Technical Compendium

Taupō District Flood Hazard Studies

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Contents

1 Introduction

Taupō District Council (TDC) engaged Opus International Consultants Ltd to assess the flood hazard posed by Lake Taupō and its six major tributaries. While there are a number of editions of some of the flood studies, the latest iterations are presented in the following reports:

- Knight, J. & McConchie, J. 2010: Taupō District Flood Hazard Study: Tauranga Taupō River. Report prepared by Opus International Consultants for Environment Waikato and Taupō District Council. July 2010. 48p.

- Maas, F. & McConchie, J. 2011. Taupō District Flood Hazard Study: Tongariro River. Report prepared by Opus International Consultants for Environment Waikato and Taupō District Council. July 2011. 59p.

- Smith, H. Paine S. & Ward, H. 2011: Taupō District Flood Hazard Study: Kuratau River. Report prepared by Opus International Consultants for Environment Waikato and Taupō District Council. July 2011. 52p.

- Paine, S. & Smith, H. 2012: Taupō District Flood Hazard Study: Hinemaiaia River. Report prepared by Opus International Consultants for Environment Waikato and Taupō District Council. June 2012. 46p

- Paine, S. & Smith, H. 2012: Taupō District Flood Hazard Study: Whareroa Stream. Report prepared by Opus International Consultants for Environment Waikato and Taupō District Council. June 2012. 48p.

- Paine, S. & Smith, H. 2012: Taupō District Flood Hazard Study: Tokaanu Stream. Report prepared by Opus International Consultants for Environment Waikato and Taupō District Council. June 2012. 50p.

- Ward, H., Morrow, F. & Ferguson, R. 2014: Taupō District Flood Hazard Study: Lake Taupō. Report prepared by Opus International Consultants. Draft for internal review. June 2014. 108p.

These reports were written largely for a 'lay' audience. Therefore the amount of technical detail provided relating to the hydrological analysis and hydraulic modelling was deliberately kept to a minimum. The only difference to this approach was the detailed technical report prepared for Waikato Regional Council relating to the Tongariro River 2D hydraulic modelling (Maas & McConchie, 2011). That report was prepared for a very different audience. The detail in that report was required because the modelling and results were a significant departure from the hydraulic modelling which had been done previously on the Tongariro River i.e., a 2-D (i.e. MIKE21) as opposed to 1-D (i.e. MIKE11) hydrodynamic model.

It has subsequently been suggested that some additional technical information, implicit in the various flood studies, might be useful to facilitate discussion, and inform hearings, relating to any proposed District Plan changes to recognise the flood hazard.

Rather than modifying each individual report, this '*Technical Compendium*' provides the background, technical detail, and analyses which underpin the individual reports. It is considered that this approach:

- Provides the level of technical detail and robust analysis necessary so that confidence can be placed in the findings and conclusions of the various individual reports; while
- Allowing the individual reports to be read easily and understood by a 'lay' audience, without a considerable amount of repetitive and potentially confusing scientific and statistical detail.

Consequently, this *Technical Compendium* addresses issues of background, approach, philosophy, assumptions and limitations, hydrology and data reliability, principles and constraints of hydraulic modelling, wave run-up analysis, combined probabilities and the residual uncertainty of the results and conclusions inherent in the studies.

The majority of this material is already available in different formats, or it is implicit in the various flood studies. However, its incorporation into this *Technical Compendium* results in a more robust explanation and justification for any proposed changes to the District Plan regarding recognition of the potential flood hazard within the Lake Taupō catchment.

2 Conceptual constraints

It is necessary that natural hazards and associated information are mapped at a scale appropriate for the end-use; in this case allowing planners to provide guidance regarding land use on or close to potentially hazardous areas. However, while generally the larger the scale the better the resolution and detail available, cost acts as a major constraint. Decisions need to be made regarding the cost of any hazard investigation and where these costs should lie. For example, which costs should be borne by the wider rating base (i.e., the Council) and which should be borne by a developer and individual landowner?

While there appears to be no standard with regard to the scale used for mapping natural hazards in New Zealand, the following have been proposed (GNS, 2015):

- National (1:1,000,000);
- Regional (1:100,000 to 1:500,000);
- Medium (1:25,000 to 1:50,000) – typically municipal or small metropolitan areas; and
- Large (1:5,000 to 1:15,000) – typically subdivision, site, or property level.

Waikato Regional Council holds broad scale (i.e. 1:50,000) flood hazard maps for the Waikato Region (WRC, 2015). These maps provide an overview of the flood hazards associated with many water bodies. This information, however, is not suitable for land-use planning processes, other than identifying potential flooding issues that may require further discussion and investigation.

It has been suggested that local authorities should map hazard information to an appropriate planning-level scale of approximately 1:10,000 to 1:20,000; with a larger scale being appropriate for 'urban' as opposed to 'rural' areas. Such an approach has been adopted in the Taupō District flood studies.

While the highest resolution data has been used in all the modelling, including LiDAR topographic information for defining the terrain, there remains some inherent uncertainty which is difficult to define without robust calibration. Robust flood calibration data only exists for the Tongariro and Tauranga Taupō Rivers; with some qualitative data also available for the Kuratau River. Even in these cases where calibration data are available, this tends to be for relatively small events when compared to the design events used in the various flood studies (i.e. the 1%AEP event plus an allowance for the potential effects of climate change). Since the scenarios modelled in the Taupō District flood study are relatively 'extreme', precise calibration is not possible currently.

It must be recognised therefore that even at the relatively large scale used in the various flood studies there remains some uncertainty regarding the flood hazard at the 'site level'. This uncertainty is a function of the resolution of the data used in any model, its calibration, changes which have occurred since the model was developed, and the constraints of the actual modelling. In addition, it must be recognised that any hydraulic model is a simplification of reality.

There are three primary issues when considering the results of any flood modelling:

1. Recognition of the resolution and uncertainty of the various input data. Essentially the resolution or scale of any results can be no higher than the resolution and scale of the lowest resolution input data;

2. Having a scale that is fine enough so that the resulting inundation maps are realistic and not pixelated; and

3. Computing restrictions, especially: the amount of data in the modelling; the computational complexity; and the run-time of the model (which can take from hours to days for an individual model).

While every endeavour was made to use the highest resolution data during the Taupō District flood studies, there remains some residual uncertainty at the specific site or property level. This uncertainty is likely to be greatest at the boundaries of any mapped inundation zone. Consequently the flood hazard areas should be regarded as 'indicative' rather than 'definitive'.

It is important to note, however, that the scale of the mapping and resolution of the various flood hazard zones tend to 'moderate' and 'smooth' the inherent uncertainties in some of the input data. For example, at the scale of the analysis the effect of a 10-20% change in the peak discharge of a design flood event, or consideration of the potential effect of climate change, has been shown to have a relatively minor effect on the extent and depth of inundation. While the absolute numbers may be different, the pattern of flooding is the same.

The potential effects of uncertainty of the input data are also moderated by the major influence of topography on the extent and depth of inundation. Rather than topography increasing gradually and evenly away from the lake or rivers, the landscape is often comprised of a series of 'steps' and terraces, or distinct 'breaks in slope'. These 'steps' in the landscape tend to constrain the extent of any inundation until the threshold of the 'step's' elevation is exceeded by the water surface and water can start to flood over the next level.

Despite some uncertainty regarding the various information used to model the potential flood hazard of Lake Taupō and its tributaries, the mapped hazard zones are considered to be robust. However, the flood hazard areas should be regarded as 'indicative' rather than 'definitive' at the property level.

3 Hydrological data quality

3.1 Introduction

The quality of the hydrological inputs to any computational hydraulic model are critical to the reliability and accuracy of the results, and any assessment of the flood hazard. It is therefore essential to assess the accuracy and reliability of the hydrological inputs to any model.

All the hydrometric data used in the various Taupō District flood studies were obtained from either the National Hydrological Archive (maintained and managed by NIWA) or the Waikato Regional Council. Both of these organisations collect and maintain their hydrometric databases to strict standards of quality control and data assurance. While there will always be some inherent uncertainty regarding hydrometric data, because of natural variability and the manner in which it is recorded, all the data used in the flood studies has been collected using industry 'best practice'.

While the various factors which affect the reliability of estimates of the design flood hydrographs were reviewed, it has been assumed that the 'raw' water level and flow data from which these estimates are derived are the best available. The recording authorities (i.e. either NIWA or the Waikato Regional Council) have comprehensive and externally audited quality assurance procedures. The hydrometric data must meet the standards of industry 'best-practice' and any departures from these standards must be noted.

3.2 Accuracy and reliability

Accurate measurements of water level, and its variation over time, are critical when assessing the risk from flooding both around Lake Taupō, and from its various tributaries. Although it is estimates of design water levels, design flows, or design flood hydrographs which are used in hydraulic models, these are all invariably derived from measurements of the water level. For a river situation water level measurements are generally converted to estimates of flow using a rating curve. Consequently, the reliability of any hydrological inputs to a flood model is a function of:

- The accuracy with which water level was recorded;
- The accuracy of the rating used to convert the water level information to estimates of flow; and
- The length of the flow record and therefore the statistical robustness of any analysis of the frequency and magnitude of flood events. This then affects the reliability of any estimates of the magnitude of design flood events.

For a lake situation, the reliability of any design flood level is also a function of the accuracy with which the lake level has been recorded.

Accuracy of water level records

As a result of changes in the technology used to measure water level over time the accuracy of water level data, and both its vertical and temporal resolution, has increased. Increased accuracy in the water level records has therefore been a response to changes in the methods by which the data are measured and recorded. For example, manual staff gauge readings are probably accurate to ±10mm while modern shaft encoders in stilling wells are accurate to ±1mm. The accepted levels of accuracy of the various level recording methods that have been used throughout New Zealand are summarised in Table 3.1.

Table 3.1: Accuracy of various water level measurement techniques.

Level measurement technique	General accuracy
Staff gauge	±10mm
Littlejohn recorder	±20mm
Kent recorder	±20mm
Lea or Foxboro recorder	±20mm
Fischer and Porter	±3mm
Digital encoder	±1mm

It is therefore important to consider the recording method when analysing variation in water level (e.g. Lake Taupō), or the flow records from the various tributaries. This is particularly important for very long records where a range of different technologies may have been used over the duration of the record.

Despite some uncertainty over the accuracy of the measurement of water level over time, it has been assumed that the data used in the various flood studies are the 'best available'. As discussed later, it is unlikely that any residual error or uncertainty in the data has a significant effect on the results of the flood hazard analyses.

Accuracy of rating curves

With most river flow records it is actually the water level which is measured quasi-continuously not the flow. The current 'standard' is to measure water level every 15-minutes, although this temporal resolution was often considerably longer during early records because of the limitations in the technology available. These water level readings are then converted to estimates of flow using a rating curve i.e. essentially a calibration which relates the water level to the rate of flow. A rating curve is developed by undertaking a series of measurements of the actual flow in the river and recording the water level at the time. A relationship is then derived (i.e. the rating curve) which allows all the water level measurements to be converted to estimates of flow.

Therefore if the form or characteristics of the channel change significantly, such as during a flood event, then the rating must also be changed. While gauging locations are generally chosen for their stability, the alluvial channels of the tributaries draining to Lake Taupō require frequent rating changes to maintain the accuracy of flow estimates.

The accuracy of flow estimates in any river is therefore a function of both the accuracy of the water level measurements (currently accepted to be ±1mm under normal conditions) and the accuracy of the rating curve (which is reviewed periodically).

The accuracy of a rating curve depends on a range of variables including: the stability of the channel; the number of actual flow gauging used to define the curve; and the range of the flows gauged. While flows measured using industry best practice are usually regarded as being ±8%, this uncertainty increases during higher flows (i.e. floods). This is because of the rapidly changing water level, changing bed form, and difficulties in measuring accurately both the depth and velocity of the flow. Consequently, during flood events the uncertainty of flow estimation can increase to ±30%.

Again, despite some uncertainty over the accuracy of the rating curves used during the flood studies to convert measurements of water level to flow, it has been assumed that the derived magnitudes of the flood events are the 'best available'. It is considered unlikely that any uncertainty in the rating curves has a significant effect on the results of the flood hazard analyses.

Stationarity

Stationarity is a key assumption in all frequency analyses, including those used in this study. Stationarity implies (and it is therefore assumed) that the annual flood maxima series used in the analysis exhibit no trends or cycles; and that the extremes are drawn randomly and independently from a single statistical distribution. Implicit in this assumption is that the same processes and relationships that existed in the past will continue to apply in the future. For example, the relationship between rainfall and runoff during particular events will be the same. However, should anything change this relationship e.g., climate or land use change, then stationarity may no longer apply. When this occurs, the reliability of the frequency analysis, and the magnitudes of any derived design flood events, may be questioned.

Longer records generally have a greater likelihood of containing information relating to extreme events. Such records also tend to smooth any errors and other 'noise' in a data set. However, long records also increase the chance of violating the basic rule of stationarity because they have the potential to be affected by land use, climate, or other changes in the catchment.

It is therefore important to check any flow record for trends or cycles; other than the usual annual pattern of greater flows during winter and spring for the tributaries of Lake Taupō. If there are no trends, and only a random pattern to the distribution and magnitude of flood events, then the record should provide the basis for robust frequency analysis. This will then allow reliable estimates of the magnitude and frequency of various design events. However, if there are periods of greater or lesser flood activity it is important to determine whether these are a function of random variability in flow, or a function of climatic oscillations. This may need to be investigated further.

With respect to the flow regimes of the various tributaries of Lake Taupō it was assumed that stationarity persists throughout the lengths of all the available records. However, the potential effects of climate change have also been considered. This creates an apparent contradiction since any climate change impact would violate the assumption of stationarity. Despite this apparent conflict between the underlying assumption of stationarity, and the possible impacts of climate change, it is considered that any potential impact of non-stationarity is likely to be very small. It would have no significant effect on the results of the flood hazard analysis.

As discussed in detail in Ward *et al.* (2014) the management regime of Lake Taupō has undergone a number of significant changes over time. These include the construction and commissioning of the Taupō Gates and a new outlet channel to the Waikato River in 1941, and changes to the inflow regime as a result of the Tongariro Power Development. These changes were certainly of sufficient magnitude to violate any assumption of stationarity over the entire length of the lake level record (i.e. since 1906).

The lake level management regime, however, is considered to have been 'stationary' since 1980; once the Tongariro Power Development was complete. Consequently, only the lake level record since 1980 was used in the analysis of the flood risk posed by extremely high lake levels (i.e. the last 35 years).

Duration of flow record

The reliability of estimates of design lake levels and flood discharges is largely a function of the length of flow record used in the analysis, and the appropriateness of the flow record to a particular flood model.

As a general rule of thumb (Davie, 2008) average recurrence intervals (i.e. ARIs), or annual exceedance probabilities (i.e. AEPs), should not be extrapolated beyond twice the length of the record of annual flood maxima i.e. a 25-year flow series should only be used to estimate flood events with an ARI of up to 50 years (i.e. 2%AEP). Uncertainty of flood estimates therefore increases rapidly for more extreme events. Given the relatively short nature of the majority of flow records used in the flood studies (Table 3.2), especially with respect to the magnitude of the design flows of interest (i.e. 100-year ARI or 1%AEP), there will always be uncertainty over the design flow estimates. This uncertainty can only be accommodated effectively by adopting conservative, but still realistic values, and applying 'best professional judgement'.

Table 3.2: Duration of the annual flood series used in the analyses.

Hydrometric site	Duration of record
Lake Taupō	~35 years
Tongariro @ Turangi	~57 years
Tauranga Taupō @ Te Kono	~38 years
Kuratau @ SH41	~36 years
Whareroa @ Fishtrap	~16 years
Hinemaiaia @ DS Dam	~28 years

Despite the uncertainty inherent in estimating the magnitudes of more extreme design flood events, a sensitivity analysis of the various Taupō flood studies indicates that the extents and depths of inundation are not extremely sensitive to the exact flood magnitude used in the model. Any uncertainty in the design flood estimates is likely to have less effect on the result than other uncertainties in the modelling as discussed later.

Alternative approaches

In situations where the flow record is extremely short (i.e. <20-years) and contains no large flood events, an alternative method to a statistical analysis of the available flow record is required. Possible methods for estimating design flows include: the regional method (McKerchar and Pearson, 1989); translation and scaling of flows from adjacent and similar catchments; or rainfall-runoff models. It should be noted that all these methods involve numerous assumptions; the effects of which are usually difficult, and often impossible, to quantify. The uncertainty of the outputs from these approaches is therefore generally significantly greater than that associated with the analysis of specific flow records.

The flow data which currently underpin the regional method outlined in McKerchar and Pearson (1989) ends in 1985. Consequently there are approximately 30-years of additional site specific data now available for the major tributaries into Lake Taupō. These data add significantly more to the robustness of estimates of the magnitudes of various design events in these specific rivers than generalised indices based on the shorter, less accurate flow series used in McKerchar and Pearson (1989). In addition, the annual flood maxima from a number of the major tributaries to Lake Taupō were used in McKerchar and Pearson (1989); for example, Tongariro @ Turangi, Tauranga Taupō @ Te Kono, and Wanganui @ Te Porere (which does not flow into Lake Taupō). To use the regional method for these rivers would appear to introduce some circularity and bias to any analysis.

The regional flood frequency indices are currently being revised and updated to include all information collected since the original report (i.e. since 1985). Once these new indices are available they are likely to add significantly to the robustness and consistency of flood frequency analyses obtained from short flow series. However, at this stage it is considered that the additional 35 years of data available for the specific rivers and streams modelled in the Taupō flood study adds more to the robustness of design flood estimates than potentially 'smoothing' the data through the use of generalised indices from McKerchar and Pearson (1989).

The reliability of rainfall-runoff models is largely determined by the quality of the calibration and validation. It also depends on the nature of the rainfall-runoff relationship within the catchment, and how this is 'captured' by the calibration and validation. Since calibration and validation are invariably undertaken on flows significantly less than the required design discharge (i.e. the 1%AEP flood), the nature of this relationship, and how it changes with flood magnitude, is critical. Consequently, the assumptions relating to rainfall distribution across the catchment, and the simplifications required within a rainfall-runoff model usually result in greater uncertainty than is likely from the use of even relatively short duration flow records.

These alternative methodologies, however, can be valuable in providing an 'independent' check on the likely reliability of design flood estimates from short flow records e.g., Whareroa @ Fishtrap and Tokaanu Stream.

3.3 Flow series

Each flow series for the rivers flowing into Lake Taupō was reviewed to assess the quality of the flow record, including: any gaps in the record; the accuracy and appropriateness of the rating curves for estimating discharge at higher flows; the annual maxima flood series; and the frequency analysis used to derive estimates of the magnitudes of the design flood events.

Tongariro River

The longest and most suitable flow series to assess the flood risk to Turangi is from the Tongariro River at Turangi site; operated and maintained by NIWA. A discussion of this flow record is available in Maas & McConchie (2011). To supplement that discussion, further analysis of the quality of the flow record is provided.

The flow record provided by NIWA shows no gaps (i.e. missing data) between 1957 and 2014. The comment file for this site, however, indicates that there was initially one gap during January

2007; of approximately eight hours. That gap was 'filled' after comparing flows with those recorded at a site located further upstream.

The rating curves for the Tongariro at Turangi flow site are shown in Figure 3.1. There have been 58 ratings used between 1957 and 2012. The gaugings used to define the various rating curves are also plotted in Figure 3.1. The highest gauged flow was on 24 February 1958 at 1470m³/s. This is also the largest flow on record. Given the historic nature of this flood, there is some uncertainty as to whether the flood was actually gauged at its peak. Also, given the extreme nature of this event there is some uncertainty over the accuracy of the gauging. However, this is the information within the 'comment file' and therefore it has been assumed to be correct. There is no basis currently to disregard this information.

An analysis of the stage data during the second largest flood on record (i.e., that of 29 February 2004) shows apparently large variations in the bed level over short periods of time (Figure 3.2). This bed instability affects the estimated flood flows and also impacts on the accuracy of any hydraulic model. The hydrological and hydraulic controls which cause this bed instability are inherent in the dynamics of flood events. Such channel instability cannot be modelled and therefore the uncertainty caused by this variability can only be incorporated through conservative design assumptions.

