4 Frequency Analysis - Gumbel Method:

A list of annual maximum events was constructed from the master list of events and this used to derive an annual average return frequency table using the Least-Mean-Squares technique outlined in the paper by BENHAM (N.Z. Inst. of Engineers Proc. 1950 p126 - 143). Results are given in Appendix 2.3.1,2. These results agree closely with those reportedly obtained by the MWD "FRAN" package (which is a mix of several Annual-Exceedence methods): (Ref MWD letter of 17/9/87 Roel von't Steen).

Method	30 year	250 year
Gumbel (LMS)	1350 Cumecs	1820 Cumecs (ECCB)
Fran (1958 - 87)	1300 - 1350	1700 - 1900 (MWD)

The control curves show reasonable confidence below 50 years, but diverge quickly in the upper range.

Ordinarily, these results would be accepted but more recent experience by the Board has shown an alarming tendency for events to seemingly occur at a greater frequency than suggested by these results. For this reason alternative methods using the complete list of all events in each year (i.e. the "Full-Duration" rather than just one event per year as with the Annual Exceedence method) are advocated. Results have shown this to be preferable as the following sections show.

2.5 Frequency Analysis - Event Probability Method:

Details of results are provided in Appendix 2.4.1, 12.

The method uses the full list of peak events obtained from the 27.17 year continuous stage-time record and using the current MWD Ratings.

Magnitude Distribution:

The list of events is first assembled into a cumulative frequency distribution of the magnitudes (i.e., the number of events that fall into various ranges of discharge are totalled). A listing showing this distribution is given in Appendix 2.4.2,4. A continuous curve is then fitted to this observed distribution. Because all of the events are assumed to belong to the same continuous distribution, this curve can then be extrapolated to higher ranges with reasonable confidence. The effect of various "Outliers" can then be determined by their effect of exclusion or inclusion on the curve "fit" which is based on standard statistical measures (R-squared and F-test). In this way, a curve which is unbiased by unusual events (such as a 100 year event occuring in a 27 year records, say) can be obtained. Details of the results of this process are shown in App. Fig 2.4.5, 6. With the Waioeka record, a further refinement was made in that the two largest events (1964 & 1967) are known to be the largest since the commencement of reasonably reliable records of floods circa 1918. The probability of these two events was therefore adjusted to the equivalent of a 70 year and 35 year average return period. The remainder of the record was left alone as the 27 year length of this record is sufficient to cover the balance. It may be noted that the magnitude distribution curve fit produces an R-square value of 0.99 which indicates a very good fit. This can be verified in Fig. 2.4.5 which shows the Actual data with the fitted distribution superimposed. Fig 2.4.6 is the same curve plotted in logarithmic form - note that the two largest points are 1964 to 1967.

Time Distribution:

A Binomial Distribution was used to fit the time-parameter of the events seperately. Fig. 2.4.7 illustrates this fitting of this theoretical curve [dashed line] to the time distribution of the data [dotted line]. It can be seen that the Binomial Distribution does fit reasonably closely to the data.

Derived Frequency Curve:

The two derived distributions are now re-combined to form a single function representative of the frequency relationship for the site that would be obtained if we had a very long record of peak events. Simple statistical methods can be applied to the list of observed events to obtain the variance, standard deviation and 95% confidence limits on this original list of data. This is detailed in App. 2.4.8 which also lists the 63% and 50% probability of exceedence of each of the ranges in peak discharge in the period shown (Years). Appendix 2.4.10 gives the resultant overall 95% confidence limits on these probabilities.

The results of the EVPROB run can be summarised in several ways. For purposes of comparison with other Annual Maximum techniques, the plot of App. 2.4.11 shows the discharges with a probability of occurrence of 63% which are similar at larger return freqs to average annual return values. For the Waioeka at Cableway, the EVPROB results fall below the two RFE method curves, but considerably above the Gumbel and FRAN curve. The 1966 design is also exceeded (as expected, as it is also based on annual maxima) but to a lesser extent. A more universal presentation of results is by producing a series of design curves which correspond to a range of commonly used Design Periods. The probability of occurence for any sized event can be read from these directly - see App. 2.4.12. A plot of the nett confidence limits is shown on Figs. 2.1.1 and 2.1.2.

2.6 Validity of Results:

A discussion on which is the "Best" method to adopt is the source of much confusion and frustration. Theoretically based arguments can be advanced and refuted on any of the methods but these remain essentially academic as the final "proof" can only be obtained after a long record is obtained - prefereably 100 years or more. However, perhaps the best test for any method is in comparison with the actual observed available record, recognising that the larger events quite likely have "real" return frequencies longer or shorter than the period of observation. (For example the largest event in a 20 year record may actually be a 30 or 50 year event, or just as easily could be only a 10 or 15 year event). However, the smaller values should certainly more closely approach expected frequencies. Therefore, any method should agree closely with observed data especially in the lower ranges.

To this end the observed data were plotted on the Return Frequency Diag (63%) Prob curve), Fig. 2.1.1. It can be seen that the EVPROB curve fits the observed data well, especially at the lower end. It is also of interest to note that the data has a strong tendency to plot as a value theory which assumes a linear relationship at upper ranges on a Log-Natural plot (i.e. Gumbel plots). Whether the EVPROB curve would gradually "flatten" as more data accumulates with time, or whether it does represent the ultimate curve can as mentioned above, only be determined by a long term hydrological record.

It is also interesting to note that the ranges above 1000 seems to plot lower than the rest, suggesting that fewer than expected events in this range have been experienced so far. If this is so, we can expect this situation to remedy itself as the observed record grows longer. Alternatively, if the 1000-1500 range is "correct" then the lower ranges (600-900) must be abnormal, with many more events occurring than expected. However, lower ranges have a much higher confidence than higher ranges. Further, Evprob runs with the '64 and '67 events deleted still result in curves which run above the 1000 cumec range due to the preponderance of lower events defining a curve which passes above the 1000 cumec values. The third alternative is, as mentioned above, that the EVPROB magnitude distribution is incorrect, and that the observed upward concavity of the smaller events is perhaps some artificiality of the transformation of the data.

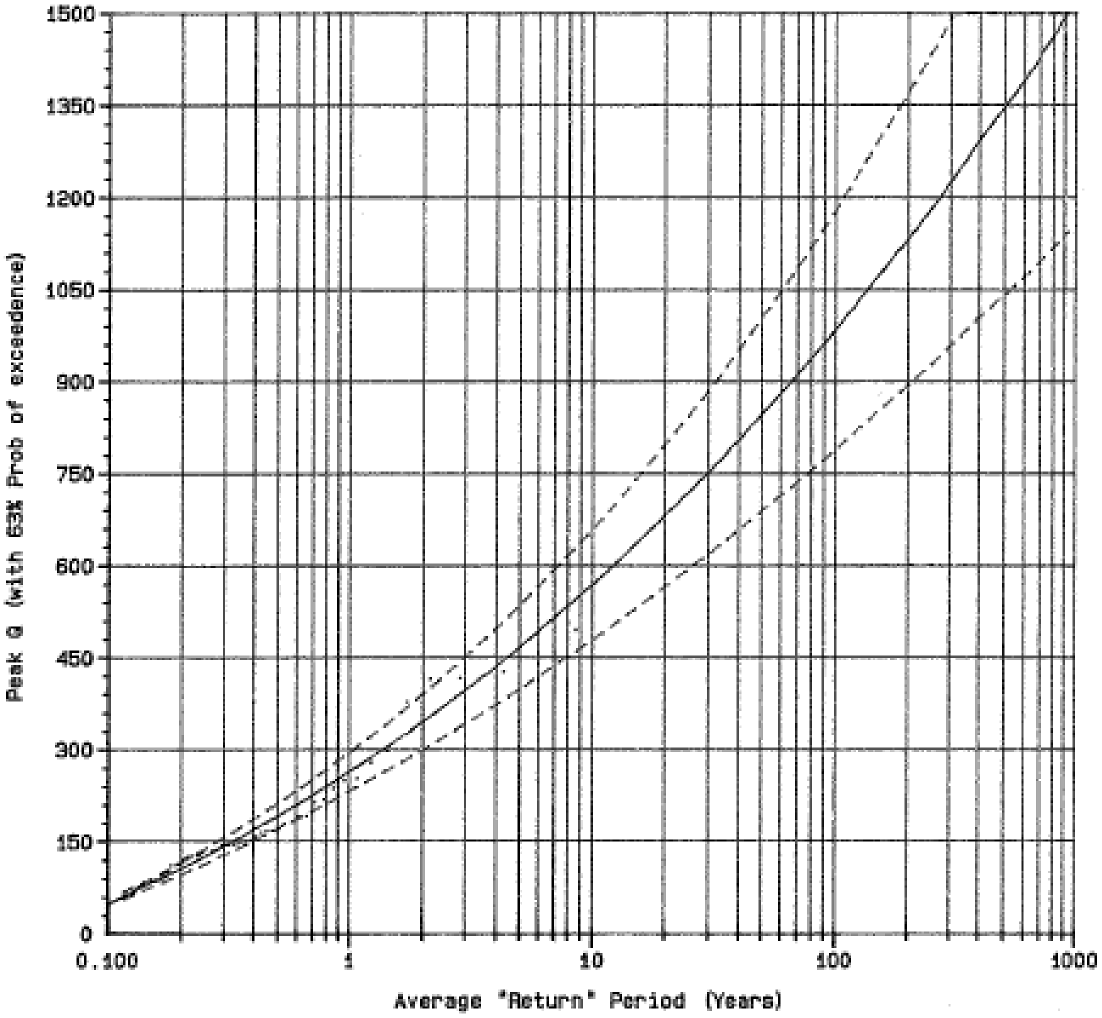
2.6 Conclusion:

Given the above results and within the limitations of the combined Confidence Limits and the Rating Curves, it is recommended that the Evprob curve should be accepted as defining an upperlimit curve, with the Gumbel curve a lower limit for design purposes.

