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# Flood hydrology of the Waiwhetu Stream

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# Contents

# 1. Introduction

This report presents an analysis of the flood hydrology of the Waiwhetu Stream. The aim of the report is to produce design flood estimates to use in a hydraulic model for defining the flood hazard in the Waiwhetu catchment. Three methods are used to assess the flood hydrology and derive design flood estimates:

- At-site flood frequency analysis using recorded flow data;
- Regional flood frequency analysis; and
- Rainfall runoff modelling.

The specific output required from this report is:

1. An updated rainfall runoff model for the Waiwhetu catchment;
2. Design rainfall events of 5 to 200 year return periods, for durations of 1 to 48 hours;
3. Design flood hydrographs for specific nodes within the Waiwhetu catchment;
4. Design estimates for Q2, Q5, Q10, Q20, Q50, Q100 and Q200[1].

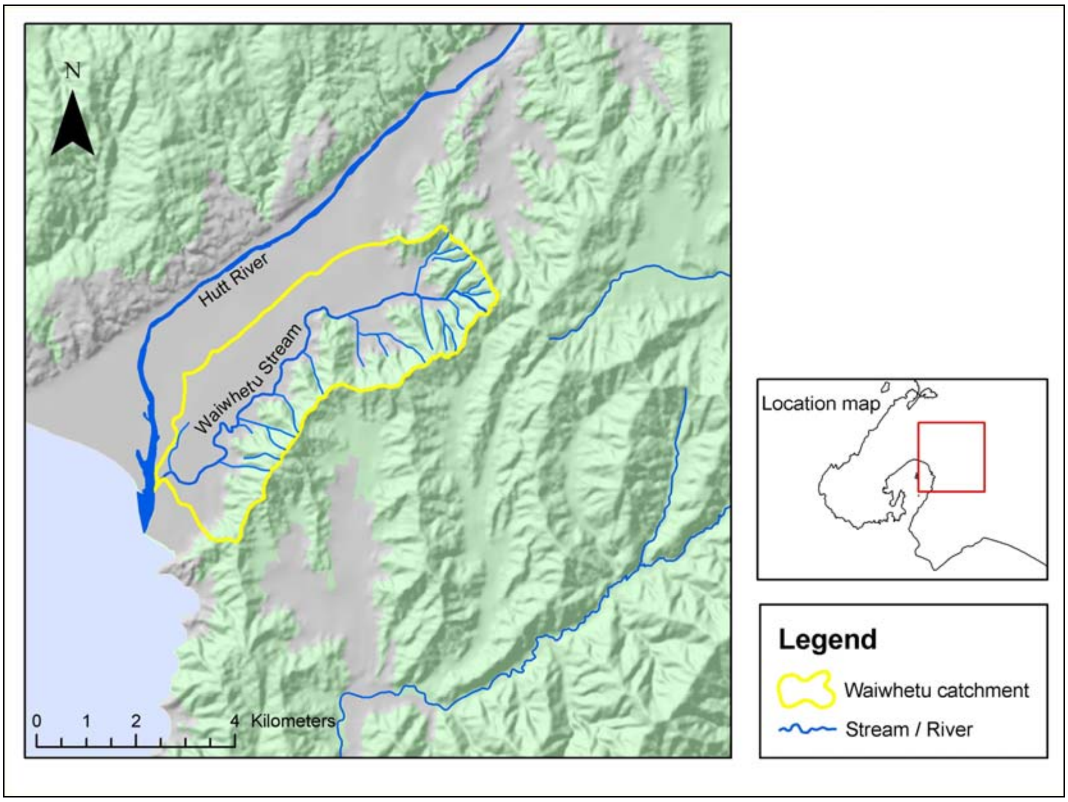
An initial review of Waiwhetu Stream flood hydrology was completed by Wellington Regional Council in the mid-1990s (Lew, 1996). This report provides an updated hydrological assessment using an additional nine years of flow data. Comparisons are made with the results of the 1996 review where appropriate.

[1] Note Qx is a flood with return period x years, i.e. a probability of occurrence of 1/x during any year.

## 2. Catchment description

The Waiwhetu Stream is located in the Hutt valley, flowing southward from its headwaters in the Eastern Hutt hills to enter the Hutt River downstream of Estuary Bridge (Figure 1). The catchment is approximately 18 $km^2$, with a main stream length of about 9 km. The headwaters of the stream, in the Eastern Hutt hills, are relatively steep but as the stream emerges onto the valley floor in Naenae the gradient reduces. An estuarine zone of about 2 km extends upstream from the Waiwhetu Stream mouth.

**Figure 1: Location of the Waiwhetu catchment, Lower Hutt**




The Waiwhetu catchment is bounded by the ridge of the Eastern Hutt hills, and by stormwater drains to the west of the main railway line. Apart from small streams from the Eastern Hutt hills and stormwater drains the only major tributary of the Waiwhetu Stream is the Awamutu Stream, which enters the Waiwhetu Stream near its mouth.

Much of the area of the Waiwhetu catchment is in residential and industrial land use, and therefore a large percentage of the catchment is impervious. The main stream channel is concrete-lined in places, and as the channel is confined the capacity is only sufficient to contain approximately a 20-year flood event. The stormwater channels that feed into the Waiwhetu Stream are unlikely to cope with flood levels greater than a 2-year return period (Minson, 1996). Due to the susceptibility of this catchment to flooding, and the subsequent risk to urban areas, it is vital that the flood hydrology of the stream is understood.

# 3. Data availability and quality

## 3.1 Rainfall data

Although rainfall data has been collected in or near the Waiwhetu catchment since the 1940s, the rainfall record is disjointed. Several daily rainfall stations that operated since the 1950s or 1960s have now been closed. The only historic automatic rainfall station near the Waiwhetu catchment was at Avalon, and this was closed in 1993. However, the Avalon rainfall record has not been completely digitised.

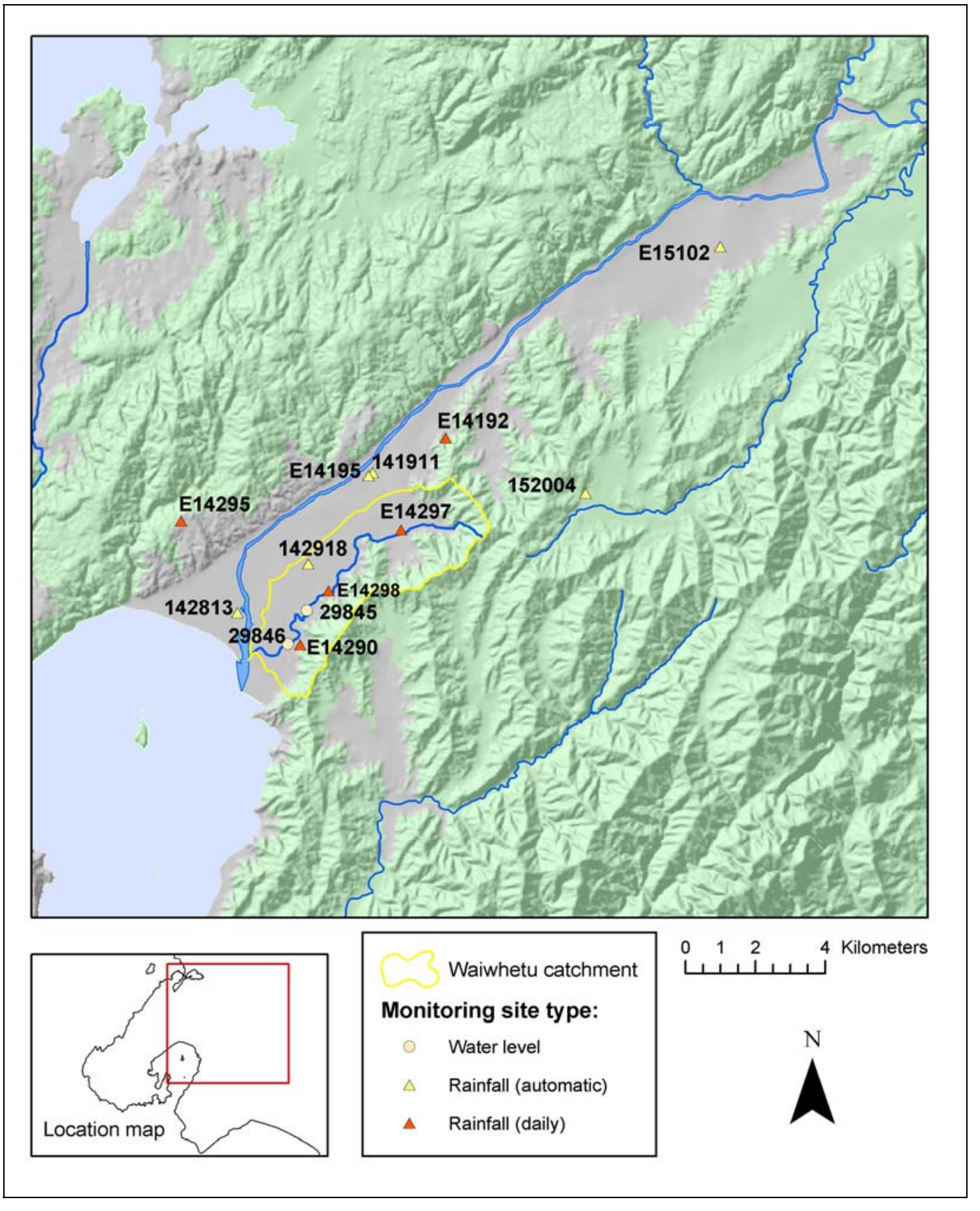
Greater Wellington installed an automatic rainfall station at Mabey Road in 1995. Thus there is a gap in automatic rainfall records for the Lower Hutt area between 1993 and 1995. Shortly before the closure of the Mabey Road rainfall station Greater Wellington installed an automatic rainfall station at Birch Lane, on the valley floor of the Waiwhetu catchment. An automatic station was also installed at Shandon, near the southern end of the Waiwhetu catchment.

Table 1 lists the rainfall data that exists for the Waiwhetu catchment and surrounding areas. The location of these sites is shown in Figure 2. Note that since 1996 Hutt City Council has operated automatic rainfall stations at several sites in and near the Waiwhetu catchment. However, as this network is not maintained to hydrometric data collection standards and the records have several gaps during major storm events, the data is not useful for rainfall runoff modelling or depth-duration frequency analyses at this time.

**Table 1: Rainfall stations in and around the Waiwhetu catchment**

| Station (site number) | Catchment | Period of record | Recorder type |
|---|---|---|---|
| Birch Lane (142918) | Waiwhetu | July 2001 – present | Automatic |
| Shandon (142813) | Hutt (Petone) | April 2000 – present | Automatic |
| Wallaceville (E15102) | Hutt (Upper Hutt) | 1939 – present | Daily / automatic |
| Tasman Vaccine (152004) | Mangaroa | 1980 – present | Automatic |
| Mabey Road (141911) | Hutt (Central) | 1995 – 2003 | Automatic |
| Avalon (E14195) | Hutt (Central) | 1948 – 1993 | Automatic |
| Taita (E14192) | Hutt (Central) | 1957 – 1993 | Daily |
| Gracefield (E14290) | Waiwhetu | 1958 – 1992 | Daily |
| Waterloo (E14298) | Waiwhetu | 1962 – 1982 | Daily |
| Maungaraki (E14295) | Hutt (Western Hills) | 1969 – 1990 | Daily |
| Naenae Park (E14297) | Waiwhetu | 1948 – 1970 | Daily |
| Waiwhetu (E14293) | Waiwhetu | 1903 – 1959 | Daily |

**Figure 2: Hydrologic monitoring sites in and near the Waiwhetu catchment**




## 3.2 Water level and stream flow data

Water level data for the Waiwhetu Stream has been recorded at Whites Line East (29845) (Figure 2) since 1978. A second water level monitoring station operated at Bell Road Bridge (29846) between 1978 and 1980. Due to the short record at Bell Road Bridge, and the lack of a rating curve for the site, this report will use the flow record from Whites Line East only.

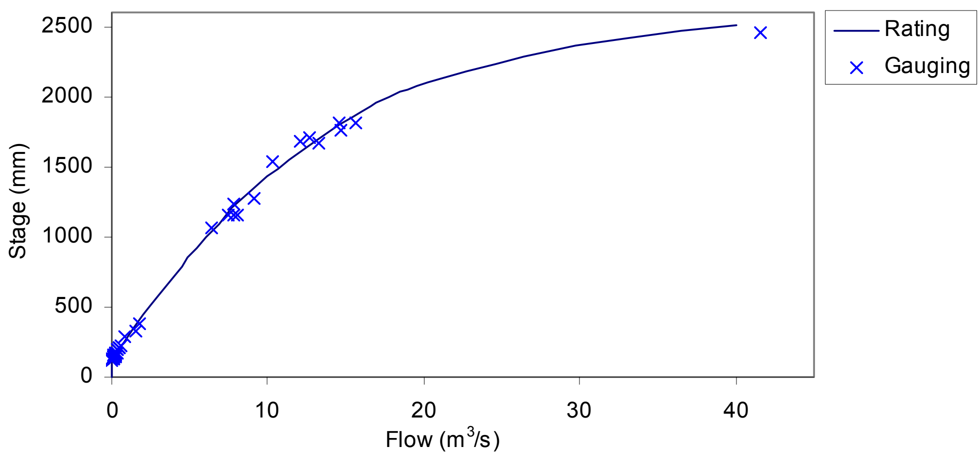
### 3.2.1 Rating curve and gaugings

Data for the Whites Line East water level monitoring station is collected in accordance with the Resource Information Quality Procedures, which meets the ISO/NZS 9002 standard for production, installation and servicing. Figure 3 shows the current rating curve for the site. The high-end of the rating curve was changed in February 2004 following a high flow slope-area gauging. All other gaugings to support the rating curve were performed with a current meter.

The slope-area gauging corresponds to the highest stage on record. The gauging is likely to have an error of +/- 20 to 30%, and this was taken into account when adjusting the rating curve. The highest flow gauged with a current meter corresponds to only 43% of the highest flow on record, thus it was important to take the slope-area gauging into consideration when fitting the rating curve.

Most of the flood events to be used for calibrating the rainfall runoff model and for the at-site frequency analysis are between 12 and 20 $m^3/s$. The rating curve is well-supported by gaugings in this range. Gaugings above 2000 mm stage height would help confirm the rating curve for more extreme flows.

**Figure 3: Rating curve for Waiwhetu Stream at Whites Line East (29845)**




### 3.2.2 Effect of the Whites Line East bridge

Previous reports have expressed concern over the effect of the Whites Line East bridge on high flows recorded at site 29845 (Lew, 1996; Minson, 1996). The bridge is likely to have the largest impact at stage heights greater than 2700 mm (the level of the bridge soffit). Below this level the culvert acts as a control for the site, resulting in a stable rating curve. In addition, the rating curve is reasonably well-supported by gaugings at stage heights less than 2500 mm, meaning that any impact of the bridge will be accounted for in the rating. Therefore it is assumed that the impact on data for calibrating the runoff model and for the at-site flood frequency analysis is minimal.

It is likely that the bridge will have an effect on recorded water level at stage heights above 2700 mm, causing a change in the shape of the rating curve. Flow gaugings (preferably current meter gaugings) at high stages should be a priority. Hydraulic modelling with an updated hydraulic model will also help confirm the rating curve at high stages and the likely impact of the bridge.

### 3.2.3 Record continuity and annual maximum series

The Whites Line East water level site initially opened in 1969 and operated until 1973, although records from that period have not been digitised. The site reopened in May 1978.

The data record from 1978 to the present has a large amount of missing record (Table 2), mostly caused by problems with the Foxboro chart recorder (which was replaced in 1989). Lew (1996) concluded that no “major” floods occurred during missing record. However, although the largest floods since 1978 have been recorded, it is possible that some annual maxima occurred during the period of missing record. The record at Whites Line East was compared with the flow record from the Hutt River at Taita Gorge (site 29202). The comparison showed that it is likely that eight annual maxima are missing from the Whites Line East flow record, and those years were removed from the series (Table 3) to reduce error associated with the flood frequency analysis in Chapter 6. Note that this report assumes the 16 February 2004 flood event will be the annual maximum for 2004.

**Table 2: Gaps in the Waiwhetu Stream at Whites Line East (29845) flow record[2]**

| Missing record start date | Missing record end date | Length of missing record (days) |
|---|---|---|
| 25/12/1978 | 4/01/1979 | 10 |
| 9/01/1979 | 19/01/1979 | 10 |
| 9/04/1979 | 17/04/1979 | 8 |
| 29/05/1979 | 11/06/1979 | 26 |
| 14/06/1979 | 12/07/1979 | 28 |
| 20/08/1979 | 10/09/1979 | 42 |
| 1/10/1979 | 3/12/1980 | 429 |
| 5/02/1981 | 9/04/1981 | 190 |
| 11/04/1981 | 29/04/1981 | 18 |
| 15/01/1982 | 22/01/1982 | 7 |
| 12/02/1982 | 8/03/1982 | 24 |
| 21/11/1983 | 7/12/1983 | 16 |
| 13/10/1986 | 22/10/1986 | 9 |
| 6/04/1987 | 21/04/1987 | 15 |
| 23/09/1987 | 18/08/1989 | 695 |
| 18/10/1989 | 2/11/1989 | 15 |
| 22/02/1990 | 9/03/1990 | 15 |
| 28/02/1991 | 8/03/1991 | 8 |
| 17/07/1991 | 22/08/1991 | 36 |
| 19/09/1991 | 15/10/1991 | 26 |
| 28/11/1991 | 11/12/1991 | 13 |
| 22/02/1992 | 3/04/1992 | 41 |
| 14/10/1992 | 2/02/1993 | 111 |
| 6/04/1993 | 28/05/1993 | 52 |
| 1/07/1993 | 19/08/1993 | 49 |
| 4/10/1993 | 21/09/1994 | 352 |
| 2/10/1995 | 28/01/1996 | 118 |
| 29/06/1996 | 22/08/1996 | 54 |
| 11/11/1996 | 19/12/1996 | 37 |
| 4/02/1997 | 14/03/1997 | 38 |
| 8/02/1998 | 1/04/1998 | 52 |
| 9/09/1998 | 29/10/1998 | 50 |
| 21/05/1999 | 27/08/1999 | 98 |
| 28/09/1999 | 24/03/2000 | 178 |

[2] Only gaps longer than 7 days included in table

**Table 3: Annual maximum series for Waiwhetu Stream at Whites Line East (29845)**

| Year | Annual maximum ($m^3/s$) |
|---|---|
| 1978 | *Year excluded* |
| 1979 | 10.1 |
| 1980 | *Year excluded* |
| 1981 | 15.6 |
| 1982 | 14.9 |
| 1983 | 13.3 |
| 1984 | 8.8 |
| 1985 | 12.1 |
| 1986 | 9.2 |
| 1987 | *Year excluded* |
| 1988 | *Year excluded* |
| 1989 | *Year excluded* |
| 1990 | 16.1 |
| 1991 | 9.2 |
| 1992 | 9.8 |
| 1993 | *Year excluded* |
| 1994 | 13.1 |
| 1995 | *Year excluded* |
| 1996 | 12.6 |
| 1997 | 19.7 |
| 1998 | *Year excluded* |
| 1999 | *Year excluded* |
| 2000 | 13.5 |
| 2001 | 23.0 |
| 2002 | 11.3 |
| 2003 | 14.8 |
| 2004 | 36.2 |

### 3.2.4 Historic flood data

Several historic events are also available for the analysis:

1. Lew (1996) reported that a flood of about 23 $m^3/s$ at Whites Line East occurred in 1950. The source of this estimate is unknown.