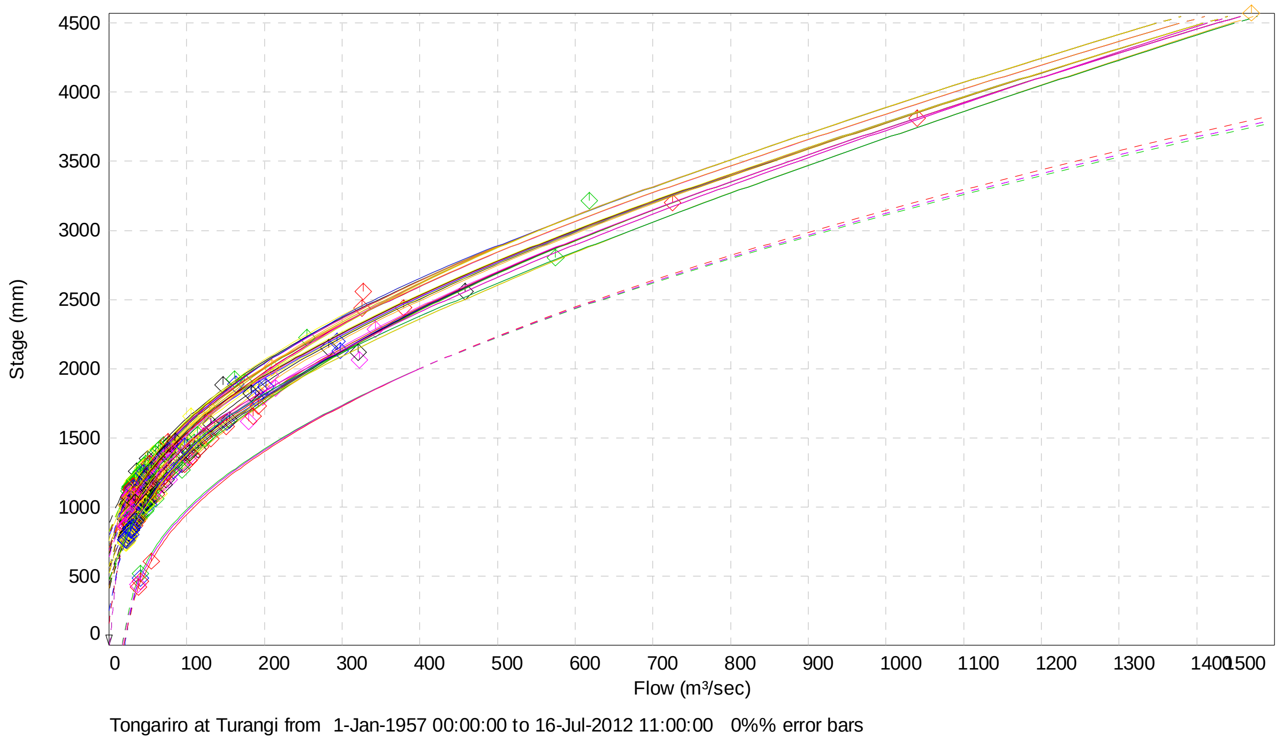

Figure 3.1: Rating curves for the Tongariro River at Turangi.

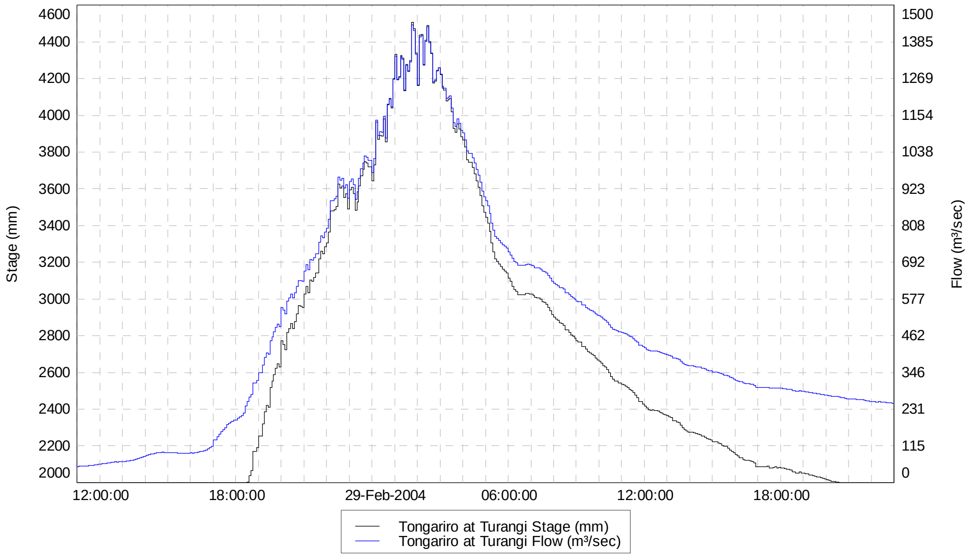

Figure 3.2: Stage and flow readings for 29 Feb 2004 flood in the Tongariro River at Turangi.

Despite these various sources of uncertainty, it has been assumed that the instrumental flow series has been collected using 'industry best practice'. While there will also be uncertainty as to the exact magnitude of flood flows, those represented in the Tongariro at Turangi record are considered robust, and are certainly the best available for any flood hazard analysis.

The annual maxima flood series for Tongariro at Turangi is shown in Figure 3.3. There is no obvious longer term trend in the data, and the magnitude and distribution of flood events appear essentially random. There appears, however, to be some indication of minor cyclic behaviour and persistence; with periods of generally larger floods interspersed with periods of smaller annual events.

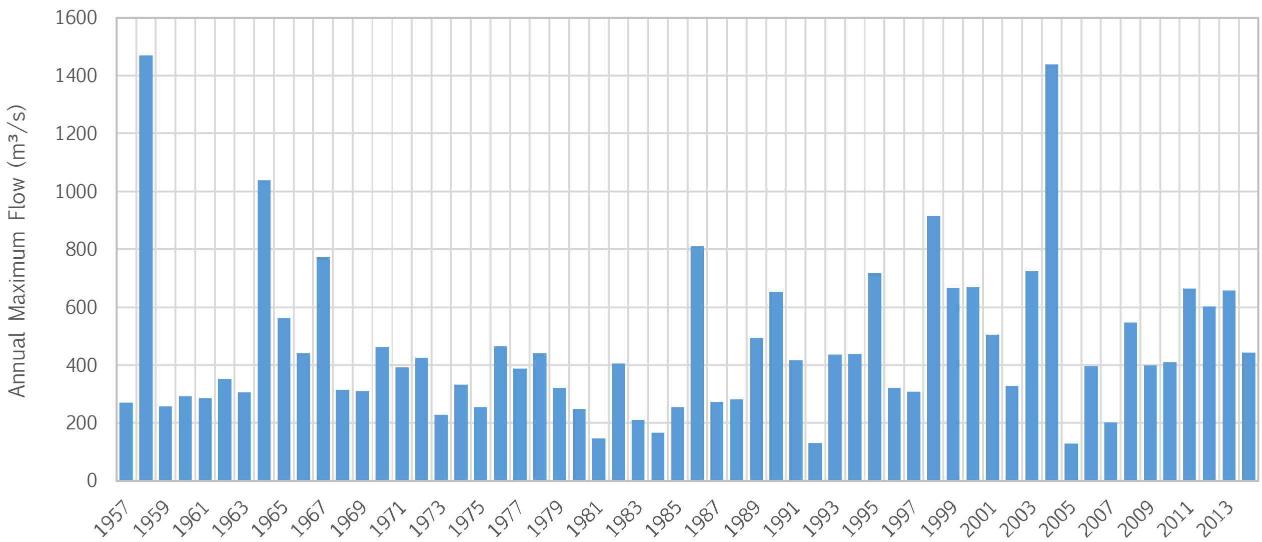

Figure 3.3: Annual maxima flood series for Tongariro River at Turangi (1957-2014).

Implicit in the frequency analysis of annual flood maxima series is that the events are independent, drawn from a single distribution, and that stationarity has persisted throughout the length of record. It would appear from Figure 3.3 that this is the case with respect to the record from the Tongariro River at Turangi. To ensure that all events in the flood series are independent, the flow series was reviewed. All the flood maxima are separated from other large events in the series by a considerable period of time indicating that all the flood maxima are independent events. The effect of any departure from the basic assumptions is therefore likely to be very small relative to the uncertainty inherent in; firstly, the magnitudes of the events in the annual maxima series, and then the magnitude and frequency of any design events.

It is problematic that if the assumption of stationarity is not valid then any robust frequency analysis would not be possible.

An updated frequency analysis of the Tongariro at Turangi record has been undertaken using the extended record up until the end of 2014. This included an analysis of the L moments (Figure 3.4) and the appropriateness of various statistical distributions when modelling the annual flood maxima series (Figure 3.5).

The L moment analysis indicates that a GEV or Generalised Logistic statistical distribution would best represent the Tongariro at Turangi flow series. However, when analysing these distributions on the frequency analysis plot, both appear to have an unrealistic shape. The GEV, Generalised Logistic and Log Normal distribution all increase 'exponentially' with increasing ARI or decreasing AEP (Figure 3.5). The statistical distribution with the most realistic and 'natural' shape is that provided by the PE3 distribution.

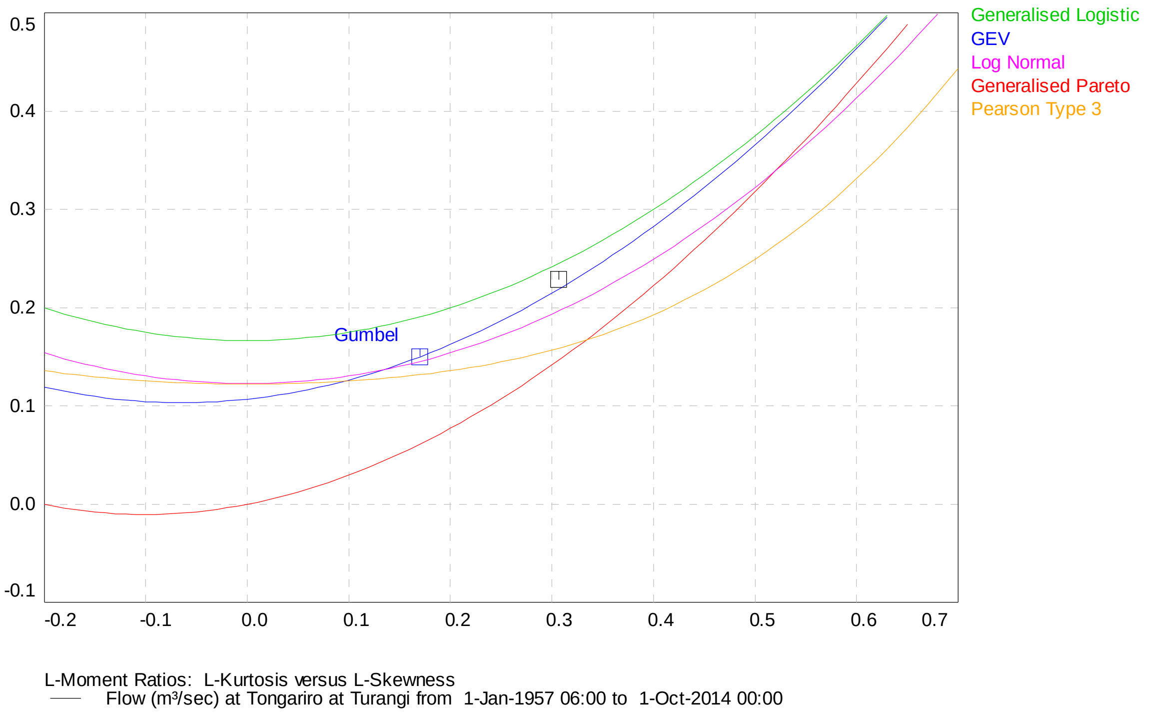

Figure 3.4: L moment analysis for the Tongariro at Turangi annual flood maxima series.

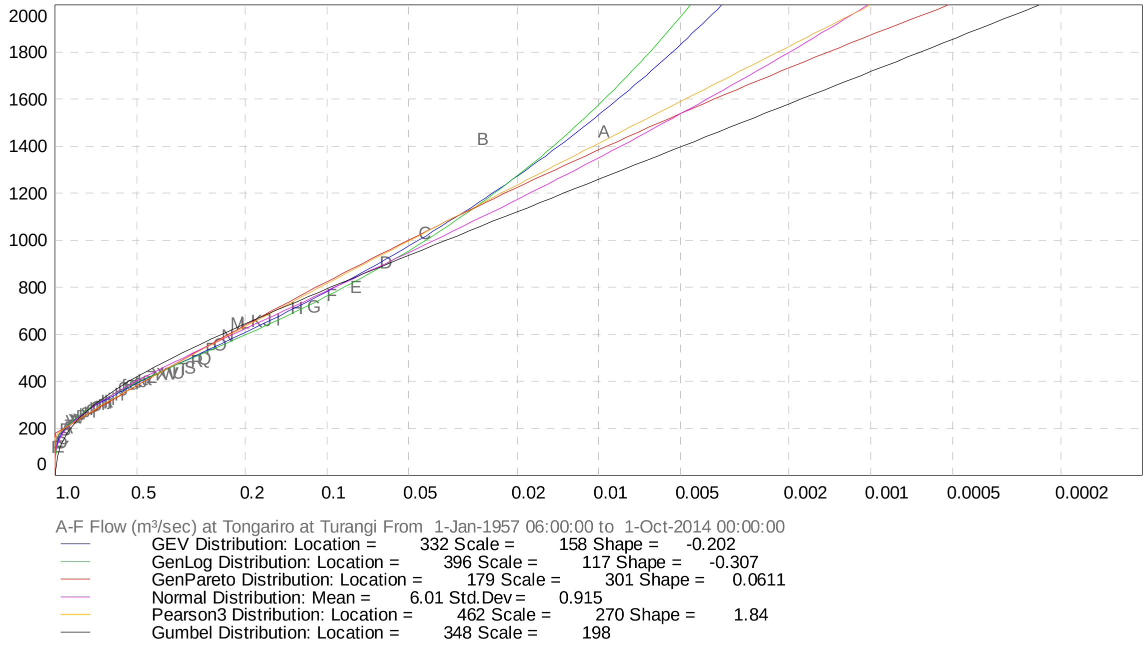

Figure 3.5: Frequency analysis for Tongariro at Turangi annual flood maxima series.

Estimates of various design flood events, assuming several different statistical distributions, are listed in Table 3.3. The PE3 results are highlighted and are considered the more realistic. It should be noted that the difference between the magnitude of the 100-year flood event (i.e. 1%AEP) assuming a PE3 distribution is 164m³/s lower than the highest estimate, and 153m³/s higher than the lowest estimate i.e. the difference is only ±11% of the estimated peak discharge. Given the uncertainties inherent in estimating the magnitudes of various design flood events, these differences are not significant. They would have no effect on the outcome of any flood hazard analysis.

Table 3.3: Results of frequency analysis for Tongariro at Turangi 1957-2014 (m³/s).

ARI	GEV	Generalised Logistic	Generalised Pareto	Log Normal	PE3	Gumbel
2.33	428	431	426	442	427	462
5	609	598	640	625	634	645
10	783	763	824	785	817	794
20	976	956	1002	949	998	937
50	1272	1274	1225	1173	1235	1121
100	1533	1577	1385	1351	1413	1260

Despite some inherent uncertainty in estimates of the magnitude of flood flows, those derived for the Tongariro River at Turangi are considered robust. They are the best available for any flood hazard analysis and hydraulic modelling.

Tauranga Taupō River

The longest and most suitable flow series to assess the flood risk posed by the Tauranga Taupō River is from Te Kono; a site operated and maintained by Waikato Regional Council. A discussion of this flow record is available in Knight (2010). To supplement that discussion, further analysis of the quality of the flow record is provided.

The flow record provided by Waikato Regional Council shows 35 gaps between 1976 and 2014. These gaps span a total of 7372 hours (i.e. 307 days). The majority of these gaps (i.e. 80%) are less than a week long (Table 3.4); however, 80% of time total time represented by the gaps is the result of longer duration periods of missing record (i.e. >1 week duration).

Table 3.4: Gap analysis of Tauranga Taupō at Te Kono flow record.

Duration	Number of gaps	Total hours
>1 week	7	5922
<1 week	15	1315
< 1day	13	135

The rating curves for the Tauranga Taupō at Te Kono flow site are shown in Figure 3.6; together with the gaugings used to define the various curves. All these data were provided by Waikato Regional Council.

There were 250 flow gaugings between 1981 and 2015. The highest gauged flow was on 12 July 2008 at 151m³/s, while the largest flow on record is 296m³/s. Consequently the highest gauged flow is only about half the estimated magnitude of the largest flood event. This results in some uncertainty regarding the estimated peak discharges of larger flood events.

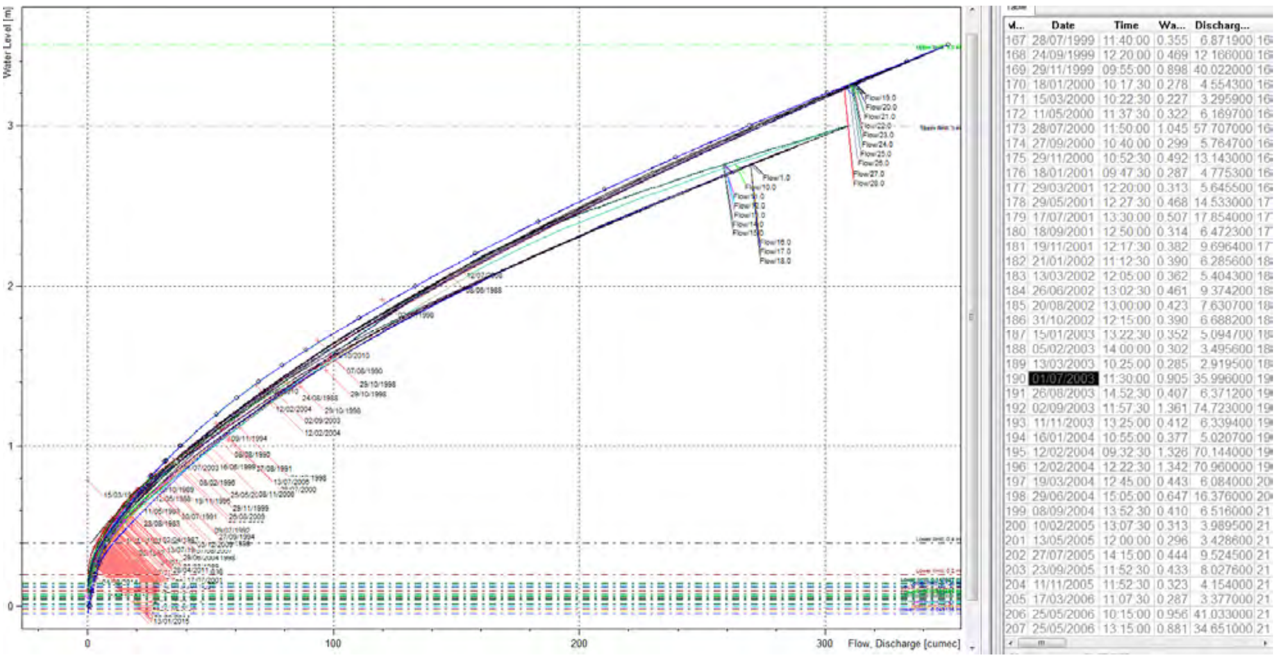

Figure 3.6: Rating curves for the Tauranga Taupō River at Te Kono.

The annual flood maxima series for the Tauranga Taupō River at Te Kono is shown in Figure 3.7. There is no obvious longer term trend in the data, and the magnitude and distribution of the flood events appear essentially random. There appears, however, to be some indication of minor cyclic behaviour and persistence; with periods of generally larger floods interspersed with periods of smaller annual events.

Implicit in any frequency analysis of annual flood maxima series is that the events are independent, drawn from a single distribution, and that stationarity has persisted throughout the length of record. It would appear from Figure 3.7 that this is the case with respect to the flow record from the Tauranga Taupō River at Te Kono. To ensure that all events in the flood series are independent, the flow series was reviewed. All the flood maxima are separated from other large events in the series by a considerable period of time indicating that all the flood maxima are independent events. The effect of any departure from these assumptions is therefore likely to be very small relative to the uncertainty inherent in; firstly, the magnitudes of the events in the annual maxima series, and then the estimated magnitude and frequency of any design events.

It is problematic that if the assumption of stationarity is not valid then any robust frequency analysis would not be possible.

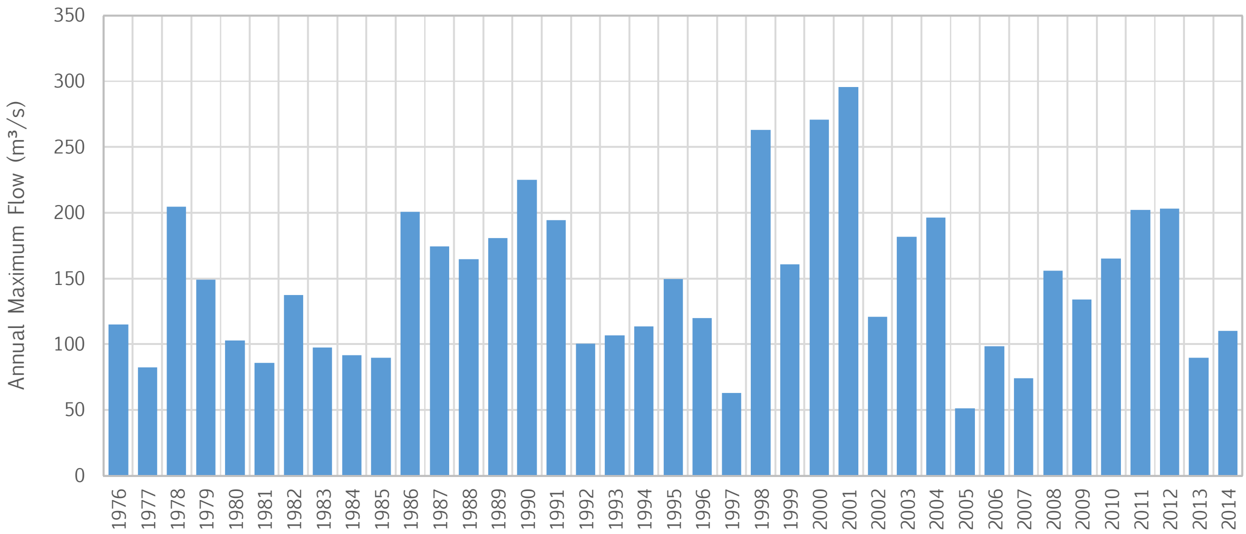

Figure 3.7: Annual maxima flood series for Tauranga Taupō River at Te Kono (1976-2014).