3.0 OTARA FLOOD FREQUENCY:

3.1 Summary: The Peak discharge which is equalled or exceeded for various discharges is as follows:

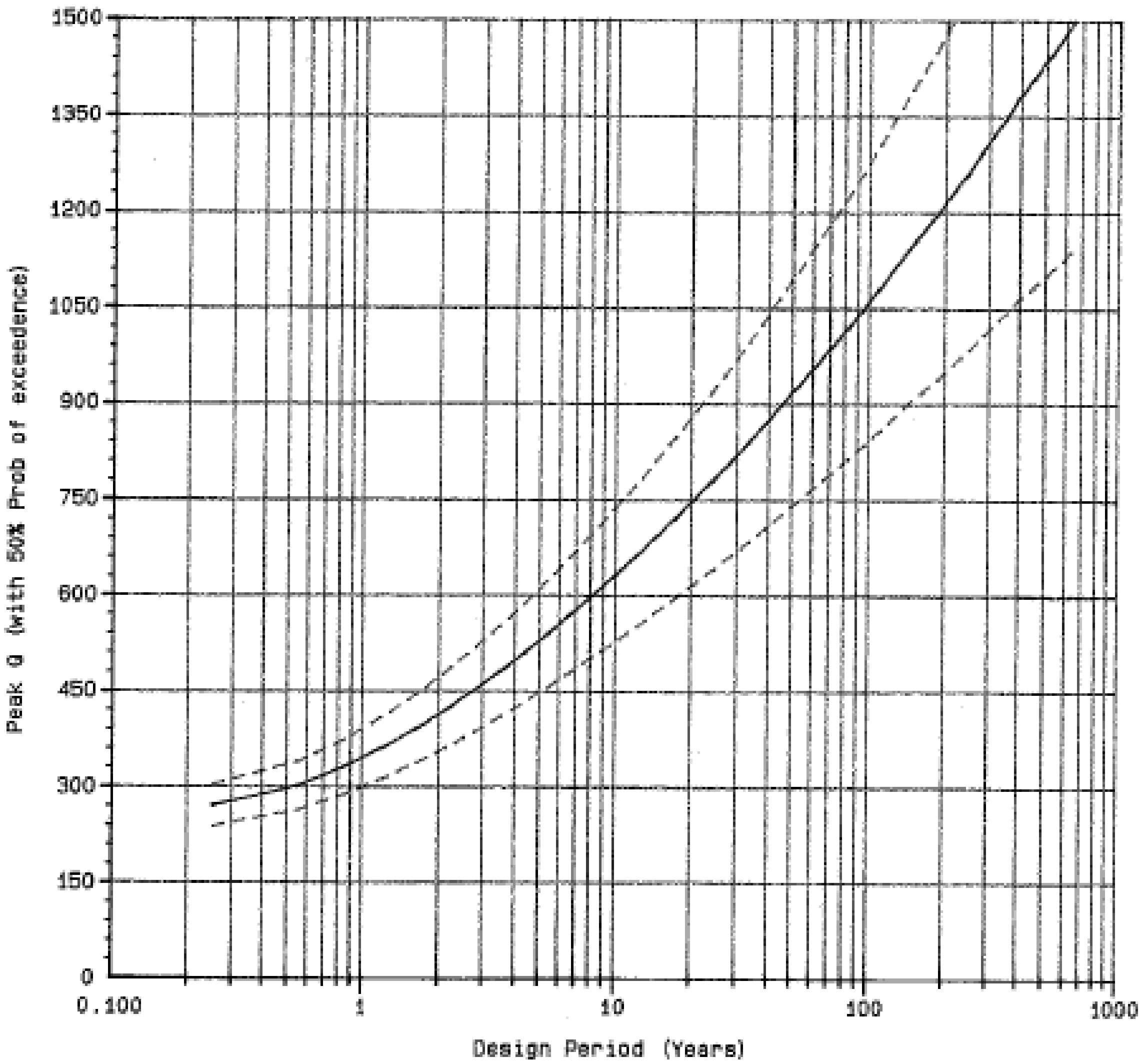
Design Period (years)	63% Prob	C.L. (95%)	50% Prob	C.L. (95%)
1	260	30	350	47
2	340	45	420	60
5	460	65	530	80
10	560	90	640	100



"RETURN" FREQUENCY CURVE
Otara at Browns Br
SITE NO.: 16002

Years.....79-87 dDays..... 3
dPeriod...12 min SG....0.0
#Trials... 1 min rise..10%

Plot: TUE, 16 FEB 1988 11:33:47
Sheet 1 of 3



50% PROBABILITY FREQUENCY CURVE
Otara at Browns Br
SITE NO.: 16002

Plot: TUE, 16 FEB 1988 11:33:51
Sheet 2 of 3

Design Period (Years)	63% Prob	C.L. (95%)	50% Prob	C.L. (95%)
20	680	115	750	130
30	750	133	800	145
50	840	150	920	180
100	980	190	1060	250
200	1125	230	1215	260
250	1180	250	1250	300
500	1350	300	1440	340
1000	1550	375	1630	400

These tables are derived from graphs 3.1.1 and 3.1.2. A full probability graph is available in Appendix 3.4.13.

3.2 <u>Stage-Time Data</u>:

The records for the Browns Bridge site are digitized from 0-10 metre Foxboro Charts. The accuracy of this is about +/- 05m. There are few gaps, with all significant Events covered.

The list of Peaks was derived using the EVENTS F77 program, using a standard 3 day minimum separation between peaks and a minimum rise in stage of 10% to eliminate insignificant rises or multiple peaks in one event. The Annual maximum event for each year was extracted from the "Full Duration" list for Gumbel analysis. However, the resulting list of Annual events is rather short (only 7 full years) and so was supplemented by diary records of peaks from 1964.

The full list of peaks is given in Appendix 3.1.1.2.

3.3 <u>Gauging Data</u>:

The current revision of the rating curves for this site consist of 3 curves applying from 1964-76, 77-80 and 1980 - current. As the stage records only apply from 1979, the first curve is not used for the full duration list of peaks. (Note that the ECCB Hydrosuite of programs automatically apply the appropriate rating curve).

The original curve use for this site was limited to gauging data up to about 4m stage. The revised curves have incorporated LATIS run to extend this range to 6m where a large proportion of over bank flow occurs, thus flattening the rating curves considerably. The derived LATIS points have been used on each of the revised curves, as ratings in this range are proportionally less affected by bed changes and hence tend to be more stable.

A graph showing a comparison of the rating curves is given in Appendix 3.2.1. Details of the current No. 2 and No. 3 curves is given in Appendix 3.2.2-8 and the Original curve is detailed in Appendix 3.2.9-11.

Although the gaugings that are used to make up the ratings show a considerable degree of variance, it is felt that the current curves will not prove too much in error as more data becomes available in future: Discharge rating curves in these rivers are particularly prone to many variables which can easily change from Flood to Flood. The ratings should be seen at best as AVERAGES of loop curves, about which actual events will apply.

3.4 Frequency Analysis:

GUMBEL METHOD: Analyses for 1964-86, 1969-1986 and the 1979-86 periods have been run. The method used is the Least Mean Squares approach outlined in the paper by Benham (see Sect. 2.4). A plot of the results is shown in Appendix 3.3.1 together with the original 1966 design curve. The 1979-86 curve plots much lower than the other curves as can be expected considering that this 8 year record is very short for any Annual Maxima method and further that it happens to contain few large events. The 1964-86 curve includes both of the largest recorded events (1964 and 1965) and although the record length is nearly 3 times longer at 23 years which should compensate to some degree, this curve still is very sensitive to inclusion or exclusion of the larger events. It is this sensitivity of the Annual Maxima method which makes rational choice of a curve very difficult.

Printouts of the Gumbel calcs is appended in App. 3.3.2-3.

3.5 Frequency Analysis - Event Probability Method:

A description of the method is given in Section 2.5 above, but refer to Appendix 3.4.1-9. This analysis for the Otara at Browns is limited to the 1979-87 period of "Full-Duration" record and so excludes the larger '64 and '65 events. This short record naturally produces a wider variation in the observed distributions, but still managed a good R-squared fit of 0.9927 on the optimised Magnitude distribution curve. Details of the resulting fit to the observed points are in App. 3.4.7-8.

Details of the derivation of the final curve are in App. 3.4.10-13.

The results of the Evprob run for this short record are encouraging. The total confidence limits are within reasonable bounds (ref. plots of Sect 3.1.1 and 3.1.2). A comparison of the 63% prob curve with other methods (App 3.5.1) shows that this curve plots closely to the RFE (Local) curve and the 64-85 Gumbel curve. The closeness with this gumbel curve is interesting as the EVPROB curve is derived from the short record (excluding the 64 and 65 events) and yet still approaches the Gumbel curve which includes them.

3.6 Validity of Results:

Using the same approach as outlined in Section 2.6 above, the actual observed events are plotted on the graph of derived curves (Fig. App. 3.5.1 and Sect 3.1.1.).

The lower part of the "actual data" plots closely to the EVPROB curve and approaches the GUMBEL 64-86 curve at mid level. (At lower levels the Gumbel curve becomes handicapped by the data being confined to the minimum resolution of 1 year). Above 300 cumecs, the data becomes somewhat eratic as can be expected with such a short record. However, all of the curves are contained in the 95% control curves. It is contended therefore that this supports the EVPROB and 64-85 Gumbel curves as being the best indication of the ultimate curve which will become further defined as the length of record increases with time.

It is recommended in this case that the EVPROB curve be adopted for design purposes at this time.

Appendix 4 –1998 Flood Frequency Analyses

File: 5720 05
Date: 22 September 2006

Thanks Phil,

These 500 year figures are close to:

Waioeka: 1.4 * Q100
Otara: 1.25 * Q100

Sorry I have not given much guidance on this. I think the Waioeka one is fine, but the Otara one looks low.

To check I linearly extrapolated on the reduced y variate using the Q100 to Q200 slope, giving respectively 2526, 1231 cumecs. As the Waioeka tips up the 2600 looks fine. I have therefore quickly carried out the following (attached).

Based on the quick draw I think Otara is 1250. HOWEVER, the Otara record is the one river scheme record that I believe needs a revised flood frequency analysis. It is biased strongly by the quiescent IPO (around 25 years of that and 10 of active) and misses both the 2003 Q30 (730 cumecs) flood and any floods in the 50s. Inspection of the attached plots show it does tend to underestimate at the top end (interestingly I only minorly altered this from the 1988 design curve – whereas the Waioeka had to be significantly dropped). It could be that the Q100 is closer to today's Q200.

Therefore, anticipating this can you run the Otara Q500 at 1300 cumecs please.

Note we have programmed formal hydrological reviews of all schemes including global warming for 2007/08 and 2008/09.

Can you run with freeboard added (or stopbanks correspondingly lowered also) please?

I am still deciding whether to go to Garry's retirement function. It fits, as we are at OPC until Wednesday am. I have discussed with Clive and I could be in Wellington easily by 1pm. Garry's farewell is 4pm, so would you be available for around 2 hours in between to discuss this and use of your MIKEFLOOD for other areas not done?

Thank you heaps.

Peter

From: Philip Wallace [mailto:philip.wallace@actrix.co.nz]
Sent: Friday, 22 September 2006 9:08 a.m.
To: Peter Blackwood
Subject: Opotiki Flooding

Hi Pete,
I've run an extreme event through the Waioeka-Otara model as you and Clive wanted. I ran two scenarios – Waioeka Q500 + Otara Q100, and Otara Q500 + Waioeka Q100. For both I used a 2.02 peak tide.

Q500 for the Otara appears to be 1160cumecs, extrapolating the current design estimates. Q500 for the Waioeka is a bit more uncertain as there seems to be an upward curve to the current design estimates – but extrapolating I get 2600m^3/s.

In the Otara Q500, there is a bit of spillage over the Otara urban banks at the d/s end of the town, and at the aerodrome. (totalling about 4cumecs, without allowing for freeboard).