2. The December 1976 flood was estimated to be about 36 $m^3/s$ at Whites Line East (Lew, 1996, following Wellington Regional Water Board, 1977), although the exact methodology for deriving the estimate was not specified. It is likely to have an error of at least +/-20%.

3. A peak for the November 1977 flood was estimated to be 24 $m^3/s$ by Lew (1996). The estimate was derived using a MIKE11 model to gain a flow for the flood level reported by Hovey (1979). The likely error of the estimate is unknown.

# 4. Rainfall analysis

The rainfall analyses required are:

- Selection of rainfall data for use in calibrating and validating the rainfall runoff model for the Waiwhetu catchment;
- Estimation of the 5, 10, 20, 50, 100 and 200 year return period design storms for the Waiwhetu catchment; and
- Estimation of the probable maximum precipitation for the Waiwhetu catchment.

## 4.1 Rainfall data for runoff modelling

For this study rainfall data is needed for eleven storm events which will be used for calibrating and validating a runoff model[3]. Table 4 shows which automatic rainfall stations were operative at the time of each event.

**Table 4: Storm events for rainfall-runoff modelling: rainfall data availability**

| Event date | Rainfall data available |
|---|---|
| 21 May 1981 | Avalon |
| 11 December 1982 | Avalon |
| 13 March 1990 | Avalon |
| 8 November 1994 | Wallaceville |
| 4 October 1997 | Mabey Road |
| 2 October 2000 | Mabey Road, Shandon |
| 22 November 2001 | Mabey Road, Birch Lane, Shandon |
| 27 December 2001 | Mabey Road, Birch Lane, Shandon |
| 9 June 2003 | Birch Lane, Shandon |
| 3 October 2003 | Birch Lane, Shandon |
| 16 February 2004 | Birch Lane, Shandon |

The Birch Lane rainfall station is the only gauge located within the Waiwhetu catchment, but this gauge was only in existence for five of the eleven events. For the remaining events Mabey Road, Wallaceville or Avalon rainfall data must be used. The issues to be addressed are:

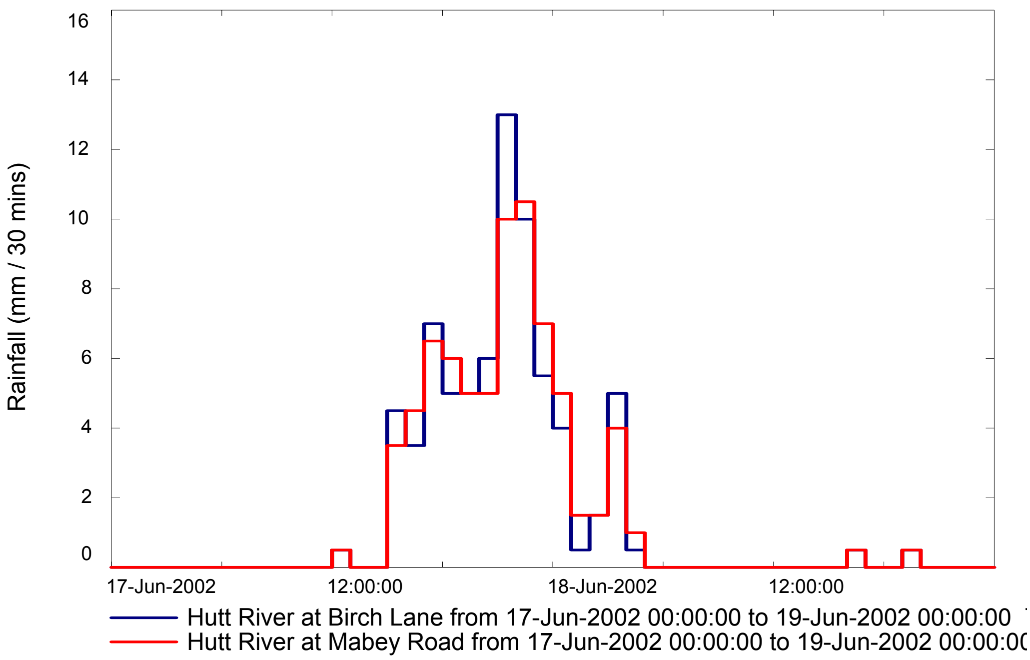
- Are the Avalon, Wallaceville, and Mabey Road rainfall stations representative of rainfall in the Waiwhetu catchment? and
- Is spatial variation within the Waiwhetu catchment likely to be significant, so that it should be incorporated into the rainfall-runoff model?

[3] The selection of these events is outlined in Chapter 4

### 4.1.1 Use of rainfall data from outside the Waiwhetu catchment

For six storm events there is no automatic rainfall data for the Waiwhetu catchment. To assess the appropriateness of using rainfall data from Mabey Road for the Waiwhetu catchment, the period of overlapping record for Birch Lane and Mabey Road was compared (July 2001 – February 2003). During this period the rainfall totals for the two sites differ by only 3%. When several storm events during this period were compared (for example see Figure 4) the storm rainfall totals tended to be very similar. The assumption was therefore made that the Mabey Road rainfall site is adequate for representing rainfall in the Waiwhetu catchment with no adjustment factor.

**Figure 4: Example of rainfall at Birch Lane compared with rainfall at Mabey Road**




To assess the use of Avalon rainfall data for use in the Waiwhetu catchment, daily records from Avalon were compared to daily records at Waterloo for the period 1969 to 1983 (when the Waterloo rainfall site was removed). During this period, the total rainfall between the sites differed by only 2%. Also, the Avalon site is very close to the Mabey Road site, which was found to record similar rainfall totals to those at Birch Lane. Therefore it is assumed that rainfall data from Avalon is appropriate to use for modelling runoff in the Waiwhetu catchment when necessary.

For the storm event of November 1994 there is no daily rainfall data available for the Waiwhetu catchment and the nearest automatic rainfall data is from Wallaceville. After comparing the annual rainfall totals at Wallaceville and at Waiwhetu (as in Wellington Regional Council, 1995), an adjustment factor of 0.9 was applied to the Wallaceville rainfall data to represent rainfall in the Waiwhetu catchment. However, as the November 1994 event is to be used for model validation only the calibration results for the flood model are not dependent on the accuracy of this adjustment factor.

### 4.1.2 Spatial variation of rainfall within the Waiwhetu catchment

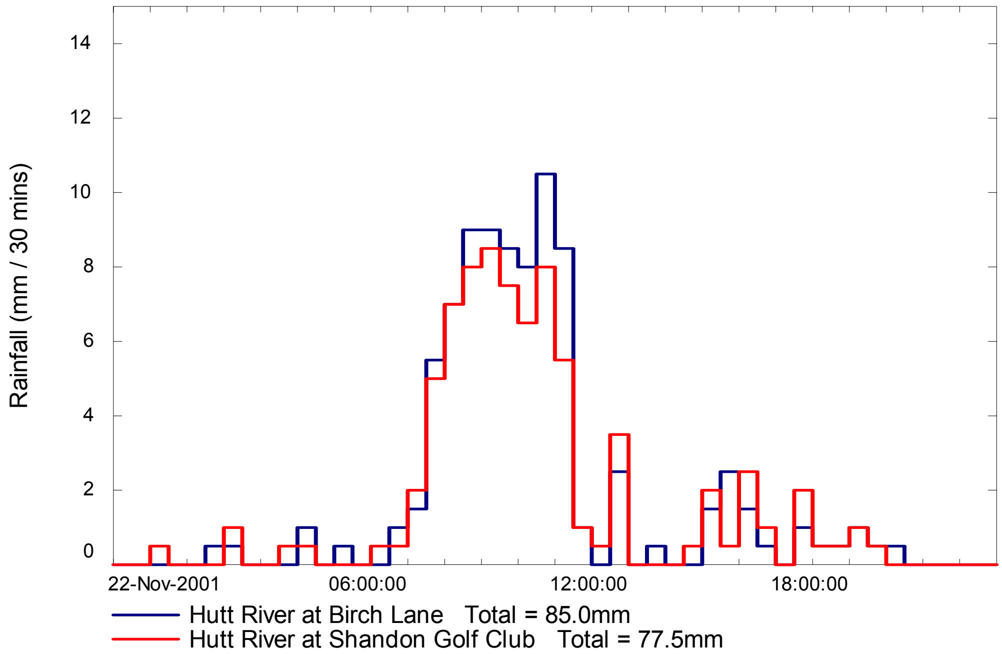
Although it was determined that the Avalon and Mabey Road rainfall stations can be used to represent rainfall in the Waiwhetu catchment, accurate runoff modelling needs to take into account any significant spatial variation of rainfall within the catchment. However, there are no detailed annual rainfall isohyets available for the Waiwhetu catchment, probably due to the disjointed nature of the rainfall records.

The Waiwhetu catchment has a significant area of low hill country, rising to elevations of approximately 300 metres. It is probable that orographic enhancement of rainfall occurs, and this should be accounted for in the rainfall runoff model. However, there are no rainfall records for the Eastern Hutt hills. The mean annual rainfall at Maungaraki (Western Hutt hills) is about 1420mm at an elevation of about 205 metres; whereas on the valley floor of the Waiwhetu catchment the mean annual rainfall is 1180 mm[4] (Wellington Regional Council, 1995). Thus it is possible that the rainfall along the Eastern Hutt hills is also about 120% of that which occurs on the valley floor. This is equal to the orographic enhancement factor estimated for the Waiwhetu catchment by Lew (1996).

Rainfall may also vary spatially along the valley floor of the Waiwhetu catchment. However, there is a lack of rain gauge coverage to accurately determine this. Comparison of rainfall totals for several storm events between 2000 and 2004 shows Shandon generally receives between 50 and 90% of the rainfall recorded further north in Lower Hutt (as in Figure 5). On average Shandon receives about 70% of the rainfall at Birch Lane or Avalon. For modelling purposes it is assumed that the southern end of the Waiwhetu catchment receives about 70% of the rainfall in the northern part of the catchment. A summary of the spatial variation in rainfall, as indicated by annual rainfall isohyets, is shown in Figure 6.

[4] Calculated using the daily rainfall records from 1903 – 1959

**Figure 5: Comparison of rainfall recorded at Birch Lane and Shandon**



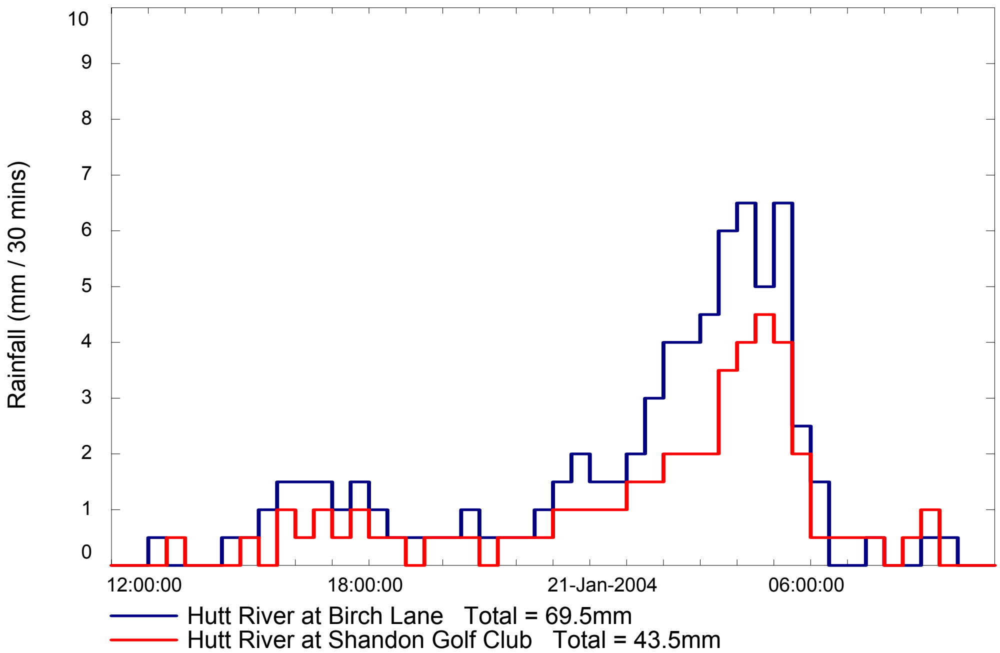
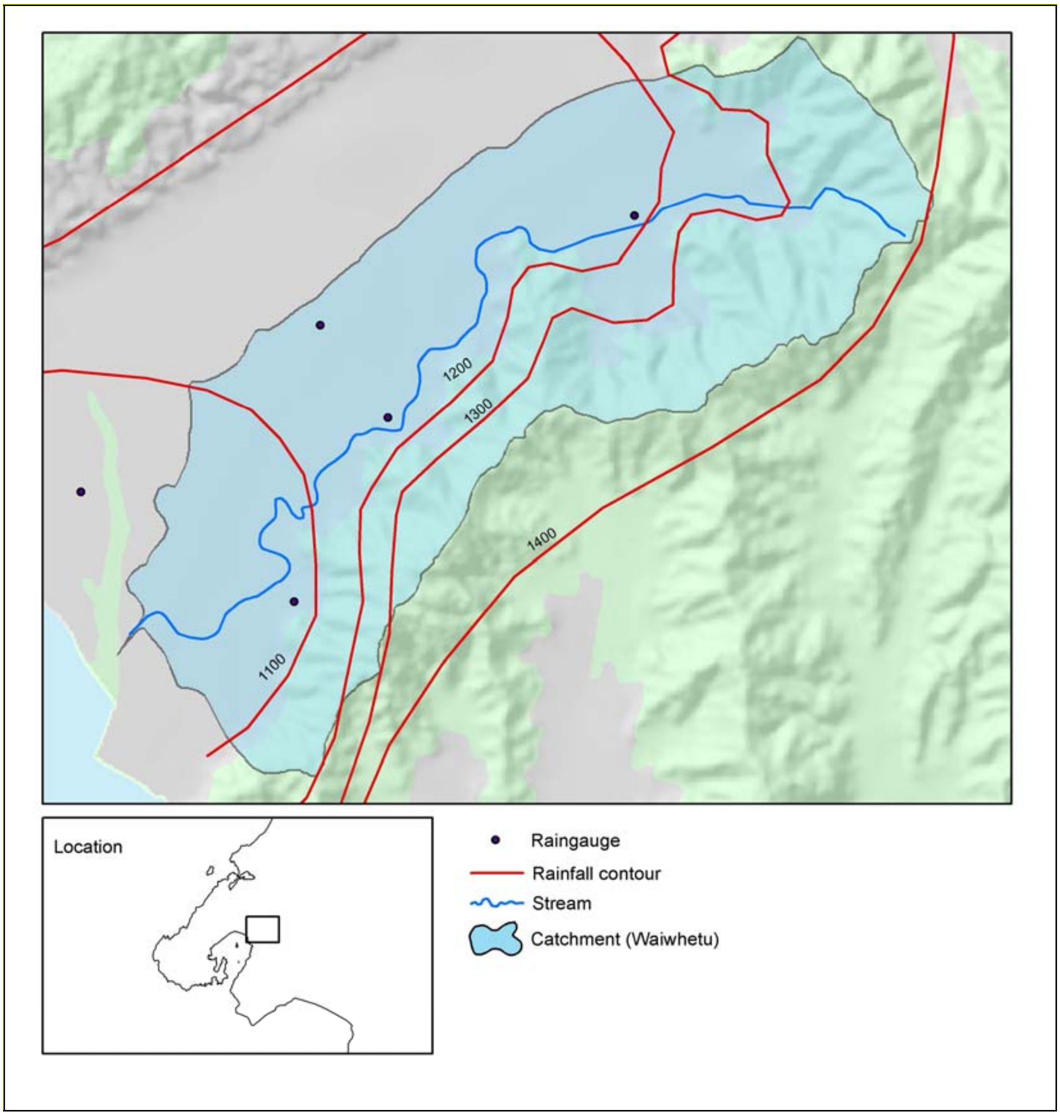



**Figure 6: Assumed annual rainfall contours (mm) for the Waiwhetu catchment**



### 4.1.3 Conclusion

From the above rainfall analysis it was determined that:

- For rainfall runoff modelling the Birch Lane rainfall data will be used to represent rainfall in the northern part of the Waiwhetu catchment. For events where Birch Lane is not present then Avalon or Mabey Road rainfall data can be used as a direct substitute, and for the November 1994 event Wallaceville data will be used with an adjustment factor of 0.9.

- For the area of Eastern Hills within the Waiwhetu catchment, an orographic enhancement factor of 1.2 will be applied to the rainfall data from the valley floor.

- The Shandon rainfall station data should be used to represent rainfall in the southern subcatchments of the Waiwhetu catchment. For events which occurred prior to the Shandon gauge being installed an adjustment factor of 0.7 should be applied to the rainfall recorded at Avalon / Mabey Road.

The poor spatial coverage of rainfall information for the Waiwhetu catchment means that it has to be assumed that no other significant spatial variation in rainfall occurs.

## 4.2 Rainfall frequency analysis

Depth-duration frequency analysis of rainfall data for the Waiwhetu catchment is necessary to determine design storms for modelling purposes. Such an analysis is constrained by the lack of long rainfall records for the Waiwhetu catchment. The previous Waiwhetu flood hydrology study used depth-duration frequency estimates from the Avalon rainfall site (Table 5), which was probably determined using an EV1 distribution.

**Table 5: Rainfall depth-duration frequency data for Avalon (1948 – 1989), from Lew (1996)**

| Return period | Rainfall depths (mm) of duration: | | | | | |
|---|---|---|---|---|---|---|
| | 1 hour | 2 hours | 6 hours | 12 hours | 24 hours | 48 hours |
| 2 years | 18 | 26 | 44 | 60 | 81 | 101 |
| 5 years | 24 | 35 | 65 | 92 | 116 | 142 |
| 10 years | 27 | 41 | 79 | 112 | 139 | 169 |
| 20 years | 31 | 47 | 93 | 132 | 162 | 194 |
| 50 years | 35 | 54 | 110 | 157 | 191 | 228 |
| 100 years | 40 | 59 | 121 | 174 | 213 | 253 |
| 200 years | 44 | 64 | 125 | 193 | 235 | 278 |

The options for this study were to:

1. Re-use the Avalon depth duration frequency estimates in Table 5;

2. Use depth-duration frequency estimates from the Mabey Road rainfall data (1995 – 2003); or

3. Use depth-duration frequency data for the Waiwhetu catchment derived using HIRDS (NIWA, 2002).

Option 3 was ruled out, as HIRDS does not need to be used if there is a long record of nearby rainfall data. In addition, initial investigation found HIRDS returned significantly lower rainfall depths (by up to 40%) than those derived using the Avalon rainfall data.