An updated frequency analysis of the Tauranga Taupō River at Te Kono flow record has been undertaken using the extended record up until the end of 2014. This included an analysis of the L moments (Figure 3.8) and the appropriateness of assuming various statistical distributions when analysing the annual flood maxima series (Figure 3.9).

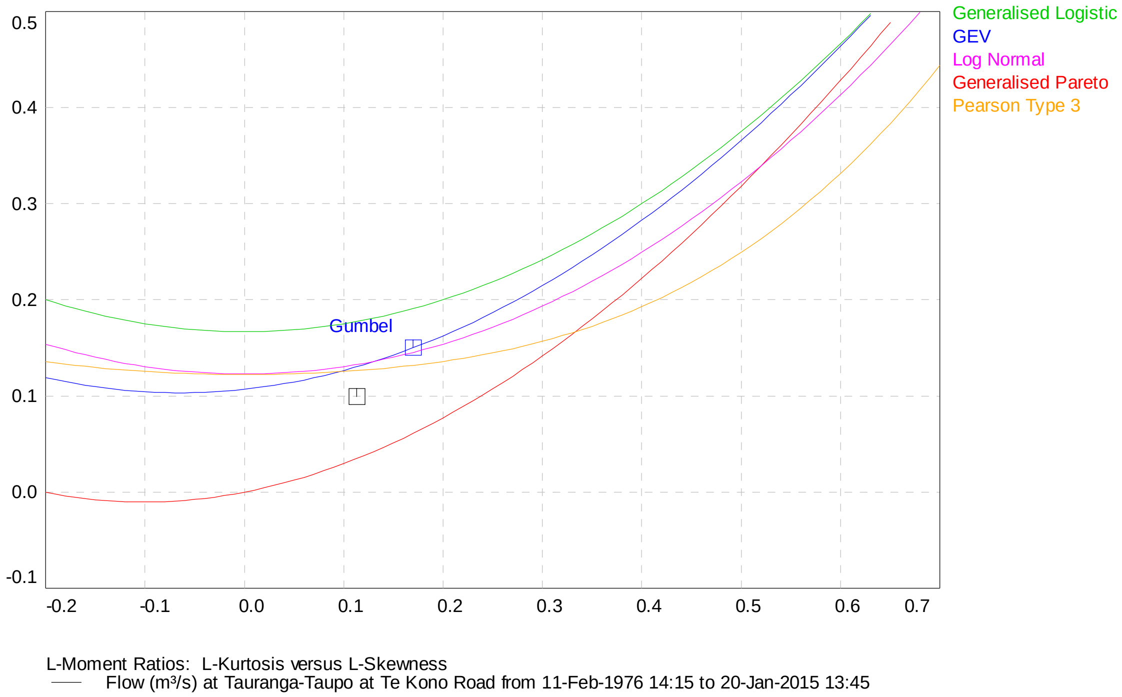

Figure 3.8: L moment analysis for Tauranga Taupō River at Te Kono annual flood maxima series.

The L moment analysis indicates that a PE3 distribution best represents the annual flood maxima series for the Tauranga Taupō River at Te Kono; although a GEV distribution gives almost the same design flows. When analysing the various statistical distributions, the PE3 statistical distribution appears to have both a realistic shape and plots closest to the largest floods.

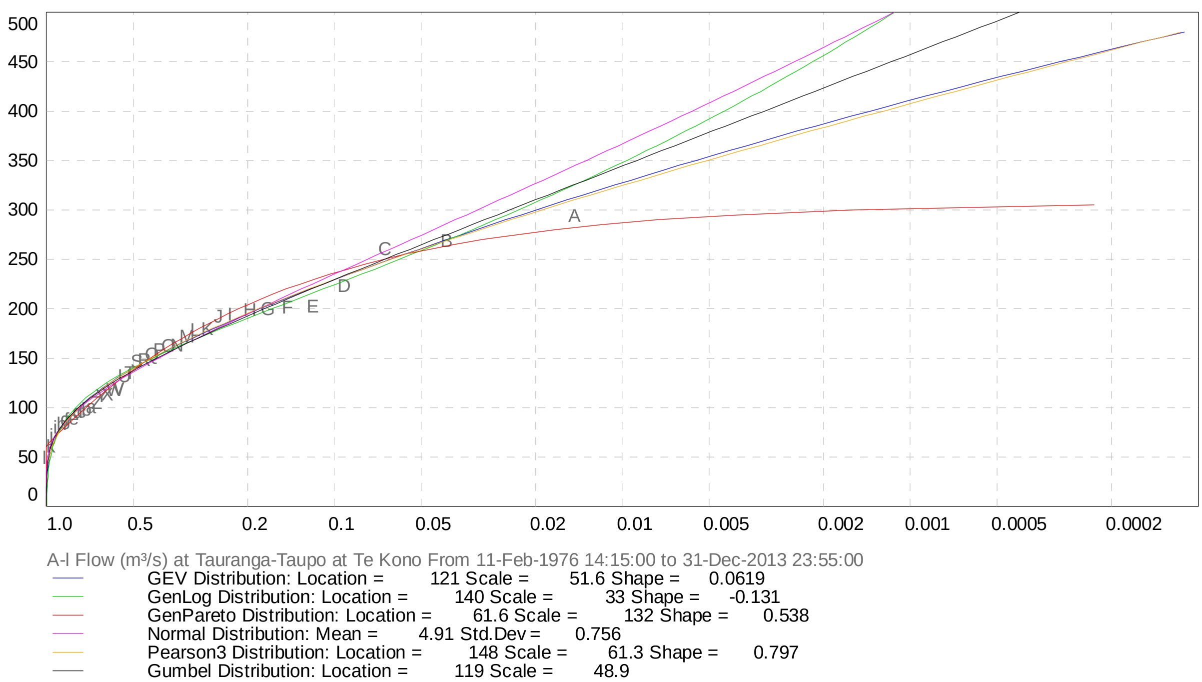

Figure 3.9: Frequency analysis for Tauranga Taupō River at Te Kono annual flood maxima series.

The results assuming several different statistical distributions are listed in Table 3.5. The PE3 results are highlighted and are considered the most reliable. It should be noted that the difference between the magnitude of the 100-year flood event assuming a PE3 statistical distribution is only 23m³/s lower than the highest estimate, and only 38m³/s higher than the lowest estimate i.e. the difference is only about ±10% of the estimated peak discharge during the 100-year ARI event. Given the uncertainties inherent in estimating the magnitudes of various design flood events, these differences are not significant. They would have no effect on the outcome of any flood hazard analysis.

Table 3.5: Results of frequency analysis for Tauranga Taupō at Te Kono 1976-2015 (m³/s).

ARI	GEV	Generalised Logistic	Generalised Pareto	Log Normal	PE3	Gumbel
2.33	147	147	149	141	148	144
5	193	188	203	193	193	190
10	227	222	234	238	227	228
20	258	256	255	282	258	264
50	295	304	272	342	294	311
100	321	343	280	389	320	346

Despite some inherent uncertainty in the estimates of the magnitude of flood flows, those derived for the Tauranga Taupō at Te Kono record are considered robust. They are the best available for any flood hazard analysis and hydraulic modelling.

Hinemaiaia River

The longest and most suitable flow series to assess the flood risk posed by the Hinemaiaia River is from a site operated and maintained for TrustPower by NIWA. A discussion of this flow record from the Hinemaiaia "Below HB dam" is available in Paine & Smith (2012a). To supplement that discussion, further analysis of the quality of the flow record is provided.

The flow record provided by NIWA for this site shows six gaps between 2000 and 2014. These gaps span a total of 349 hours (i.e. 15 days). One gap lasts for less than a day while the others are up to a week duration (Table 3.4). Gaps of between one day and one week make up 94% of the total period of missing data.

Table 3.6: Gap analysis of Hinemaiaia River at Below HB Dam flow record.

Duration	Number of gaps	Total hours
>1 week	0	0
<1 week	5	327
< 1day	1	22

The rating curves for the Hinemaiaia River at Below HB Dam flow site are shown in Figure 3.10. Eight different ratings were used between 2000 and 2013. The gaugings used to define the various rating curves are also plotted in Figure 3.10.

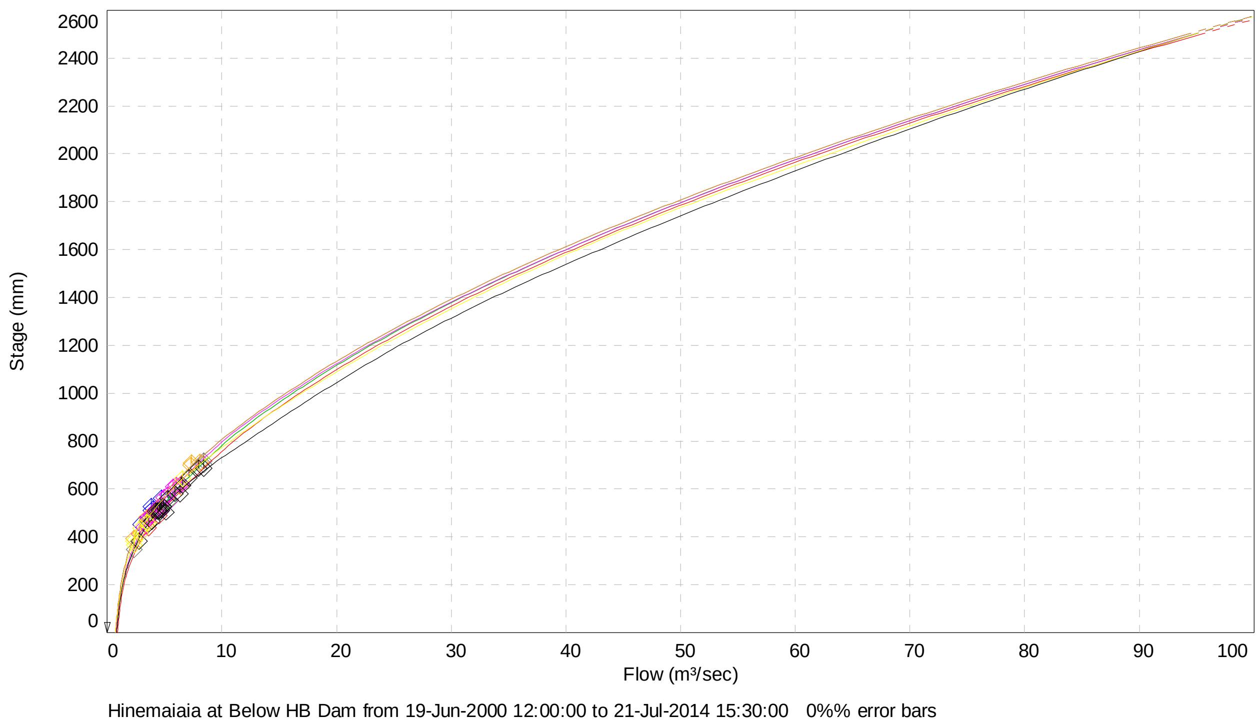

Figure 3.10: Rating curves for the Hinemaiaia River at Below HB Dam.

The highest gauged flow was on 24 July 2000 at 8.433m³/s, while the largest flow on record is estimated at 87.5m³/s. Consequently the highest gauged flow is only about 10% of the largest estimated flow. This is likely to result in some uncertainty regarding the estimated peak discharges of larger flood events. Despite this, as discussed in Paine and Smith (2011), the Hinemaiaia at Below HB Dam flow record, augmented with earlier flows measured at an adjacent site, provides the most suitable annual flood maxima series for use when assessing the flood risk posed by the Hinemaiaia River.

The annual flood maxima series for Hinemaiaia River at Below HB Dam is shown in Figure 3.11. There is no obvious longer term trend in the data, and the magnitude and distribution of flood events appear essentially random. The flow record at this site is relatively short and therefore any pattern is difficult to determine. However, the extended annual flood maxima series discussed in Paine & Smith (2012a) also shows no longer term trend in either flood frequency or magnitude.

Implicit in the frequency analysis of an annual flood maxima series is that the events are independent, drawn from a single distribution, and that stationarity has persisted throughout the length of record. It would appear from Figure 3.11, and the detailed discussion provided in Paine & Smith (2012a), that this is the case with respect to the flow record from the Hinemaiaia River. To ensure that all events in the flood series are independent, the flow series was reviewed. All the flood maxima are separated from other large events in the series by a considerable period of time. This indicates that all the flood maxima are independent events. The effect of any departure from these assumptions is therefore likely to be very small relative to the uncertainty inherent in; firstly, the magnitudes of the events in the annual maxima series, and then the estimated magnitude and frequency of any design events.

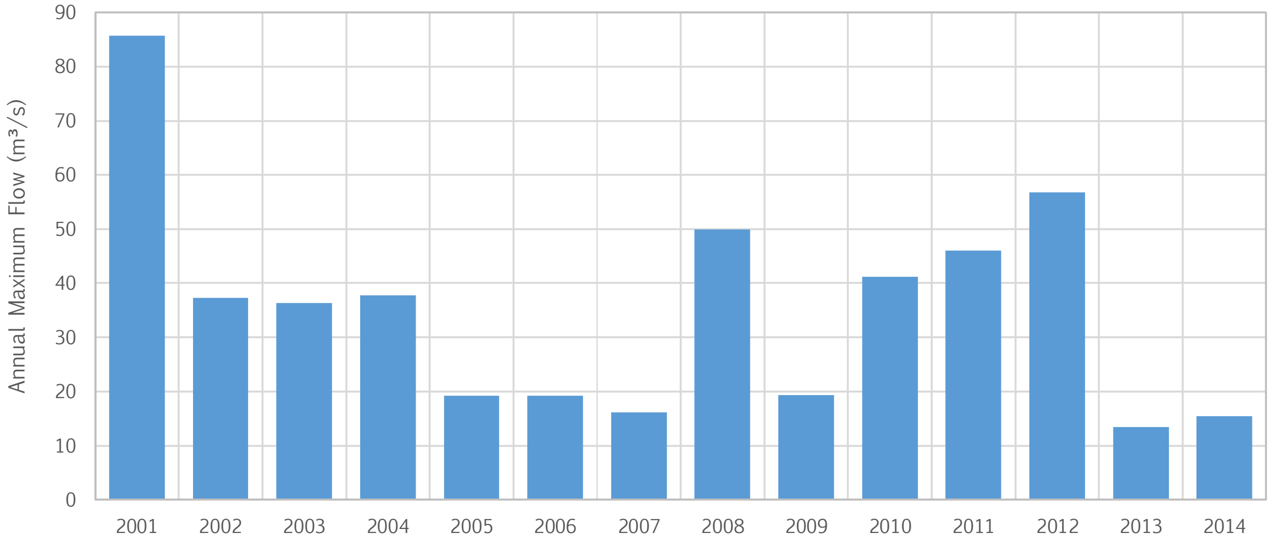

Figure 3.11: Annual maxima flood series for Hinemaiaia River at Below HB Dam (2000-2014).

An updated frequency analysis of the Hinemaiaia River at Below HB Dam record from 2000-2014 was undertaken. This included an analysis of the L moments (Figure 3.12) and the appropriateness of assuming various statistical distributions when analysing the annual flood maxima series (Figure 3.13).

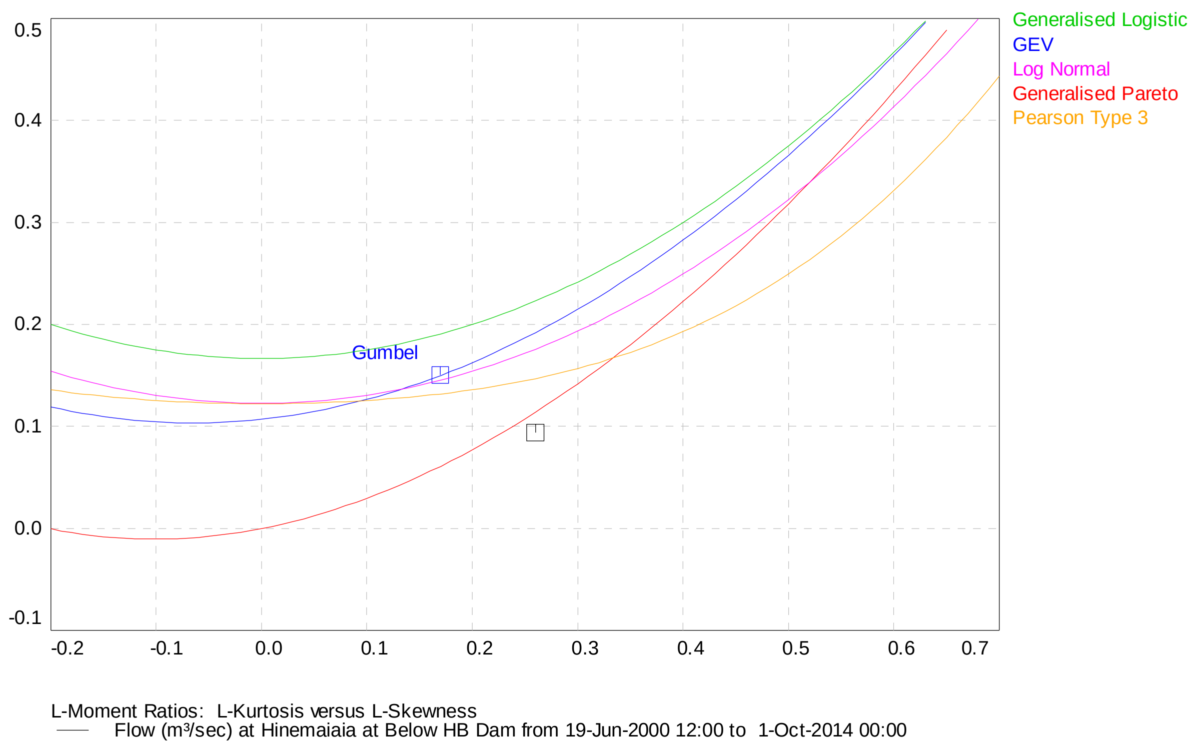

Figure 3.12: L moment analysis for Hinemaiaia River at Below HB Dam annual flood maxima series.

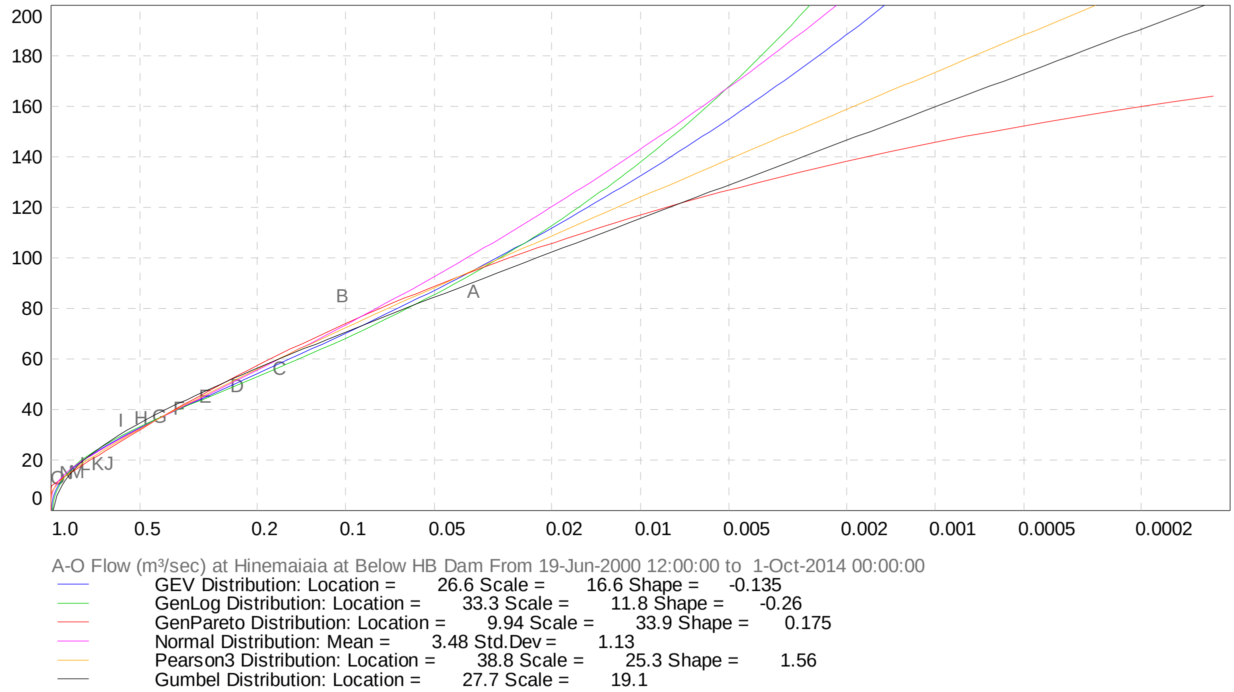

Figure 3.13: Frequency analysis for Hinemaiaia River at Below HB Dam annual flood maxima series.

The L moment analysis indicates that a Generalised Pareto distribution best represents the annual flood maxima series for the Hinemaiaia River at Below HB Dam; although both the PE3 and Gumbel statistical distributions also provide almost identical design flows. When reviewing these distributions on the frequency analysis plot (Figure 3.13), the Generalised Pareto statistical distribution appears to flatten off significantly at higher flows when compared with the other distributions. Such a characteristic appears atypical of accepted flood behaviour in New Zealand. The most realistic statistical distribution is therefore considered to be either the PE3 or Gumbel. The adoption of the PE3 statistical distribution is slightly conservative. It results in a slightly higher estimate of the magnitude of the 1%AEP design flood event.