However, the Waieoka Q500 is the bigger threat. Some very minor spillage occurs at the d/s end of the town (insignificant amount), but about 160cumecs in total spills over the Mill Stream (RB) banks into the urban area.

I think the reason for this is that in the stopbank upgrades, we only topped up the areas according to the following:
- in lower reaches, we topped up to 1% (+ f/b), with the expectation that if the confluence was ever realigned that would then provide 0.5% AEP protection
- further upstream (where any confluence realignment would have no effect), if the banks needed topping up to get to 1% AEP, then we topped up to 0.5% AEP.

Thus u/s stopbanks that were already at 1% AEP level that needed topping up, we left alone. So water would spill first in these areas. We never topped up those Mill Stream stopbanks.

Anyway, in that extreme Q500 case, water would spill through the town, including over the High School site – although less flooding there would be less. Flow would spill into Duke St, then into town centre area, and into the Domain area.

In both scenarios, water spills into Woodlands Rd area as well.

Let me know if you want to take this any further, or if you want any more info. I could maybe prepare an animation or floodmap, although the model does crash after the peak has passed (as the Domain area fills up).

On other matters:
FYI – I've now bought my own full version (unlimited) of MIKEFLOOD software (ie full Mike11, MIKe21 & mikeflood). So I can do larger &/or more detailed models than previously (before I had limited H-point & 80000 limit m21cells version).

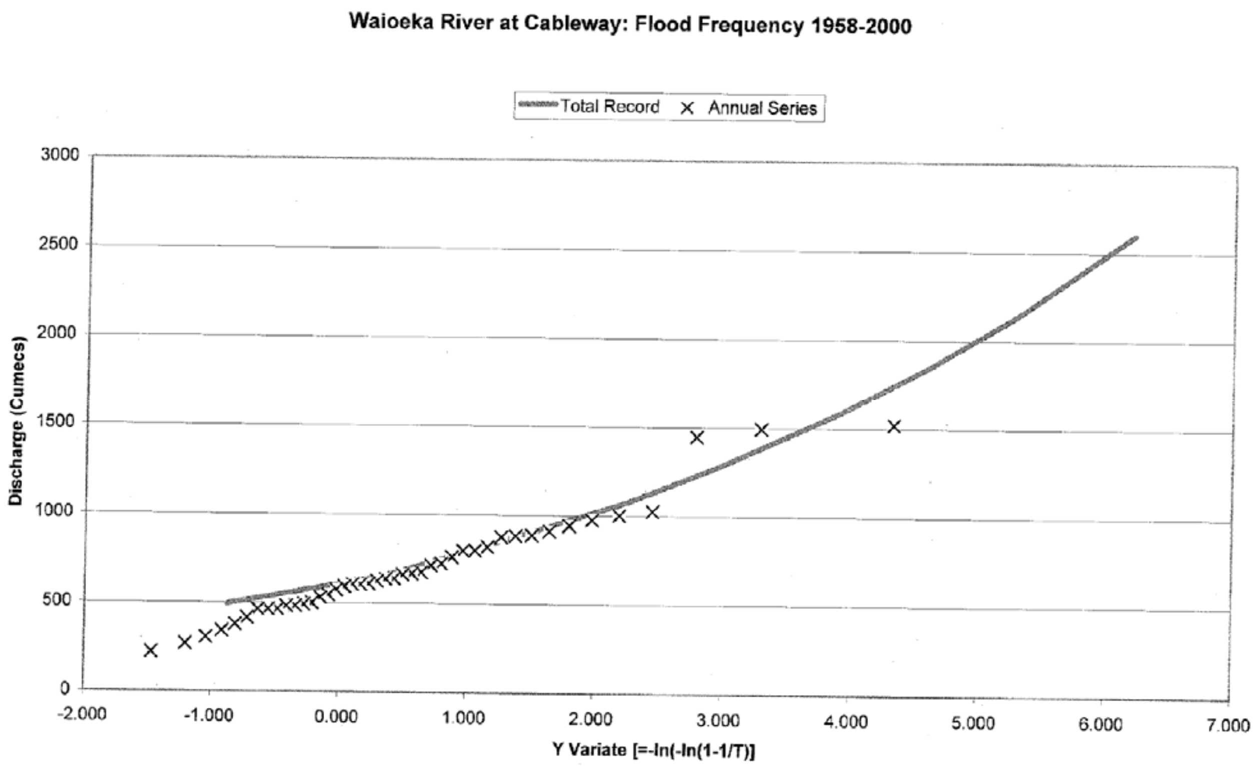
Also a question on the South Waitohu stopbank. Do you recall why the decision was made to align the Sth Waitohu stopbank along the Mangapouri Stream (Along line of existing "stopbank/drain diggings bank") – rather than along a different alignment eg Tasman Rd.

Regards
Phil

Waioeka at Cableway

Annual Extremes

Discharge Total Record (Cumecs)	Rank	Return Period (Years)	Y Variate
2600		500.000	6.214
2140		200.000	5.296
1845		100.000	4.600
1583		50.000	3.902
1279		20.000	2.970
1075		10.000	2.250
904		5.000	1.500
656		2.000	0.367
494		1.100	-0.875
1521	1	77.000	4.337
1494	2	27.641	3.301
1447	3	16.844	2.794
1026	4	12.112	2.451
1002	5	9.456	2.191
976	6	7.755	1.980
943	7	6.573	1.802
911	8	5.704	1.646
887	9	5.037	1.508
882	10	4.510	1.384
876	11	4.083	1.270
821	12	3.730	1.164
800	13	3.433	1.066
799	14	3.180	0.974
762	15	2.962	0.887
725	16	2.771	0.804
714	17	2.604	0.724
675	18	2.456	0.648
667	19	2.323	0.575
662	20	2.204	0.503
639	21	2.097	0.434
636	22	2.000	0.367
625	23	1.911	0.300
610	24	1.830	0.235
606	25	1.756	0.171
605	26	1.687	0.107
594	27	1.623	0.044
575	28	1.565	-0.019
546	29	1.510	-0.082
532	30	1.459	-0.146
500	31	1.411	-0.210
492	32	1.366	-0.275
484	33	1.324	-0.341
483	34	1.285	-0.410
465	35	1.248	-0.481
464	36	1.213	-0.555
463	37	1.179	-0.633
416	38	1.148	-0.717
379	39	1.118	-0.809
342	40	1.090	-0.914
304	41	1.063	-1.038
269	42	1.038	-1.200
221	43	1.013	-1.469

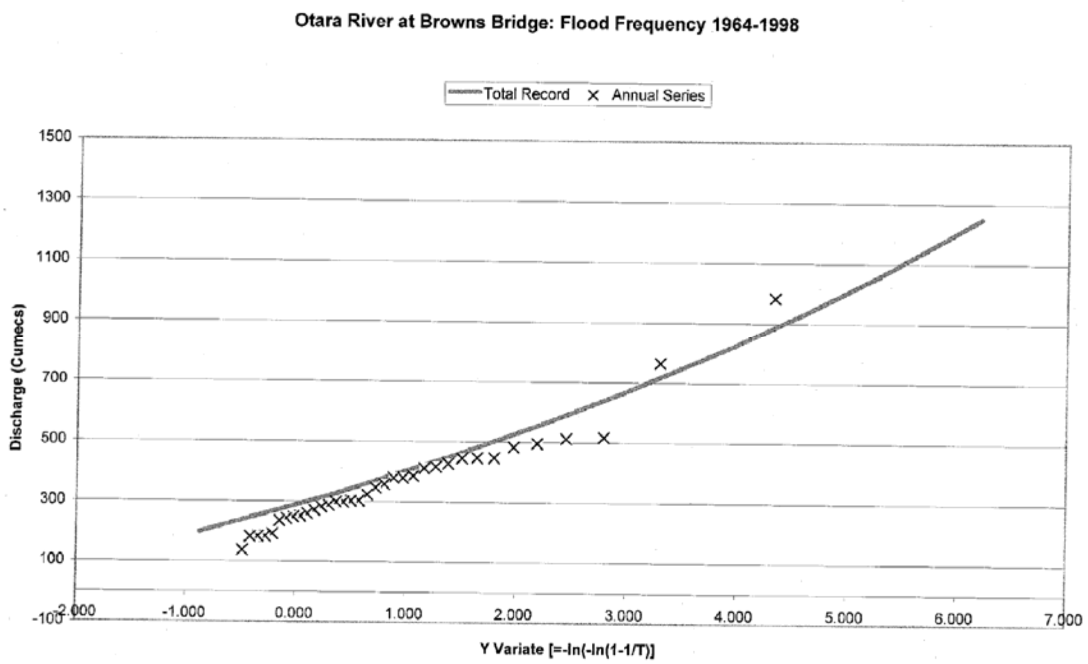

Waioeka River at Cableway: Flood Frequency 1958-2000
Total Record
Annual Series
Discharge (Cumecs)
3000
2500
2000
1500
1000
500
0
-2.000
-1.000
0.000
1.000
2.000
3.000
4.000
5.000
6.000
7.000
Y Variate [=-ln(-ln(1-1/T)]