A depth-duration frequency analysis of the Mabey Road rainfall record was performed, using a GEV distribution and L-moments method of fitting. The resulting rainfall depths are shown in Table 6. The rainfall depths are generally higher than those in Table 5, except for the 48-hour storm duration.

**Table 6: Rainfall depth-duration frequency at Mabey Road (1995 – 2003)**

| Return period | Rainfall depths (mm) of duration: | | | | | |
|---|---|---|---|---|---|---|
| | 1 hour | 2 hours | 6 hours | 12 hours | 24 hours | 48 hours |
| 2 years | 22 | 30 | 50 | 63 | 72 | 89 |
| 5 years | 28 | 39 | 67 | 86 | 105 | 121 |
| 10 years | 32 | 46 | 81 | 106 | 130 | 144 |
| 20 years | 36 | 54 | 98 | 129 | 159 | 167 |
| 50 years | 42 | 66 | 123 | 166 | 204 | 197 |
| 100 years | 46 | 78 | 147 | 199 | 244 | 221 |

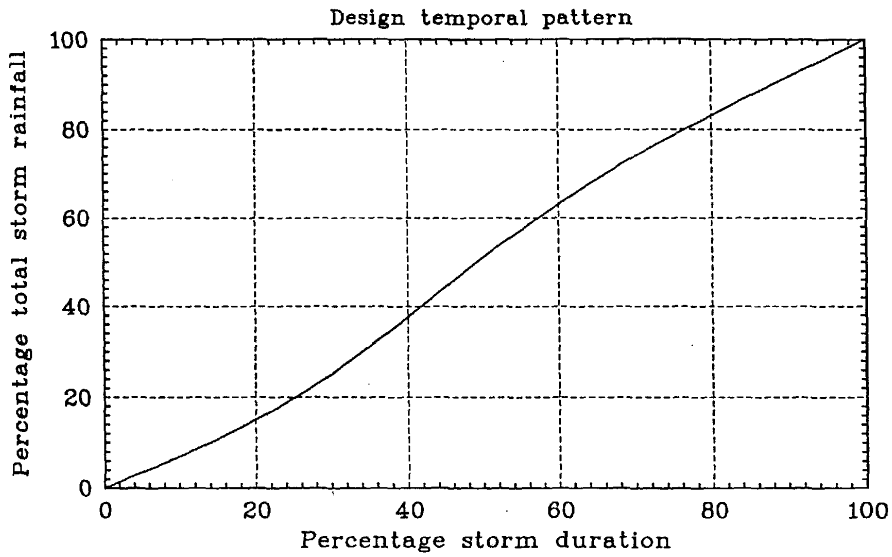
Due to the short length of record on which the Mabey Road depth-duration frequency analysis is based (8 years), it was decided that the Avalon analysis should be used for this study. Although the Avalon data is not as recent, the 40-year record length is likely to yield a more accurate depth-duration frequency analysis than the 8-year record at Mabey Road.

## 4.3 Design rainfall

Design rainfalls can be distributed in time according to the pattern of average variability, as proposed by Pilgrim & Cordery (1975). The assumption implicit in this method is that design rainfalls induce floods of the same return period. Using the temporal distribution graph in Figure 7, design rainfall events were determined from the design depths (Table 5). Design storms for durations of 3, 4, 8 and 10 hours were also derived, by interpolating the rainfall depths from Table 5 using intensity duration frequency curves. The resulting design storms for the Waiwhetu catchment are contained in Appendix 1.

To account for spatial variation within the Waiwhetu catchment, the design rainfalls were multiplied by a factor of 0.7 to derive rainfall for the subcatchments near the Waiwhetu Stream mouth (following the findings of Section 4.1). The orographic enhancement factor was not applied to the design rainfall, as this factor is incorporated into the rainfall runoff model (Chapter 5).

**Figure 7: Temporal pattern of rainfall within a design storm (Source: Pilgrim & Cordery, 1991)**




## 4.4 Probable maximum precipitation

Probable maximum precipitation (PMP) is theoretically the greatest depth of rainfall for a given duration that is meteorologically possible over a given duration at a particular time of year (World Meteorological Organization, 1986). The return period of the PMP is considered to be about 10,000 years. Thompson & Tomlinson (1993) determined a method for estimating the PMP for small areas in New Zealand, and there has been no nationally accepted alternative methodology since that time.

The PMP for the Waiwhetu catchment was derived by Lew (1996) (Table 7). The method uses catchment area, a moisture reduction factor, and depth-duration adjustment to determine the PMP. The depth-duration adjustment used by Lew (1996) was based on a 6:1 hour rainfall depth ratio of 3.62. To check if this adjustment factor is still appropriate for the Waiwhetu catchment, 6:1 hour rainfall depth ratios were derived using a range of intense short storms recorded at Birch Lane. The four most intense short events returned an average ratio of 3.5, with the most intense event having a ratio of 3.69. Therefore it was decided that the values derived by Lew (1996) using a ratio of 3.62 were appropriate to re-use for the Waiwhetu catchment.

**Table 7: Probable maximum precipitation values for the Waiwhetu catchment (Lew, 1996)**

| Duration (hours) | Rainfall (mm) |
|---|---|
| 0.5 | 99 |
| 1 | 162 |
| 2 | 266 |
| 3 | 354 |
| 4 | 466 |
| 5 | 512 |
| 6 | 586 |

The PMP depths were converted to rainfall events using the temporal distribution method outlined in Section 4.3. Note that the PMP values represent average rainfall over the catchment; i.e. taking into account areas of higher elevation of the Eastern Hutt hills. Thus the orographic and elevation adjustment factors derived in Section 4.1 will not be applied to these PMP depths.

# 5. Rainfall runoff modelling

To determine design hydrographs for the Waiwhetu Stream a rainfall runoff model was developed for the catchment. The process involved:

- Building a network model to represent the Waiwhetu catchment;
- Calibrating the hydrologic parameters of the model using observed flood events;
- Validating (testing) the model using observed flood events; and
- Modelling design rainfall to produce design flood hydrographs.

## 5.1 Model description

Greater Wellington Regional Council uses the TimeStudio Modelling package developed by the Hydro-Electric Corporation (2000). TimeStudio is a storage routing model for estimating the flood hydrograph. TimeStudio Modelling was previously known as Hydrol[5], and version 3.6.2.0 was used for this work.

A TimeStudio model is made up to two basic elements, nodes and links, which are connected together to form a network. Nodes represent subcatchment inflow points, stream confluences and other locations of interest in the catchment, and links represent the channel network. Each node and link has an operating rule that defines how it operates at each time step. Operating rules for nodes and links are defined using TimeStudio Basic script language.

Storage routing models consist of two separate steps. The first step is a loss function, which estimates how much rainfall becomes runoff. TimeStudio provides the option of three types of loss functions: the Australian Water Balance Model (AWBM), Initial-Continuing Loss Model and Proportional Runoff Model. The second step is a non-linear flow routing procedure for moving the runoff through the catchment and predicting the shape of the hydrograph at one or more points in the catchment.

## 5.2 Model configuration

The Waiwhetu catchment was divided into nineteen subcatchments for modelling purposes, ten of which are located upstream of Whites Line East (Figure 8). Subcatchment delineation was based on contour information. The model layout was similar to that used in the previous model for the catchment (Lew, 1996). However, in the previous model, all stormwater from the area between the Waiwhetu Stream at Whites Line East and the railway line was routed east to the Waiwhetu Stream. After checking the digital terrain model and stormwater maps for Lower Hutt, it was decided that some of this stormwater (from subcatchments P and Q) actually flows south-west to the Awamutu Stream.

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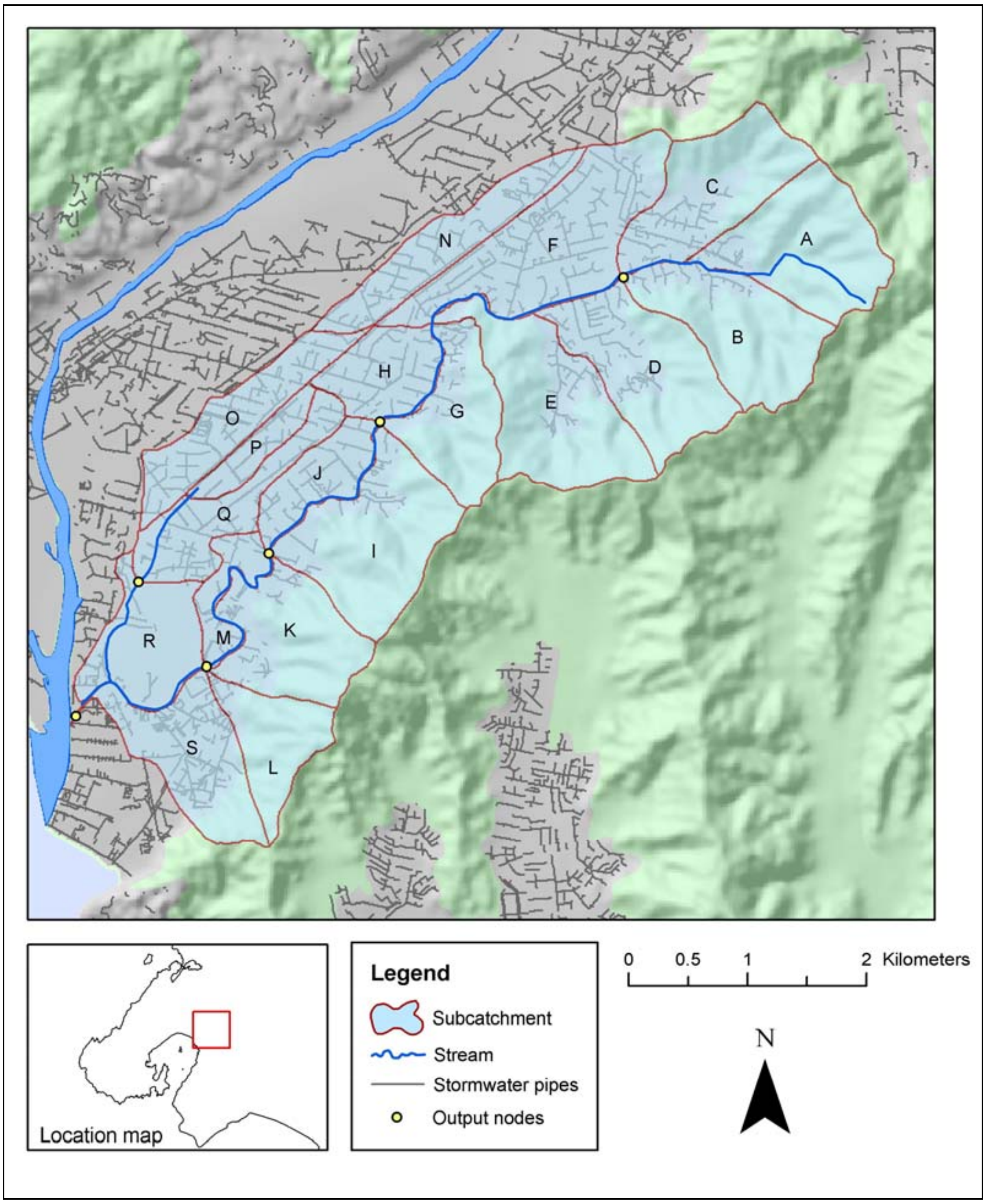
[5] The most recent version (2003) is known as HYDSTRA modelling

Hutt City Council stormwater maps indicate that pipes cross the railway line, taking stormwater from subcatchments O and N eastward. However, as previously mentioned, the capacity of the stormwater system is limited and probably only able to cope with low magnitude rainfall events (Minson, 1996). The digital elevation model for the area shows that, if the stormwater system is inundated, water will flow southwards following the railway line. The stormwater will then enter the top of the Awamutu system through a large pipe under the railway line near Woburn Station (at the south end of subcatchment P). Thus in the rainfall runoff model it was decided to route the surface runoff from N and O through subcatchments Q and R to the Waiwhetu Stream.

**Figure 8: Waiwhetu catchment subcatchment delineation for rainfall runoff modelling**

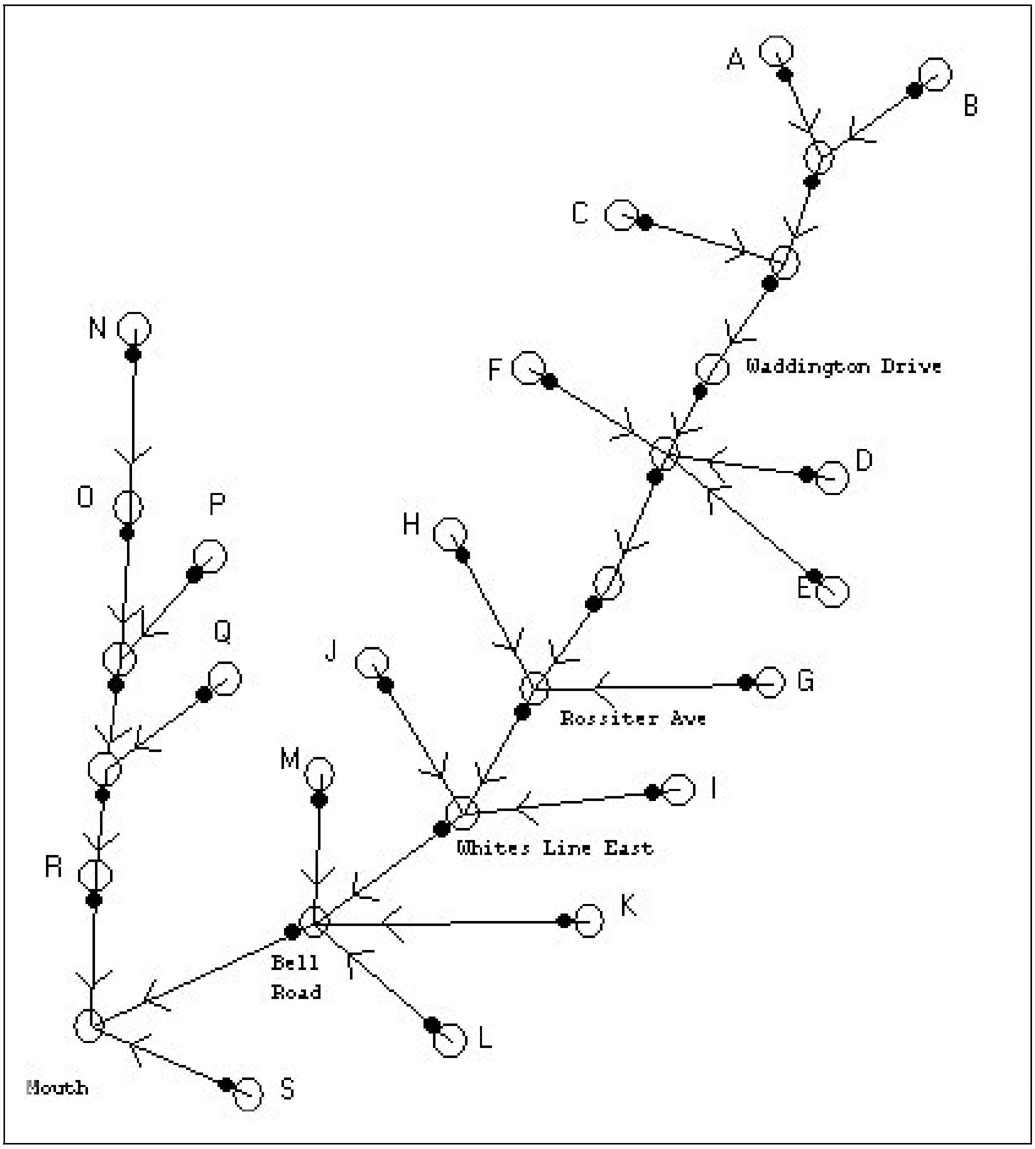


The Waiwhetu catchment model configuration of nodes (representing subcatchments and stream confluences) and channel links is shown in Figure 9. Model output nodes (which export hydrographs at that point) were placed at six points in the model:

- Waiwhetu Stream at Waddington Drive;
- Waiwhetu Stream at Rossiter Ave;
- Waiwhetu Stream at Whites Line East;
- Waiwhetu Stream at Bell Road;
- Waiwhetu Stream at Mouth; and
- Awamutu Stream at Railway Line.

The model was set to 15-minute time steps, which is equal to the data recording interval at Whites Line East.

**Figure 9: Schematic diagram of TimeStudio model for the Waiwhetu catchment**




## 5.3 Storm events for rainfall runoff modelling

The rainfall runoff model for the Waiwhetu catchment was calibrated and validated using the flow data from Whites Line East. The storm events used for calibration and validation were selected from the annual maximum series. To ensure adequate model calibration and testing, it was decided that five or six events were required for model calibration and five or six events for model validation.

To select the events, all annual maxima less than 12 $m^3/s$ were excluded, as the aim was to ensure that the model performed well for extreme storm events. Next, all flood events for which there was no automatic rainfall data available were excluded (this included some events from the 1980s for which Avalon rainfall data has not been digitised), leaving nine annual maxima storm events. Two non-annual maxima events were also selected (October 2003 and December 2001). The resulting storm events for model calibration and validation are listed in Table 8. Note that the largest event on record (February 2004) was used for model validation rather than calibration, because of the uncertainty surrounding the peak flow estimate for that event.

**Table 8: Storm events for Waiwhetu catchment rainfall runoff modelling**

| Event date | Flood peak ($m^3/s$) | Calibration / Validation |
|---|---|---|
| 16 February 2004 | 36.2 | Validation |
| 22 November 2001 | 23.0 | Calibration |
| 4 October 1997 | 19.7 | Validation |
| 13 March 1990 | 16.1 | Calibration |
| 21 May 1981 | 15.6 | Validation |
| 11 December 1982 | 14.9 | Validation |
| 9 June 2003 | 14.8 | Calibration |
| 3 October 2003 | 14.3 | Calibration |
| 2 October 2000 | 13.5 | Calibration |
| 8 November 1994 | 13.1 | Validation |
| 27 December 2001 | 12.4 | Calibration |

## 5.4 Model calibration

### 5.4.1 Model parameters and calibration procedure

The Waiwhetu catchment TimeStudio model was calibrated using an AWBM loss function and non-linear channel routing. The model parameters required are listed in Table 9. Initial values of several other floating parameters are also required (such as groundwater storage contents and baseflow). However, as the model was run for a month prior to each storm event with actual rainfall data (to ensure accurate antecedent catchment conditions) the initial values chosen did not affect the calibration or validation results.