The results assuming several different statistical distributions are listed in Table 3.7. The PE3 results are highlighted and are considered the more reliable. It should be noted that the difference between the magnitude of the 1%AEP design flood event using the PE3 statistical distribution is only 19m³/s lower than the highest estimate, and only 7m³/s higher than the lowest estimate i.e. the difference is only from 6-15% of the estimated peak discharge during the design event. Given the uncertainties inherent in estimating the magnitudes of various design flood events, these differences are not significant. They would have no effect on the outcome of any flood hazard analysis.

Table 3.7: Results of frequency analysis for Hinemaiaia at Below HB Dam 2000-2014 (m^3/s).

ARI	GEV	Generalised Logistic	Generalised Pareto	Log Normal	PE3	Gumbel
2.33	37	37	37	36	37	39
5	54	53	57	56	56	56
10	70	68	74	74	72	71
20	87	86	89	93	88	85
50	112	113	106	120	109	102
100	133	138	117	143	124	116

Despite some uncertainty inherent in the estimates of the magnitude of flood flows, those derived for the Hinemaiaia River using all available flow data are considered robust. They are the best available for any flood hazard analysis and hydraulic modelling.

Tokaanu Stream

There have been two flow sites on the Tokaanu Stream downstream of the power station tail race. However, both sites were closed after only a relatively short period of time (i.e. 22 and 60 months respectively).

Following discussions with the local NIWA Field Team, who have an extensive understanding of the hydrology of the wider area, it was decided that the most realistic flood frequency estimates would be obtained by scaling results from the adjacent Whanganui at Te Porere flow site. The appropriateness and limitations of this approach are discussed in Paine & Smith (2012b). To supplement that discussion, further analysis of the quality of the flow record from Whanganui at Te Porere is provided.

The flow record provided by NIWA for the Whanganui at Te Porere flow site between 1966 and 2014 shows one gap. The period of missing record lasts only 1.26 days. This gap therefore represents 0.007% of the total length of record and is likely to be inconsequential.

The rating curves for the Whanganui at Te Porere flow site are shown in Figure 3.14; together with the gaugings used to define the various curves. There have been 37 different ratings used between 1966 and 2014. The highest gauged flow was on 9 March 1990 at 51.8m^3/s, while the largest estimated flow on record is 54.0m^3/s. Therefore the highest gauged flow is approximately 96% of the largest estimated flow. This allows a high level of confidence to be placed in the estimates of larger flood events, and consequently the annual flood maxima series.

The annual flood maxima series for Whanganui at Te Porere is shown in Figure 3.15. There is no obvious longer term trend in the data, and the magnitude and distribution of the flood events appear essentially random. There appears, however, to be some indication of minor cyclic behaviour or persistence; with periods of generally larger floods interspersed with periods of smaller annual events.

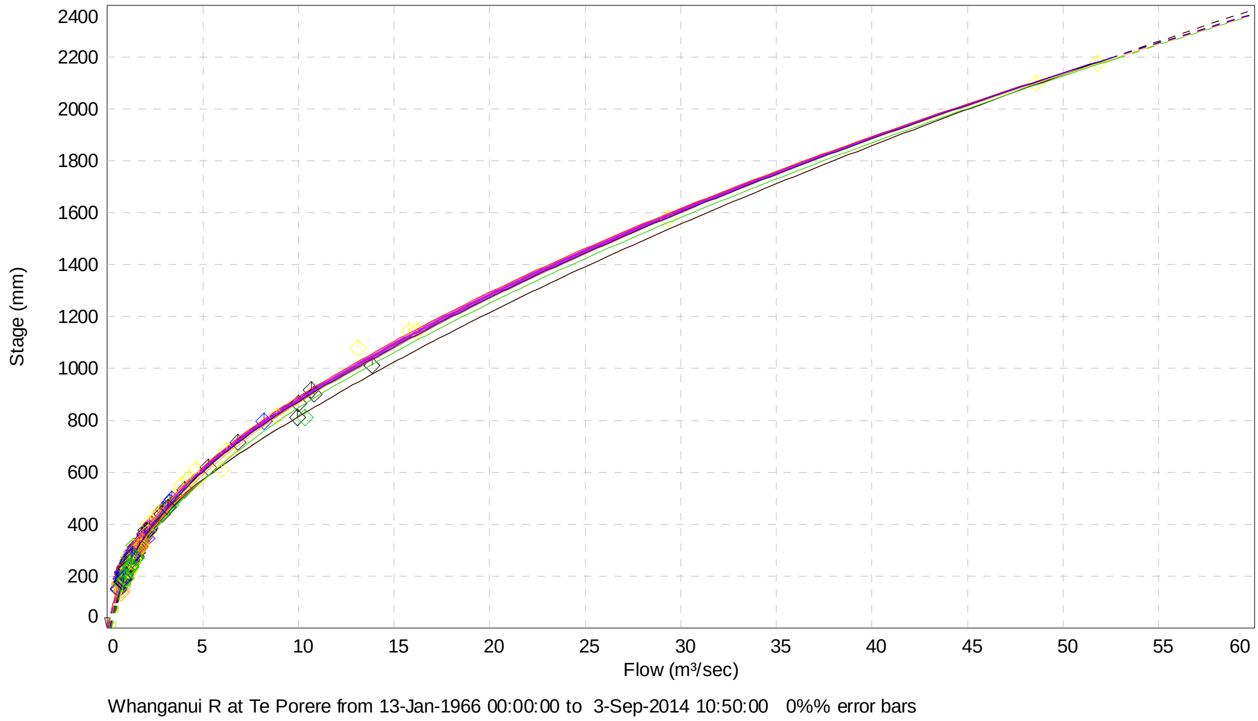

Figure 3.14: Rating curves for the Whanganui at Te Porere.

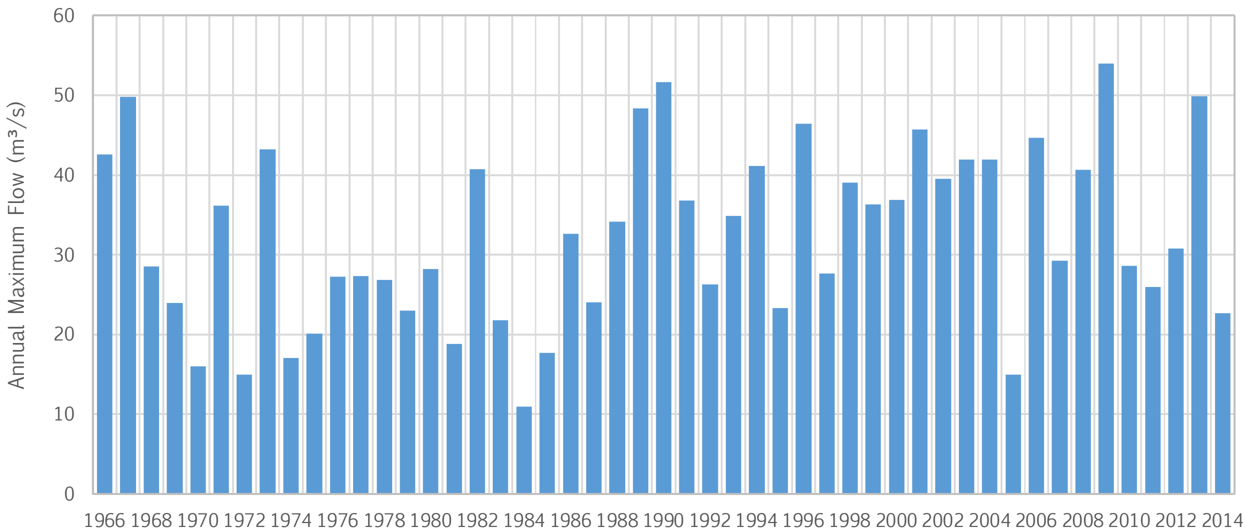

Figure 3.15: Annual maxima flood series for Whanganui at Te Porere (1966-2014).

Implicit in any frequency analysis of an annual flood maxima series is that the events are independent, drawn from a single distribution, and that stationarity has persisted throughout the length of record. It would appear from Figure 3.15 that this is the case with respect to the flow record from the Whanganui at Te Porere. To ensure that all events in the flood series are independent, the flow series was reviewed. All the flood maxima are separated from other large events in the series by a considerable period of time. This indicates that all the flood maxima are independent events. The effect of any departure from these assumptions are likely to be very small relative to the uncertainty inherent in; firstly, the magnitudes of the events in the annual maxima series, and then the magnitude and frequency of any design events. It is problematic that if the assumption of stationarity is not valid then any robust frequency analysis would not be possible.

An updated frequency analysis of the Whanganui at Te Porere annual flood maxima series was undertaken using the extended record up until the end of 2014. This included an analysis of the L moments (Figure 3.16) and various frequency distributions (Figure 3.17).

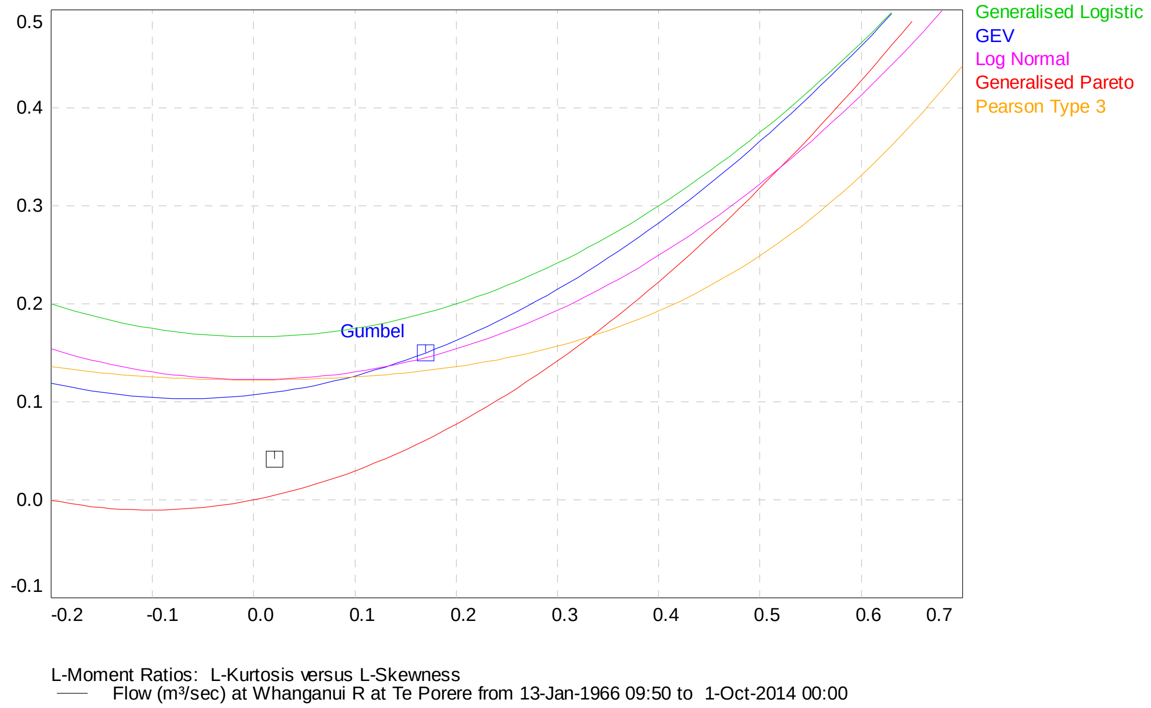

Figure 3.16: L moment analysis for Whanganui at Te Porere annual flood maxima series.

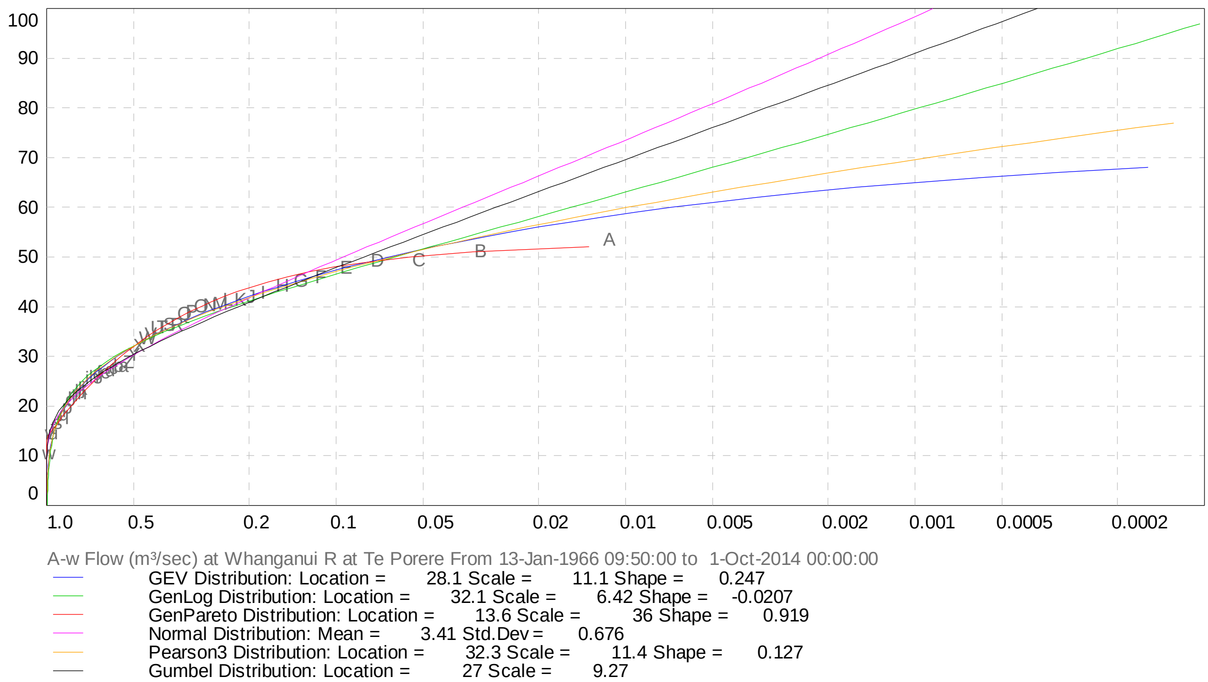

Figure 3.17: Frequency analysis for Whanganui at Te Porere annual flood maxima series.

The L moment analysis indicates that a Generalised Pareto statistical distribution best represents the annual flood maxima series for the Whanganui at Te Porere. When reviewing the various statistical distributions, however, the Generalised Pareto appears to flatten off significantly at higher flows. This is considered to be unrealistic. The most realistic statistical distributions are considered to be either the GEV or PE3. The assumption of either of these statistical distributions results in almost identical estimates of the magnitude of the design flood.

The results of the frequency analyses, assuming several different statistical distributions, are listed in Table 3.8. The estimates assuming the GEV and PE3 statistical distributions are highlighted and are considered the more reliable.

Table 3.8: Results of frequency analysis for Whanganui at Te Porere 1966-2014 (m^3/s).

ARI	GEV	Generalised Logistic	Generalised Pareto	Log Normal	PE3	Gumbel
2.33	34	34	35	32	34	32
5	42	41	44	42	42	41
10	47	47	48	49	47	48
20	52	52	50	57	51	55
50	56	58	52	66	57	63
100	59	63	52	74	60	70

It should be noted that the difference between the magnitude of the 1%AEP flood event assuming a GEV statistical distribution is about $15m^3/s$ lower than the highest estimate, and about $7m^3/s$ higher than the lowest estimate i.e. the difference is between 11 and 25% of the peak discharge of the design event. Given the uncertainties inherent in estimating the magnitudes of various design flood events, these differences are not significant. They would have no effect on the outcome of any flood hazard analysis, especially when down-scaled to Tokaanu Stream. To allow for the difference in catchment areas the various design flood estimates derived from the Whanganui at Te Porere annual flood maxima series have been scaled as a function of the catchment areas to the power of 0.8 (Table 3.9). The justification for this is provided in Paine & Smith (2012b) and McKerchar & Pearson (1989).

Table 3.9: Results of frequency analysis for Tokaanu at below tail race (m^3/s).

ARI	Whangarei at Te Porere ($28.2km^2$)	Tokaanu at below tail race ($5.4km^2$)
2.33	34	9
5	42	11
10	47	13
20	52	14
50	56	15
100	59	16

Despite the inherent uncertainty in any estimate of the magnitudes of design flood flows, those derived for Tokaanu Stream by down-scaling those from the Whanganui at Te Porere are considered robust. They are the best available for any flood hazard analysis and hydraulic modelling.

Kuratau River

Flows of the Kuratau River at the SH41 Bridge have been monitored continuously since November 1978. There have been a number of other flow sites monitored within the Kuratau catchment at various times, and for various durations, but the one at SH41 has the longest record. The recorder at the SH41 Bridge is upstream of Lake Kuratau and Kuratau Dam. Consequently, there are a number of significant tributaries downstream. Flows recorded at the SH41 Bridge therefore need to be scaled to account for catchment inflows downstream.

Previous work (McKerchar and Pearson, 1989) has shown that magnitudes of 1%AEP flood events vary as a function of catchment area to the power of 0.8, rather than simply by catchment area. While the exact reasons for this have not be discussed, they are likely to relate to the average rainfall and storm intensity which both decrease with increasing catchment size.

The flows measured in the Kuratau River at the SH41 Bridge therefore need to be scaled by a function of $Area^{0.8}$ to provide an estimate of flows that could potentially affect the lower valley. Given that the total catchment area of the Kuratau River is $198km^2$, while that above the SH41 Bridge is $119km^2$, the necessary scale factor is 1.50.

To supplement the discussion provided in Smith *et al.* (2011) further analysis of the quality of the flow record from the Kuratau River at the SH41 Bridge record is provided.

The flow record provided by NIWA contains three gaps between 1978 and 2014 which last for a total of 18.8 days. These gaps represents 0.14% of the total length of record.

The rating curves for the Kuratau River at the SH41 Bridge flow site are shown in Figure 3.18; together with the gaugings used to define the various curves. There have been 16 different ratings used between 1976 and 2014. The highest gauged flow was on 29 Jun 1977 at $22.9m^3/s$; this was apparently prior to the start of the continuous water level record. The highest estimated recorded flow has been $60.1m^3/s$. The highest gauged flow is therefore only about 38% of the largest recorded flow. This results in increased uncertainty over the estimates of the magnitudes of larger flood events, and consequently the larger floods contained in the annual flood maxima series.

The annual maxima flood series for Kuratau River at the SH41 Bridge is shown in Figure 3.19. There is no obvious longer term trend in the data, and the magnitude and distribution of the flood events appear essentially random. There appears, however, to be some indication of minor cyclic behaviour or persistence; with a period of generally larger floods interspersed with periods of smaller annual events.

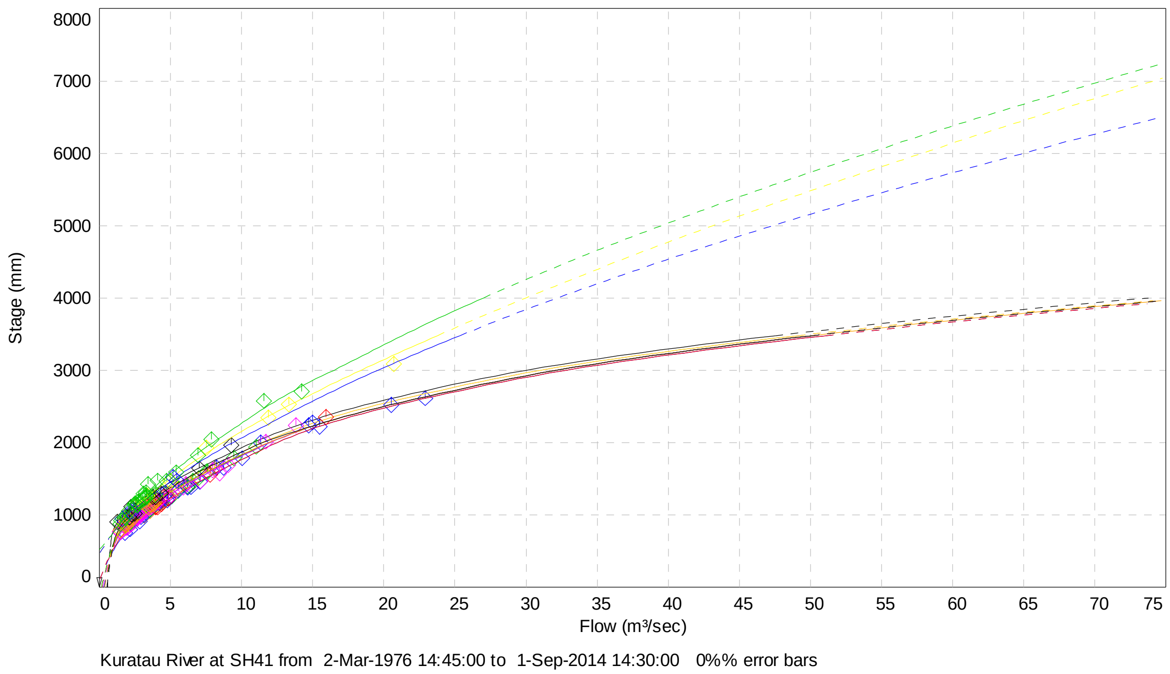

Figure 3.18: Rating curves for the Kuratau River at the SH41 Bridge.