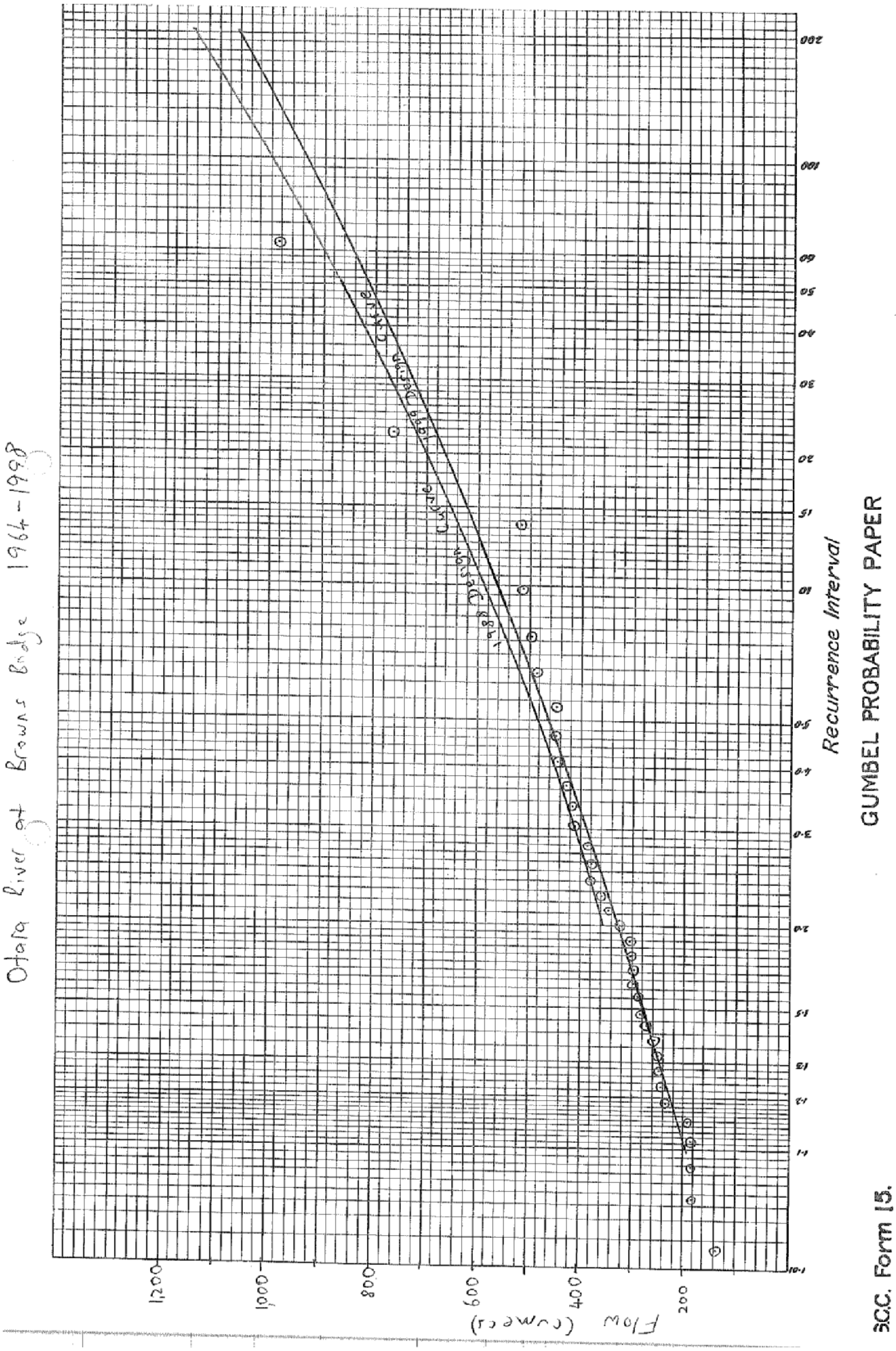
Otara at Browns Bridge

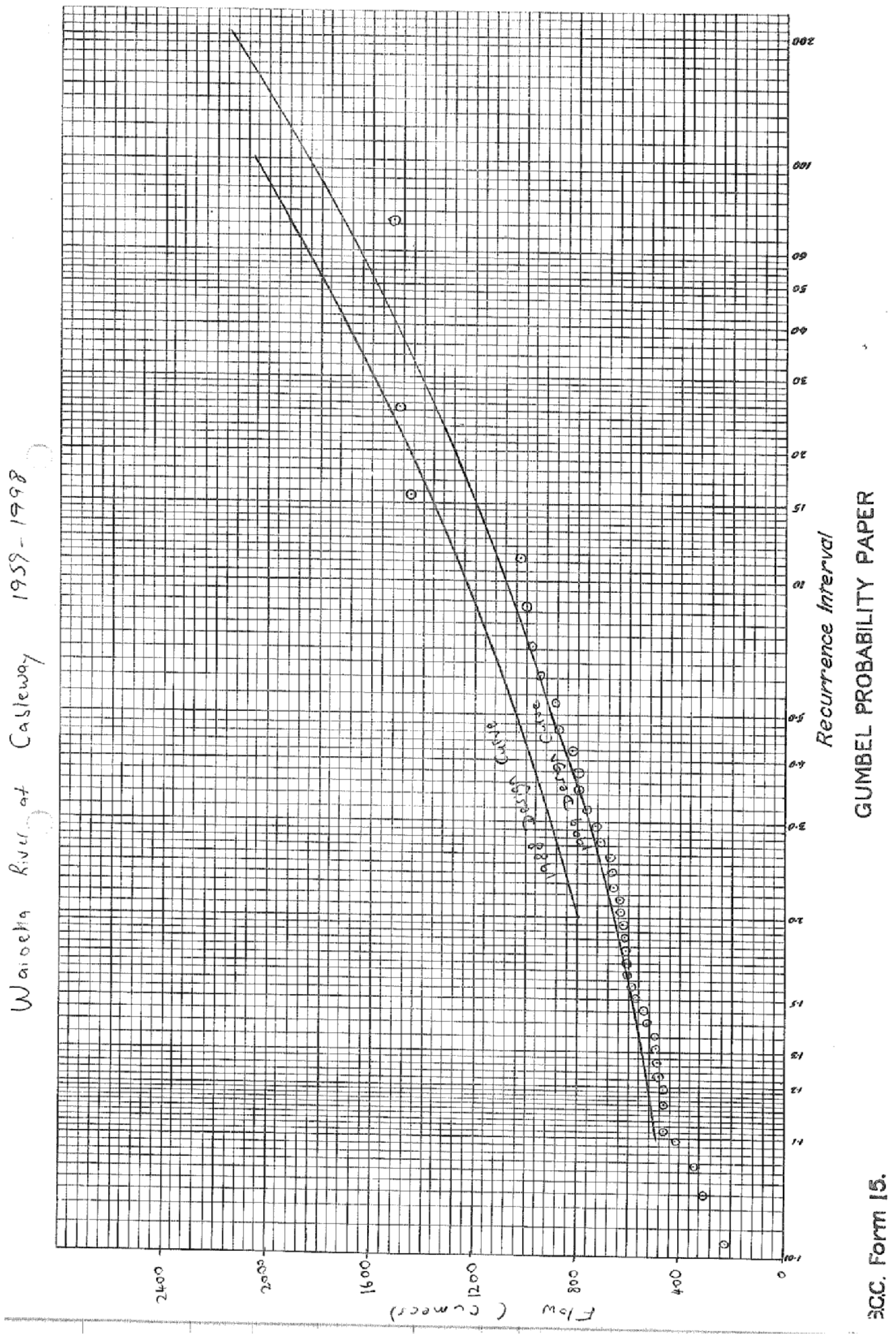
Annual Extremes

Discharge Total Record (Cumecs)	Discharge Total Record (Cumecs)	Rank	Return Period (Years)	Y Variate
1250	1300		500.000	6.214
1062	1062		200.000	5.296
932	932		100.000	4.600
812	812		50.000	3.902
666	666		20.000	2.970
562	562		10.000	2.250
463	463		5.000	1.500
327	327		2.000	0.367
197	197		1.100	-0.875
984		1	77.000	4.337
765		2	27.641	3.301
517		3	16.844	2.794
512		4	12.112	2.451
496		5	9.456	2.191
483		6	7.755	1.980
447		7	6.573	1.802
447		8	5.704	1.646
445		9	5.037	1.508
427		10	4.510	1.384
417		11	4.083	1.270
411		12	3.730	1.164
386		13	3.433	1.066
379		14	3.180	0.974
379		15	2.962	0.887
358		16	2.771	0.804
346		17	2.604	0.724
323		18	2.456	0.648
303		19	2.323	0.575
302		20	2.204	0.503
298		21	2.097	0.434
298		22	2.000	0.367
288		23	1.911	0.300
282		24	1.830	0.235
271		25	1.756	0.171
259		26	1.687	0.107
250		27	1.623	0.044
248		28	1.565	-0.019
243		29	1.510	-0.082
235		30	1.459	-0.146
192		31	1.411	-0.210
183		32	1.366	-0.275
183		33	1.324	-0.341
182		34	1.285	-0.410
137		35	1.248	-0.481

362.17
166.29


Otara River at Browns Bridge: Flood Frequency 1964-1998
Total Record
Annual Series
Discharge (Cumecs)
1500
1300
1100
900
700
500
300
100
-100
-2.000
-1.000
0.000
1.000
2.000
3.000
4.000
5.000
6.000
7.000
Y Variate [=-ln(-ln(1-1/T)]


Otara River at Browns Bridge 1964-1998
1988 Design Curve
1998 Design Curve
Flow (cumecs)
200
400
600
800
1000
1200
Recurrence Interval
GUMBEL PROBABILITY PAPER
SCC. Form 15.


Waioeka River at Cableway 1959-1998
GUMBEL PROBABILITY PAPER
Recurrence Interval
1988 Design Curve
Flow (cumecs)
2400
2000
1600
1200
800
400
0
1.01
1.1
1.2
1.3
1.5
2.0
3.0
4.0
5.0
10
15
20
30
40
50
60
100
200
BCC. Form 15.

Appendix 5 –1998 Flood Frequency Analyses FORTRAN Results

```
FREQUENCY PLOT FOR THE JENKINSON METHOD (QJENK5).
NOTE: XRET IS THE RETURN PERIOD IN YEARS (LOG SCALE) AND GDATA IS THE SERIES OF ANNUAL EXTREMES.

           %%  WAIOEKA RIVER AT CABLEWAY BIENNIAL ANALYSIS (1959-1998)              %%
THE DATA   %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

1. LOG-P   DATA READ INTO THE COMPUTER
2. LOG-P   *************************

3. LOG-P   LENGTH OF ANNUAL EXTREME SERIES: NYEAR=   20 YEARS
4. LOG-N   NUMBER OF DESIGN RETURN PERIODS: PERIOD=   8
5. GENER   ANNUAL EXTREMES READ IN: (IN  CUMECS )
6. EXTRE       611.00     725.00    1494.00     876.00    1447.00     976.00     762.00     625.00
7. GUMBE       821.00     800.00     575.00     492.00     675.00     799.00    1026.00     714.00
8. JENKI       887.00     610.00    1002.00    1521.00

           SPECIFIED RETURN PERIODS: (IN YEARS)

               2.00                 2.33
               5.00                10.00
              20.00                50.00
REFERENC     100.00               200.00

            *    *    *    *    *    *    *    *    *    *
METHOD                                                                                     )UENCIES.
1. U.S.
   BULLE   OUTPUT FROM ANALYSIS OF DATA
2. SAME    ****************************
   AND L   THIS PROGRAM USES THE PLOTTING-POSITION FORMULA PROPOSED BY I. GRINGORTEN,1963,
   WATER   JNL OF GEOPHYS. RESEARCH,VOL 68,NO 3,IE: T=(N+.12)/(I-.44),FOR ALL THE EXTREME
3. BOBEE   VALUE DISTRIBUTIONS.
   WATER   A COMPROMISE FORMULA PROPOSED BY THE NAT. ENVIRONMENT RES. COUNCIL,1975,FLOOD
4. U.S.    STUDIES REPORT,VOL 1,IE:T=(N+.2)/(I-.4),IS USED FOR THE LOG-PEARSON 3 AND
   BULLE   LOG-NORMAL DISTRIBUTIONS.
5. NATUR   THE WEIBULL (ING. VETENSKAPS AKAD. HANDL.,1939,VOL 151) PLOTTING POSITIONS,
   VOLUM   IE:T=(N+1)/I,ARE ALSO GIVEN - ALTHOUGH NOT USED IN THIS PROGRAM.
6. SAME    THE Y-VARIATE FOR THE EXTREME VALUE TYPE 1 DISTRIBUTION IS
7. ROBER   CALCULATED USING THE RETURN PERIODS GIVEN BY THE GRINGORTEN FORMULA.
   N.Z.    THE RATIO OF EACH EXTREME (X) TO THE MEAN OF THE EXTREMES
8. SAME    IS ALSO LISTED.
   HYDRC
   NORDI   ANNUAL EXTREMES (IN   CUMECS ) WITH THEIR RETURN PERIODS AND PROBABILITIES AND Y-VARIATES.
```