**Table 9: Fixed-value parameters required for a TimeStudio model of the Waiwhetu catchment**

| Parameter | Description |
|---|---|
| Cap1 | Capacity of storage 1 in AWBM |
| Cap2 | Capacity of storage 2 in AWBM |
| Cap3 | Capacity of storage 3 in AWBM |
| A1 | Areal proportion of storage 1 in AWBM |
| A2 | Areal proportion of storage 2 in AWBM |
| A3 | Areal proportion of storage 3 in AWBM |
| K | Daily recession constant in AWBM |
| Inf | Infiltration parameter in AWBM |
| $\alpha$ | Channel lag parameter in non-linear channel routing |
| n | Non-linearity parameter in non-linear channel routing |
| Area | Area of each subcatchment |
| L | Channel length (specified in each link) |

Values of all parameters (with the exception of channel length and subcatchment area) were assumed to be constant over the Waiwhetu catchment. The area of each subcatchment (*Area*) and the channel lengths (*L*) were calculated using ArcGIS.

Initially values of the storage capacities (*Cap1*, *Cap2* and *Cap3*) and areal extent of the storage capacities (*A1*, *A2* and *A3*) were set equal to those in the TimeStudio model for the Hutt River. However, preliminary results found that the modelled hydrograph shapes did not mimic the observed hydrographs well. The storage capacities and their areal proportions were then adjusted to account for the large extent of urban area in the catchment, with some storage provided by the Eastern Hutt hills and permeable parts of the catchment (*Cap1* = 2mm, *Cap2* = 10mm, *Cap3* = 50mm, *A1* = 0.5, *A2* = 0.3, *A3* = 0.2).

The calibration process focussed on the parameters *K*, *Inf*, $\alpha$ and *n*. The model was calibrated by comparing modelled flow with the recorded flow at Whites Line East. The calibration process consisted of two steps: firstly, determining the model parameters which gave the best-fit of modelled versus recorded flow for each storm event, and secondly, determining the set of model parameters which gave the best fit for all six calibration events. The first step was carried out using the parameter estimation software PEST2000 (Doherty, 2000). The second step in the calibration process was carried out by manually varying parameter values within the likely range. The calibration was determined by visual assessment of the similarity of hydrograph shape, calculation of error in peak flow, and calculation of mean absolute error (which is the average of the difference in modelled flow and recorded flow for each time step in the model).

Trialling of the model during the calibration process indicated a timing error, with the modelled peak flow occurring prior to the recorded peak at Whites Line East. Hence a delay of one hour was added to the model upstream of the Whites Line East node prior to model calibration.

### 5.4.2 Calibration results

The combination of parameters that provided the best fit for each event, as determined using PEST2000, are shown in Table 10. The parameter values varied significantly between the events, particularly the values of *K* (which affects hydrograph recession) and $\alpha$ (which affects the steepness of the hydrograph).

**Table 10: Best-fit parameters for each calibration event, as determined using PEST2000**

| | Event date | | | | | |
|---|---|---|---|---|---|---|
| | **November 2001** | **March 1990** | **June 2003** | **October 2003** | **October 2000** | **December 2001** |
| K | 0.86 | 0.71 | 0.85 | 0.23 | 0.099 | 0.60 |
| Inf | 0.50 | 0.54 | 0.77 | 0.60 | 0.72 | 0.65 |
| N | 0.73 | 0.70 | 1.15 | 0.62 | 1.11 | 0.82 |
| $\alpha$ | 1.28 | 0.92 | 0.39 | 0.95 | 0.20 | 0.44 |

To obtain the single set of parameter values that provided the best results for all calibration events the parameters were varied within the ranges of values shown in Table 10. The final set of values that best modelled the calibration events are shown in Table 11, along with average error statistics. Note that during this process it was found that runoff during the June 2003 event was vastly over predicted by the TimeStudio model. Further investigation suggested that this may be due to an unusual degree of spatial variation in rainfall during that event, meaning rainfall was overestimated for the catchment. Thus the June 2003 storm event was excluded from the calibration process, and the results in Table 11 are based on the five remaining calibration events.

**Table 11: Calibration results for Waiwhetu catchment TimeStudio model (using Waiwhetu Stream at Whites Line East)**

| Parameter | Best fit value | Average error in modelled peak flow (%) | Average mean absolute error ($m^3/s$)[6] |
|---|---|---|---|
| K | 0.70 | 13.2 | 1.11 |
| Inf | 0.51 | | |
| n | 0.75 | | |
| $\alpha$ | 0.90 | | |

The recorded and modelled hydrographs, and error statistics for each calibration event, are contained in Appendix 2.

## 5.5 Model validation

Five storm events were used for model validation. The events were modelled with the parameter values shown in Table 11, and the model was run for a month prior to the start of each event to ensure accurate antecedent conditions. The modelled and recorded hydrographs for the validation events are shown in Figures 10 to 14, and the model performance statistics are shown in Table 12.

[6] Mean absolute error is an indication of the difference between modelled and observed flow over the entire storm event (i.e. the difference is calculated for each time step and averaged for the entire hydrograph).

**Figure 10: Waiwhetu catchment model validation results 1 (February 2004 event)**

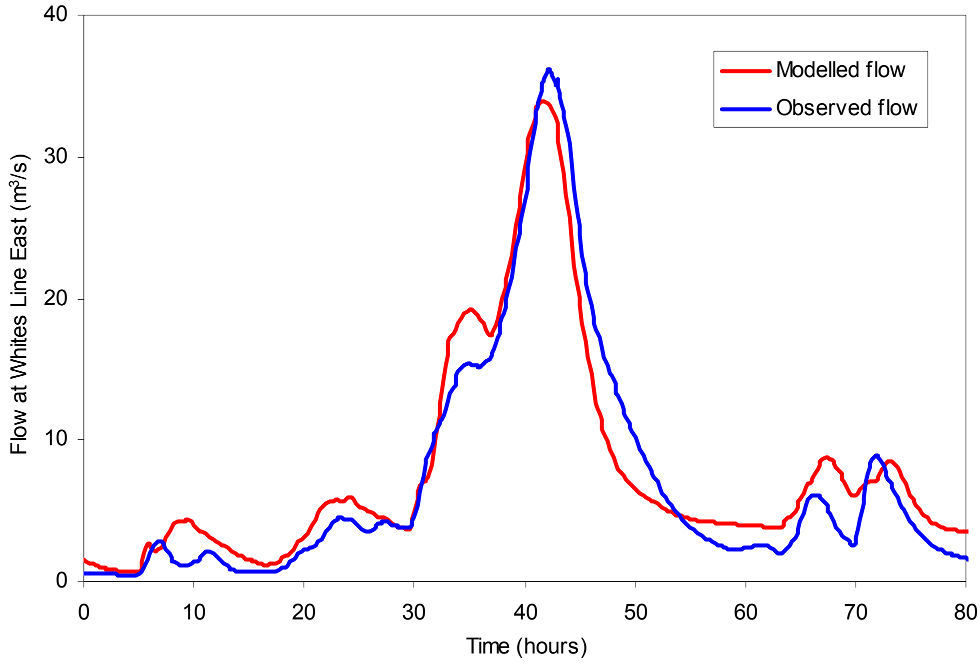



**Figure 11: Waiwhetu catchment model validation results 2 (October 1997 event)**

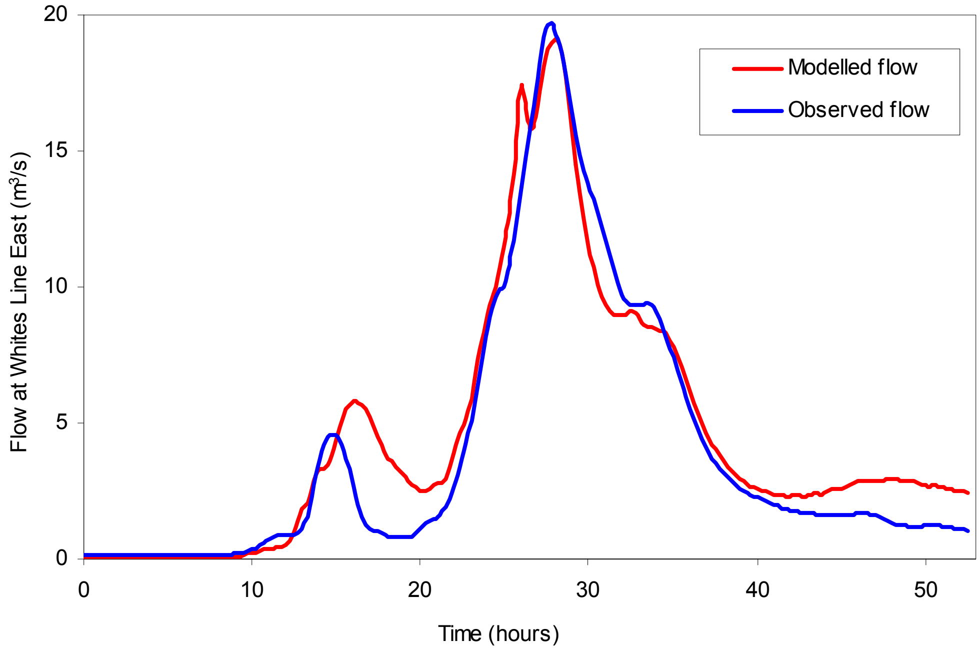


**Figure 12: Waiwhetu catchment model validation results 3 (May 1981 event)**

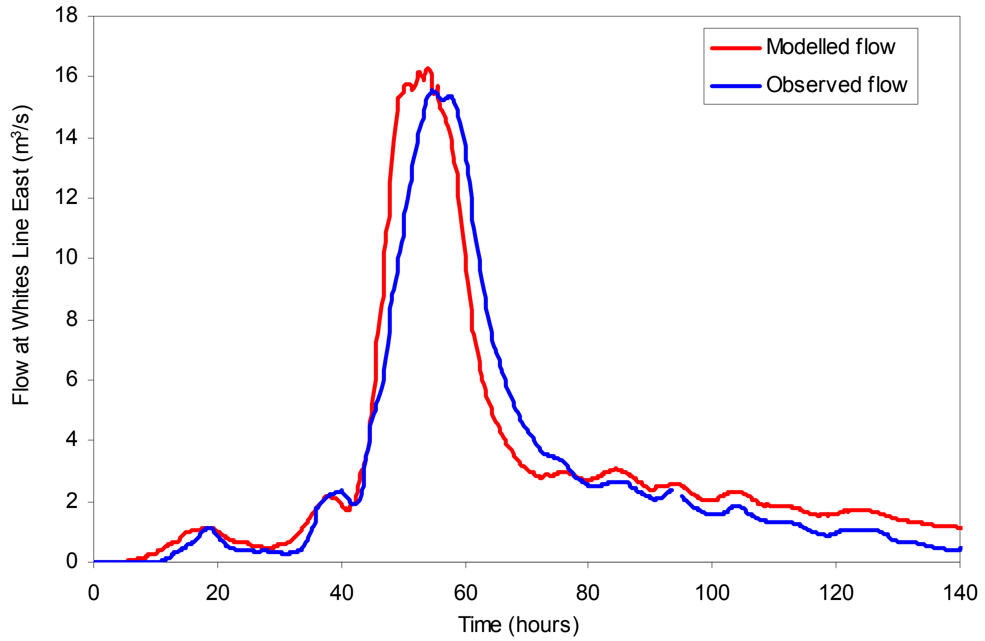



**Figure 13: Waiwhetu catchment model validation results 4 (December 1982 event)**

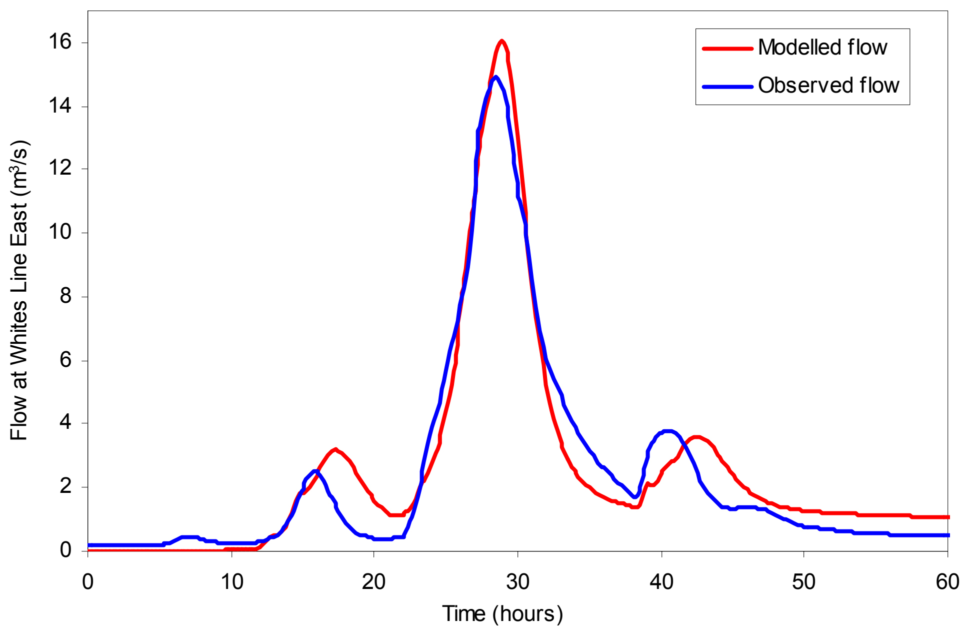


**Figure 14: Waiwhetu catchment model validation results 5 (November 1994 event)**

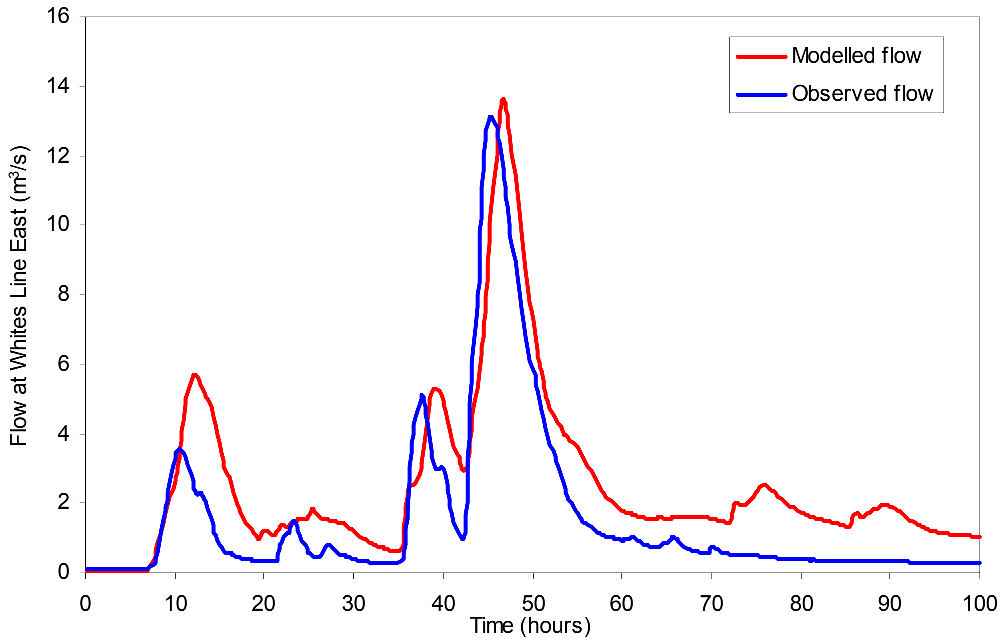



**Table 12: Model validation results for Waiwhetu Stream at Whites Line East**

| | Event date | | | | |
|---|---|---|---|---|---|
| | **February 2004** | **October 1997** | **May 1981** | **December 1982** | **November 1994** |
| Error in peak flow (%) | -6.1 | -3.0 | 4.5 | 7.4 | 3.8 |
| Mean absolute error ($m^3/s$) | 1.8 | 0.9 | 0.8 | 0.6 | 1.6 |
| Error in timing of peak flow | + 15 mins | - 15 mins | + 45 mins | -45 mins | - 1 hour 15 mins |

The TimeStudio model produced good results for the validation events at Whites Line East. For all five events the error in the modelled peak flow was less than 10%, with the average (absolute) error in the peak flow being 5.0%, and the standard error of the modelled peak flow estimate being 1.55 $m^3/s$.

In all cases the modelled hydrographs were of a similar shape to the recorded hydrographs, as indicated by the low mean absolute error. The timing errors were not consistent between events, but the timing error is considered minor in relation to event length. The successful model validation means that the rainfall runoff model can be used for modelling runoff from the design rainfall events.

## 5.6 Design event modelling

The design rainfall events in Appendix 1 were run through the calibrated TimeStudio model to obtain the design hydrographs. 'Average' initial conditions were assumed – i.e. a moderate groundwater saturation (10%) which relates to a realistic starting baseflow at Whites Line East of about 0.5 $m^3/s$. Tables 13 to 19 show the modelled peak flows at each output node for each storm duration[7]. The critical duration (the duration that gives the maximum peak) for each location is shaded. Critical duration flood hydrographs are contained in Appendix 5.