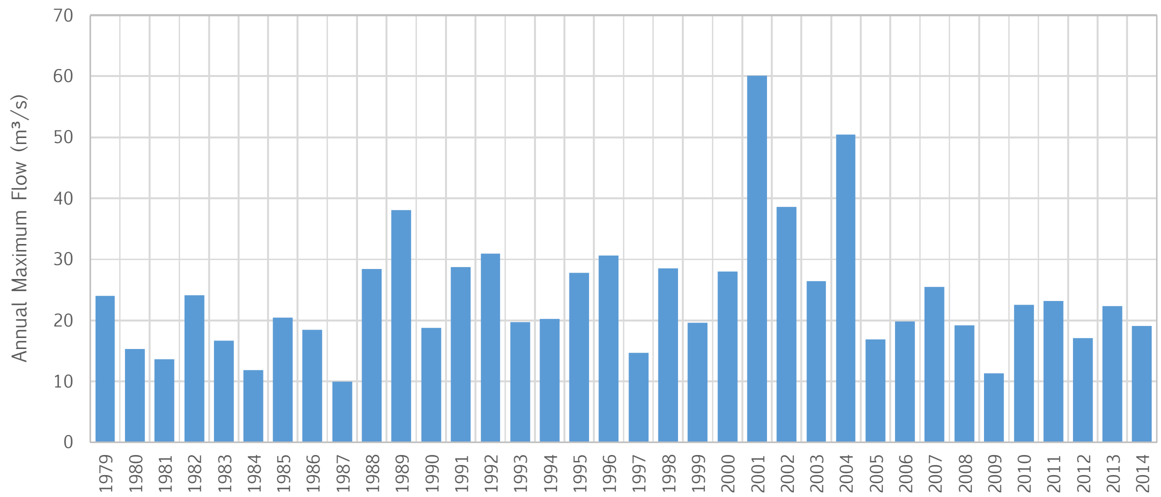

Figure 3.19: Annual flood maxima series for Kuratau River at the SH41 Bridge (1979-2014).

Implicit in any frequency analysis of an annual flood maxima series is that the events are independent, drawn from a single distribution, and that stationarity has persisted throughout the length of record. It would appear from Figure 3.19 that this is the case with respect to the record from the Kuratau River at the SH41 Bridge. To ensure that all events in the flood series are independent, the flow series was reviewed. All the flood maxima are separated from other large events in the series by a considerable period of time. This indicates that all the flood maxima are independent events. The effect of any departure from the assumption of stationarity is likely to be very small relative to the uncertainty inherent in; firstly, the magnitudes of the events in the annual maxima series, and then the magnitude and frequency of any design events. Further uncertainty is introduced when the design floods are scaled to take account of the difference in catchment area between SH41 and Lake Taupō. It is problematic that if the assumption of stationarity is not valid then any robust frequency analysis would not be possible.

An updated frequency analysis of the Kuratau River at the SH41 Bridge annual flood maxima series was undertaken using the extended record up until the end 2014 (Figure 3.20 & Figure 3.21).

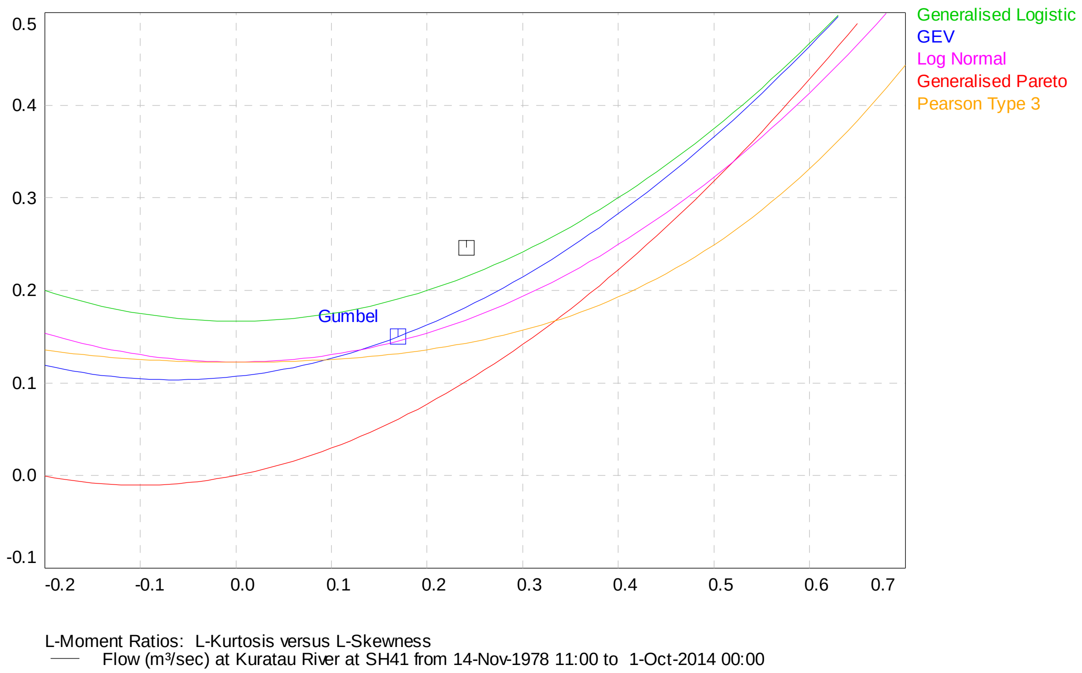

Figure 3.20: L moment analysis for Kuratau River at the SH41 Bridge annual flood maxima series.

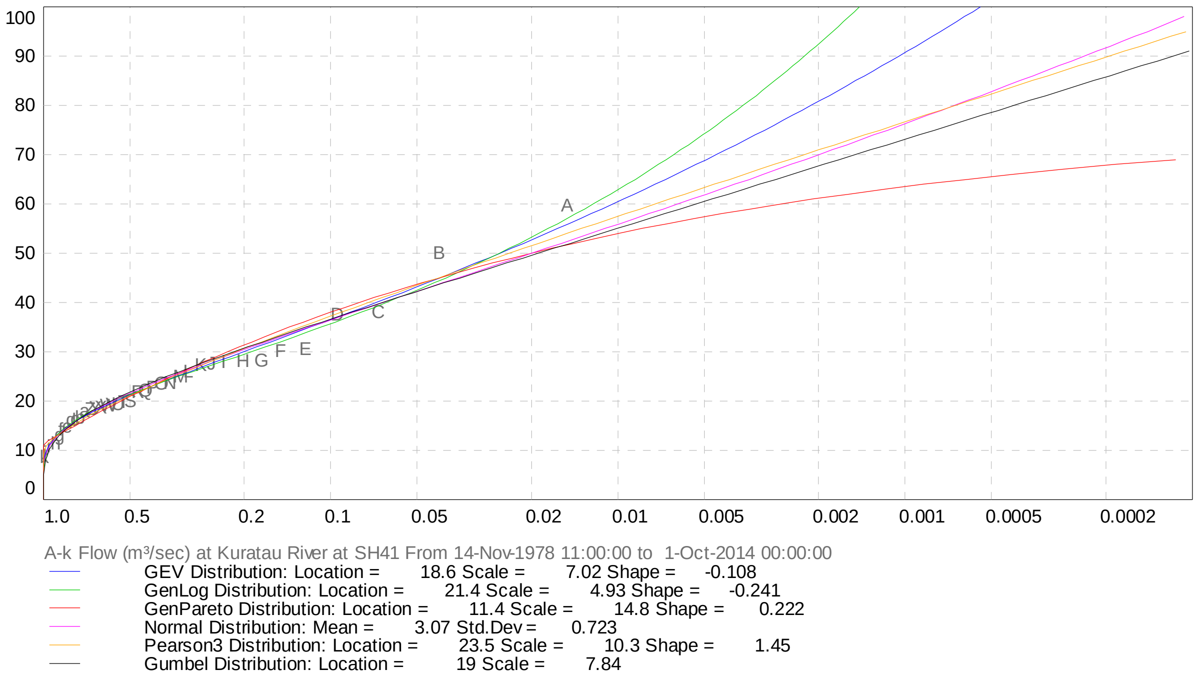

Figure 3.21: Frequency analysis for Kuratau River at the SH41 Bridge annual flood maxima series.

The L moment analysis indicates that a Generalised Logistic statistical distribution best represents the annual flood maxima series for the Kuratau River at the SH41 Bridge. However, when reviewing the various statistical distributions, the Generalised Logistic distribution appears to have an unrealistic shape. Peak discharges appear to increase at an increasing rate with decreasing AEP of the event (Figure 3.21). Flood magnitudes do not appear to 'flatten off' with increasing recurrence interval as is normally the case for floods in New Zealand. The statistical distributions with the more realistic 'fit' to the annual flood maxima series are either the GEV or PE3. The use of a GEV distribution produces slightly conservative (i.e. higher) estimates of the magnitudes of various design flood events (Table 3.10). These design flood estimates are considered the most reliable when assessing the potential flood hazard.

Table 3.10: Results of a frequency analysis of the annual flood maxima series for the Kuratau River at the SH41 Bridge 1978-2014 (m^3/s).

ARI	GEV	Generalised Logistic	Generalised Pareto	Log Normal	PE3	Gumbel
2.33	23	23	23	23	23	24
5	30	30	31	30	31	31
10	37	36	38	36	37	37
20	43	43	44	42	44	42
50	53	53	50	50	52	50
100	60	63	54	56	57	55

It should be noted that the difference between the magnitude of the 1%AEP event assuming a GEV statistical distribution is only 2.4m^3/s lower than the highest estimate, and only 6.5m^3/s higher than the lowest estimate i.e. the difference is only about ±7%. Given the uncertainties inherent in estimating the magnitudes of various design flood events, these differences are not significant and would have no effect on any flood hazard assessment.

The results from a frequency analysis of the annual flood maxima series from the Kuratau River at the SH41 Bridge, assuming a GEV statistical distribution, were scaled by the difference in area to the power of 0.8 to estimate the discharge at Lake Taupō (Table 3.11).

Table 3.11: Results of frequency analysis for Kuratau River at mouth (m^3/s).

ARI	Kuratau River at the SH41 Bridge (119km²)	Kuratau River at mouth (198km²)
2.33	23	34
5	30	45
10	37	55
20	43	65
50	53	79
100	60	91

Despite some inherent uncertainty in the estimates of the magnitude of flood flows, those derived from the Kuratau River at the SH41 Bridge record, and then scaled downstream to Lake Taupō, are considered robust. They are the best available for any flood hazard analysis and hydraulic modelling.

Whareroa Stream

The only flow series on Whareroa Stream to assess the flood risk is from a site called "FishTrap". The site is operated and maintained by the Waikato Regional Council. A discussion on this flow record is available in Paine and Smith (2012c). Flows were monitored for a 3-year period starting in 1977, and then since 2002. Consequently, the combined flow record is now about 15 years long. This record is therefore of limited duration for a robust frequency analysis; particularly for larger and less frequent events. The relatively short flow record from Whareroa Stream acts as a constraint on the accuracy and confidence that can be placed in estimates of the potential magnitude of various design flood events, and consequently the results of any flood modelling. To mitigate any potential effects of this uncertainty, a comparison with the flow record from the adjacent catchment i.e. Kuratau at SH41, scaled to account for differences in catchment area, is also provided.

The flow record has 30 periods of missing data; including the 22 year gap between 1980 and 2002. Over the remainder of the flow record there are missing data for a total of approximately 6135 hours (i.e. 256 days). The characteristics of these gaps are summarised in Table 3.12.

Table 3.12: Gap analysis of Whareroa at Fish Trap flow record.

Duration	Number of gaps	Total hours
>1 week	9	195423
<1 week	17	1182
< 1day	1	15

The rating curves for the Whareroa at Fish Trap are shown in Figure 3.22; together with the various gaugings used to define the curves. These data were provided by the Waikato Regional Council.

There have been 73 flow gaugings between 2002 and 2014. The highest gauged flow of 2.36m³/s was on 15 September 2008, while the largest flow on record is estimated to have been 13.66m³/s. The fact that the highest gauged flow is only 17% of the largest flow on record indicates considerable uncertainty regarding the accuracy and reliability of the estimates of peak discharge during large flood events. This uncertainty has a potential impact on the reliability of the annual flood maxima series, and the magnitude of any design flood events derived from these data.

The annual maxima flood series from the Whareroa at Fish Trap is shown in Figure 3.23. Although the available data are limited, there is no obvious longer term trend in either the magnitudes or frequencies of large flood events. The annual flood maxima appear essentially random. This is consistent with the lack of any trend apparent in the longer flow records from the other tributaries to Lake Taupō.

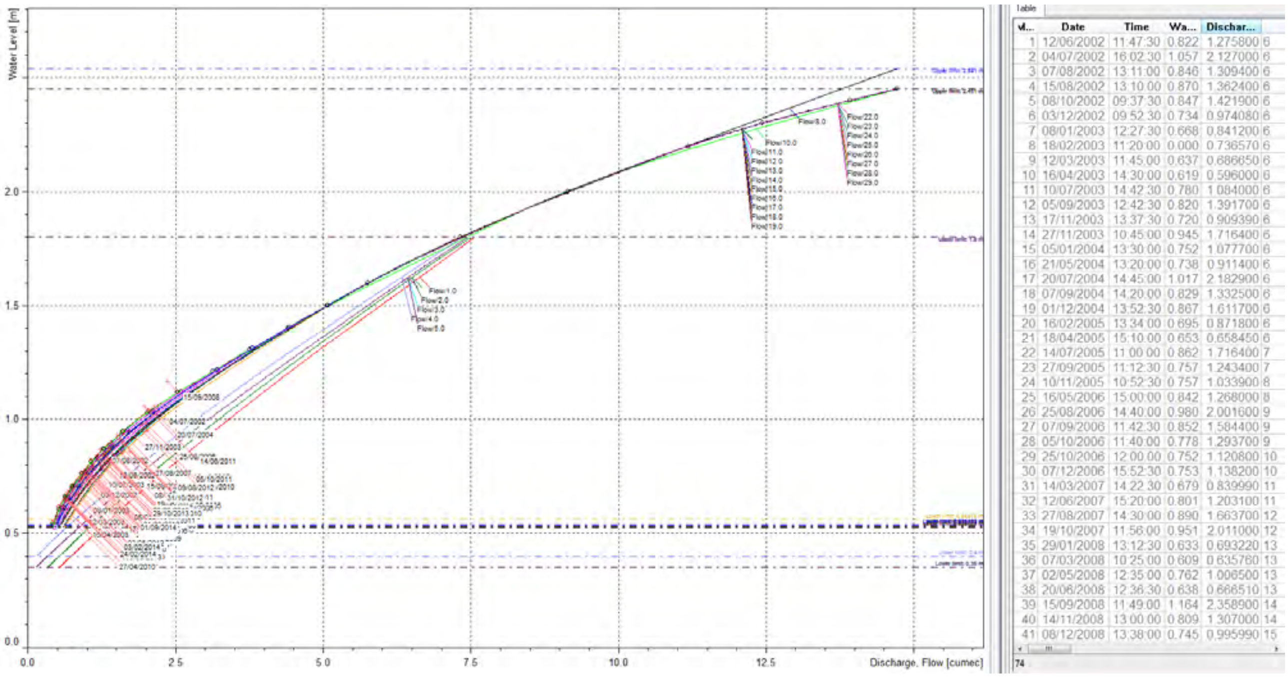

Figure 3.22: Rating curves for the Whareroa at Fish Trap.

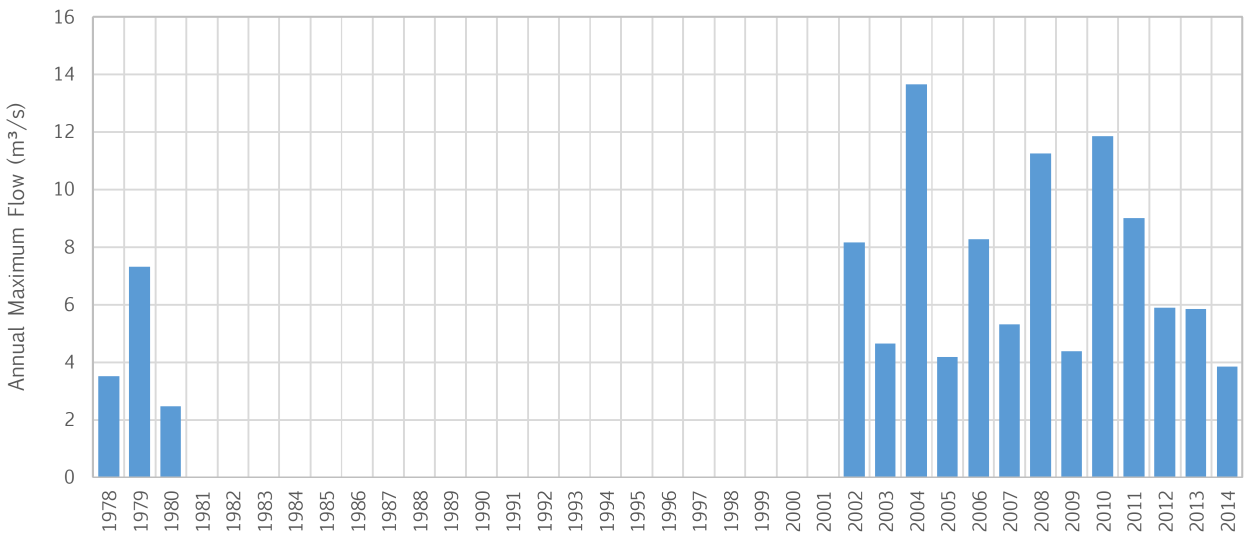

Figure 3.23: Annual maxima flood series for Whareroa at Fish Trap (1978-2014).

Implicit in any frequency analysis of an annual flood maxima series is that the events are independent, drawn from a single distribution, and that stationarity has persisted throughout the length of record. It would appear from Figure 3.23 that this is the case with respect to the flow record from the Whareroa at Fish Trap. To ensure that all events in the flood series are independent, the flow series was reviewed. All the flood maxima are separated from other large events in the series by a considerable period of time. This indicates that all the flood maxima are independent events. The effect of any departure from the assumption of stationarity is likely to be very small relative to the uncertainty inherent in; firstly, the magnitudes of the flood events in the annual maxima series, and then the magnitude and frequency of any design events. It is problematic that if the assumption of stationarity is not valid then any robust frequency analysis would not be possible.

An updated frequency analysis of the annual flood maxima series from the Whareroa at Fish Trap was undertaken using the extended record up until the end of 2014. This included an analysis of the L moments (Figure 3.24), and the appropriateness of various different statistical distributions when modelling the annual flood maxima series (Figure 3.25).

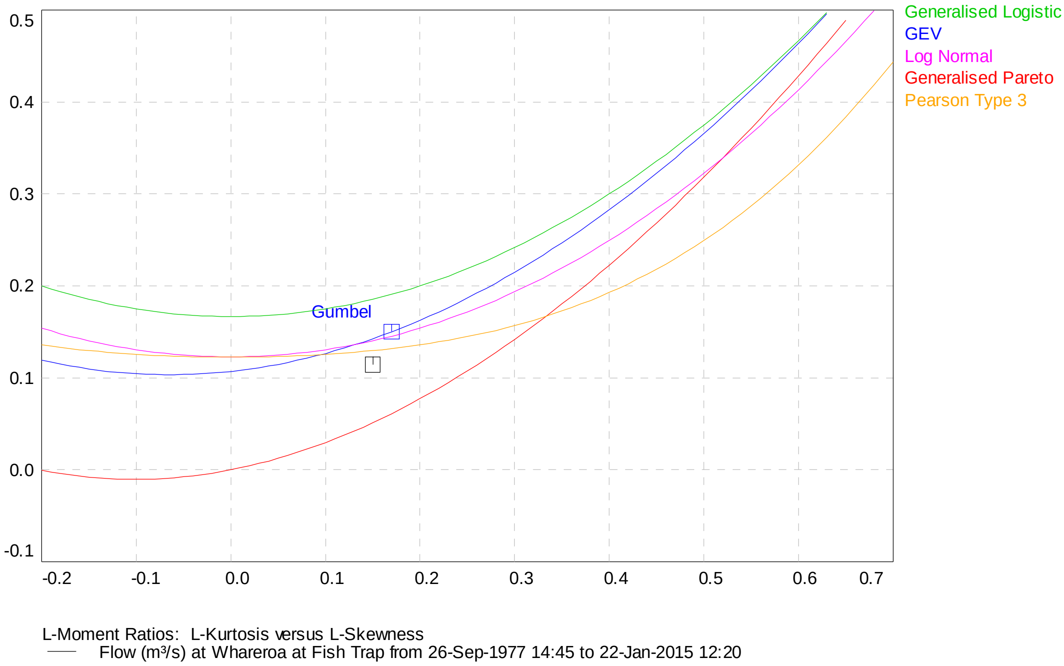

Figure 3.24: L moment analysis of the Whareroa at Fish Trap annual flood maxima series.