EXTREME VALUES	GRINGORTEN RET. PER.	GRINGORTEN PROBS.	NERC RET. PER.	NERC PROBS.	WEIBULL RET. PER.	WEIBULL PROBS.	Y-VARIATE	X/MEAN
492.00	1.029	0.972	1.031	0.970	1.050	0.952	-1.276	0.564
575.00	1.084	0.922	1.086	0.921	1.105	0.905	-0.939	0.659
610.00	1.146	0.873	1.148	0.871	1.167	0.857	-0.724	0.700
611.00	1.215	0.823	1.217	0.822	1.235	0.810	-0.549	0.701
625.00	1.293	0.773	1.295	0.772	1.313	0.762	-0.395	0.717
675.00	1.382	0.724	1.384	0.723	1.400	0.714	-0.252	0.774
714.00	1.484	0.674	1.485	0.673	1.500	0.667	-0.114	0.819
725.00	1.602	0.624	1.603	0.624	1.615	0.619	0.021	0.832
762.00	1.740	0.575	1.741	0.574	1.750	0.571	0.157	0.874
799.00	1.905	0.525	1.906	0.525	1.909	0.524	0.296	0.916
800.00	2.105	0.475	2.104	0.475	2.100	0.476	0.439	0.918
821.00	2.350	0.425	2.349	0.426	2.333	0.429	0.590	0.942
876.00	2.661	0.376	2.658	0.376	2.625	0.381	0.752	1.005
887.00	3.067	0.326	3.061	0.327	3.000	0.333	0.930	1.017
976.00	3.619	0.276	3.607	0.277	3.500	0.286	1.129	1.119
1002.00	4.412	0.227	4.391	0.228	4.200	0.238	1.359	1.149
1026.00	5.652	0.177	5.611	0.178	5.250	0.190	1.636	1.177
1447.00	7.859	0.127	7.769	0.129	7.000	0.143	1.994	1.660
1494.00	12.897	0.078	12.625	0.079	10.500	0.095	2.517	1.713
1521.00	35.929	0.028	33.667	0.030	21.000	0.048	3.567	1.744

RETURN PERIODS		STANDARD ERROR
2.00		52.59
2.33		65.00
5.00		121.13
10.00		167.75
20.00		212.61
50.00		270.73
100.00		314.29
200.00		357.69

```
BASIC PARAMETERS OF DATA
------------------------

MEAN              =  871.90
STD. DEVIATION =  301.87
COEFF. OF SKEW =  1.1963

LOGARITHMS OF DATA:

MEAN              = 6.7204
STD. DEVIATION = 0.3168
COEFF. OF SKEW = 0.6163
ADJUSTED COEFF.
OF SKEW           = 0.9511
(NOTE:ADJUSTED TO CORRECT FOR BIAS DUE TO LENGTH OF RECORD
SEE B.BOBEE & R.ROBITAILLE,1977,WATER RES.RESEARCH,VOL 13,N0 2,P427.)

REDUCED Y-VARIATE (FOR LEAST SQUARES GUMBEL METHOD):

MEAN           =0.5570
STD. DEVIATION =1.1892

 *    *    *    *    *    *    *    *    *    *

 PARAMETERS FOR GEV DISTRIBUTION:

U =  743.320  ALPHA =  205.097  KAY =    0.004

 PARAMETERS FOR EV1 DISTRIBUTION:

U =  743.320  ALPHA =  205.097
```

y – ln 2

2
1.5   —.7872   1.125
1.1   —1.5677   1.0083

%% WAIOEKA RIVER AT CABLEWAY (1959-1998) %%
%%

DATA READ INTO THE COMPUTER
****

LENGTH OF ANNUAL EXTREME SERIES: NYEAR= 40 YEARS
NUMBER OF DESIGN RETURN PERIODS: PERIOD= 8

ANNUAL EXTREMES READ IN: (IN CUMECS)

611.00	605.00	221.00	725.00	464.00	1494.00	876.00	532.00
1447.00	546.00	416.00	976.00	662.00	762.00	342.00	625.00
639.00	821.00	463.00	800.00	500.00	575.00	492.00	304.00
675.00	636.00	799.00	465.00	1026.00	943.00	594.00	714.00
887.00	483.00	484.00	610.00	1002.00	667.00	606.00	1521.00

SPECIFIED RETURN PERIODS: (IN YEARS)

2.00	2.33
5.00	10.00
20.00	50.00
100.00	200.00

* * * * * * * * * *

OUTPUT FROM ANALYSIS OF DATA
****

THIS PROGRAM USES THE PLOTTING-POSITION FORMULA PROPOSED BY I. GRINGORTEN,1963, JNL OF GEOPHYS. RESEARCH,VOL 68,NO 3,IE: T=(N+.12)/(I-.44),FOR ALL THE EXTREME VALUE DISTRIBUTIONS.
A COMPROMISE FORMULA PROPOSED BY THE NAT. ENVIRONMENT RES. COUNCIL,1975,FLOOD STUDIES REPORT,VOL 1,IE:T=(N+.2)/(I-.4),IS USED FOR THE LOG-PEARSON 3 AND LOG-NORMAL DISTRIBUTIONS.
THE WEIBULL (ING. VETENSKAPS AKAD. HANDL.,1939,VOL 151) PLOTTING POSITIONS, IE:T=(N+1)/I,ARE ALSO GIVEN - ALTHOUGH NOT USED IN THIS PROGRAM.
THE Y-VARIATE FOR THE EXTREME VALUE TYPE 1 DISTRIBUTION IS CALCULATED USING THE RETURN PERIODS GIVEN BY THE GRINGORTEN FORMULA.
THE RATIO OF EACH EXTREME (X) TO THE MEAN OF THE EXTREMES IS ALSO LISTED.

ANNUAL EXTREMES (IN CUMECS) WITH THEIR RETURN PERIODS AND PROBABILITIES AND Y-VARIATES.

EXTREME VALUES	GRINGORTEN RET. PER.	GRINGORTEN PROBS.	NERC RET. PER.	NERC PROBS.	WEIBULL RET. PER.	WEIBULL PROBS.	Y-VARIATE	X/MEAN
221.00	1.014	0.986	1.015	0.985	1.025	0.976	-1.452	0.316
304.00	1.040	0.961	1.041	0.960	1.051	0.951	-1.178	0.434
342.00	1.068	0.936	1.069	0.935	1.079	0.927	-1.012	0.488
416.00	1.097	0.911	1.098	0.910	1.108	0.902	-0.885	0.594
463.00	1.128	0.886	1.129	0.886	1.139	0.878	-0.777	0.661
464.00	1.161	0.861	1.162	0.861	1.171	0.854	-0.681	0.663
465.00	1.195	0.836	1.196	0.836	1.206	0.829	-0.594	0.664
483.00	1.232	0.812	1.233	0.811	1.242	0.805	-0.512	0.690

THE STANDARD ERROR GIVES AN ORDER OF MAGNITUDE ONLY OF THE STANDARD ERROR OF ESTIMATE.
IT IS CALCULATED FROM A FORMULA IN THE FLOOD STUDIES REPORT (NERC, 1975, VOL.1, P.170) ASSUMING A COEFFICIENT OF VARIATION OF 0.40.

NOTE: THE GEV DISTRIBUTION IS AN EXTREME VALUE TYPE 3

THE METHOD WHICH PROVIDES THE BEST LINE OF FIT TO THE PLOTTED DATA CAN BE ESTIMATED BY REFERENCE TO THE DISCHARGE VS RET.PERIOD GRAPHS FOLLOWING.IF A CHI-SQUARED GOODNESS OF FIT TEST IS DESIRED,THE FOLLOWING DATA GIVES CHI-SQUARE VALUES AND DEGREES OF FREEDOM FOR EACH METHOD.

NOTE: *** DUE TO THE SMALL SIZE OF THE SAMPLE,THIS TEST WILL NOT BE SUFFICIENTLY SENSITIVE FOR GREAT RELIANCE TO BE PLACED ON THE FOLLOWING RESULTS:

VALUE OF CHI-SQUARE FOR METHOD 1 = 0.737
DEGREES OF FREEDOM ARE: 1.

VALUE OF CHI-SQUARE FOR METHOD 2 = 0.737
DEGREES OF FREEDOM ARE: 1.

VALUE OF CHI-SQUARE FOR METHOD 3 = 0.737
DEGREES OF FREEDOM ARE: 1.

VALUE OF CHI-SQUARE FOR METHOD 4 = 0.737
DEGREES OF FREEDOM ARE: 2.

VALUE OF CHI-SQUARE FOR METHOD 5 = 0.737
DEGREES OF FREEDOM ARE: 2.

VALUE OF CHI-SQUARE FOR METHOD 6 = 0.737
DEGREES OF FREEDOM ARE: 2.

VALUE OF CHI-SQUARE FOR METHOD 7 = 0.737
DEGREES OF FREEDOM ARE: 2.

VALUE OF CHI-SQUARE FOR METHOD 8 = 6.000
DEGREES OF FREEDOM ARE: 2.

%% OTARA RIVER AT BROWN'S BRIDGE (1964-1998) %%

FREQUENCY ANALYSIS ESTIMATES (IN CUMECS) FOR THE SPECIFIED RETURN PERIODS:

RETURN PERIODS	LOG-PEARSON	L.PEAR. ADJUSTED	L.PEAR. BOBEE	LOG-NORMAL	GENERAL EV DIST	EV1 DIST.	GUMBEL	JENK.	STANDARD ERROR	
1.10	193.94	195.01	192.52	190.68	197.01	194.97	166.30	152.75	-1.44	
1.50	271.26	270.49	271.22	274.10	276.86	283.07	272.52	257.21	8.57	
2.00	326.38	324.62	327.11	332.40	327.29	335.06	335.20	318.85	16.51	
2.33	355.23	353.57	355.85	360.79	351.38	359.00	364.06	347.23	20.41	
5.00	467.26	466.16	467.36	470.35	462.81	462.99	489.45	470.53	38.04	
10.00	569.60	571.10	567.92	563.91	562.27	547.70	591.57	570.96	52.67	
20.00	683.91	690.05	678.83	662.51	665.69	628.95	689.53	667.30	66.76	
50.00	821.20	835.05	810.26	775.25	812.29	734.12	816.33	791.99	85.01	7
100.00	939.47	961.81	922.49	867.25	932.40	812.93	911.35	885.43	98.69	6.620
200.00	1065.36	1098.46	1040.78	961.41	1061.59	891.45	1006.02	978.53	112.31	5291

500 1271 linear extrap 6.214

THE STANDARD ERROR GIVES AN ORDER OF MAGNITUDE ONLY OF THE STANDARD ERROR OF ESTIMATE.
IT IS CALCULATED FROM A FORMULA IN THE FLOOD STUDIES REPORT (NERC, 1975, VOL.1, P.170) ASSUMING A COEFFICIENT OF VARIATION OF 0.40.