**Table 13: Modelled Q2 peak flows ($m^3/s$)**

| | Duration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 hour | 2 hour | 4 hour | 6 hour | 8 hour | 10 hour | 12 hour | 24 hour |
| Waddington Drive | 3.8 | 5.0 | 4.9 | 4.1 | 3.7 | 3.5 | 3.3 | 2.3 |
| Rossiter Ave | 4.5 | 7.3 | 9.6 | 9.3 | 8.8 | 8.4 | 7.9 | 5.7 |
| Whites Line East | 4.0 | 6.8 | 9.3 | 10.1 | 10.0 | 9.7 | 9.2 | 6.7 |
| Bell Road | 3.7 | 6.3 | 9.0 | 11.0 | 11.3 | 10.9 | 10.4 | 7.8 |
| Waiwhetu Mouth | 4.1 | 4.0 | 10.5 | 14.3 | 13.9 | 13.6 | 13.1 | 10.0 |
| Railway (Awamutu) | 1.6 | 2.3 | 2.5 | 2.3 | 2.2 | 2.1 | 2.1 | 1.5 |

**Table 14: Modelled Q5 peak flows ($m^3/s$)**

| | Duration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 hour | 2 hour | 4 hour | 6 hour | 8 hour | 10 hour | 12 hour | 24 hour |
| Waddington Drive | 6.2 | 7.3 | 6.9 | 6.0 | 5.5 | 5.7 | 5.4 | 3.6 |
| Rossiter Ave | 7.2 | 11.4 | 15.0 | 14.5 | 14.0 | 14.1 | 13.3 | 9.1 |
| Whites Line East | 6.3 | 10.2 | 15.0 | 16.5 | 16.7 | 16.4 | 15.5 | 10.9 |
| Bell Road | 5.7 | 9.5 | 14.5 | 18.1 | 19.2 | 18.8 | 18.0 | 12.7 |
| Waiwhetu Mouth | 6.3 | 10.6 | 17.1 | 22.5 | 23.9 | 23.8 | 22.7 | 16.3 |
| Railway (Awamutu) | 2.5 | 3.4 | 3.8 | 3.6 | 3.6 | 3.7 | 3.4 | 2.4 |

**Table 15: Modelled Q10 peak flows ($m^3/s$)**

| | Duration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 hour | 2 hour | 4 hour | 6 hour | 8 hour | 10 hour | 12 hour | 24 hour |
| Waddington Drive | 7.3 | 8.7 | 8.4 | 7.5 | 7.3 | 7.3 | 7.3 | 4.6 |
| Rossiter Ave | 8.6 | 14.1 | 19.1 | 18.8 | 18.4 | 18.2 | 17.3 | 11.4 |
| Whites Line East | 7.5 | 12.8 | 19.6 | 21.6 | 21.5 | 21.1 | 20.2 | 13.6 |
| Bell Road | 6.8 | 11.8 | 19.2 | 23.6 | 24.9 | 23.9 | 23.0 | 15.9 |
| Waiwhetu Mouth | 7.4 | 13.1 | 22.9 | 29.6 | 31.3 | 30.5 | 29.2 | 20.2 |
| Railway (Awamutu) | 3.0 | 4.2 | 4.8 | 4.7 | 4.7 | 4.8 | 4.5 | 3.0 |

[7] Durations of 3 and 48 hours are not shown in the tables, but these durations were not critical for any of the locations or return periods

**Table 16: Modelled Q20 peak flows ($m^3/s$)**

| | Duration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 hour | 2 hour | 4 hour | 6 hour | 8 hour | 10 hour | 12 hour | 24 hour |
| Waddington Drive | 8.9 | 10.0 | 10.0 | 9.2 | 9.1 | 9.3 | 8.9 | 5.6 |
| Rossiter Ave | 10.6 | 16.9 | 23.6 | 23.2 | 22.4 | 22.5 | 21.0 | 13.7 |
| Whites Line East | 9.2 | 15.3 | 24.1 | 27.0 | 26.2 | 26.2 | 24.7 | 16.2 |
| Bell Road | 8.3 | 14.2 | 23.8 | 29.0 | 30.1 | 29.4 | 28.1 | 18.9 |
| Waiwhetu Mouth | 9.0 | 15.6 | 28.6 | 36.7 | 38.0 | 37.3 | 35.4 | 24.1 |
| Railway (Awamutu) | 3.6 | 4.9 | 6.0 | 5.8 | 5.7 | 5.8 | 5.6 | 3.6 |

**Table 17: Modelled Q50 peak flows ($m^3/s$)**

| | Duration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 hour | 2 hour | 4 hour | 6 hour | 8 hour | 10 hour | 12 hour | 24 hour |
| Waddington Drive | 10.4 | 11.6 | 12.2 | 11.5 | 11.3 | 11.3 | 10.8 | 6.6 |
| Rossiter Ave | 12.5 | 20.3 | 28.7 | 28.3 | 27.3 | 27.4 | 26.1 | 16.4 |
| Whites Line East | 11.0 | 18.4 | 29.3 | 33.3 | 32.1 | 32.2 | 30.2 | 19.5 |
| Bell Road | 9.9 | 17.1 | 29.4 | 36.5 | 36.4 | 36.5 | 34.6 | 22.6 |
| Waiwhetu Mouth | 10.6 | 18.8 | 35.2 | 45.1 | 46.3 | 45.9 | 43.7 | 28.7 |
| Railway (Awamutu) | 4.2 | 5.8 | 7.2 | 7.2 | 7.1 | 7.1 | 6.9 | 4.3 |

**Table 18: Modelled Q100 peak flows ($m^3/s$)**

| | Duration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 hour | 2 hour | 4 hour | 6 hour | 8 hour | 10 hour | 12 hour | 24 hour |
| Waddington Drive | 12.2 | 12.9 | 14.3 | 13.0 | 12.9 | 12.6 | 11.9 | 7.4 |
| Rossiter Ave | 15.2 | 23.2 | 33.3 | 31.6 | 30.9 | 30.3 | 29.4 | 18.3 |
| Whites Line East | 13.2 | 21.1 | 34.1 | 37.4 | 36.7 | 35.9 | 33.7 | 21.9 |
| Bell Road | 11.9 | 19.6 | 34.5 | 41.4 | 41.6 | 40.9 | 38.8 | 25.3 |
| Waiwhetu Mouth | 12.7 | 21.7 | 41.3 | 50.4 | 52.6 | 51.6 | 49.2 | 32.3 |
| Railway (Awamutu) | 5.1 | 6.6 | 8.4 | 8.1 | 8.1 | 8.0 | 7.7 | 4.8 |

**Table 19: Modelled Q200 peak flows ($m^3/s$)**

| | Duration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 hour | 2 hour | 4 hour | 6 hour | 8 hour | 10 hour | 12 hour | 24 hour |
| Waddington Drive | 13.7 | 14.4 | 14.7 | 13.1 | 13.2 | 13.7 | 13.2 | 8.1 |
| Rossiter Ave | 17.5 | 26.2 | 34.2 | 32.2 | 32.7 | 33.0 | 32.6 | 20.2 |
| Whites Line East | 15.1 | 23.9 | 35.1 | 37.7 | 38.9 | 39.1 | 37.5 | 24.1 |
| Bell Road | 13.5 | 22.2 | 35.6 | 40.8 | 44.0 | 44.8 | 43.1 | 27.8 |
| Waiwhetu Mouth | 14.4 | 24.6 | 42.6 | 50.7 | 55.2 | 56.3 | 54.7 | 35.6 |
| Railway (Awamutu) | 5.8 | 7.4 | 8.6 | 8.1 | 8.4 | 8.6 | 8.5 | 5.3 |

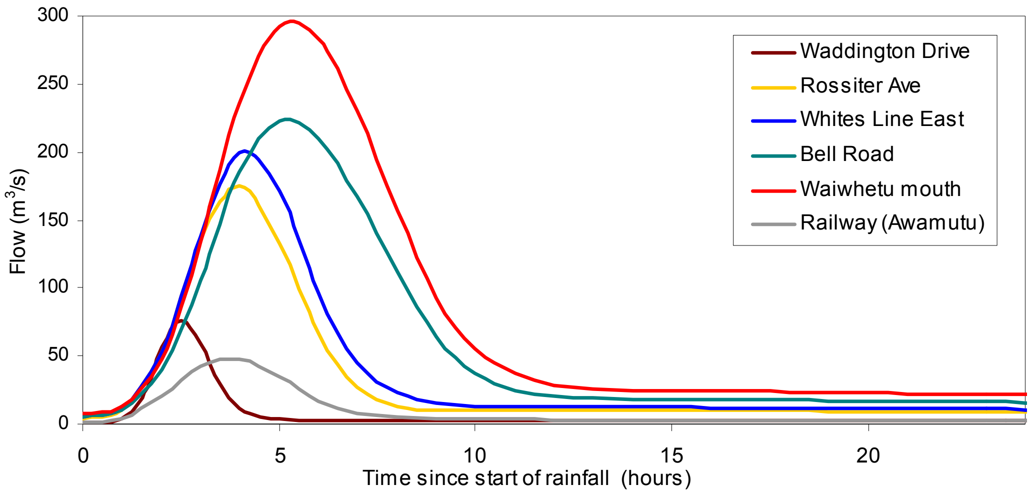
## 5.7 Probable maximum flood modelling

The PMP for events of 1 to 6 hour durations were run through the TimeStudio model to determine the probable maximum flood (PMF) peaks for the catchment. A worst-case scenario was assumed for the initial conditions – i.e. a high initial groundwater storage (100%) resulting in a high initial baseflow, so that the highest flows possible in the catchment would result. The PMF peaks are shown in Table 20, with the critical duration for each location highlighted. The PMF hydrographs for the critical durations are shown in Figure 15.

**Table 20: Probable maximum flood peaks ($m^3/s$) for the Waiwhetu catchment**

| | Duration | | | | |
|---|---|---|---|---|---|
| | 1 hour | 2 hour | 3 hour | 4 hour | 6 hour |
| Waddington Drive | 74 | 77 | 71 | 66 | 62 |
| Rossiter Ave | 116 | 163 | 173 | 175 | 159 |
| Whites Line East | 102 | 154 | 187 | 199 | 193 |
| Bell Road | 92 | 150 | 189 | 214 | 223 |
| Waiwhetu Mouth | 106 | 172 | 235 | 276 | 292 |
| Railway (Awamutu) | 40 | 47 | 48 | 48 | 45 |

**Figure 15: Probable maximum flood hydrographs for the Waiwhetu catchment**




## 5.8 Discussion of design flood modelling results

### 5.8.1 Critical durations

The critical duration for maximising flood peaks varies between the locations in the catchment, but tends to increase with distance downstream, as expected. The critical durations for the design events were found to be:

- 2 to 4 hours at Waddington Drive;
- 2 to 4 hours at Rossiter Ave;
- 6 to 10 hours at Whites Line East;
- 6 to 10 hours at Bell Road and the Waiwhetu Stream mouth; and
- 4 hours for the Awamutu Stream at Railway.

The time of concentration at the mouth of the Waiwhetu catchment, calculated using the Bransby Williams formula, is about 3.5 hours. The calculated time of concentration is much less than the critical durations determined by rainfall runoff modelling. This anomaly is due to storage within the catchment that is not accounted for by time of concentration formulae.

### 5.8.2 Maximised flood peaks

The maximum flood peaks from the rainfall runoff modelling are shown in Table 21. The validation of the TimeStudio model found that the mean error in the modelled peak flow was 5%, with a maximum error of 7.4%. Thus it would be conservative to assume that error introduced by the rainfall runoff model in deriving the peak flows could be up to 10%.

**Table 21: Maximum modelled flood peaks for the Waiwhetu catchment ($m^3/s$)**

| Location | Q2 | Q5 | Q10 | Q20 | Q50 | Q100 | Q200 | PMF |
|---|---|---|---|---|---|---|---|---|
| Waddington Drive | 5.0 | 7.3 | 8.7 | 10.0 | 12.2 | 14.3 | 14.7 | 77 |
| Rossiter Ave | 9.6 | 15.0 | 19.1 | 23.6 | 28.7 | 33.3 | 34.2 | 175 |
| Whites Line East | 10.1 | 16.7 | 21.6 | 27.0 | 33.3 | 37.4 | 39.1 | 199 |
| Bell Road | 11.3 | 19.2 | 24.9 | 30.1 | 36.5 | 41.6 | 44.8 | 223 |
| Waiwhetu Mouth | 14.3 | 23.9 | 31.3 | 38.0 | 46.3 | 52.6 | 56.3 | 292 |
| Railway (Awamutu) | 2.5 | 3.8 | 4.8 | 6.0 | 7.2 | 8.4 | 8.6 | 48 |

The flood peaks in Table 21 are considerably lower than those derived using the RORB model (Lew, 1996), even though the same design rainfall depths were used. Use of a different loss model should not result in a difference in flood peaks of up to 50% as observed. The RORB model was calibrated assuming no orographic enhancement of rainfall in the Waiwhetu catchment. However, when Lew (1996) modelled the design storms an orographic enhancement factor was applied to the design rainfall. Thus the conditions assumed during model calibration were not kept constant, and the runoff resulting from the design storms was probably over-predicted. The results in Table 21 should therefore be used in preference to the modelled flood peaks presented in Lew (1996).

# 6. Flood frequency analysis

## 6.1 At-site flood frequency analysis

### 6.1.1 Data and method

The annual maximum series for the Waiwhetu Stream at Whites Line East has several years of missing flood peaks, as discussed in Section 3. To determine the likely magnitude of the missing annual maxima a correlation between 6-hour rainfall at Avalon / Mabey Road and peak flow at Whites Line East was determined. Of the nine missing annual maxima, the most significant is likely to be the October 1998 event. Using the rainfall runoff model to estimate the peak flow for this event gave a result of 20 $m^3/s$. This event was probably one of the largest since 1976 and should not be omitted from the analysis, therefore the modelled peak flow was included in the annual maximum series for the site.

Historical flood peaks from 1950, 1976 and 1977 have been estimated (see section 3.2.4). There is considerable uncertainty over how the 1950 peak was determined, and whether or not there were any other significant peak flows between 1950 and 1976. Because the calculations could not be confirmed the 1950 event was not included in this at-site analysis. Similarly, because the calculations of the 1977 peak flow could not be checked, and the error associated with this estimate was uncertain, the 1977 event was also excluded.

The 1976 flood estimate (36 $m^3/s$) is likely to have considerable error (Lew, 1996). As a check of the estimate, the event was modelled using the calibrated TimeStudio model for the catchment; this derived a peak flow of 48 $m^3/s$. Similarly, Lew (1996) suggested that the Wellington Regional Water Board (1977) estimate of 36 $m^3/s$ might not be high enough. However, because it is certain that the event was either the largest or the second largest between 1976 and 2004 it was included in some scenarios of the at-site flood frequency analysis.

The 1970 – 1972 annual maxima are available from chart data. This period was not included in the annual maximum series due to the missing annual maxima from 1973 – 1976. However, it can be included when the data is analysed as a discontinuous series.

The following flood frequency analyses were carried out using the at-site data:

Scenario 1. Analysis of the annual maximum series for 1978 – 2004 using the L-moments method and EV1 and EV2 frequency distributions in Hilltop, both including and excluding the 1976 event.

Scenario 2. Analysis of the annual maximum flood data for 1970 – 2004, treating the data as a discontinuous series using the MAX software (maximum likelihood method of fitting with EV1 and EV2 frequency distributions).

Scenario 3. Partial duration series analysis of the at-site record (1978 – 2004) using the exponential and GPA distributions.

### 6.1.2 Scenario 1 – Annual maximum series analysis using Hilltop

Table 22 shows the results of the at-site analysis using the annual maximum series with and without the 1976 event (treating the data as a continuous series). The EV1 and EV2 distributions produce significantly different results, particularly for the high return periods. In both cases the EV2 distribution produced a better fit to the data than the EV1. The inclusion of the 1976 event (assigned a value of 36 $m^3/s$) significantly increases flood magnitudes as expected, particularly for return periods of 20 years and greater. Plots of the annual maxima and fitted distribution curves can be found in Appendix 3.

**Table 22: At-site flood frequency estimates ($m^3/s$) for Waiwhetu Stream at Whites Line East assuming a continuous annual maximum series**

| | 1978 - 2004 | | 1976 - 2004 | |
|---|---|---|---|---|
| | **EV1** | **EV2** | **EV1** | **EV2** |
| Q2 | 13.5 | 13.1 | 14.8 | 13.4 |
| Q5 | 19.2 | 17.9 | 21.2 | 19.3 |
| Q10 | 22.7 | 22.1 | 25.5 | 24.6 |
| Q20 | 26.1 | 27.2 | 29.6 | 31.0 |
| Q50 | 30.4 | 35.6 | 35.0 | 42.0 |
| Q100 | 33.7 | 43.5 | 39.0 | 52.8 |
| Q200 | 37.0 | 53.2 | 43.0 | 66.3 |

### 6.1.3 Scenario 2 – Discontinuous series analysis using MAX

The MAX software (Stedinger *et al.*, 1988) accounts for a discontinuous data series, and uses the maximum likelihood method of fitting. The program also allows error margins or likely ranges to be placed on annual maxima that have uncertain magnitudes. The following conditions were specified in the MAX analysis:

- There are eleven years of missing record between 1970 and 2004;
- The 1976 annual maximum is within the range 36 – 50 $m^3/s$; and
- The 1998 annual maximum is within the range 18 – 22 $m^3/s$.

Table 23 shows the results of this method when EV1 and EV2 distributions are used. The EV1 distribution produces lower flood magnitudes, and the EV2 distribution provides a better fit to the data despite the higher standard errors. A plot of the EV2 distribution fitted to the data using the maximum likelihood method is contained in Appendix 3.

**Table 23: At-site flood frequency estimates (m³/s) for Waiwhetu Stream at Whites Line East assuming a discontinuous annual maximum series**

| | EV1 distribution | | EV2 distribution | |
|---|---|---|---|---|
| | Estimate | Standard error | Estimate | Standard error |
| Q2 | 13.0 | 0.9 | 12.4 | 0.9 |
| Q5 | 17.8 | 1.4 | 17.4 | 1.6 |
| Q10 | 20.9 | 1.8 | 21.5 | 2.2 |
| Q20 | 24.0 | 2.2 | 26.1 | 2.9 |
| Q50 | 27.9 | 2.8 | 33.1 | 4.1 |
| Q100 | 30.8 | 3.2 | 39.3 | 5.1 |
| Q200 | 33.7 | 3.6 | 46.5 | 6.3 |

### 6.1.4 Scenario 3 – Partial duration series analysis

Partial duration series analyses are sometimes considered more appropriate for flood frequency estimation, as all flood events on record are considered (Stedinger *et al*., 1993). A partial duration series analysis was performed to compare with the estimates determined using the annual maximum series. An arbitrary threshold of 8 m$^3$/s was selected, and between 1978 and 2004 there were 45 events over this threshold. The GPA and exponential frequency distributions were used (as recommended by Pearson, 2003). The two distributions produced significantly different results, particularly for return periods of 50 to 200 years (Table 24).

**Table 24: Partial duration series flood frequency estimates (m³/s) for the Waiwhetu Stream at Whites Line East**

| | GPA distribution | Exponential distribution |
|---|---|---|
| Q2 | 12.6 | 13.1 |
| Q5 | 16.5 | 16.9 |
| Q10 | 19.9 | 19.9 |
| Q20 | 23.8 | 22.8 |
| Q50 | 29.8 | 26.7 |
| Q100 | 35.0 | 29.6 |
| Q200 | 41.0 | 32.5 |

### 6.1.5 Comparison of at-site flood frequency results

The analyses that excluded the 1976 flood event (Scenario 1, 1978 – 2004 and the partial duration series) returned lower flood frequency estimates than those that included the event. Because the 1976 event is about equal to the largest event in the systematic record (1978 – 2004) it will obviously have a large impact on the results. Even though there is uncertainty surrounding the magnitude of the 1976 peak, the analyses that include the event are preferred to those that do not because they reflect the reality that there were two events greater than 35 m$^3$/s in 29 years.

The partial duration series analysis for the Waiwhetu Stream is not considered as accurate as the annual maximum series analysis. This is because the partial duration series analysis does not include the 1976 event, and because of the large number of gaps in the flow record (meaning that not all events above the threshold would have been recorded). Therefore the rate of event occurrence, which has a large impact on the results, may not be correct. For this reason the partial duration series should not be used as the preferred at-site flood frequency method for the Waiwhetu Stream.

When the annual maximum series is used the EV2 distribution provides the best fit to the data, thus this is the preferred distribution for the Waiwhetu Stream at Whites Line East. The two options using the EV2 distribution are the Hilltop results (which treat the 1976 – 2004 annual peaks as a continuous series) and the MAX results (which treat the 1970 – 2004 annual peaks as a discontinuous series). As shown by Table 25, the two methods give significantly different results, with the MAX results being lower particularly for return periods of 10 years and greater. The main reason for the continuous analysis returning higher values is that the gaps in the record are not accounted for, i.e. the analysis assumes that there were two floods over 35 $m^3/s$ in 20 years rather than the correct 29 years.