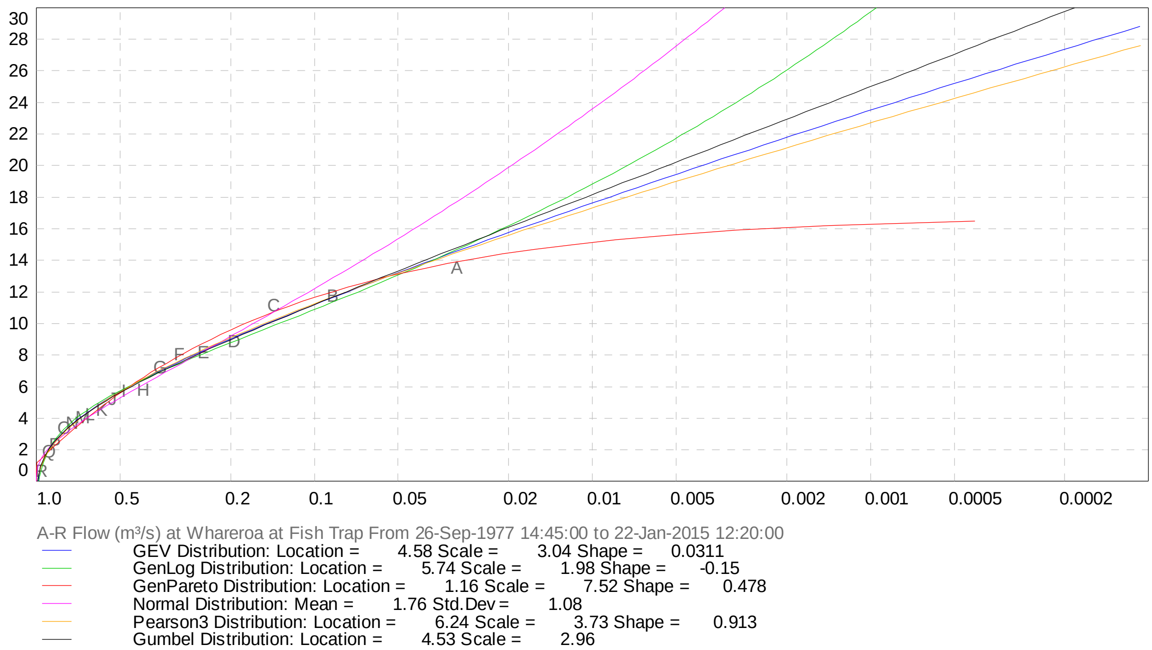

Figure 3.25: Frequency analysis of the Whareroa at Fish Trap annual flood maxima series.

The L moment analysis indicates that a PE3 statistical distribution best represents the annual flood maxima series from Whareroa Stream at Fish Trap. Reviewing the various statistical distributions shows that the assumption of a PE3 distribution, or a GEV distribution, provides a slightly conservative (i.e. higher) estimate of the magnitude of the 1%AEP design flood event.

Estimates of the magnitudes of various design flood events, assuming several different statistical distributions, are listed in Table 3.13. The PE3 results are highlighted and are considered the more reliable; although they are almost identical to those assuming a GEV distribution. The difference between the magnitudes of the 1%AEP flood event assuming a PE3 distribution is 6.3m³/s lower than the highest estimate, and 2.2m³/s higher than the lowest estimate. The design flood estimates obtained assuming a Log Normal statistical distribution are significantly higher than any other estimate and should be disregarded. Given the uncertainties inherent in estimating the magnitudes of various design flood events, the uncertainty caused by the choice of either the PE3 or GEV statistical distribution is not likely to have a significant impact on any subsequent analysis.

Table 3.13: Results of frequency analysis for Whareroa at Fish Trap 1977-2015 (m³/s).

ARI	GEV	Generalised Logistic	Generalised Pareto	Log Normal	PE3	Gumbel
2.33	6.3	6.3	6.4	6.0	6.3	6.2
5	9.0	8.8	9.6	9.2	9.1	9.0
10	11.2	10.9	11.7	12.2	11.2	11.2
20	13.2	13.1	13.1	15.4	13.2	13.3
50	15.8	16.2	14.5	19.9	15.6	16.1
100	17.6	18.8	15.1	23.6	17.3	18.2

As a result of the relatively short nature of the Whareroa at Fish Trap annual flood maxima series, the results from the frequency analysis of the Kuratau at SH41, the adjacent catchment, were also scaled as a function of catchment area (i.e. $A^{0.8}$).

The estimated peak discharge during a 1%AEP event in Whareroa Stream derived by scaling the results of a flood frequency analysis of the longer-term Kuratau River at SH41 Bridge flow record by $A^{0.8}$ is approximately twice as large as that obtained from the local flow series (Table 3.14). An extensive discussion of the reasons for this difference, and the potential effect of the results on the assessment of the flood hazard is provided in Paine and Smith (2012c).

However, the incised nature and local topography of the lower Whareroa Stream means that any 'error' in estimating the magnitude of design floods is likely to have little effect on the results of any flood hazard assessment.

Table 3.14: Results of frequency analysis for Kuratau River at mouth (m³/s).

ARI	Kuratau River at the SH41 Bridge (119km²)	Whareroa at Fish Trap scaled from Kuratau at SH41 (58.8km²)	Whareroa at Fish Trap from actual record
2.33	23	13	6.3
5	30	17	9.1
10	37	21	11.2
20	43	25	13.2
50	53	30	15.6
100	60	34	17.3

4 Hydraulic modelling

The hydraulic modelling undertaken as part of the Taupō District Flood Study covered five watercourses: the Tongariro River; Kuratau River; Hinemaiaia River; Whareroa Stream; and Tokaanu Stream. The extents of each of the hydraulic models are described in the respective 'Taupō District Flood Hazard Study' reports (Maas & McConchie, 2011; Paine & Smith, 2012a; Paine & Smith, 2012b; Smith *et al.*, 2011; Paine & Smith, 2012c). The hydraulic modelling of the Tongariro River is covered by its own separate report (Maas, 2009).

4.1 Modelling software

All the computational hydraulic modelling was undertaken using *"MIKE software"* developed by DHI Water and Environmental (formerly the Danish Hydraulic Institute and some other organisations). MIKE hydrodynamic modelling software tools are widely used, throughout New Zealand and internationally, to aid engineers and other professionals in the assessment and management of river flooding. Hydrodynamic modelling allows the user (with enough input data) to estimate water levels, flows, and velocities at discreet locations and points in time over the duration of a flood. Effects on water level can be carried both upstream and downstream over time and so these models are termed dynamic.

Hydrodynamic modelling tools are generally classified as either one-dimensional (1D) or two-dimensional (2D). The difference between the two types of models relates to the mathematical approach adopted to describe the river and floodplain.

4.2 1D and 2D models

One-dimensional models (e.g. MIKE11) generally represent the river or stream as a series of cross-sections. Flow, velocity and depth are averaged across each cross-section of the river channel or floodplain which comprises the model's river network. There are various methods to improve the answers given by averaging, but in essence a 1D model requires the user to fully specify the direction of water flow. This assumption is generally good when flow is predominantly uni-directional, such as in a well-defined river channel.

In a 2D hydrodynamic model (e.g. MIKE21) flow, velocity and depth are able to be calculated across a near-horizontal planar surface. Localised values of flow velocity (averaged over the depth) and depth are obtained across a grid network rather than over a channel cross-section. This approach is especially useful when modelling flow across a floodplain where the horizontal flow direction is difficult to predict. MIKE21 also more accurately simulates energy losses as any flow moves across the floodplain than a 1D model. It can also estimate the energy losses which occur when flow either re-enters or exits the main channel. Consequently, 2D models are much more appropriate for modelling flow across floodplains.

The MIKE suite of hydrodynamic modelling software provides the following tools:

- MIKE11 – a 1D tool generally used to model flow within a defined uni-directional channel;
- MIKE21 – a 2D tool used to model flow across a floodplain or large channel where the landscape can be represented by a 'grid' rather than cross-sections. Flow direction may be difficult to predict, and there may be significant differences in flow depth, velocity and direction between adjacent cells forming the grid. A two-dimensional MIKE21 model can simulate multi-directional flow over a floodplain when combined with high-resolution topographic information, such as that from a LiDAR survey; and
- MIKEFLOOD - is able to more realistically model a river-floodplain system where overtopping of the riverbanks, and overland flow occurs. MIKEFLOOD dynamically links together a one-dimensional MIKE11 model of a river channel with a two-dimensional MIKE21 model of a floodplain.

With respect to the Taupō District Flood Study the flood hazard of the Tongariro River and Tokaanu Stream was modelled in MIKE21. In these cases the channel was either large enough to be represented as a 'mesh' of grid cells (e.g. the Tongariro River) or so small as to have no significant effect of the flooding (e.g. Tokaanu Stream). In both cases the key concern was to assess the flood risk to the floodplain rather than the river itself.

For the Kuratau River, Hinemaiaia River and the Whareroa Stream MIKEFLOOD was used to combine a one-dimensional representation of the channel in MIKE11 with a two-dimensional model of the floodplain in MIKE21.

MIKEFLOOD models are generally concerned with assessing potential inundation of the floodplain rather than the flood hazard within the actual river channel. Consequently, the results from a MIKEFLOOD model often do not include the actual channel. This was the case with respect to the Tauranga Taupō River. The flood hazard of the actual river channel was therefore assessed intuitively rather than as a function of the depth and velocity. Given the depth and velocity within the active channel the flood hazard was assessed as 'High'.

4.3 Level datum

All ground levels, river and stream bed levels, lake levels, and predicted flood levels are expressed in terms of mean sea level relative to the Moturiki datum. It should be noted, however, that the flood hazard is essentially modelled using the 'relative elevations' of the river and adjacent floodplain not the absolute height.

4.4 Floodplain description

The Digital Terrain Models (DTMs) used in the modelling had a maximum spatial resolution of 5m. All the terrain models were based on the June-July 2006 LiDAR data supplied by Environment Waikato. This was supplemented with later data provided by Taupō District Council for two locations: in the vicinity of Waihi, towards the southern end of Lake Taupō; and the foreshore adjacent to the Taupō township. All the LiDAR data has a spatial resolution of 1m and a vertical accuracy of ±0.1m. The LiDAR information was used to provide a three-dimensional model of the shoreline of Lake Taupō, and floodplain topography adjacent to various rivers and streams. Using a 5m resolution DTM allowed relatively shorter computer run-times without significantly reducing the model accuracy in terms of the prediction of flood levels and extents.

4.5 LiDAR accuracy

When raw LiDAR data are processed, part of the filtering process is to remove the effects of any vegetation cover so that the data better represents the elevation of the ground surface. Over heavily vegetated areas this is difficult and may lead to errors in the final inferred topography, and any derived DTM.

The various algorithms used to process LiDAR data produce consistent data, with a high level of reliability and accuracy. While dense tall vegetation can cause calibration problems, the relative heights are consistent across the terrain model. Since a flood surface is developed from the relative topographic information slight errors in the absolute height, usually within ±0.15m, have little quantifiable effect. LiDAR information is now the international standard for generating terrain models, and consequently flood extents, across floodplains. That is, despite the possibility of minor errors, LiDAR information is the best available for floodplain modelling.

The collection of LiDAR information does not extend deeper than approximately 10cm below any water surface. Therefore in areas where there is an abrupt change in elevation close to the shoreline of Lake Taupō, such as in the Western Bays, this leads to instability in any DTM derived from the LiDAR information i.e. accurate elevation data for the lake edge are not available because of the steep scarp. However, given the nature of the terrain where this is a potential problem i.e. steep cliffs dropping to the lake, it has no effect on the assessed flood risk.

4.6 River channel description

No surveyed cross-sections were available for the various hydraulic models; except for the Tongariro River as discussed in Maas (2009), and the Kuratau River as discussed in Smith *et al.* (2011). The LiDAR elevation measurements are of the water surface at the time the data were 'captured'. Hence, the river bed and lake bed below the water surface was not represented in the supplied data. Therefore the river bed needed to be 'burnt' into the DTM created from the LiDAR information.

For the Tongariro River, the average bed levels at each cross-section over the 5.5km reach of the river through Turangi were extracted from the previous MIKE11 model of the Tongariro River (T&T, 2004) and used to adjust the DTM at the relevant locations.

The river bed 'burnt' into the MIKE21 model included all the current gravel shoals without vegetation. This is because these shoals are considered likely to become mobile when there are significant flows in the river i.e. during the large events considered during the flood hazard analysis.

The volume of the channel of Tokaanu Stream below the water surface when the LiDAR was captured was ignored. Given that the width and volume of water in the stream during low flows, when the LiDAR was likely to have been captured, are very small relative to those during the extreme events considered in this study, this assumption is reasonable. Any error caused by this assumption is likely to be small when compared to the inherent uncertainty of other inputs to the hydraulic model.

The Kuratau River contains a deep, narrow channel in places which is difficult to represent in a two-dimensional grid. To do so requires an extremely high resolution (high number of grid cells) model. To run such a model then requires prohibitively long computer processing time. The LiDAR data was therefore augmented with cross-sections extracted from a previous Cheal Consultants HEC-RAS model. These cross-sections were generated from a terrain model derived from GPS survey contours.

The cross-sections derived from LiDAR data were compared to those where survey data were also available. A conservative depth value was then added to the LiDAR-sourced channel data. This was to ensure a smooth transition along the channel between reaches where survey data were available (lower channel), and other reaches where only LiDAR data were available.

For the Hinemaiaia River hydraulic model, cross-sections were extracted from a high resolution 1.0m DTM, and then checked carefully against the original LiDAR data used to construct the DTM. These cross-sections were then implemented in the one-dimensional MIKE11 portion of the hydraulic model.

The same process was followed for the Whareroa Stream hydraulic model, but using a higher resolution 0.5m DTM.

4.7 Lake Taupō description

Lake Taupō forms the downstream boundary for all the models so it was necessary to represent part of the lake in each model. This was done by adding the bed level of Lake Taupō to each DTM. The lake bed levels were based on available bathymetric data.

4.8 Vegetation effects

When raw LiDAR data are processed, part of the filtering process is to remove the effects of any vegetation cover. Over heavily vegetated areas this is difficult and may lead to errors in the final inferred topography. In some areas, the original LiDAR data had not been corrected adequately for the effect of floodplain vegetation.

For the Tongariro River model this led to water levels that were considerably higher than those predicted by the MIKE11 model for the calibration event (T&T, 2004). The DTM was therefore lowered over the floodplain to more accurately reflect the ground levels in these locations.

The corrections made to the DTM were primarily based on a comparison between corresponding cross-sections from the MIKE21 DTM and cross-sections from the MIKE11 model (T&T, 2004).

For the other models, no additional adjustments were made for the effects of vegetation.

4.9 Structures

Stopbanks on the Tongariro River were included in the model through the DTM. Stopbanks constructed prior to the calibration event (29 February 2004) show up in the model bathymetry and were therefore all assumed to have been included in the LiDAR data. These were not checked further, nor amended. Stopbanks constructed after the calibration event were also included in the LiDAR data. Therefore these were removed from the DTM when calibrating the hydraulic model against the 2004 flood event.

Once the model was calibrated, all the stopbanks constructed since 29 February 2004 were added back into the DTM to represent the present (2008) situation. Care was taken to ensure that each stopbank formed a continuous linear feature in the DTM. The levels used for each stopbank were based on as-built drawings provided by the Waikato Regional Council. No surveying of the stopbanks was undertaken for this study and the as-built drawings were assumed to be correct.

Consideration was given to modelling the flow through the specific design characteristics of the SH41 Bridge over Tokaanu Stream. However, investigation of preliminary model results showed that flood levels remain well below the soffit of the bridge. Also, the pier arrangement (one narrow pier in the centre of the stream) is such that no undue flow energy losses occur which would require special treatment of the bridge within the hydraulic model. Losses resulting from contraction and expansion of flow through a narrow opening were accounted for by default by using a two-dimensional modelling approach. Other small bridges were not considered to require special treatment as they are located within a highly vegetated main channel which was modelled with a relatively high resistance factor.

No additional adjustments to include specific structures were made to the Kuratau River, Hinemaiaia River and Whareroa Stream hydraulic models.

4.10 Surface roughness

Each land use in the model was assigned a Manning's n value to represent the ground surface roughness. The following roughness values were used in the models:

Table 4.1: Values of Manning's n roughness values used in the MIKE21 models.

Land cover	Manning's n	Manning's M (1/n)
River bed	0.03	33.33
Trees/vegetation along banks	0.07	14.29
Floodplain trees	0.10	10.00
Floodplain pasture	0.03	33.33
Floodplain delta grass	0.05	20.00
Urban floodplain areas	0.15	6.67
Streets/roads	0.025	40.00

The land use was determined from aerial photography and known land uses. The Manning's n value for each land use was estimated using values reported in standard literature references, past experience, and knowledge obtained from the previous calibration of the Tongariro River hydraulic model (Maas, 2009).

The relatively higher Manning's n value for 'Urban floodplain areas' accounts for the resistance to flow caused by buildings in the urban centres.

4.11 Boundary conditions

To produce the predicted flood extents for a 1%AEP design flood event on each of the rivers modelled, the 100-year peak discharge was applied to the model at the upstream end and the 1%AEP level of Lake Taupō was applied to the downstream end.

This combination of extreme events (i.e. a 1%AEP flood and a 1%AEP lake level) has a lower effective probability of exceedance than 'once in 100 years'. However, the level of Lake Taupō changes very slowly, and by only a very small amount, with decreasing frequency of an event. For example, the lake level changes only by 14cm between a 10%AEP event and a 1%AEP event. Any effect of this relatively small change in lake level is within the resolution of the hydraulic models.

4.12 Assumptions and uncertainty

There are a number of assumptions necessary during the construction of the various hydraulic models. These include simplifications of reality during model build, assumptions regarding the suitability of provided data, and the type of vegetation and materials present. These assumptions are discussed below:

- The Manning's n values applied for each of the different types of surface roughness in the models were refined during the calibration of the Tongariro River hydraulic model (Maas, 2009). Manning's n values for different surface roughness types in the other models were assigned based on values from standard references, past experience and local knowledge;
- The materials and vegetation present during the survey/site visit/aerial photography are assumed to be typical of the materials in the channel, and on the banks, at all times of year and represent the worst case;
- The measured lake level data are assumed to be accurate as they are regularly audited and subject to continuous scrutiny and peer-review;
- LiDAR data does not provide information beneath water bodies such as lakes, rivers and ponds. In the hydraulic models where this was adjusted using information from other sources, the data have been assumed to be correct;
- For the hydraulic models where the portion of wetted channels beneath the water surface was ignored in constructing a DTM, any baseflow is likely to occupy only a very small proportion of the total channel volume active during a large flood. This assumption is reasonable and any resulting error in flood level predictions is likely to be small when compared to the inherent uncertainty of other inputs to the hydraulic model;
- When LiDAR information is processed, algorithms are used to adjust elevation values to remove or smooth out the effects of vegetation. Over heavily vegetated areas this is difficult and may lead to errors in the provided LiDAR data. Other than the Tongariro model where, in some areas, the elevation of the DTM has been manually adjusted during the calibration process, the DTMs were assumed to be accurate;
- The 1%AEP flow and lake levels have been incorporated into the models to provide the design scenario flood inundation extents. This combination would be a rare event and has a lower probability than 1 in 100 years. Therefore the model results are conservative;
- The cross-sections defining the main river or stream channel in each model are assumed to remain unchanged over the course of a flood. In reality, the river or stream bed sediment material will be mobilised in large floods. However, the assumption of a "fixed" bed will generally be conservative; and

- The flood inflows to each model assumed steady flows rather than unsteady flow flood discharge hydrographs. This use of a constant steady flow makes no difference to flood level predictions where rivers or streams are hydraulically steep and where there is minimal channel storage. Such characteristics are typical of all the rivers and streams draining to Lake Taupō which were modelled.

There is some uncertainty common to all hydraulic models. This is because of the difficulty in defining real world processes precisely by mathematical expressions, and because of the assumptions made regarding model parameters and inputs. For this study, however, the assumptions made to counter or allow for this uncertainty are considered to produce results that are conservative but still realistic. Tests were carried out to assess the sensitivity of the model predictions to parameters for which there is greater uncertainty regarding their values. The results of these sensitivity tests confirmed the appropriateness of the predicted flood hazard zone in each case.

4.13 Non-contiguous areas – river-based flooding

The 2-D hydraulic modelling within MIKE21 and MIKEFLOOD assesses the movement of water across a network of cells. In the various models used for the Taupō District Flood Hazard Study cell sizes were either 2m or 5m depending on the model extent and situation. The final results were presented within a common reference grid of 5m.

Although the model consists of a network of cells, flooding is only shown when the depth of water flowing over the ground surface represented by the cell is greater than 3cm. Consequently, it is possible to have water flowing over the surface, but the area not appearing at risk of flooding, as long as any water is less than 3cm deep.

The effect of this is that it is possible to have what appear to be non-contiguous areas of flooding i.e. areas where water depths exceed 3cm but which are separated from other areas of flooding by zones of very shallow surface flow. Although these areas appear to be non-contiguous on the flood hazard map, they are actually connected by zones of shallow surface runoff and flooding.

4.14 Tongariro River model update

The digital terrain model used for the flood hazard assessment of the Tongariro River, reported in Maas (2009) and Maas & McConchie (2011), was derived from LiDAR information flown in June-July 2006 and supplied by Waikato Regional Council. The LiDAR topographic information was combined with surveyed river cross-sections to establish a two-dimensional MIKE21 model. The model covers an area of approximately 7.8km by 11.5km, and extends 19km downstream from Poutu Pool to Lake Taupo.

The digital terrain model was initially adjusted by removing certain stopbanks so that it reflected the river and its floodplain prior to the 29 February 2004 flood. The model was calibrated against this flood event to ensure that all the parameters were appropriate and the results realistic.

All the stopbanks constructed since February 2004 were then added to the pre-29 February 2004 digital terrain model, using information from the 'as built' drawings, so that the model represented the present (i.e. 2009) situation. It is the results from this model which are reported in Maas (2009) and Maas & McConchie (2011).

As part of a *'Service Level Review'* Waikato Regional Council used the MIKE21 model with the same parameters and boundary conditions (Grant, 2014). Two changes, however, were made. These were:

- Using a flood hydrograph rather than a constant flood discharge; and
- Updating the representation of the stopbanks within the model using their actual surveyed heights.

These changes, particularly the inclusion of what are essentially slightly higher stopbanks, resulted in some changes to the pattern of flooding previously presented in Maas (2009) and Maas & McConchie (2011).