IMPORTANT *** THE TREND IN THE LOWER END OF THE DATA SERIES HAS CAUSED A TYPE 2 E.V. DISTRIBUTION TO BE FITTED TO THE DATA IN METHOD 5. SUCH A DISTRIBUTION HAS NO UPPER BOUND AND THUS THE RESULTS FROM THE METHOD SHOULD BE REGARDED WITH CAUTION. IT IS RECOMMENDED THAT THE EV2 CURVE SHOULD BE COMPARED WITH THE REGIONAL FLOOD FREQUENCY CURVE IN CHECKING ITS VALIDITY.

THE METHOD WHICH PROVIDES THE BEST LINE OF FIT TO THE PLOTTED DATA CAN BE ESTIMATED BY REFERENCE TO THE DISCHARGE VS RET.PERIOD GRAPHS FOLLOWING.IF A CHI-SQUARED GOODNESS OF FIT TEST IS DESIRED,THE FOLLOWING DATA GIVES CHI-SQUARE VALUES AND DEGREES OF FREEDOM FOR EACH METHOD.

VALUE OF CHI-SQUARE FOR METHOD 1 = 4.400
DEGREES OF FREEDOM ARE: 3.

VALUE OF CHI-SQUARE FOR METHOD 2 = 4.400
DEGREES OF FREEDOM ARE: 3.

VALUE OF CHI-SQUARE FOR METHOD 3 = 4.400
DEGREES OF FREEDOM ARE: 3.

VALUE OF CHI-SQUARE FOR METHOD 4 = 5.200
DEGREES OF FREEDOM ARE: 4.

VALUE OF CHI-SQUARE FOR METHOD 5 = 3.600
DEGREES OF FREEDOM ARE: 4.

VALUE OF CHI-SQUARE FOR METHOD 6 = 2.800
DEGREES OF FREEDOM ARE: 4.

VALUE OF CHI-SQUARE FOR METHOD 7 = 5.600
DEGREES OF FREEDOM ARE: 4.

VALUE OF CHI-SQUARE FOR METHOD 8 = 9.200
DEGREES OF FREEDOM ARE: 4.

FREQUENCY PLOT FOR THE JENKINSON METHOD (QJENK5).
NOTE: XRET IS THE RETURN PERIOD IN YEARS (LOG SCALE) AND GDATA IS THE SERIES OF ANNUAL EXTREMES.

THE DATA ARE ANALYSED BY THE FOLLOWING METHODS:

1. LOG-PEARSON TYPE 3 - FITTED BY THE METHOD OF MOMENTS.
2. LOG-PEARSON TYPE 3 (WITH AN ADJUSTED COEFFICIENT OF SKEW) - FITTED BY THE METHOD OF MOMENTS.
3. LOG-PEARSON TYPE 3 (BOBEE METHOD) - FITTED BY THE METHOD OF MOMENTS.
4. LOG-NORMAL - FITTED BY THE MAXIMUM LIKELIHOOD (ML) METHOD.
5. GENERAL EXTREME VALUE - FITTED BY THE ML METHOD
6. EXTREME VALUE TYPE 1 (EV1 OR GUMBEL) - FITTED BY THE ML METHOD.
7. GUMBEL - FITTED BY THE LEAST SQUARES METHOD.
8. JENKINSON - FITTED BY THE ML METHOD.

REFERENCES FOR THE ABOVE METHODS :

METHOD
1. U.S. WATER RESOURCES COUNCIL, 1967: A UNIFORM TECHNIQUE FOR DETERMINING FLOOD FLOW FREQUENCIES. BULLETIN NO.15, DECEMBER, 15PP.
2. SAME AS FOR 1. ALSO BOBEE AND ROBITAILLE, 1977:THE USE OF THE PEARSON 3 AND LOG PEARSON TYPE 3 DISTRIBUTIONS REVISITED. WATER RESOURCES RESEARCH, VOL. 13, NO. 2, PP.427-443.
3. BOBEE, 1975: THE LOG PEARSON TYPE 3 DISTRIBUTION AND ITS APPLICATION IN HYDROLOGY. WATER RESOURCES RESEARCH, VOL. 11, NO. 5, P.681-689.
4. U.S. WATER RESOURCES COUNCIL, 1977: GUIDELINES FOR DETERMINING FLOOD FLOW FREQUENCY. BULLETIN 17A OF THE HYDROLOGICAL COMMITTEE, JUNE.
5. NATURAL ENVIRONMENTAL RESEARCH COUNCIL, 1975: FLOOD STUDIES REPORT. VOLUME 1, HYDROLOGICAL STUDIES. LONDON.
6. SAME AS FOR 5.
7. ROBERTSON, 1963: THE FREQUENCY OF HIGH INTENSITY RAINFALLS IN N.Z. N.Z. METEOROLOGICAL SERVICE, MISC. PUBLN. 118.
8. SAME AS FOR 5. ALSO SAMUELSSON, 1972: STATISTICAL INTERPRETATION OF HYDROMETEOROLOGICAL EXTREME EVENTS. NORDIC HYDROLOGY, VOL. 3, NO. 4, PP.199-233.

Appendix 5 –Technical Review of the "Waioeka-Otara: Flood Frequency Analyses"

7 April 2009

Mr Jonathan Freeman
Engineering Hydrologist
Evironment BoP
P.O. Box 364
WHAKATANE

Dear Jonathan,

Re: Technical review of *"Waioeka-Otara Rivers: Flood Frequency Analyses"*

I have reviewed your draft paper on *"Waioeka-Otara Rivers: Flood Frequency Analyses"* and would make the following comments:

1. Overall your recommendations as to the design flows would appear to be consistent with my own analysis, although your values are slightly more conservative (higher). This would result in a slightly higher flood levels and therefore more robust design.

2. It is not clear from your report whether the emphasis is on the various analyses used, and how these have changed over time, or on providing the best estimates of the potential flood events for various return periods. Personally, I would like to see the emphasis squarely on the analysis of the current flow data set using our understanding of the most appropriate techniques. While reference to previous work is important, I would see this more as providing a chronology and more robustness to your estimates. It is important to review the historic estimates and to postulate various explanations as to why current estimates tend to be significantly lower – but that these estimates are robust and will not lead to under-designed flood schemes or assessments of flood risk.

3. The critical element to me in this study is that the methodology is robust, accepted, and that the results have been critiqued. All these aspects have been met but detailed reference to previous work tends to 'dilute' the confidence that the reader places in your results.

4. The referencing is 'non-standard' but that is OK given the nature of the report, and the fact that your approach still provides sufficient information for any interested party to find the material if necessary.

5. I would like to see the report start with a graphical display of the various data sets used in the analysis – for the two sites (Figure 1). A short discussion of the various patterns in the flow data would then be helpful. Are there any patterns present? How might these patterns affect the analysis, and more critically how might they be used to explain the reduction in apparent flood magnitudes overtime.

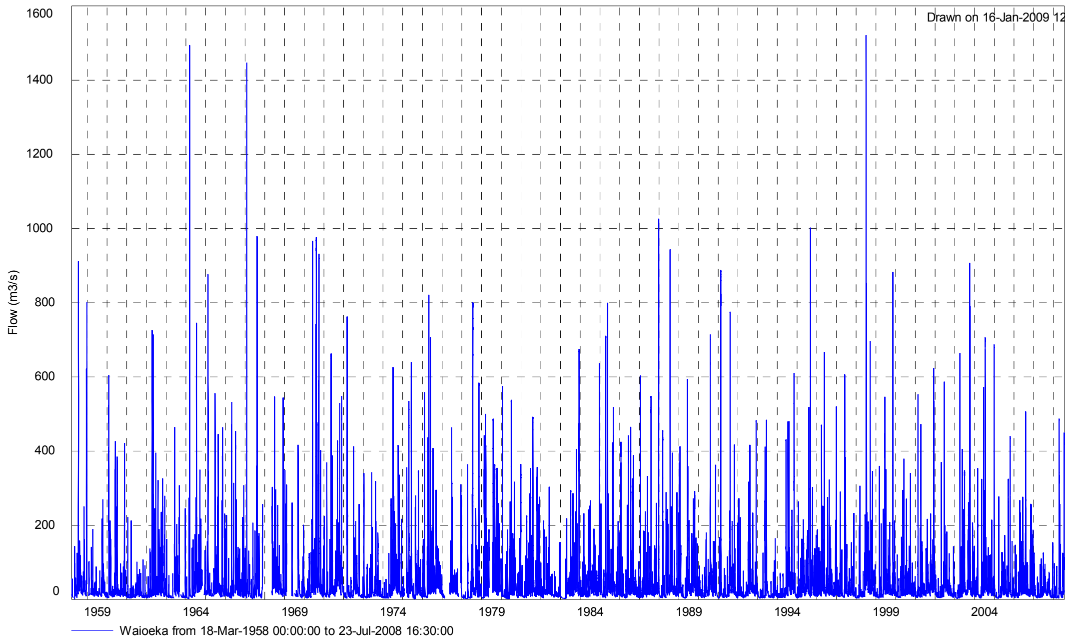


Figure 1: Flow regime of the Waioeka showing the significance of the three largest flood events and the cyclic flow behaviour caused by the variations in the IPO.

6. A statistical summary of the data would also be helpful. Something similar to that below produced by using PSUM in Tideda or Hilltop. The nature of the two flow regimes could then be discussed with similarities and differences of the two records highlighted. Given the adjacent nature of the catchments, but the significant differences found later when relating the Regional results to the Frequency Analysis, this might be quite useful.

Table 1 Summary of flow variability of the Waioeka.

Min	Max	Mean	Std Dev	L.Q.	Median	U.Q.
0.93	1520.54	31.36	50.17	9.32	16.69	32.78

7. How do the flow regimes of the two rivers relate one to the other? The relationship between the maximum annual discharges of the two catchments would appear to show that the control of flood peak is not solely a function of catchment area. What are the other controls? Why is rainfall in the two catchments so different? Do you have any idea of the time of concentration of the catchments? This would affect the duration of the critical storms, and consequently the rainfall intensities and runoff volumes.

8. I am a little concerned about the use of the 1918 event in the analysis. This is not really the largest event in *'recorded history'* since it is unlikely that the event was actually recorded. It would be interesting to know how the estimates of flood peak for this event were derived, and therefore how much confidence can be placed in these data.