**Table 25: Comparison of at-site flood frequency estimates ($m^3/s$) for the Waiwhetu Stream at Whites Line East, derived using a EV2 distribution**

| | Hilltop estimates (1976 – 2004) | MAX estimates (1970 – 2004) |
|---|---|---|
| Q2 | 13.4 | 12.4 |
| Q5 | 19.3 | 17.4 |
| Q10 | 24.6 | 21.5 |
| Q20 | 31.0 | 26.1 |
| Q50 | 42.0 | 33.1 |
| Q100 | 52.8 | 39.3 |
| Q200 | 66.3 | 46.5 |

The MAX analysis is preferred because:

- It takes into account data collected in the early 1970s;
- It allows for the gaps in the data record so that the correct number of years is specified; and
- It takes into account the high error associated with the December 1976 peak flow estimate.

The at-site flood frequency estimates are considerably higher than those derived in the previous Waiwhetu flood hydrology review (Lew, 1996). There are three reasons for the apparent increase:

1. Several large flood events have occurred since 1996 – of the five largest flood events since 1976 four have occurred since 1996[8];

2. Lew (1996) assumed that all annual maxima were measured (i.e. record gaps were not taken into account). This assumption will bias the results downward, as flood events that were not true annual maxima were included in the analysis; and

3. The preferred at-site results presented in Lew (1996) were derived using an EV1 distribution, but a EV2 distribution provides a better fit to the data.

## 6.2 Regional flood frequency analysis

Pearson (1991) developed a regionalised methodology for small catchments based on rainfall depth duration frequency, catchment area, and New Zealand Land Resource Inventory data. The method was used to derive regional flood frequency estimates for the Waiwhetu Stream by Lew (1996).

New regional estimates were calculated for this study using the mean annual flood from 1978 to 2004 (15.1 $m^3/s$) and an average of the flood frequency ratios recommended for low slope (<19°) and higher slope (>19°) catchments by Pearson (1991). The updated regional estimates (Table 26) are slightly higher than those calculated by Lew (1996) due to the increase in the mean annual flood since 1996.

**Table 26: Regional flood frequency estimates for the Waiwhetu Stream at Whites Line East**

| | Flow ($m^3/s$) |
|---|---|
| Q2 | 12.8 |
| Q5 | 20.4 |
| Q10 | 27.6 |
| Q20 | 34.3 |
| Q50 | 43.1 |
| Q100 | 49.9 |
| Q200 | 55.6 |

## 6.3 Rainfall runoff model flood frequency estimates

The flood frequency estimates for the Waiwhetu Stream at Whites Line East derived using the calibrated rainfall runoff model were shown in Section 5 (Table 27). These are lower than the RORB design flood peaks derived by Lew (1996) for the reasons discussed in Section 5.7.2.

[8] This assumes the modelled annual maxima for 1998 (20 $m^3/s$) is accurate to within +/-10% error.

**Table 27: Modelled (TimeStudio) flood frequency estimates for the Waiwhetu Stream at Whites Line East**

| | Flow ($m^3/s$) | Model standard error[9] ($m^3/s$) |
|---|---|---|
| Q2 | 10.1 | 1.55 |
| Q5 | 16.7 | |
| Q10 | 21.6 | |
| Q20 | 27.0 | |
| Q50 | 33.3 | |
| Q100 | 37.4 | |
| Q200 | 39.1 | |

## 6.4 Flood frequency discussion

### 6.4.1 Comparison of results

Table 28 shows a summary of the flood frequency estimates for the Waiwhetu Stream at Whites Line East. McKerchar & Pearson (1989) recommend combining at-site and regional flood frequency estimates to obtain pooled estimates, where the at-site record is greater than 10 years.

The at-site, regional and pooled flood frequency estimates are higher than the modelled estimates, and the difference is significant at the 95% confidence level. Because the model validation produced good results (and model error is taken into account in the 95% confidence interval), the difference between the modelled and at-site / regional flood frequencies is more likely to be a result of underestimated rainfall depths rather than rainfall runoff model inaccuracy. The depth-duration frequency analysis was based on Avalon rainfall data from 1948 – 1989, but there have been several significant rainfall events since 1989. In addition, the design rainfall was produced using an EV1 distribution, which will produce lower rainfall depths for high return periods than the EV2 distribution.

### 6.4.2 Options for design flood frequency estimates

The options for producing design flood frequency estimates for the Waiwhetu catchment are:

1. Use the pooled at-site / regional estimates, and scaling up the modelled design hydrographs produced in Chapter 5.

2. Average the modelled estimates with the regional or at-site estimates, and scaling up the modelled design hydrographs produced in Chapter 5.

[9] Estimated standard error of flood peak, derived from the model validation results. The error refers to that introduced by the rainfall runoff modelling process only. The error for each return period cannot be specified due to the small number of validation events, thus an average standard error is assumed.

3. Discount the at-site and regional estimates and use the modelled flood frequency estimates and unaltered design hydrographs produced in Chapter 5.

The previous Waiwhetu Stream flood hydrology study (Lew, 1996) took the approach of option 2, i.e. an average of the regional and modelled flood frequencies. However, there is no scientific justification for such an approach. Similarly, the use of the modelled flood frequency estimates alone was ruled out (option 3). There is a good length of at-site data for the Waiwhetu Stream at Whites Line East which should not be ignored.

By pooling the at-site data with regional estimates as recommended by McKerchar & Pearson (1989) the relatively large error associated with the at-site analysis (caused by gaps in the record and uncertainty surrounding the magnitude of some of the events) is taken into account. Thus the confidence interval surrounding the pooled estimate is narrower than if the at-site estimates are used alone. At this stage the preferred approach is therefore option 1 – using the pooled estimates and scaling the modelled design hydrographs as necessary so that the peaks at Whites Line East match the pooled estimates.

**Table 28: Waiwhetu Stream at White Line East flood frequency estimates ($m^3/s$)**

| | Preferred at-site | | Regional | Pooled at-site / regional | | Modelled | |
|---|---|---|---|---|---|---|---|
| | Estimate | 95% confidence interval | | Estimate | 95% confidence interval | Estimate | 95% confidence interval |
| Q2 | 12 | 10.5 – 14.3 | 13 | 13 | 10.8 – 14.2 | 10 | 5.8 – 14.4 |
| Q5 | 17 | 14.1 – 20.7 | 20 | 19 | 16.8 – 21.2 | 17 | 12.4 – 21.0 |
| Q10 | 22 | 16.9 – 26.1 | 28 | 25 | 21.8 – 28.2 | 22 | 17.3 – 25.9 |
| Q20 | 26 | 20.1 – 32.1 | 34 | 30 | 25.8 – 34.2 | 27 | 22.7 – 31.3 |
| Q50 | 33 | 24.6 – 41.5 | 43 | 38 | 32.2 – 43.8 | 33 | 29.0 – 37.6 |
| Q100 | 39 | 28.7 – 49.9 | 50 | 45 | 37.8 – 52.2 | 37 | 33.1 – 41.7 |
| Q200 | 47 | 33.4 – 59.6 | 56 | 51 | 42.1 – 60.1 | 39 | 34.8 – 43.4 |
| PMF | n/a | n/a | n/a | n/a | n/a | 199 | n/a |

# 7. Summary and recommendations

This investigation into the flood hydrology of the Waiwhetu Stream catchment included rainfall analyses, calibration and validation of a rainfall runoff model, modelling of design rainfall events and flood frequency analyses using at-site and regional methods.

The rainfall runoff model for the catchment calibrated using flood events between 1978 and 2004 produced good results when tested on five validation events. This model can be used with confidence to model flood flows in the catchment, but model performance should be continually assessed as floods occur.

The flood frequency estimates produced for Whites Line East using the rainfall runoff model were lower than the at-site and regional flood frequency results. The discrepancy is probably due to the design rainfall being based on historic data, and also because the design rainfall was derived using an EV1 distribution.

The at-site flood frequency estimates derived in this study are considerably higher than those previously derived for the Waiwhetu Stream at Whites Line East. Because a good length of data is now available, and record gaps and annual maxima error were taken into account, the at-site analysis can be used with greater confidence than the previous results.

The recommended flood frequency estimates for the Waiwhetu Stream at Whites Line East are those which were derived by pooling the at-site and regional estimates (Table 29) as recommended by McKerchar & Pearson (1989). Appendix 5 contains the recommended design hydrographs[10] for the Waiwhetu catchment.

**Table 29: Recommended flood frequency estimates for the Waiwhetu Stream at Whites Line East**

| | Annual exceedance probability (%) | Flow ($m^3/s$) | 95% confidence interval |
|---|---|---|---|
| Q2 | 50 | 13 | 10.8 – 14.2 |
| Q5 | 20 | 19 | 16.8 – 21.2 |
| Q10 | 10 | 25 | 21.8 – 28.2 |
| Q20 | 5 | 30 | 25.8 – 34.2 |
| Q50 | 2 | 38 | 32.2 – 43.8 |
| Q100 | 1 | 45 | 37.8 – 52.2 |
| Q200 | 0.5 | 51 | 42.1 – 60.1 |
| Probable maximum flood | 0.01 | 197 | n/a |

[10] These are the hydrographs from rainfall runoff modelling of the design rainfall events, scaled up so that the flood peaks at Whites Line East fit the recommended flood frequency estimates in Table 29.

# References

Doherty, J., 2000: PEST2000 Model-independent parameter estimation. Watermark Numerical Computing, Australia.

Hovey, C., 1979: Waiwhetu Stream hydrology and design comments. River control notes, Wellington Regional Water Board.

Hydro-Electric Corporation, 2000: Hydrol Model version 3.6.2.0. Tasmania, Australia.

Lew, D., 1996: Waiwhetu Stream Flood Hydrology. Resource Investigations Department, Wellington Regional Council. Publication WRC/RI-T-96/17.

McKerchar, A. and Pearson, C., 1989: Flood frequency in New Zealand. Hydrology Centre Publication 20, Christchurch.

Minson, R., 1996: Waiwhetu Stream Management Plan Flood Hazard Assessment. Landcare Division, Wellington Regional Council. Publication WRC/RI-T-96/67.

NIWA, 2002: HIRDS V2.0 – High Intensity Rainfall Design System.

Pearson, C., 1991: Regional flood frequency analysis for small New Zealand basins. 2. Flood frequency groups. Journal of Hydrology (New Zealand) 30(2): 77 – 92.

Pearson, C., 2003: Hydrological Statistics Course notes, NIWA Environmental Monitoring Training Course 03S. NIWA, Christchurch.

Pilgrim D. and Cordery, I., 1975: Rainfall temporal patterns for design flood estimation. Journal of Hydraulic Engineering 100(1): 81-95.

Stedinger, J., Surani, R. and Therivel, R., 1988: MAX – A program for flood frequency analysis using systematic-record, historical, botanical, physical paleohydrologic and regional hydrologic information using maximum likelihood techniques. Department of Environmental Engineering, Cornell University.

Stedinger, J., Vogel, R. and Foufoula-Gergiou, E., 1993: Frequency analysis of extreme events. In Maidment, D (ed): Handbook of Hydrology. McGraw-Hill, USA.

Thompson, C. and Tomlinson, A., 1993: Probable maximum precipitation in New Zealand for small areas and short durations. Journal of Hydrology (New Zealand) 31(1): 78 – 90.

World Meteorological Organisation (WMO), 1986: Manual for estimation of probable maximum precipitation. Operational Hydrology Report No.1, WMO No 332.

Wellington Regional Council, 1995: Hydrology of the Hutt Catchment, Volume 1 Surface water. Hydrological Services Group Publication WRC/CI-T-95/38.

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# Appendix 1: Design rainfall events

| Storm duration | Increment (minutes) | % of rainfall total | Return period | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 2 years (mm) | 5 years (mm) | 10 years (mm) | 20 years (mm) | 50 years (mm) | 100 years (mm) | 200 years (mm) |
| One hour | 15 | 20 | 3.6 | 4.8 | 5.4 | 6.2 | 7.0 | 8.0 | 8.8 |
| | 30 | 30 | 5.4 | 7.2 | 8.1 | 9.3 | 10.5 | 12.0 | 13.2 |
| | 45 | 30 | 5.4 | 7.2 | 8.1 | 9.3 | 10.5 | 12.0 | 13.2 |
| | 60 | 20 | 3.6 | 4.8 | 5.4 | 6.2 | 7.0 | 8.0 | 8.8 |
| Two hour | 30 | 20 | 5.2 | 7.0 | 8.2 | 9.4 | 10.8 | 11.8 | 12.8 |
| | 60 | 30 | 7.8 | 10.5 | 12.3 | 14.1 | 16.2 | 17.7 | 19.2 |
| | 90 | 30 | 7.8 | 10.5 | 12.3 | 14.1 | 16.2 | 17.7 | 19.2 |
| | 120 | 20 | 5.2 | 7.0 | 8.2 | 9.4 | 10.8 | 11.8 | 12.8 |
| 3 hour | 30 | 12 | 3.7 | 5.2 | 6.2 | 7.2 | 8.4 | 9.2 | 9.8 |
| | 60 | 18 | 5.6 | 7.7 | 9.4 | 10.8 | 12.6 | 13.9 | 14.8 |
| | 90 | 22 | 6.8 | 9.5 | 11.4 | 13.2 | 15.4 | 16.9 | 18.0 |
| | 120 | 19 | 5.9 | 8.2 | 9.9 | 11.4 | 13.3 | 14.6 | 15.6 |
| | 150 | 16 | 5.0 | 6.9 | 8.3 | 9.6 | 11.2 | 12.3 | 13.1 |
| | 180 | 13 | 4.0 | 5.6 | 6.8 | 7.8 | 9.1 | 10.0 | 10.7 |
| 4 hour | 30 | 8 | 2.9 | 4.2 | 5.0 | 5.8 | 6.7 | 7.5 | 7.7 |
| | 60 | 12 | 4.3 | 6.2 | 7.6 | 8.8 | 10.1 | 11.3 | 11.5 |
| | 90 | 15 | 5.4 | 7.8 | 9.5 | 11.0 | 12.6 | 14.1 | 14.4 |
| | 120 | 15 | 5.4 | 7.8 | 9.5 | 11.0 | 12.6 | 14.1 | 14.4 |
| | 150 | 16 | 5.8 | 8.3 | 10.1 | 11.7 | 13.4 | 15.0 | 15.4 |
| | 180 | 14 | 5.0 | 7.3 | 8.8 | 10.2 | 11.8 | 13.2 | 13.4 |
| | 210 | 10 | 3.6 | 5.2 | 6.3 | 7.3 | 8.4 | 9.4 | 9.6 |
| | 240 | 10 | 3.6 | 5.2 | 6.3 | 7.3 | 8.4 | 9.4 | 9.6 |
| Six hour | 60 | 12 | 5.3 | 7.8 | 9.5 | 11.2 | 13.2 | 14.5 | 15.0 |
| | 120 | 18 | 7.9 | 11.7 | 14.2 | 16.7 | 19.8 | 21.8 | 22.5 |
| | 180 | 22 | 9.7 | 14.3 | 17.4 | 20.5 | 24.2 | 26.6 | 27.5 |
| | 240 | 19 | 8.4 | 12.4 | 15.0 | 17.7 | 20.9 | 23.0 | 23.8 |
| | 300 | 16 | 7.0 | 10.4 | 12.6 | 14.9 | 17.6 | 19.4 | 20.0 |
| | 360 | 13 | 5.7 | 8.5 | 10.3 | 12.1 | 14.3 | 15.7 | 16.3 |
| 8 hour | 60 | 8 | 4.1 | 6.2 | 7.5 | 8.8 | 10.4 | 11.7 | 12.2 |
| | 120 | 12 | 6.1 | 9.2 | 11.3 | 13.2 | 15.6 | 17.5 | 18.4 |
| | 180 | 15 | 7.7 | 11.6 | 14.1 | 16.5 | 19.5 | 21.9 | 23.0 |
| | 240 | 15 | 7.7 | 11.6 | 14.1 | 16.5 | 19.5 | 21.9 | 23.0 |
| | 300 | 16 | 8.2 | 12.3 | 15.0 | 17.6 | 20.8 | 23.4 | 24.5 |
| | 360 | 14 | 7.1 | 10.8 | 13.2 | 15.4 | 18.2 | 20.4 | 21.4 |
| | 420 | 10 | 5.1 | 7.7 | 9.4 | 11.0 | 13.0 | 14.6 | 15.3 |
| | 480 | 10 | 5.1 | 7.7 | 9.4 | 11.0 | 13.0 | 14.6 | 15.3 |