To ensure that the latest information is used to inform the District planning process, the results from this latest modelling have been adopted (i.e. those from Grant, 2014). These results are considered to best represent current conditions likely to affect the flood risk to land adjacent to the Tongariro River.

5 Waves run-up analysis

5.1 Model adopted

Wave run-up around Lake Taupō has been estimated previously by Hicks *et al.* (2000) using the hindcast model '*LakeWave*'. Output from the model was used successfully to predict lakeshore geomorphic processes, shoreline erosion, and near-shore flooding.

LakeWave, however, has a number of limitations, including:

a. It has no nearshore dissipation of wave energy, and is therefore liable to over-estimate wave run-up on shelving shores; and

b. It does not formally refract or diffract waves other than an empirical exposure weighting on the effective fetch. Therefore the results are less reliable within embayed beaches.

Constraints relating to the Taupō District Flood Study meant that it was not feasible to develop a more sophisticated model (e.g. SWAN) for assessing the potential for wave run-up around the entire shoreline. Likewise the expense of developing a more sophisticated model could not be justified. Irrespective of the model used to assess wave run-up at the District-scale, it is likely that the results would still not be 'accurate' when enlarged to the scale of specific sites or properties.

Consequently the approach adopted for the Taupō District Flood Study was to continue to use *LakeWave*, but recognise fully its constraints and limitations when reviewing and applying the results within the context of the District Plan.

While *LakeWave* has limitations as listed above, a qualitative assessment of the accuracy of the model was undertaken by Hicks *et al.* (2000). They concluded that the agreement between modelled and actual wave conditions was "*quite reasonable*". At a broader scale they also found qualitative agreement between the modelled run-up height and the amount of surf observed at the time.

Subsequently, it was recognised by Opus (2008) that *LakeWave* functions acceptably at the broad scale but that the results may be limited within small embayments where wave condition are strongly affected by refraction and diffraction.

5.2 Alternative models

SWAN2D is a more recent software model which can account for detailed bathymetric variations around the shoreline. However, constraints on existing data availability, and data resolution, would necessitate significant expenditure to realise any advantages over the results from *LakeWave*.

A more sophisticated modelling approach, such as SWAN2D, could be explored in the future. In particular, it might be appropriate for site-specific studies at critical locations highlighted by the District-scale assessment using *LakeWave*.

5.3 Model inputs

Representative beach characteristics used in Opus (2008) were initially a standard beach slope of 7 degrees and a sediment size of 2mm. These parameters were used to provide estimates of indicative potential wave run-up around the entire shoreline of the lake. The shoreline was then refined into the ten distinct wave run-up environments; within which similar wave run-up behaviour can be expected.

The considerable spatial variability in wave run-up around the shoreline means that the single value was not appropriate when quantifying the wave run-up in each of the ten zones. The complexity of the system, and constraints of the project, however meant that detailed individual site analysis was impractical. Therefore the wave run-up at ten sites was analysed using site-specific values of beach-face slope as detailed in Hicks *et al.* (2000). These locations were: Taupō Beach (slope 0.12); Five Mile Bay (slope 0.13); Waitahanui (slope 0.12); Hatepe (slope 0.16); Te Rangaiita (slope 0.13); Waihi (slope 0.13); Kuratau (slope 0.14); Whanganui Bay (0.13); Kinloch (slope 0.145); and Acacia Bay (slope 0.15). Sediment size, however, was assumed constant at 2mm across all the sites. This was because the majority of these sites were described as being composed of coarse or gritty sand with some gravel, and more detailed size descriptions were not available.

5.4 Wind data

The use of the Taupō Airport wind record as being representative of the entire lake is likely to have produced conservative estimates of run-up at the southern end of the lake. However, the lack of any high resolution long-term wind record from Turangi, or anywhere at the southern end of the lake, precluded the use of an alternative record when analysing wave run-up in that vicinity.

The shorter wind record from Turangi which does exist shows more calm periods and less severe extremes than at Taupō Airport. Therefore the wave run-up around the southern shoreline is likely to be over-predicted. This is not considered to be a significant issue because of the District-scale of the analysis, and the inherent advantages of being conservative rather than under-estimating the risk. It should also be noted that wave run-up is only one element of the hazard assessment at any location. Consequently any effect from over-estimating wave run-up is mitigated and moderated in the final analysis of flood risk at any location.

The approach adopted in the Taupō District Flood Study is consistent with that adopted in Hicks *et al.* (2000), Hicks (2006) and Macky & Bowler (1998). Those studies concluded:

> *The* Taupō *airport wind record was assumed to be representative of the whole lake. In fact, the short wind record from Turangi shows that the southern shore experiences more calm periods, and its extremes are less severe than at* Taupō *Airport (Macky & Bowler, 1998). Given this, the wind-wave modelling results presented in the following sections will over-predict the wave energy around the southern shore; although the magnitude/frequency estimates are probably still reasonable (Hicks, 2006).*

5.5 Calibration

There are currently no empirical studies of the waves and wave run-up on Lake Taupō. It is therefore not possible to establish an empirical relationship between the wind regime and the resultant wave and wave run-up response. However, a qualitative calibration of the modelled wave run-up has been done using photographic evidence e.g. Hicks *et al.* (2000) and Hicks (2006). In addition, field evidence of extreme water levels; such as erosion trim lines, vegetation edge lines, and beach ridge crests etc. can be used to verify the estimates of effective water level and their effect at the shore.

Therefore the vegetation edge lines and beach ridge crests reported in Hicks *et al.* (2000) and Cheal (2012 & 2013) have been compared to the modelled effective water levels (i.e. a combination of lake level and wave run-up) for verification purposes.

Grigg (2010 & 2011) provide useful qualitative records of flood and wave damage during specific storms. However, there are no vertical elevations recorded against the damage at specific sites. These data could therefore not be used for robust calibration purposes.

Table 5.1 includes the elevations of these various indicators of 'wave attack'. These are overlain on the estimated effective water levels for each site (Figure 5.1 through Figure 5.9). There are no surveyed levels for Whanganui Bay therefore wave run-up that site could not be calibrated.

Table 5.1: Levels indicative of extreme effective water levels around Lake Taupō from Hicks *et al.* (2000) and Cheal (2012 & 2013).

	Hicks *et al. (2000)*			Cheal (2012)	Cheal (2013)
Site	**Level of 1998 swash mark (m)**	**Ridge crest level (m) Nov. 1999**	**Vegetation edge (m) Nov. 1999**	**Vegetation edge (m) May 2012**	**Vegetation edge (m) May 2013* June 2013** July 2013*****
Taupō Beach	358.22	358.24	358.24	357.82 (Section 3)	357.73 (Section 3**
Five Mile Bay	358.14	358.80	357.95	-	-
Waitahanui	358.07	358.67	357.82	358.53 (Section7)	358.44 (Section 7)**
Hatepe	357.89	358.06	357.89	357.54 (Section 1)	356.71 (Section 1)***
Te Rangaiita	358.00	358.10	358.10	-	-
Waihi	357.60	357.66	356.83	-	-
Kuratau	357.98	357.98	357.83	357.57 (Section 11)	357.31 (Section 11)*
Whanganui Bay	-	-	-	-	-
Kinloch east x	358.05	358.04	358.04	-	-
Acacia Bay	357.58	357.41	357.15	-	-

1999 comparison

Lake Taupō was reasonably high compared to the previous years during the five month period leading up to November 1999 when the vegetation and ridge crest levels were surveyed. Therefore the effective water levels were also relatively high.

In general, the estimated effective water levels were relatively close to the indicators of high water levels during late 1999. Kinloch and Hatepe show the best 'correlation' between the calculated effective water levels and the various indicators. A few discrete wave run-ups appear to be slightly higher than the indicators; although this could be expected, and occasional high wave run-ups are unlikely to modify the shoreline. Five Mile, Waitahanui, and Kuratau all show effective water levels to be ~0.2m above the 1999 vegetation lines, and close to the swash lines. The ridge crest in 1999 was significantly higher than the modelled wave run-up at the Five Mile and Waitahanui sites.

There was a period of high effective water levels during 1998 when there were three successive large inflow events. It is likely that this period of high effective water levels resulted in the beach ridge and other features observed during 1999.

The effective water levels at two sites are shown to be below the vegetation and ridge crest levels. At Te Rangiita the modelled effective water level is ~0.5m below the limit of vegetation and ridge crest level. At Taupō foreshore the effective water level is ~0.2-0.3m below these lines.

The effective lake levels at Waihi and Acacia Bay are ~1m and ~0.5m above the vegetation line respectively over this period; and ~0.4m and ~0.2m above the ridge crests. Since these sites are at opposite ends of the lake this may indicate the bias of *LakeWave* to over-estimate wave run-up in sheltered locations. It may also indicate the use of the 'exposed' Taupō wind record for the whole lake tends to over-estimate effective water levels in sheltered locations.

2012/2013 comparison

The vegetation line was surveyed at few locations during 2012 -2013. Again, for those sites where data are available the modelled effective water levels were within ~0.2-0.3m of the vegetation lines at Kuratau and Hatepe. It should be noted that the vegetation line in 2013 is inexplicably lower than in 2012.

The effective water levels at Taupō and Waitahanui are ~0.4-0.5m and ~1m below the vegetation lines respectively. However, during this period lake levels were generally lower than over the preceding years.

Overall, there is a relatively high degree of consistency between the modelled effective water levels and various physical indicators of the effect of high water levels at the shoreline. The majority of the differences appear to be within the range of uncertainty associated with the derivation of effective water levels. However, some of the differences between modelled and observed extreme water levels could be attributed to the District-scale nature of *LakeWave* and differences in the exact location of survey lines and corresponding point from which data were extract from *LakeWave* for comparison.

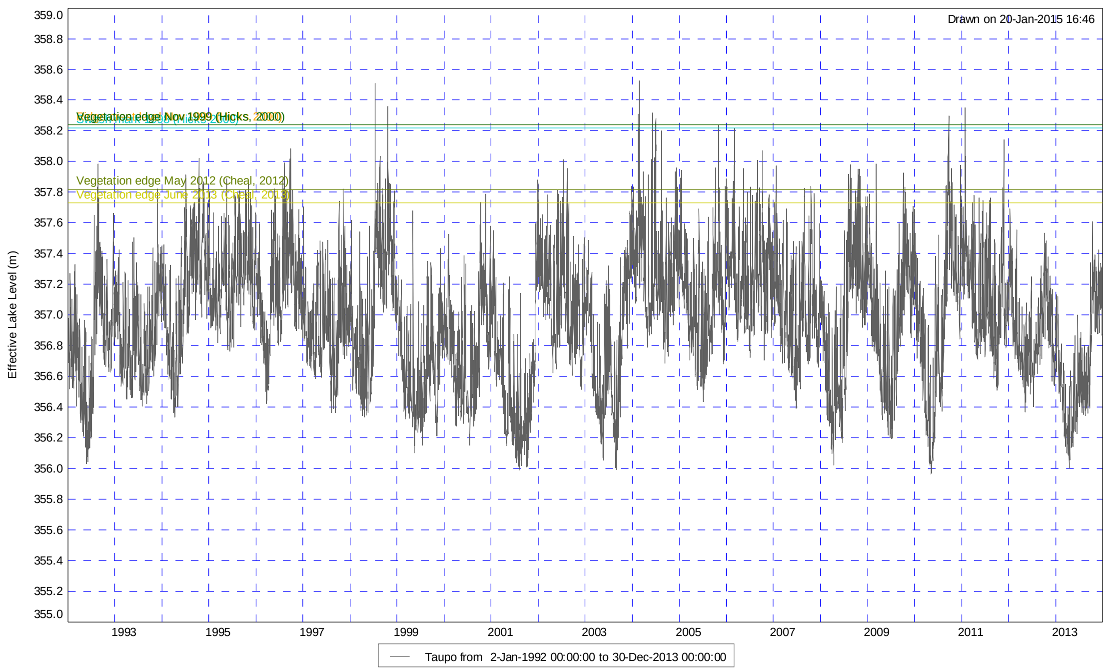

Figure 5.1: Extreme effective water elevation markers on Taupō Beach.

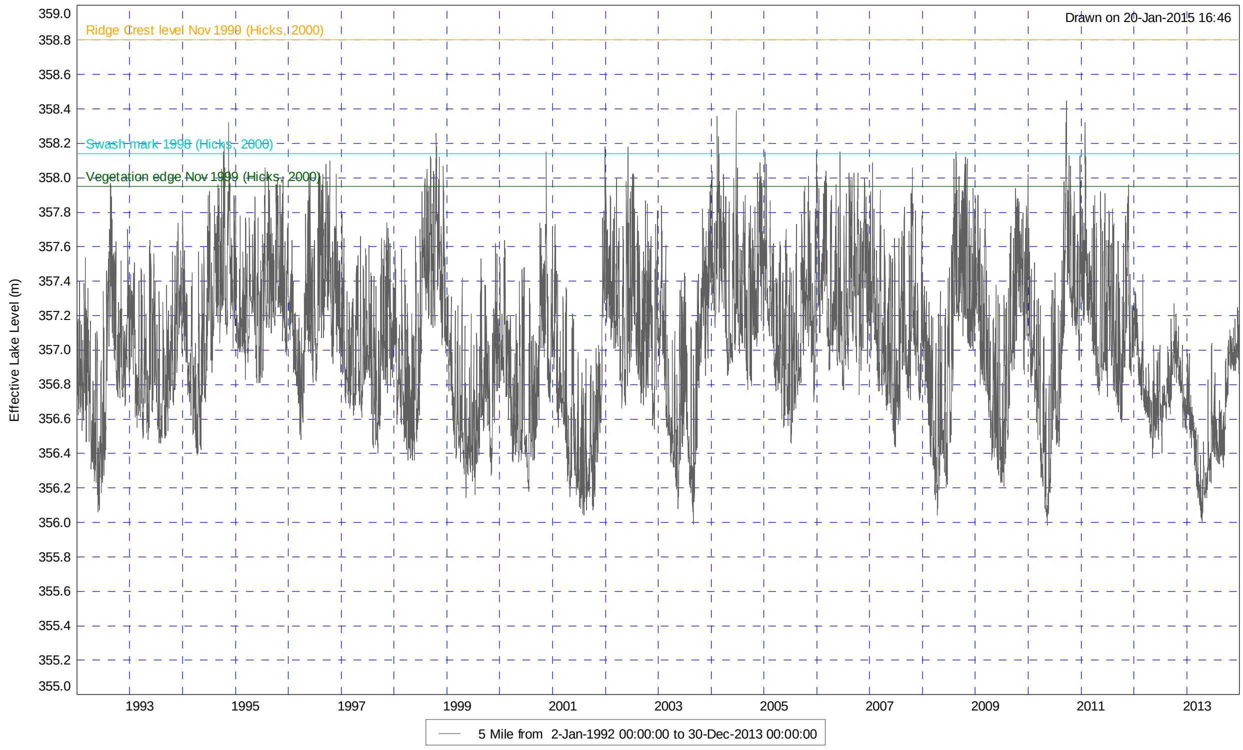

Figure 5.2: Extreme effective water elevation markers on 5 Mile Beach.

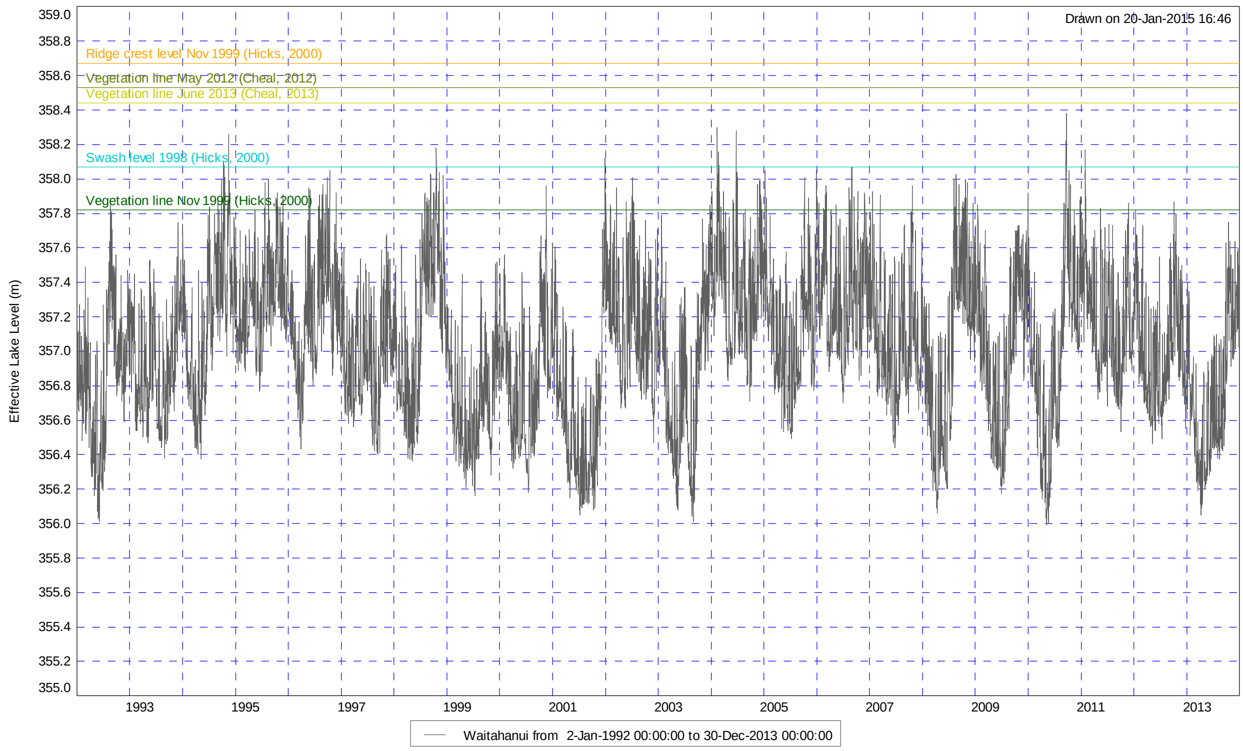

Figure 5.3: Extreme effective water elevation markers on Waitahanui.

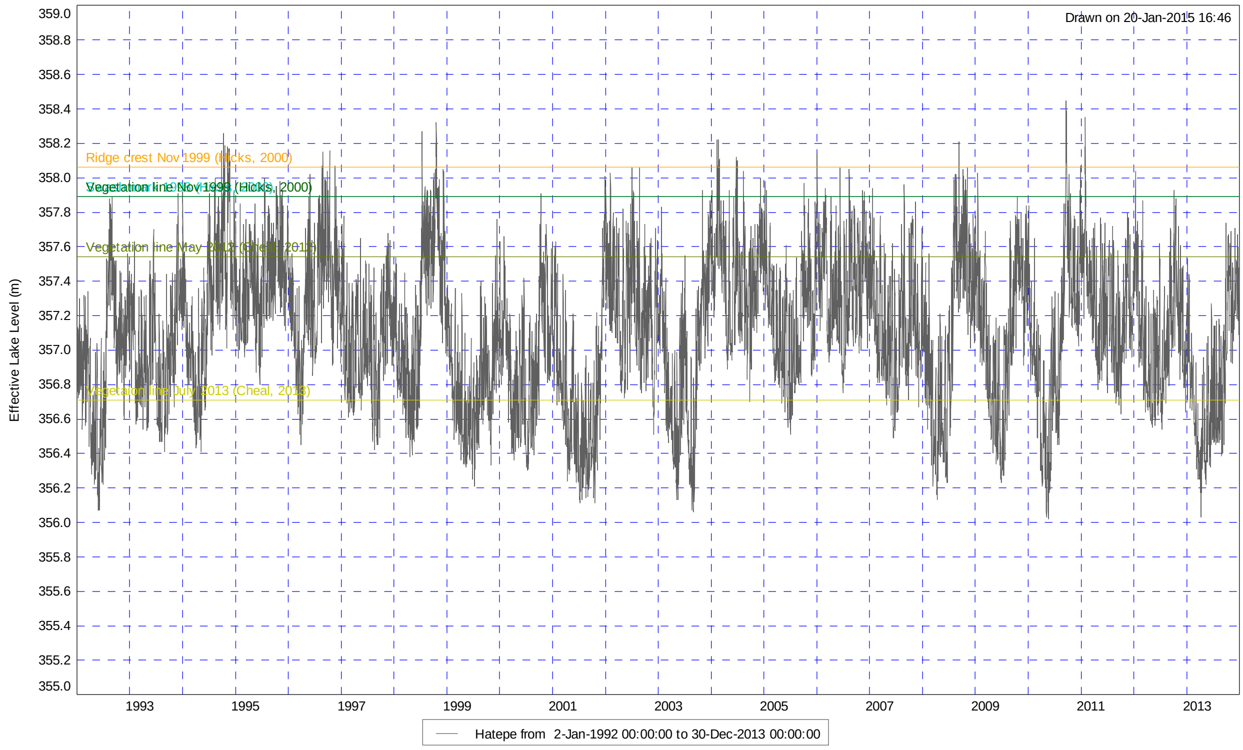

Figure 5.4: Extreme effective water elevation markers on Hatepe.

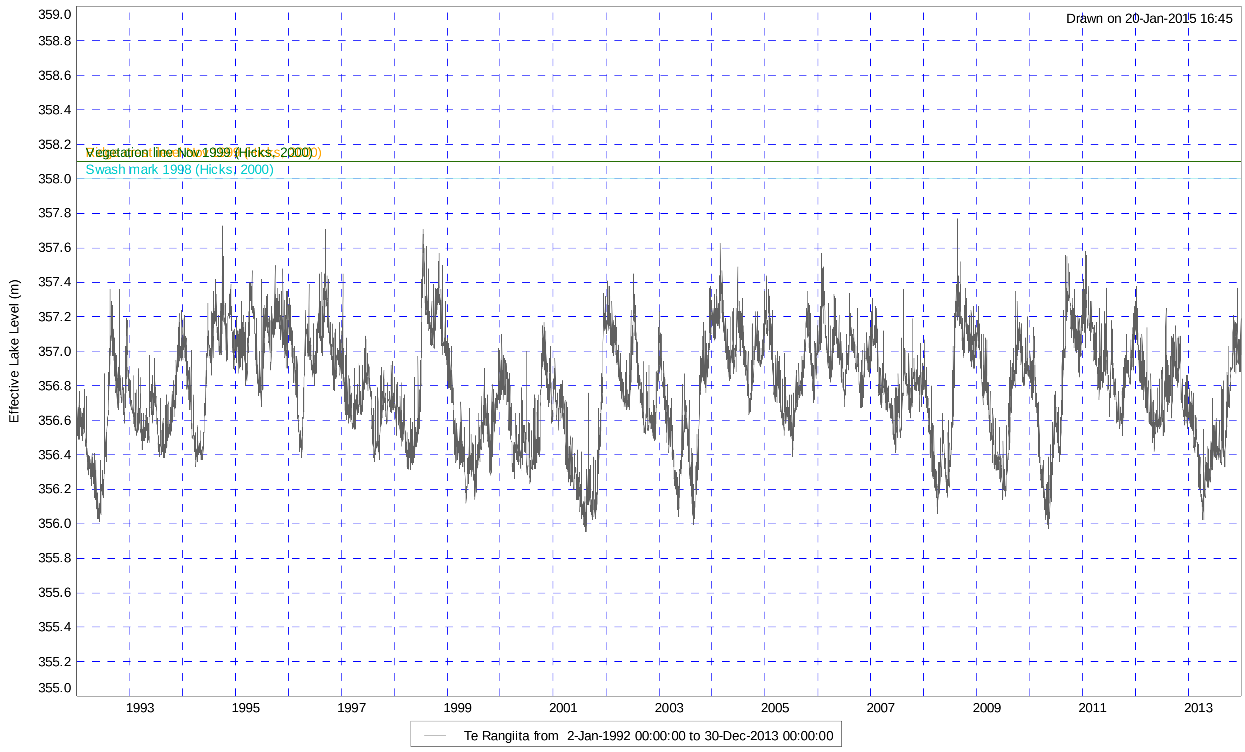

Figure 5.5: Extreme effective water elevation markers on Te Rangiita.

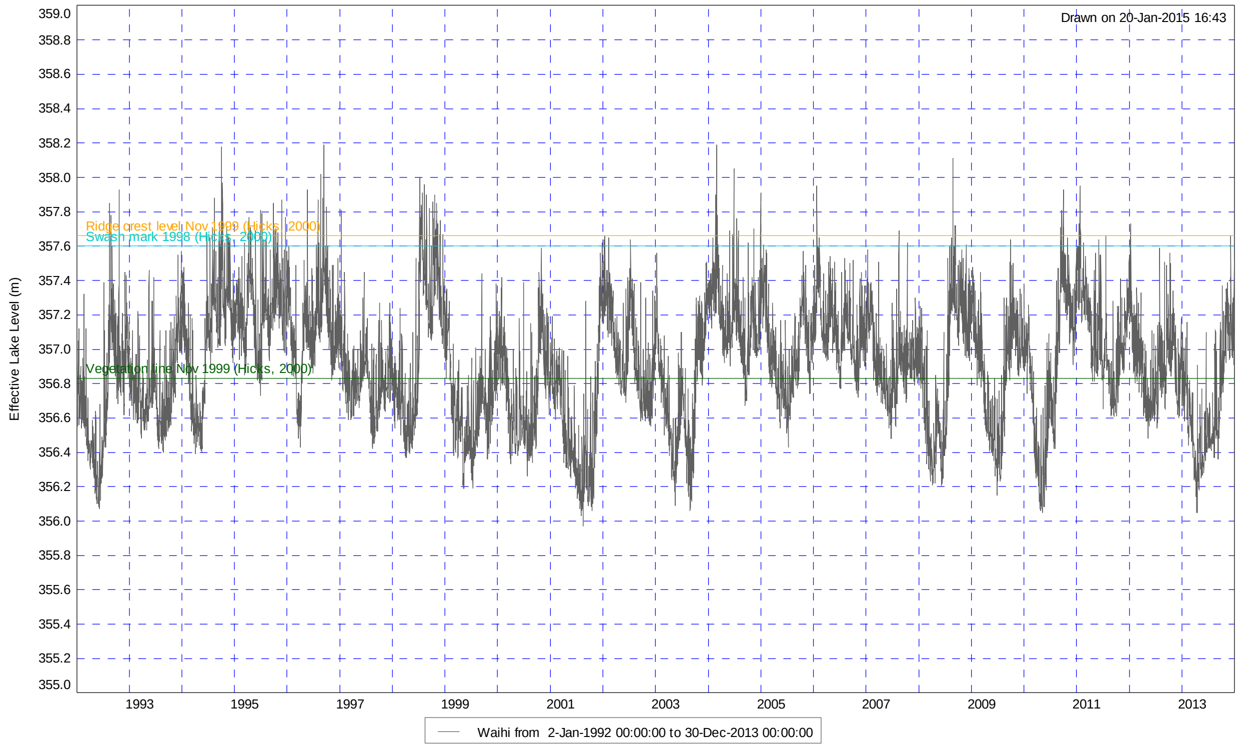

Figure 5.6: Extreme effective water elevation markers on Waihi.

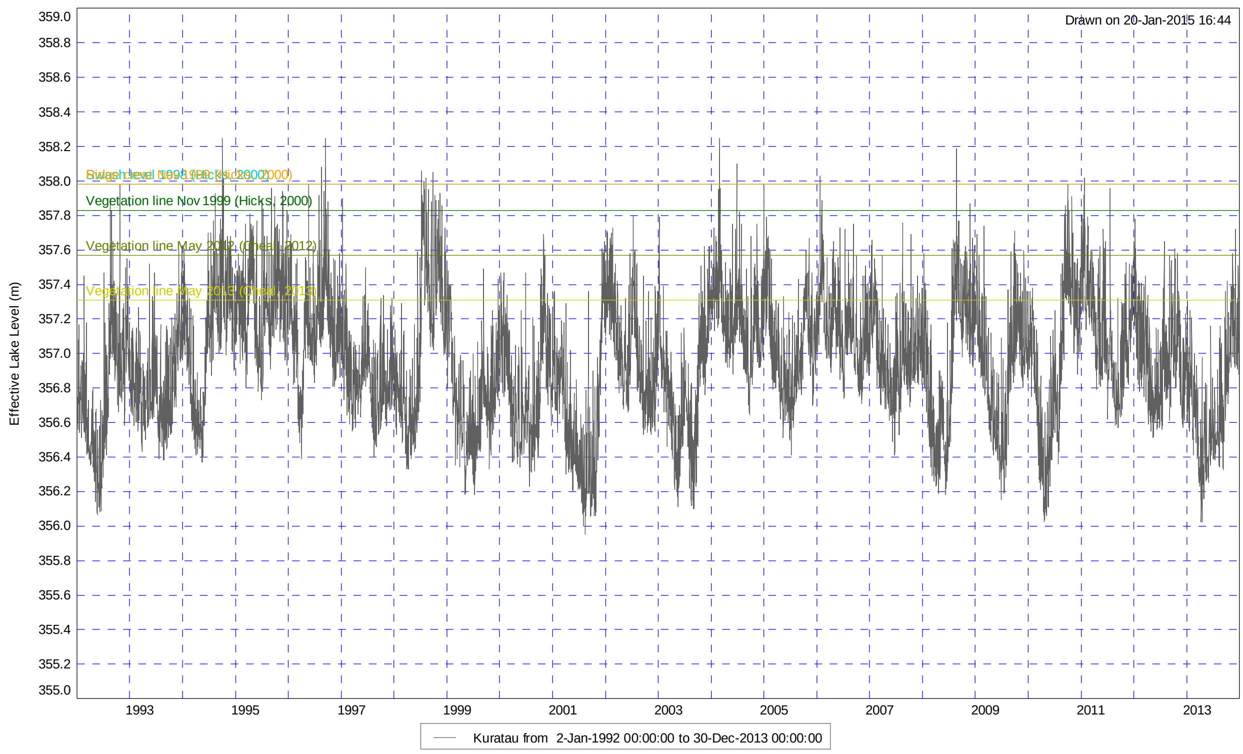

Figure 5.7: Extreme effective water elevation markers on Kuratau.

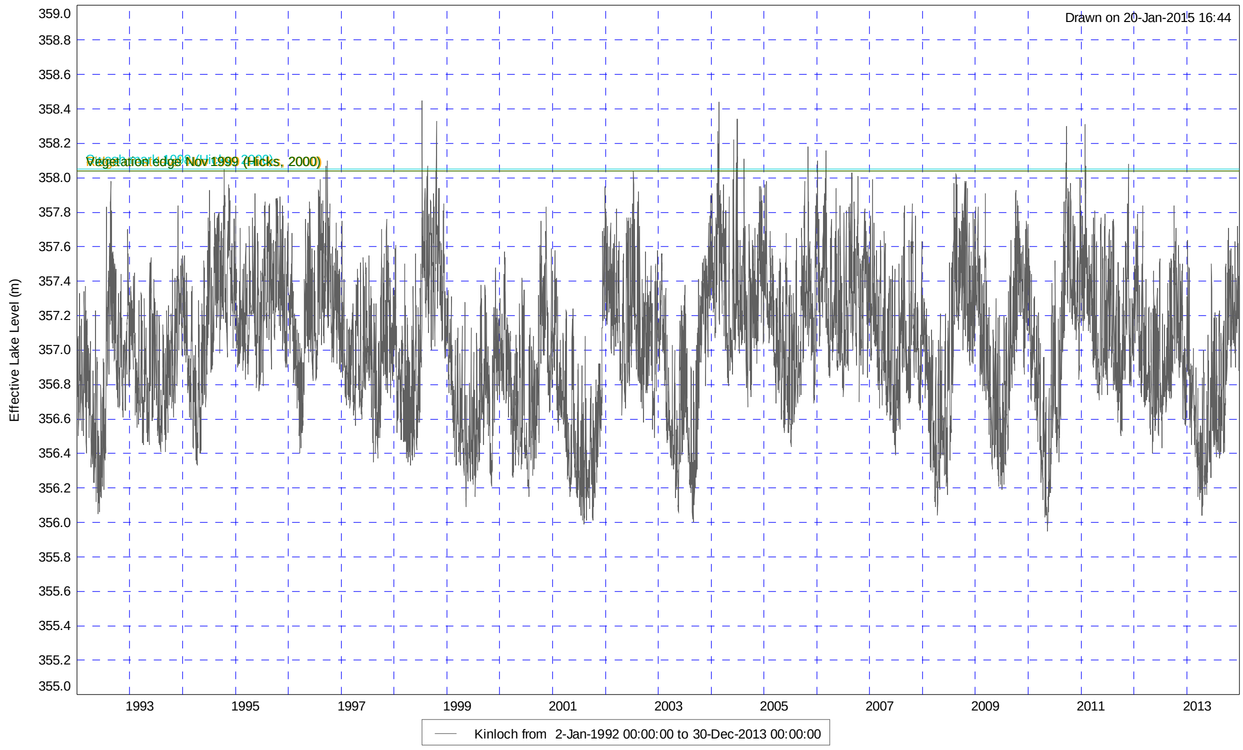

Figure 5.8: Extreme effective water elevation markers on Kinloch.

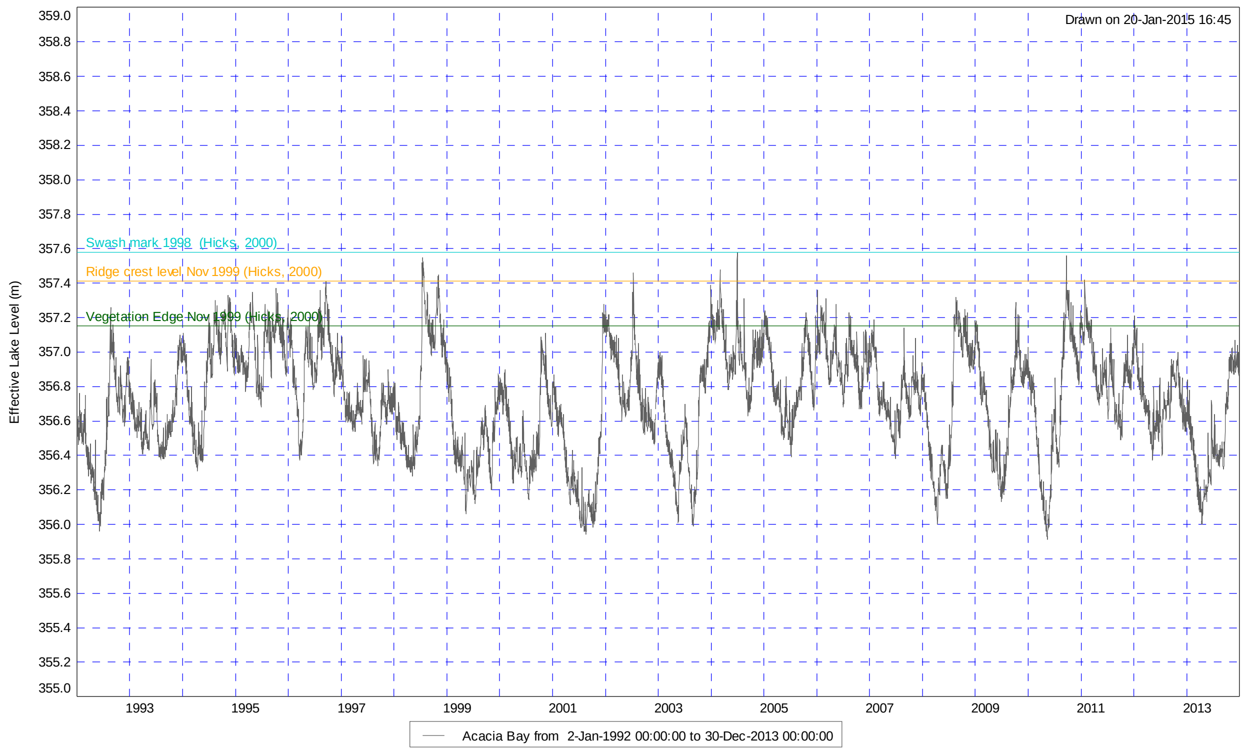

Figure 5.9: Extreme effective water elevation markers on Acacia Bay.

5.6 Non-contiguous areas – lake flooding

Flooding of the shoreline of Lake Taupō as a result of high lake levels was assessed in a different manner to the river-based flooding. Critical water levels were overlaid on the LiDAR-derived DTM, and adjusted for any effects of tectonic subsidence. Any areas where the elevation of the ground is less than the water level were initially indicated as 'flooded', and the depth of flooding derived by subtracting the ground elevation from the water level.

This approach to modelling the flood risk, however, assumes that the high water levels can actually 'connect' to any low lying areas inland from the shore. In some cases this 'connection' exists i.e. where there are rivers, streams, drains or culverts. However, in other cases the low-lying areas inland from the shoreline are separated by relatively impermeable beach ridges, berms, and road embankments etc. In these situations water in the lake cannot move inland to these areas despite being at a higher elevation.

Consequently all the areas of flooding adjacent to the lake shore were reviewed. Where a hydraulic connection could be identified between the lake and areas of flooding these were mapped as having a flood hazard. However, non-contiguous areas of flooding, where no apparent hydraulic connection could be identified, were removed from the flood hazard map.

As a result there are some areas of low-lying topography inland from the shoreline of Lake Taupō which are not shown as having a flood risk resulting from high water levels within the lake. It should be recognised that these areas are, however, likely to be susceptible to flooding and impeded drainage during localised rainstorm events.

6 Differences between wave and river flooding

A key difference between the wave run-up and the river-derived flood modelling is the scale at which the analysis was undertaken. While the river-derived flood modelling considered a 2m or 5m cell size, the wave run-up analysis considered only 10 typical sites or wave environments around the shoreline of Lake Taupō. It did not consider small lengths of the shoreline. Consequently the results of the two kinds of modelling have different resolutions and accuracy.

Overall, there is a relatively high degree of consistency between the modelled effective water levels (i.e. water levels combined with wave run-up) and various physical indicators of the effect of these high water levels at the shoreline within the various wave environments. The majority of the differences appear to be within the range of uncertainty associated with the derivation of effective water levels. However, some of the differences between modelled and observed extreme water levels could be attributed to the District-scale nature of the modelling and differences in the exact location of survey lines and corresponding point from which data were extracted for comparison.

Despite the consistency between the modelled and observed results it is not possible to apply the results of the wave run-up analysis at the site-scale. The wave run-up at any particular site can be affected by a wide range of parameters e.g. vegetation, large rocks, protection works, beach slope and character etc. All of these various parameters were not included in the wave run-up analysis and cannot be included at the scale necessary for a site-specific analysis. The cost, scale and logistical constraints of such modelling is beyond the scope and resources available for a District-scale flood assessment.

Consequently, a different approach is necessary to recognise and accommodate the potential risk from wave run-up compared to the risk from river-based flooding. The risk from wave run-up, however, will occur on top of and above, the simple flood risk caused by high water levels in Lake Taupō.

7 Combined risk

The various studies provided a holistic assessment of the potential flood risk posed by each of the tributaries and Lake Taupō. Since the flood risk is generally a function of a number of variables there is the potential for the elements of risk to cumulate, resulting in an over-estimation of the actual flood risk.

In the case of river flooding there are three main controls: the peak discharge of the design event; the potential effect of climate change; and any tectonic deformation over the longer term. The peak discharge of the design event is the major control on the extent and depth of any inundation. While climate change might increase the peak discharge by ~20%, the effect of this on the extent and depth of flooding is generally small. In almost every tributary the increase in flow resulting from climate change tends to 'fill in' those areas within the flood extent which remained 'dry' during the 100-year event. The change in the flood extent was generally within the grid resolution of the floodplain component of the hydraulic model (i.e. 5m). Because the floodplains of the various tributaries are 'large' relative to the flood discharge, the change in flood depth is also small. For example, a 1km² floodplain such as that of the Tongariro River requires 10,000m³ of flood water to raise the level by only 1cm. It would require 100,000m³ to raise the level by 10cm which is still less than the likely resolution of the various hydraulic models. Consequently, with respect to the flood hazard posed by the rivers, the potential effect of taking a cumulative and conservative approach is relatively small, and within the resolution of the models.

With regard to the risk posed by high water levels in Lake Taupō, there are a number of factors which affect the flood risk. These include the variability in lake level, seiche, the potential effect of climate change on periods when inflows exceed the discharge capacity of the Taupō Gates, and any longer term deformation of the shore. The linear addition of all these variables results in a 100-year water level only 30cm higher than that estimated from the lake level record alone. Again, this difference in potential water level, or assessed risk, is likely to be at the limit of the resolution of the terrain model used in the analysis. About two-thirds of the additional water level is caused by increased inflows caused by climate change. While this effect is likely to increase only in the longer term it must be added to the contemporary situation so as to provide a 'realistic expectation' of the flood risk towards the end of the longer term planning framework i.e. 100 years hence.

Obviously there is considerable uncertainty as to the potential effects of climate change over the longer term. However, by adopting a slightly conservative approach it is possible for the various flood hazard zones to 'contract' over time as more robust information becomes available. It would be significantly more difficult to 'expand' the flood hazard zones in the future once development had occurred if this was to become necessary.

The inclusion of a tectonic component to the flood risk is also considered reasonable and realistic. The available information shows that while tectonic deformation around the lake shore is variable, it tends to be relatively consistent at specific locations i.e. either uplift or subsidence. Consequently it is highly likely that the relative elevation of the ground will change over the longer term planning time-frame. It is considered appropriate therefore that this element of the flood risk is added linearly to those other elements affecting the overall flood risk.

Obviously the two principal components of the overall risk of high effective water levels in Lake Taupō are the relative level of the lake and the waves. The approach taken for deriving the frequency and magnitude of these combined factors on water levels is considered robust. While the two elements of risk are combined, they are essentially combined in a 'Monte Carlo' fashion by modelling the combination of the two random variables over time and then assessing the frequency and magnitude of the resulting effective water level. Again, this approach is considered reasonable when assessing the potential risk from higher effective water levels over the longer term.

8 Purpose

The purpose of the various flood studies was to provide a District-scale assessment of the potential flood risk over the longer term. The studies were never intended to provide robust flood risk assessments at the level of individual sites or building platforms. In effect, the studies were developed largely as a screening tool to identify those areas where flood risk is not a consideration, and those where some further investigation may be warranted. The uncertainty inherent in both flood modelling of extreme design events, and a District-scale assessment, mean that the resulting flood maps should not be regarded as 'definitive'. While the maps are robust, given the various assumptions and the contemporary situation, should either of these change then so too might the flood hazard maps.

It is suggested that the flood hazard maps therefore provide guidance as to what level of planning control might be appropriate, rather than restricting or denying specific activities. The maps also indicate where detailed, site-specific studies, might be required before any major capital works are undertaken.

That is, the various flood hazard maps should be regarded as a planning tool and a guide for further investigation rather than necessarily providing the 'exact answer' to the nature and magnitude of the flood risk throughout Taupō District.

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