9. In the catchment descriptions it would be useful to include the percentage (area) of the catchment upstream and downstream of the gauging sites. It would also be useful to include the locations of the gauging sites so the reader has a better understanding of the arrangement of the catchment, recorders, flow data, flood estimates etc. What is the percentage of the catchment that is ungauged? What are the implications to this with respect to the estimates of flood magnitudes and downstream flooding at Opotiki? This is important since I assume that this was the principal aim of your study.

10. The effect of vegetation on flood magnitude tends to decrease as the magnitude increases (and therefore the return period). This is because the bigger floods are when all the catchment storage is full i.e., the effect of vegetation is saturated, and therefore limited.

11. I am not sure why you have included the discussion of slips. This is really a separate issue, unless they have been responsible for some of the larger flood events. If their effect is not known then be careful that the discussion does not give more weight to this issue than it deserves. If landslides into the rivers had blocked the flow in the past, and created major flood events, then this would be important. For example, if such a mechanism was known to explain the three largest events on record then this would be a critical consideration. I can see no such suggestion in you report however.

12. With regard to your Figure 1; describe the strong cyclic pattern apparent in the distribution and magnitude of the maximum annual floods. It would appear that this relates to the IPO index. You could add a line on the graph showing the 'phase' of the IPO index. Although you discuss the impact of the IPO later, it might be worth at least highlighting the cyclic pattern. It is also important to point out that there do not appear to be sequences of either high or low flows – rather the pattern is cyclic and oscillating rather than abrupt. It is also important to point out that the total length of record includes at least two complete cycles. Therefore, analysing the entire record has incorporated this cyclic behaviour into your analysis. This would be an issue if you had a short length of record. The evidence in the graph is that your analysis will be robust and provides good estimates of the magnitude of events with particular return periods.

13. I would be tempted to just summarise the various estimates of flood events for particular return periods, and discuss how these have changed through time. I might even go so far as to suggest that you start with your analysis and then relate these earlier estimates to yours explaining and accounting for the differences. Fundamentally, I think the emphasis should be on your results – the best available – and too much doubt should not be cast on your analysis by over-emphasising earlier studies – which suffered from significantly more and greater limitations than your present study. How relevant are these earlier analyses? Do you really want to focus attention on them? Your results were derived using the longest data set available, and using a range of standard methodologies. Therefore, they must be considered the most reliable estimates available; and their use should be encouraged,

14. I am not sure of the relevance of the stage records as opposed to the gauged (flow) records. The stage records only show the water level relative to the recorder. I would assume that the beds of these two rivers are highly mobile and variable. Therefore, the stage record can vary even if the volume of water does not. This just creates confusion. You could use BEDPLOTS to look at the relative stability of the river beds at the gauging sites to provide some data to support your discussion here. I would be tempted to discuss the flow records only as these are the ones used in the analysis. Since no reference to the records from the other sites is made in the text, why are they discussed in the data section? If you were to extrapolate your analyses to other sites using these data then their reference might be justified, but again they just seem to add confusion. Their inclusion would appear unnecessary.

15. The design curves referred to in Appendix 3 are not really much use as they do not contain the data from which the curves were derived. That is, we do not know how good a fit the curves were, and therefore the reliability of the estimates. Given the degree of scatter using the most recent and longest records, and the fact that the more powerful statistical routines still have difficulty fitting a 'standard' curve to the flow data, one can only hazard a guess as to how reliable these curves actually were. It is very important to plot the data to show the reliability of the curve, and by inference the estimates of peak discharge (as you have done in your own analysis).

16. FORTRAN is actually a programming language and not a software package – at least not by my definition. That is, a FORTRAN based program was written to calculate the plotting positions and estimate the relationship between flood magnitude and return period. This is now done internally by routines included in hydrometric software such as Tideda and Hilltop. The use of these standard programs, using standard routines, produces consistent and reliable estimates without the risk of error – at least in terms of the calculations etc.

17. The use of historic data is particularly problematic. While the inclusion of the 1918 event is attractive, since it was the biggest event experienced, it does tend to distort the distribution and make fitting a standard statistical relationship more difficult. As mentioned previously, we do not really know how reliable the estimates of the magnitude of this flood actually were. Given that it is the biggest on record it tends to tie down the whole relationship, and the fit of the line. If this event was actually not as big as believed then the entire distribution would flatten and the estimates of flood magnitude would drop considerably.

18. In general, it would have to be argued that the longer the data record and the more annual floods the more reliable the analysis. I am therefore a little dubious about using the biennial analysis – particularly when it is justified in one case and rejected in the other. This is particularly the case given that the two catchments are adjacent, and therefore one would expect them to be affected by the same processes etc. Also, in a couple of places you say bi annual rather than biennial.

19. I would like to see a Table that summarises the changes in the estimates of the various return period events through time. This would make it quite clear as to how your estimates relate to those of earlier studies. These could then be referred to just with references to the various reports etc. rather than having all the detail that is in the appendix. What is the purpose of the appendix? Is it really helpful? You could then explain how the various estimates change, the causes of these changes, how the data available relate to the estimates, and therefore why your current estimates are considered the best.

20. As discussed previously I am not sure of the value of discussing all the various data sets available if you did not use them. If there are only two reliable flow records, as opposed to stage records, then this is why they were the focus of your analysis. Also, you can't estimate flood magnitudes from a stage record – certainly not if the bed is mobile and subject to change. Perhaps you could add a map showing the relationship between the various sites – how do the two used in the analysis relate, why are the other sites there etc. Again, I would be reluctant to see too much discussion of sites that are not used in the analysis.

21. When discussing the reliability of the data and the rating curves it might be useful to include a BEDPLOT of the two sites. This can be quickly and easily done within either Tideda or Hilltop. It basically shows for each gauging the difference between the actual stage and the predicted stage for that discharge. The better and more stable the rating the smaller the deviation about "0", and the variation tends to fluctuate about the "0" rather than being biased in one particular direction. How much error is associated with the rating curve? You state 5-10% but how is this derived? Each gauging, assuming best practice, is only to within ±8%. However, the errors are significantly higher under flood conditions; the flow is more variable, the river is unlikely to be gauged at the peak, and the bed is more mobile. All these factors affect the accuracy of the gauging and ultimately the rating curve. How do the gaugings relate to the high flows? If you plot the gaugings on the ratings you can see the highest flows gauged. These are critical in controlling the top end of the rating curve and ultimately its reliability at estimating the size of specific flood events. This point is really critical when discussing the reliability of the estimates of the maximum annual flood series.

22. Tideda is actually a software program and not data! Tideda can be used to store data that is all.

23. Why did you want to maintain comparability of methodology with previous studies? While this might be useful if you are trying to replicate their results, what you wanted for this study was the best and most robust and accurate methodology i.e., you wanted to apply current best practice to get the best and most reliable estimates of flood magnitude and return period. It may be difficult to quality control (or even replicate) previous studies and therefore the use of those routines is perhaps a little suspect.

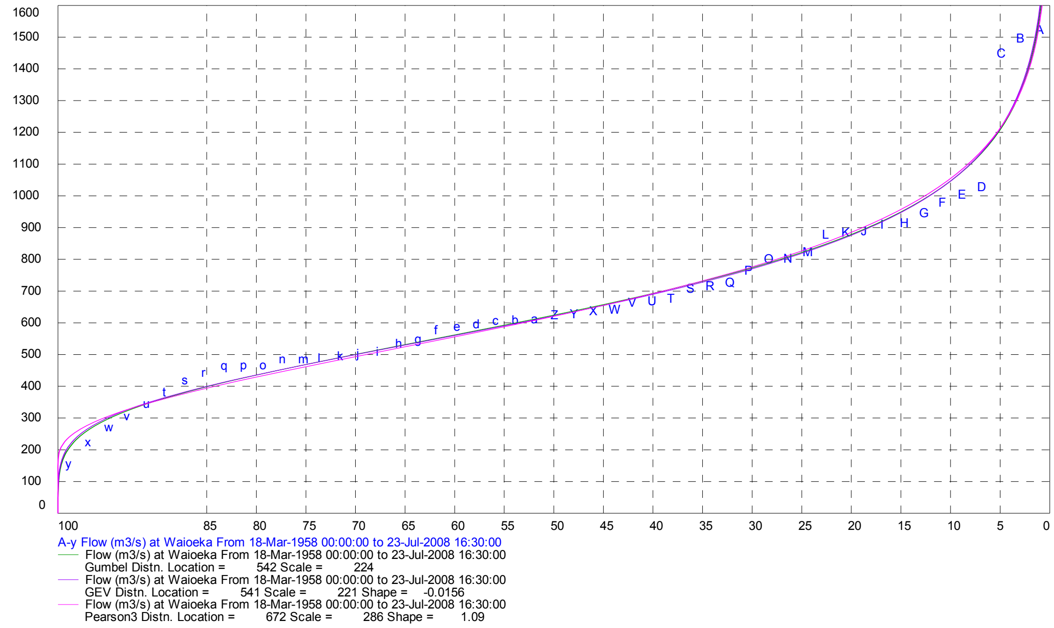


Figure 2 Annual flood series and the goodness of fit of three common statistical distributions.

24. When discussing the various statistical distributions i.e., EV1 (Gumbel), EV2 (GEV), and EV3 (PE3) it might be useful to stick to the one term rather than switching back and forward. It might also be worthwhile to point out that these are ‘standard’ statistical distributions which one hopes will approximate the actual distribution of our flood series.

25. Having fitted the distributions, the critical element is how well the ‘theoretical’ curve fits the actual data, especially at the higher end (more extreme) of the flow distribution.

26. To check your results I ran the same analysis but used the entire Waioeka record. I did not remove the ‘suspect’ years where you could not be sure that the major flood in a particular year was actually recorded because of gaps in the data. As a consequence, my results are slightly different to yours.

27. The first thing that is apparent is that the EV1, EV2, and EV3 distributions all produce very similar ‘fits’ to the data (Figure 2). It is also apparent that none of these distributions fit the actual distribution of annual flood maxima very well. They tend to under-estimate the low return period events, and over-estimate the higher return period flows.

28. It would appear that the Wakeby distribution actually fits the overall distribution better (Figure 3). It is particular good at modelling all flows up to about 900m^3/s. It then mirrors the EV1 distribution although it is perhaps better at fitting the extreme flood events – minimising the deviations of the actual data from the line.

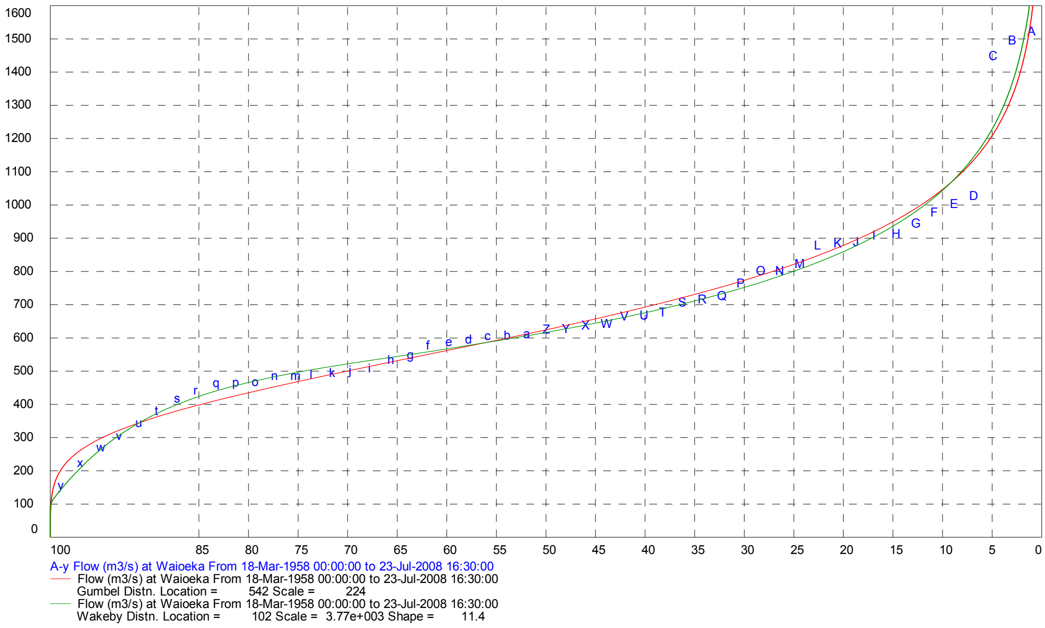


Figure 3 The Wakeby distribution would appear to fit the annual flood series best over the entire range of flows experienced.

29. The effect of which distribution is chosen to best represent the annual flood series has a significant impact on the magnitude of the various flood estimates (Table 2). It can be seen that at return periods less than about 50 years the choice of using either a EV1, EV2, or EV3 distribution has very little impact on the resulting flood magnitude. The Wakeby distribution produces a smaller estimate for the ‘mean annual flood’ often estimated from the 2.33-year RP event. The estimates for 5-20-year RPs are very similar to those predicted from the other distributions. For return periods above about 50-years, the Wakeby distribution estimates flows about 10% higher than the other distributions. They are, however, still about 10% lower than the estimates using your methodology. This difference is not great given all the uncertainty in the analysis and data. It is also perhaps good that your values are the more conservative (i.e., larger) giving a greater margins of safety.

Table 2 *Estimates of the flood magnitude for various return periods using different distributions for the Waioeka.*

Return Period	Gumbel	GEV	PE3	Wakeby	Freeman (2009)
2.33	672	669	670	657	678
5	878	876	885	860	868
10	1047	1047	1055	1044	1040
20	1208	1212	1213	1228	1221
50	1417	1430	1410	1473	1480
100	1574	1595	1553	1657	1695
200	1730	1761	1692	1843	1928
500	1936	1983	1871	2088	2268
1000	2091	2152	2004	2273	

30. It is also possible to estimate the return period for known events using the various distributions (Table 3). Again, while the EV1, EV2, and EV3 distributions all produce very similar estimates, the Wakeby distribution would suggest that these events are actually not as ‘rare’ as some may have argued.

Table 3 *Estimates of the return periods of recorded flood events on the Waioeka.*

Specific floods	Actual discharge (m^3/s)	Estimated return period			
		Gumbel	GEV	PE3	Wakeby
2/07/1998	1521	79	73	86	60
11/03/1964	1494	70	66	75	54
3/02/1967	1447	57	54	60	45

31. In summary, the estimates of the flood magnitudes you recommend would appear to be reasonable if perhaps a little conservative. Given that they tend to be lower than the estimates provided in earlier studies such an approach is sensible – it could even be argued that this allows for some influence of climate change!

32. With regard to the regional method it would be interesting to know which rivers in this area were actually used in developing the methodology. One assumes that they were most likely the ones in this study so the data should be reasonably reliable and valid. It might be useful to include this comparison in your report. Your appendix actually contains the data, and if my reading is correct, the regional method gives estimates for the 100-year event on the Waioeka that is only 10% different (1441m^3/s vs 1550-1650m^3/s) to that derived from statistical analysis of the annual flood series. This shows a high degree of comparability and reliability. I think the documentation of the methodology suggests errors of up to 30%!

33. The regional method requires that the rainfall-runoff relationships are the same for the catchments being compared and that they lie in the same ‘zone’ on the maps. These criteria are certainly met for adjacent catchments as in the current study.
34. The estimates based on transposition by area also seem reasonable. It would be useful to check whether there is a strong orographic component to rainfall (and therefore potentially

flood magnitude) in these catchments. Given the location of Tutaetoko between the other two catchments this should not be an issue, but it would appear from the maps provided that this catchment lies at slightly lower elevation. Therefore, if there is a strong orographic component to storm rainfall you might expect floods to vary not as a simple function of area i.e., if the rainfall is less then you might expect a smaller flood than indicated as a function of area alone. Any orographic effect in the Otara catchment would bias the data to higher flows at Brown's Bridge giving higher estimated flows for the Tutaetoko.

35. I think that it is important to link the discussion of the IPO in section 5.1 back to your Figure 1. What would appear to be the influence of the IPO? What is the effect of this on the distribution of flood events over time? What is the effect of this on the frequency analysis? How might this have affected the results of previous reports? It would appear to me that the IPO has a very powerful influence on the flood magnitudes and their distribution through time. As a result it has had an influence on previous estimates of flood magnitude. This would explain the difference between the earlier flood estimates and your own.

36. The inclusion of the effects of climate change is particularly problematic. As you point out there is some guidance as to how warming temperatures might affect rainfall, but how this will translate into flood magnitude is unknown. Your approach therefore is sensible. It addresses the issue but leaves the final choice as to exactly what to do up to those using the information. Which temperature increases would you recommend to adjust the rainfall? Is there any seasonal pattern to your flood maxima data? If there is, then perhaps you can use that seasonal increase. If the largest floods each year are randomly distributed with respect to season then to be conservative you should use the largest seasonal increase. The relationship of the percentage increase in rainfall to a percentage increase in flood magnitude has still to be defined. Some adjust the percentages in the same manner i.e., 15% increase in rainfall equates to a 15% increase in flood magnitude. Peter Blackwood used a larger percentage, although I have never seen his justification. Another approach I have seen used by a number of Councils is to use the 200-year RP storm as the estimate for the 100-year storm adjusted for climate change. There is no definitive answer to this question so your approach seems appropriate.

37. I have already discussed how my estimates of flood magnitude compare to yours, and that there is a high degree of similarity. Therefore, I would argue that your methodology has been applied correctly. The choice as to which statistical distribution to fit to your data remains a little problematic. No distribution appears to fit the largest 3 floods very well, and yet these are the events that really control the estimation of all the flood frequency analysis. I would suggest that the Wakeby fits slightly better but the effect is relatively small i.e., <10% and this is certainly within the margin of error of the data and other steps in the process.

38. The three largest floods on record do appear to have a distinctly different distribution to the rest. It may be that you were just 'unlucky' to get three big events early in the record. The longer the record the more these extreme values will be drawn back into the overall distribution. It is also possible that the mechanism that created these floods i.e., the rainfall generating and storm conditions; were different. It might be worth checking this to make sure that the same rainfall-runoff relationships were operating throughout the entire record. It is also possible that the estimation of these large events is in error. Again, it might be worth checking whether these estimates are distinctly different in any way. Did these events go overbank whereas the others did not? Assuming that these events are real, and caused by the same mechanisms, then they must be included and managed as best as possible. Certainly given the size and magnitude of these events they cannot be ignored but they must be understood.

39. The use of 'censored' analysis and historic data are both very subjective and prone to large differences in interpretation. While such considerations may be appropriate, they should be used with caution. They both add considerable uncertainty to the analysis which otherwise is based on sound, 'accurate' data. If such approaches cannot be justified, I would recommend not using them. It would appear that you have tended to move away from them and therefore their inclusion and discussion perhaps applies more weight and emphasis than they warrant.

40. If you are going to use the biennial data and a GEV (EV2) distribution in your final recommendation it really needs to be well justified. Personally, I would prefer to see the entire data set used. I would also prefer to see a consistent methodology applied to both catchments unless you can provide strong justification for doing something else. Since the two catchments are adjacent you would expect the same processes and rainfall-runoff relationships to apply. Therefore, to use different methods in each catchment just because they seem to work 'better' seems a little hard to justify.

41. As I mentioned above, I would like to see you compare your regional estimates for the two rivers for which you have flow data to those estimates derived from frequency analysis. This will provide a measure of the appropriateness or otherwise of using the regional approach. Since the regional approach has essentially averaged and smoothed the data, those values derived from 'at site' analysis must be better. However, a simple comparison will allow you to justify the use of the approach on the Tutaetoko River.

42. In your transposition by area analysis I am surprised by the big difference between the Otara and Waioeka catchments. Given that they are on either side of the Tutaetoko I would have thought the comparison would have been closer. This brings us back to the discussion of the flood records of both catchments. Perhaps some simple comparison using yields (flow/unit area) might help to identify why this large difference occurs. The difference is about 20% (244m^3/s for the Waioeka and 303m^3/s for the Otara) and this seems high. This would suggest that the yield from the Otara is significantly higher than that from the Waioeka. This may relate to either the rainfall pattern affecting the catchments, or catchment physical conditions. It might be worth trying to establish exactly what causing the difference.

43. I think in terms of providing an estimate of flood flows for the Tutaetoko catchment you would be better to just average the two transposition values i.e., for the 100-year event 244m^3/s and 303m^3/s. This would give a value of 274m^3/s which is about 20% more than when you include the regional and previous estimates in your analysis. This would ensure that the recommendation is based on the actual flow records from the adjacent catchments and that the flood estimation is conservative.

44. I agree with your conclusions regarding the IPO. It obviously has a significant affect on the flow regime and flood history of these catchments. However, the flow record is now long enough to include the full effect of the IPO during both positive and negative phases. Therefore, I would argue that the full effect of the IPO is built into your analysis. Your analysis is not biased by a predominance of data from one particular phase which might have been the case in earlier analyses when the data record was significantly shorter.

45. With regard to climate change you might like to emphasise your 'best guess' since this is basically what it will be. Even the changes in the MfE recommendations between 2004 and 2008 are so great as to make 'predictions' risky. Again, refer to the seasonal distribution of floods, and use the appropriate increase in temperature and rainfall to get a 'most likely' increase in runoff.

46. The 10% reduction in the estimated size of specific flood events since 1998 is consistent with the flow data and annual maxima series shown in your Figure 1. It would appear that these catchments were 'just unlucky' to get some really large floods early in their records. This tends to bias earlier analyses.

47. I thought your discussion was good but include the various issues I have mentioned above. As mentioned, I don't like the 'censored' analysis and I think your discussion should be firm on which values to use. Don't leave any doubt in the readers mind once you get to the end.

I hope that these comments are helpful. Please give me a call if you would like to discuss any of my comments in more detail, or if you need any clarification.

Yours sincerely

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