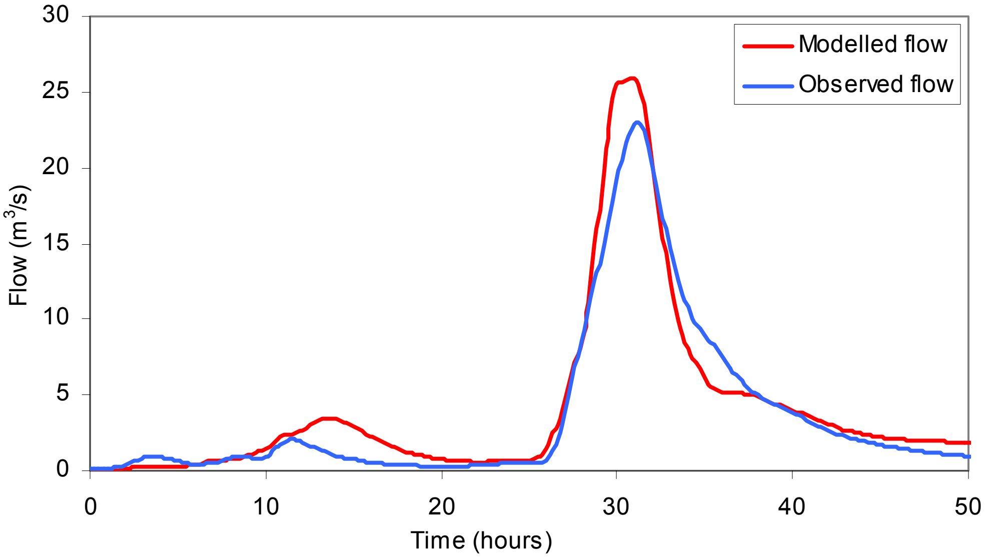
| Storm duration | Increment (minutes) | % of rainfall total | Return period | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 2 years (mm) | 5 years (mm) | 10 years (mm) | 20 years (mm) | 50 years (mm) | 100 years (mm) | 200 years (mm) |
| 10 hour | 60 | 7 | 3.9 | 6.0 | 7.3 | 8.6 | 10.3 | 11.3 | 12.4 |
| | 120 | 9 | 5.0 | 7.7 | 9.4 | 11.1 | 13.2 | 14.6 | 15.9 |
| | 180 | 10 | 5.6 | 8.6 | 10.4 | 12.3 | 14.7 | 16.2 | 17.7 |
| | 240 | 11 | 6.2 | 9.5 | 11.4 | 13.5 | 16.2 | 17.8 | 19.5 |
| | 300 | 13 | 7.3 | 11.2 | 13.5 | 16.0 | 19.1 | 21.1 | 23.0 |
| | 360 | 14 | 7.8 | 12.0 | 14.6 | 17.2 | 20.6 | 22.7 | 24.8 |
| | 420 | 12 | 6.7 | 10.3 | 12.5 | 14.8 | 17.6 | 19.4 | 21.2 |
| | 480 | 8 | 4.5 | 6.9 | 8.3 | 9.8 | 11.8 | 13.0 | 14.2 |
| | 540 | 8 | 4.5 | 6.9 | 8.3 | 9.8 | 11.8 | 13.0 | 14.2 |
| | 600 | 8 | 4.5 | 6.9 | 8.3 | 9.8 | 11.8 | 13.0 | 14.2 |
| 12 hour | 60 | 6 | 3.6 | 5.5 | 6.7 | 7.9 | 9.4 | 10.4 | 11.6 |
| | 120 | 6 | 3.6 | 5.5 | 6.7 | 7.9 | 9.4 | 10.4 | 11.6 |
| | 180 | 8 | 4.8 | 7.4 | 9.0 | 10.6 | 12.6 | 13.9 | 15.4 |
| | 240 | 9 | 5.4 | 8.3 | 10.1 | 11.9 | 14.1 | 15.7 | 17.4 |
| | 300 | 9 | 5.4 | 8.3 | 10.1 | 11.9 | 14.1 | 15.7 | 17.4 |
| | 360 | 14 | 8.4 | 12.9 | 15.7 | 18.5 | 22.0 | 24.4 | 27.0 |
| | 420 | 11 | 6.6 | 10.1 | 12.3 | 14.5 | 17.3 | 19.1 | 21.2 |
| | 480 | 9 | 5.4 | 8.3 | 10.1 | 11.9 | 14.1 | 15.7 | 17.4 |
| | 540 | 8 | 4.8 | 7.4 | 9.0 | 10.6 | 12.6 | 13.9 | 15.4 |
| | 600 | 8 | 4.8 | 7.4 | 9.0 | 10.6 | 12.6 | 13.9 | 15.4 |
| | 660 | 6 | 3.6 | 5.5 | 6.7 | 7.9 | 9.4 | 10.4 | 11.6 |
| | 720 | 6 | 3.6 | 5.5 | 6.7 | 7.9 | 9.4 | 10.4 | 11.6 |
| 24 hour | 60 | 3 | 2.4 | 3.5 | 4.2 | 4.9 | 5.7 | 6.4 | 7.1 |
| | 120 | 3 | 2.4 | 3.5 | 4.2 | 4.9 | 5.7 | 6.4 | 7.1 |
| | 180 | 3 | 2.4 | 3.5 | 4.2 | 4.9 | 5.7 | 6.4 | 7.1 |
| | 240 | 3 | 2.4 | 3.5 | 4.2 | 4.9 | 5.7 | 6.4 | 7.1 |
| | 300 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 360 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 420 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 480 | 5 | 4.1 | 5.8 | 7.0 | 8.1 | 9.6 | 10.7 | 11.8 |
| | 540 | 5 | 4.1 | 5.8 | 7.0 | 8.1 | 9.6 | 10.7 | 11.8 |
| | 600 | 6 | 4.9 | 7.0 | 8.3 | 9.7 | 11.5 | 12.8 | 14.1 |
| | 660 | 6 | 4.9 | 7.0 | 8.3 | 9.7 | 11.5 | 12.8 | 14.1 |
| | 720 | 5 | 4.1 | 5.8 | 7.0 | 8.1 | 9.6 | 10.7 | 11.8 |
| | 780 | 5 | 4.1 | 5.8 | 7.0 | 8.1 | 9.6 | 10.7 | 11.8 |
| | 840 | 5 | 4.1 | 5.8 | 7.0 | 8.1 | 9.6 | 10.7 | 11.8 |
| | 900 | 5 | 4.1 | 5.8 | 7.0 | 8.1 | 9.6 | 10.7 | 11.8 |
| | 960 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 1020 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 1080 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 1140 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 1200 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 1260 | 4 | 3.2 | 4.6 | 5.6 | 6.5 | 7.6 | 8.5 | 9.4 |
| | 1320 | 3 | 2.4 | 3.5 | 4.2 | 4.9 | 5.7 | 6.4 | 7.1 |
| | 1380 | 3 | 2.4 | 3.5 | 4.2 | 4.9 | 5.7 | 6.4 | 7.1 |
| | 1440 | 3 | 2.4 | 3.5 | 4.2 | 4.9 | 5.7 | 6.4 | 7.1 |

| Storm duration | Increment (minutes) | % of rainfall total | Return period | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 2 years (mm) | 5 years (mm) | 10 years (mm) | 20 years (mm) | 50 years (mm) | 100 years (mm) | 200 years (mm) |
| 48 hour | 60 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 120 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 180 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 240 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 300 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 360 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 420 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 480 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 540 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 600 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 660 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 720 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 780 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 840 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 900 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 960 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1020 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1080 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1140 | 3 | 3.0 | 4.3 | 5.1 | 5.8 | 6.8 | 7.6 | 8.3 |
| | 1200 | 3 | 3.0 | 4.3 | 5.1 | 5.8 | 6.8 | 7.6 | 8.3 |
| | 1260 | 3 | 3.0 | 4.3 | 5.1 | 5.8 | 6.8 | 7.6 | 8.3 |
| | 1320 | 3 | 3.0 | 4.3 | 5.1 | 5.8 | 6.8 | 7.6 | 8.3 |
| | 1380 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1440 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1500 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1560 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1620 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1680 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1740 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1800 | 2.5 | 2.5 | 3.6 | 4.2 | 4.9 | 5.7 | 6.3 | 7.0 |
| | 1860 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 1920 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 1980 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2040 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2100 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2160 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2220 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2280 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2340 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2400 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2460 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2520 | 2 | 2.0 | 2.8 | 3.4 | 3.9 | 4.6 | 5.1 | 5.6 |
| | 2580 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 2640 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 2700 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 2760 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 2820 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |
| | 2880 | 1.5 | 1.5 | 2.1 | 2.5 | 2.9 | 3.4 | 3.8 | 4.2 |

# Appendix 2: Rainfall runoff model calibration plots

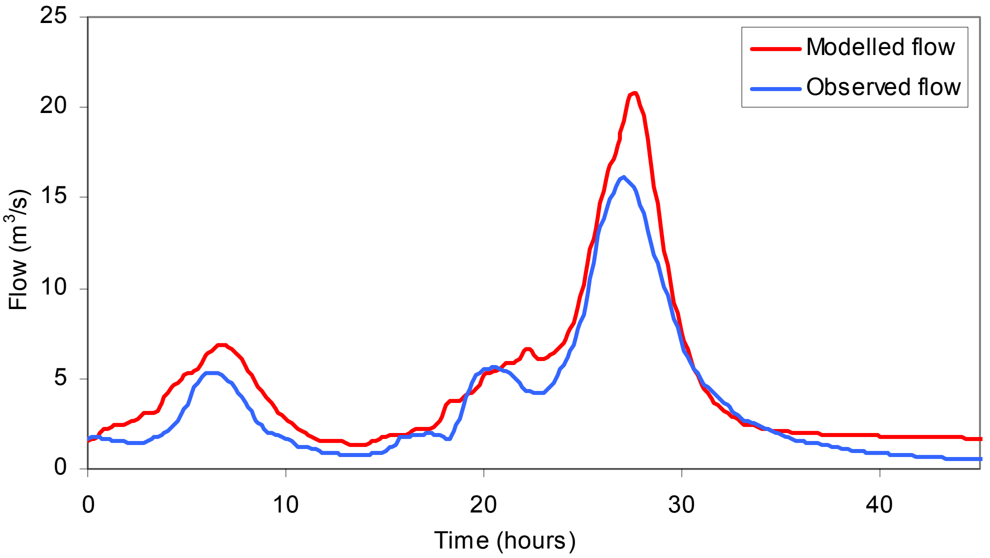
**November 2001 calibration event**

Error in peak: +12.6%
Error in timing: - 15 minutes
Mean absolute error for event: 1.2 $m^3/s$



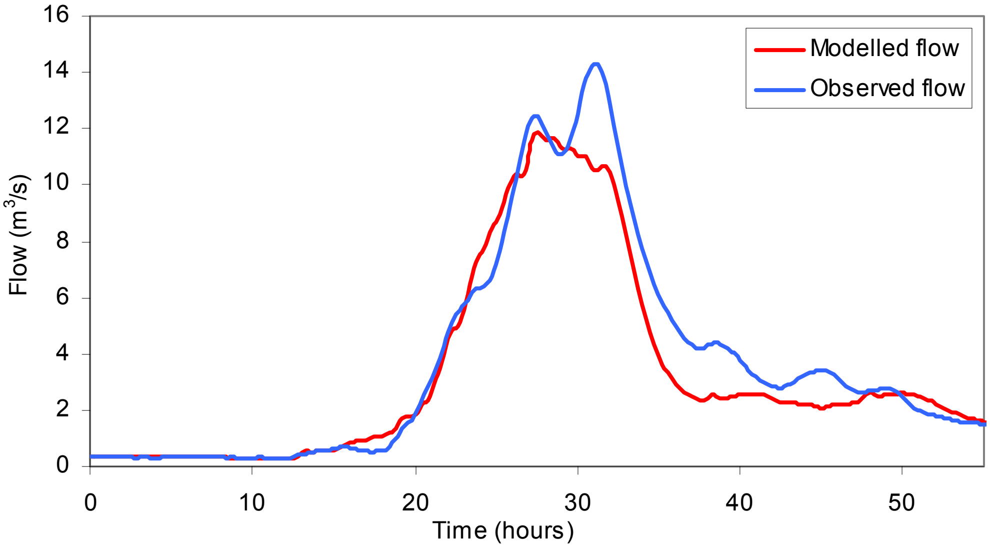

**March 1990 calibration event**

Error in peak: +29.2%
Error in timing: + 30 mins
Mean absolute error for event: 1.17 $m^3/s$



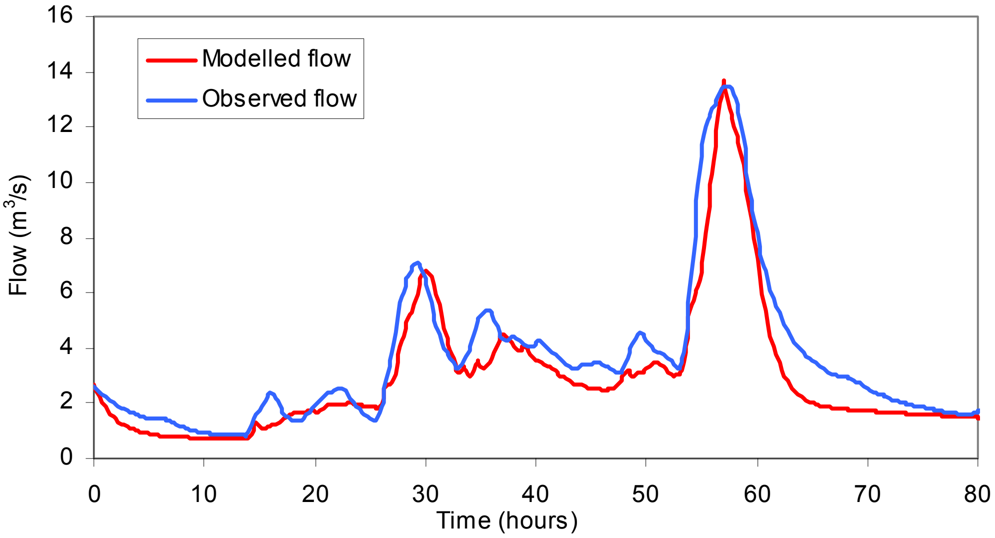
**October 2003 calibration event**

Error in peak: -16.8%
Error in timing: - 3 hours 30 mins
Mean absolute error for event: 0.91 $m^3/s$



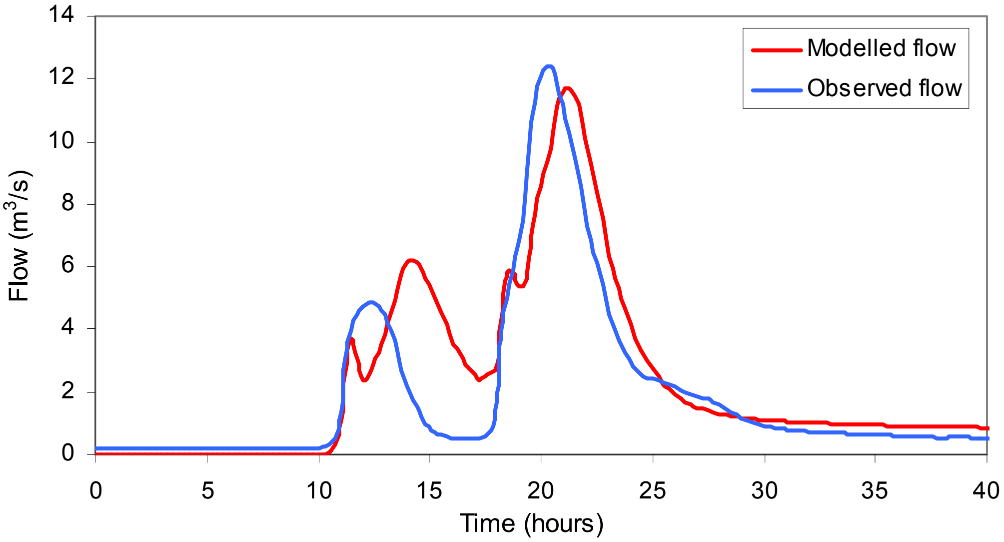

**October 2000 calibration event:**

Error in peak: 1.6%
Error in timing: + 30 mins
Mean absolute error for event: 0.80 $m^3/s$



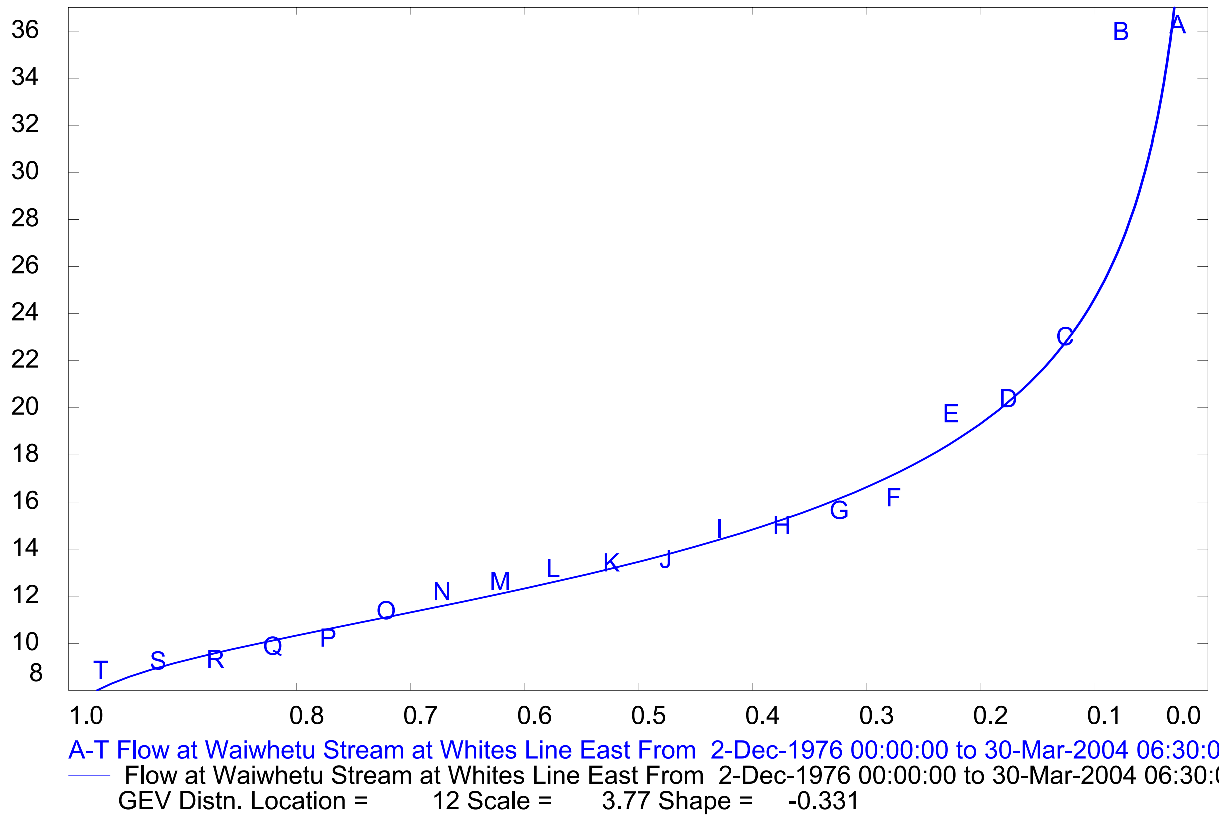
**December 2001 calibration event:**

Error in peak: -5.6%
Error in timing: + 1 hour
Mean absolute error for event: 1.45 $m^3/s$

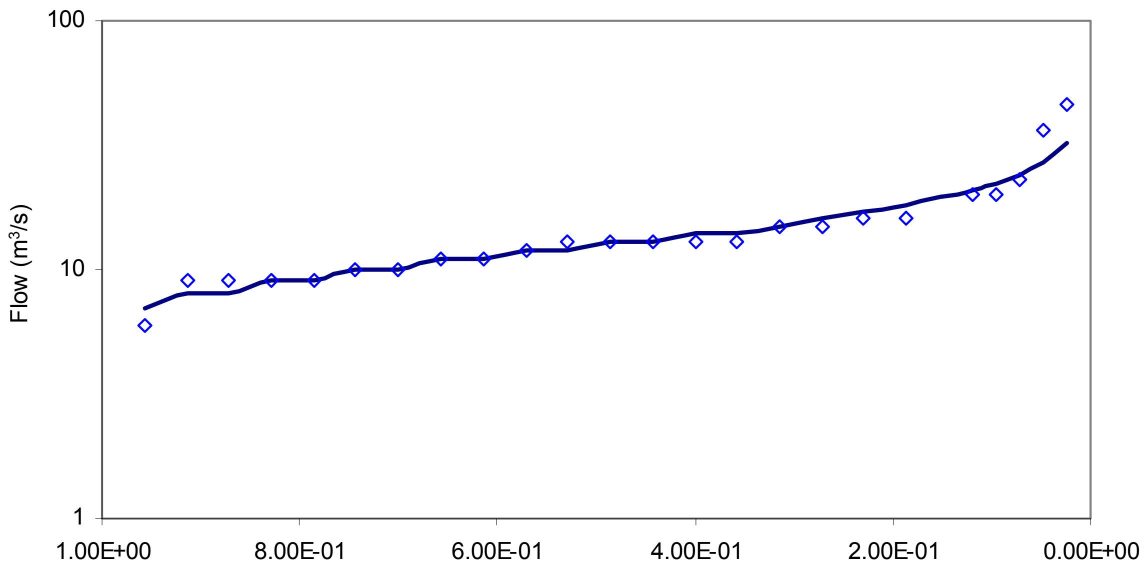


## Appendix 3: Flood frequency distribution plots

**GEV distribution fitted to Waiwhetu Stream at Whites Line East annual maxima (1976 – 2004) using L-moments method in Hilltop:**




**GEV distribution fitted to Waiwhetu Stream at Whites Line East annual maxima (1970 – 2004) using maximum likelihood method in MAX:**



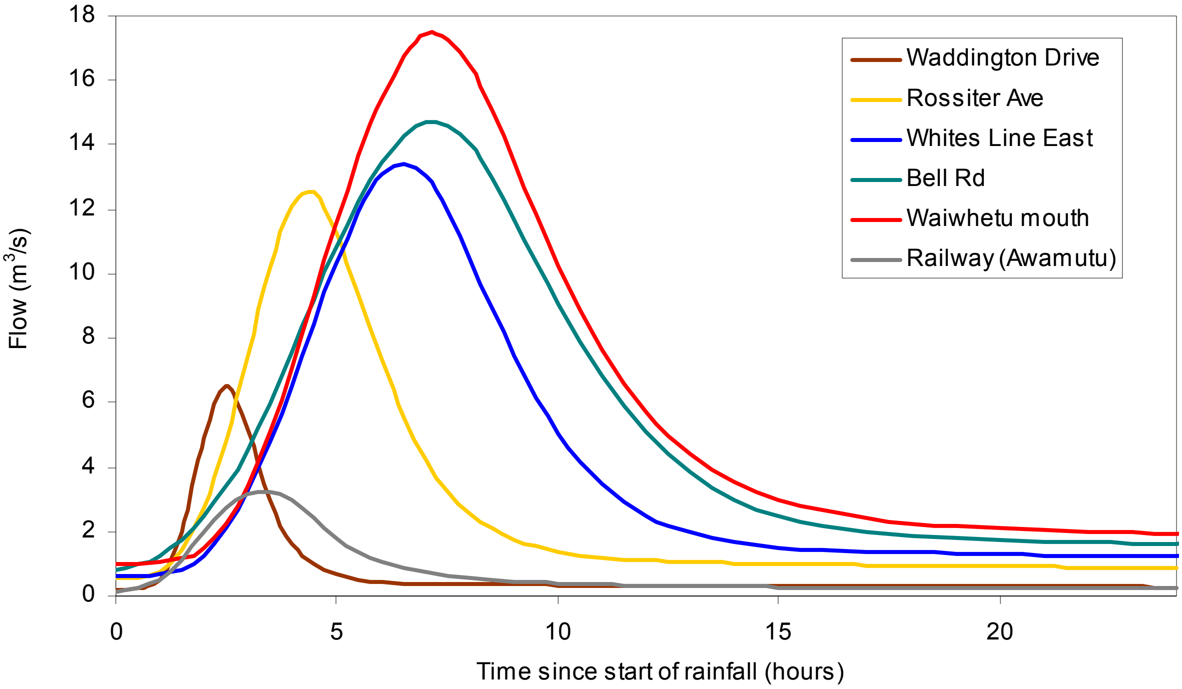
# Appendix 4: Maximised design flood peaks

The flood peaks for the output nodes within the Waiwhetu catchment are those from Table 21, scaled so that the flood peaks at Whites Line East equal the recommended flood frequency estimates in Table 29.

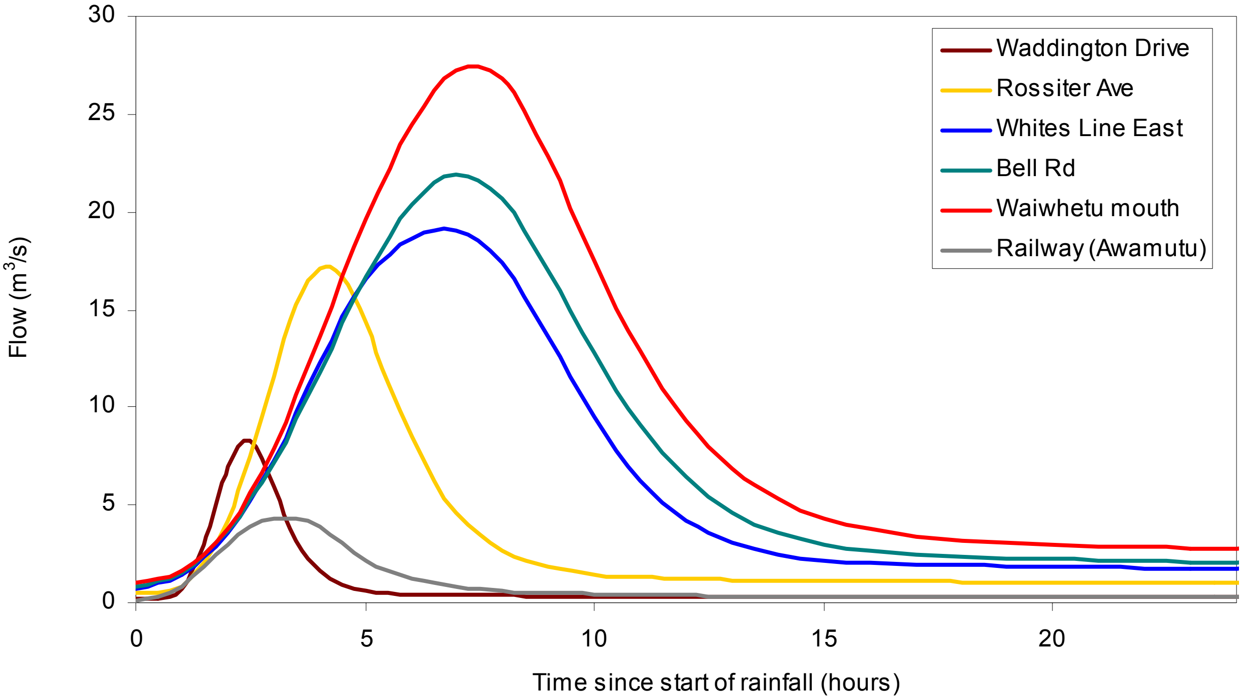
| Location | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q200 ($m^3/s$) | PMF ($m^3/s$) |
|---|---|---|---|---|---|---|---|---|
| Waddington Drive | 6.5 | 8 | 10 | 11 | 14 | 17 | 19 | 77 |
| Rossiter Ave | 12.5 | 17 | 22 | 26 | 33 | 40 | 45 | 175 |
| Whites Line East | 13 | 19 | 25 | 30 | 38 | 45 | 51 | 199 |
| Bell Road | 15 | 22 | 29 | 33 | 42 | 50 | 58 | 223 |
| Waiwhetu mouth | 18 | 27 | 35 | 42 | 53 | 63 | 73 | 292 |
| Awamutu at Railway | 3 | 4 | 6 | 7 | 8 | 10 | 11 | 48 |

# Appendix 5: Design flood hydrographs

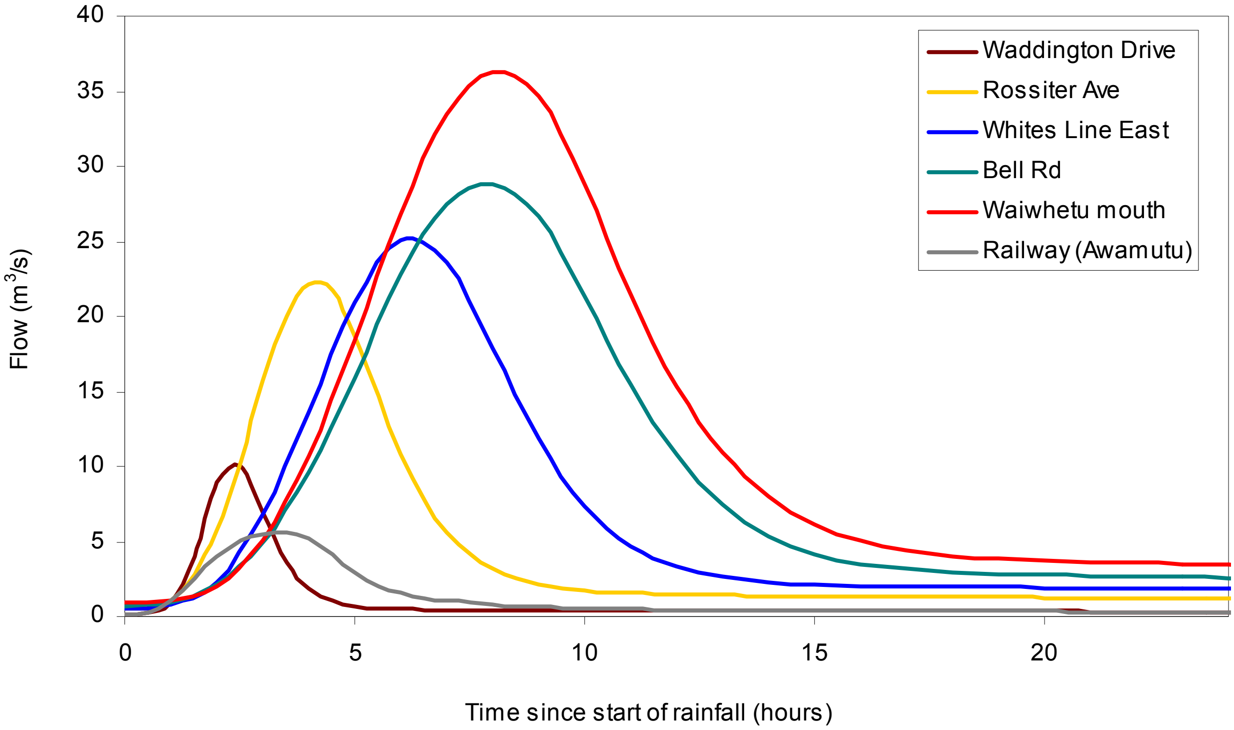
## Q2 design hydrographs:



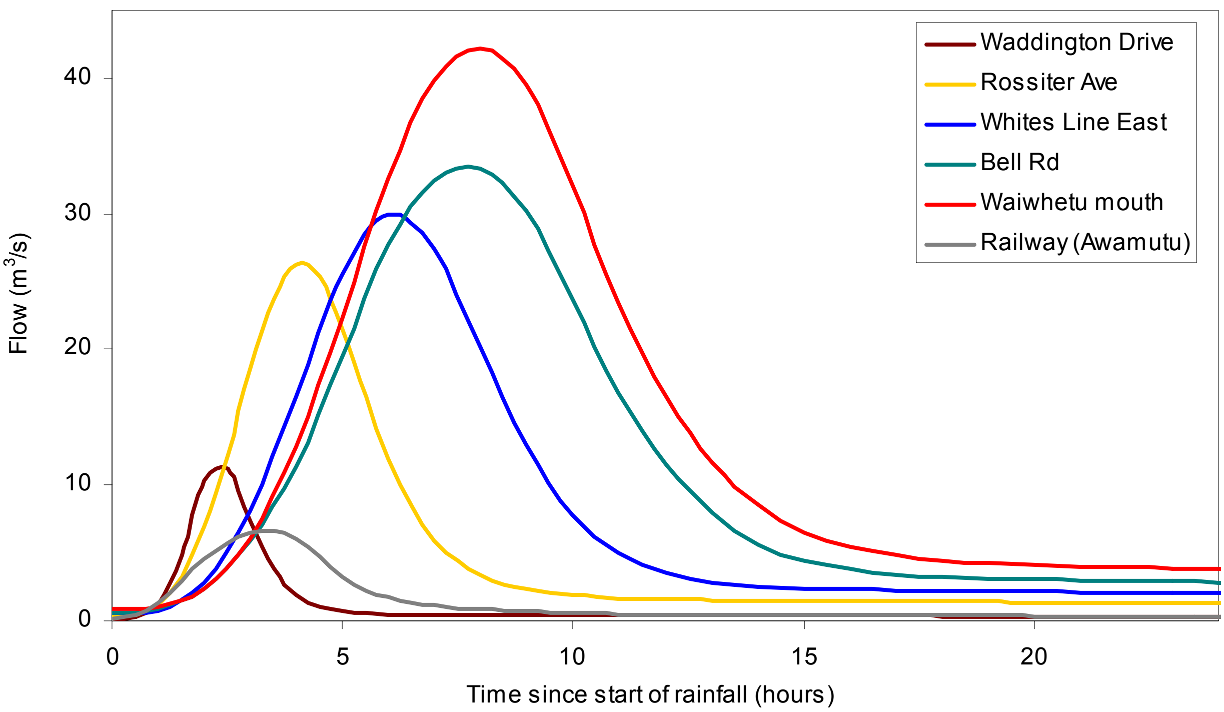

## Q5 design hydrographs:



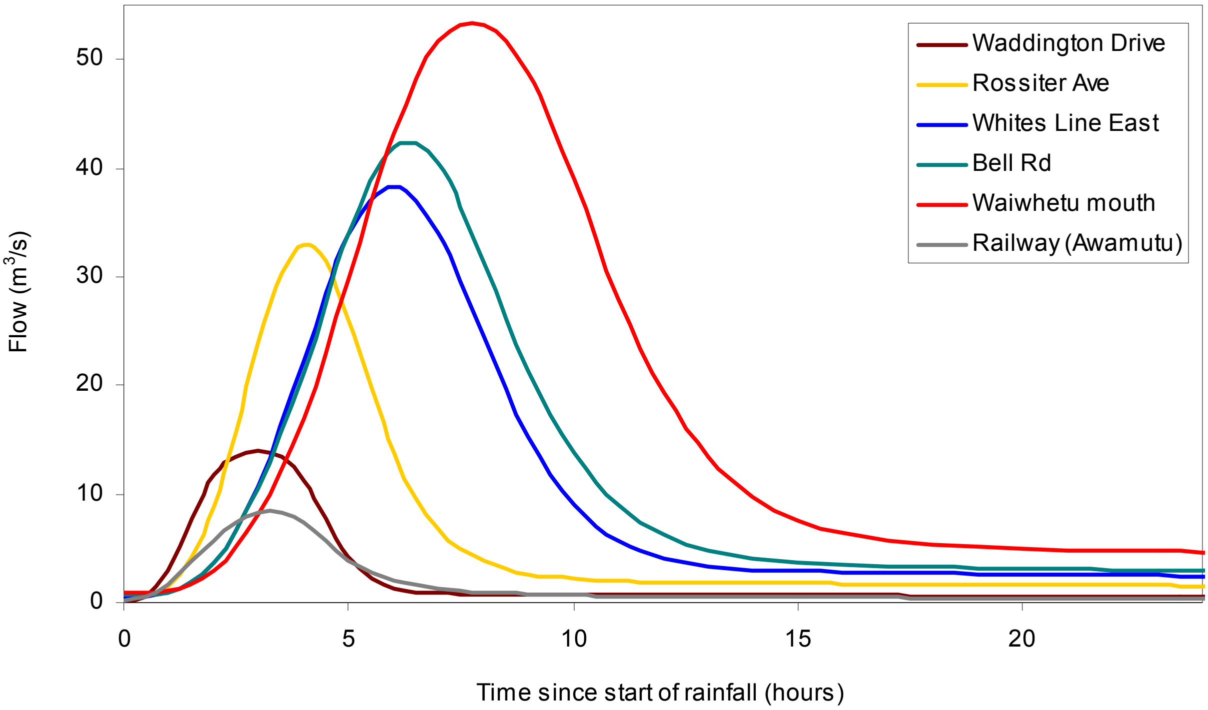
**Q10 design hydrographs:**



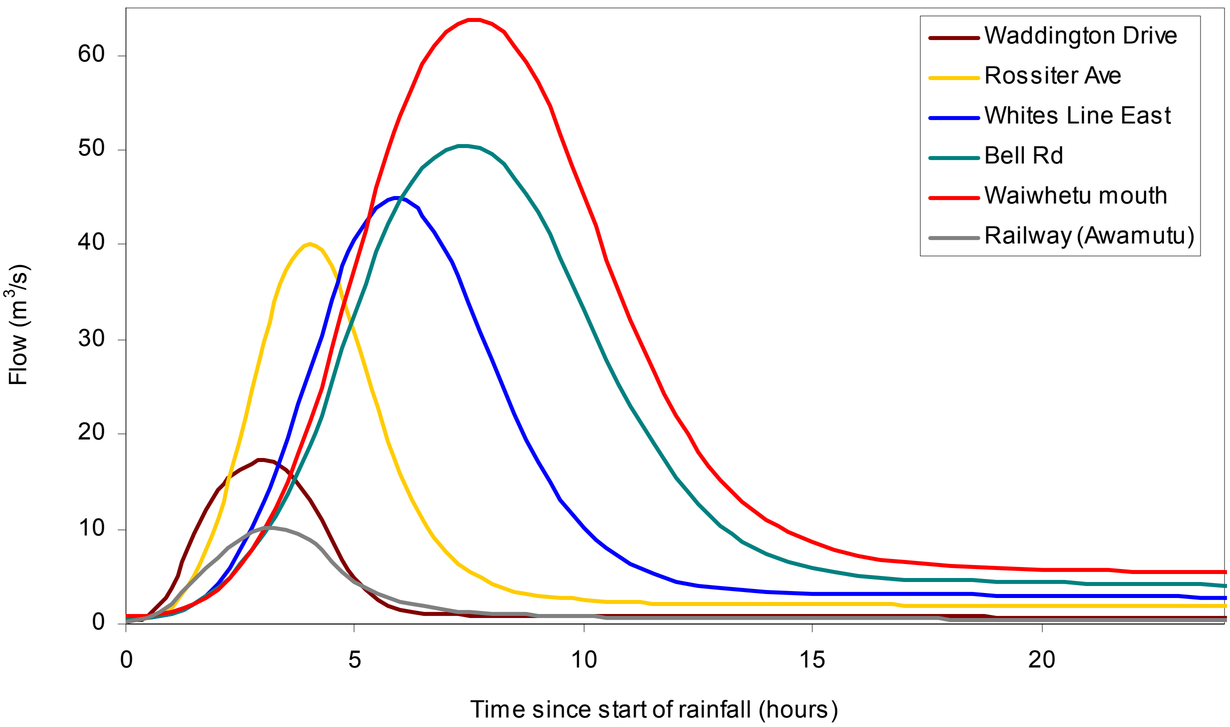

**Q20 design hydrographs:**



## Q50 design hydrographs:




## Q100 design hydrographs:



## Q200 design hydrographs: