Before Independent Hearings Commissioners
At Wellington

Under	the Resource Management Act 1991 (the Act)
In the matter of	Applications for resource consents, and a Notice of Requirement for a Designation, by Wellington Water Limited on behalf of Upper Hutt City Council, for the construction, operation and maintenance of the structural flood mitigation works identified as the Pinehaven Stream Improvements Project

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Memorandum of counsel for Wellington Water Limited in response to 2nd Minute of Independent Hearing Panel

Dated 16 July 2020

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May it please the Commissioners:

1 The purpose of this memorandum is to respond, on behalf of Wellington Water Limited ('**WWL**'), to the 2nd Minute of the Independent Hearing Panel dated 10 July 2020 ('the **Minute**').

2008 Hydrological model

2 The Minute requested the following information from WWL in relation to the 2008 hydrological model using HYDSTRA software and its calibrations ('the **2008 Model**'):[1]

a Who prepared the 2008 Model and the calibrations;

b Whether the 2008 Model and the calibrations against the 31 July 2008 storm event and independent flow estimation have been peer reviewed either internally or externally; and

c Relevant details of the peer review(s), if any.

3 The 2008 Model was prepared and calibrated by MWH New Zealand Limited (now known as Stantec New Zealand). The 2008 Model is provided as **Appendix A** to this memorandum.

4 The 2008 Model has been peer reviewed by DHI Water and Environment, Jacobs New Zealand Limited and Beca Limited.

5 The DHI Water and Environment review is provided as **Appendix B** to this memorandum.

6 Jacobs reviewed the model to satisfy themselves that it was 'fit for purpose', it did not prepare a report.

7 Beca Limited's peer review is the same document requested below at paragraph 8.

Beca technical review of flooding

8 The Minute also required information from WWL in relation to the Beca technical review of flooding dated 2 December 2019. In particular, the Minute requested

[1] Paragraph 20, 2nd Minute of Independent Hearing Panel dated 10 July 2020.

that WWL describe the nature of the 2015 Beca model and mapping review ('the **2015 Audit**') which was discussed in the 2019 review above.[2]

9 The Minute also requested a copy of the 2015 Audit.[3] This is provided as **Appendix C** to this memorandum.

10 The purpose of the 2015 Audit was to undertake a comprehensive audit of the hydrological and hydraulic modelling carried out from 2008 to 2010 to determine whether it was fit for purpose.[4] Flood extent and hazard maps created by Greater Wellington Regional Council were based on the outputs of the modelling described above.[5]

11 The main outcome of the 2015 Audit was that the hydrological and hydraulic modelling is fit for purpose.[6]

Speaker information

12 The Minute also required WWL to provide the following information to the Panel by 5pm 22 July:[7]

a A list of speakers, including their technical expertise (if any) and the topics on which they will be presenting;

b Whether written evidence (for technical experts) or written representations (for non-experts) will be provided prior to the hearing for each of those persons; and

c A proposed order of appearance and length of time for each of speaker.

13 This information is set out in **Appendix D** to this memorandum.

Nicola McIndoe
Counsel for Wellington Water Limited

[2] Paragraph 21, 2nd Minute of Independent Hearing Panel dated 10 July 2020.
[3] Paragraph 21, 2nd Minute of Independent Hearing Panel dated 10 July 2020.
[4] Appendix C, Executive summary.
[5] Appendix C, Executive summary.
[6] Appendix C, Executive summary.
[7] Paragraph 8, 2nd Minute of Independent Hearing Panel dated 10 July 2020.

Appendix A 2008 Hydrological model

Greater Wellington Regional Council

Pinehaven Stream Flood Hydrology

4 November 2008

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Quality Assurance Statement	
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Revision Schedule

Rev No	Date	Description	Prepared By	Reviewed By	Approved By

Executive Summary

This report presents the results of a flood hydrology investigation into the Pinehaven Stream catchment in Upper Hutt City. Greater Wellington Regional Council (GWRC) has commissioned MWH to complete this study.

The Pinehaven catchment has a total area of approximately 4.7 km^2 and rises to an elevation of 380 m. The stream originates at the boundary with the Mangaroa catchment with major tributaries flowing down the Pinehaven Rd and Elmslie Rd valleys, before reaching lower gradient residential areas of Pinehaven and Silverstream. The aim of this investigation is to undertake a flood hydrology assessment of the Pinehaven Stream and derive design flood flows for use in a hydraulic model. The design flood estimates presented in this report are for the Pinehaven Stream opposite Chatsworth Rd (catchment area of 4.4 km^2).

Design rainfall estimates are derived for the 2-year to 100-year Average Recurrence Interval (ARI) and Probable Maximum Precipitation (PMP) is estimated using standard New Zealand based methodology.

There is very limited recorded flow data in the Pinehaven Stream. Flood frequency analyses are therefore carried out using three regional methods. The rational method is also used as a check on the results.

A rainfall-runoff model is developed for the catchment. Limited calibration data is available – only one flood event was available to be used to calibrate the model. The design rainfall totals are input to the rainfall-runoff model with an appropriate temporal pattern, and modelled flood estimates and hydrographs are output for locations along the catchment.

Despite the lack of calibration data available for the model the results obtained are similar to those derived by the regional methods. This provides confidence in the use of the modelled results and the design flood hydrographs for further hydraulic modelling.

It is recommended that the rainfall-runoff model results be adopted as the design flood estimates.

Table E1 details the recommended design flood estimates for the Pinehaven Stream at Chatsworth Rd.

Table E1. Final Design Flood Estimates for the Pinehaven Stream

ARI (Years)	Flow (m^3/s)
5	15
10	16
20	18
50	20
100	22
PMF	86

After publication of this report in November 2008, the rainfall-runoff model was able to be updated with newly collected flood data on the Pinehaven Stream. Design flood estimates presented have been updated as a result. Table E1 details the updated results while Appendix B contains the relevant updated Sections (6, 7 and 8) of this original report.

Greater Wellington Regional Council

Pinehaven Stream Flood Hydrology

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Appendix B Revision of Rainfall-Runoff Model and Design Flood Estimates, 25 November 2009

1 Introduction

This report presents the results of a flood hydrology investigation into the Pinehaven Stream catchment in Upper Hutt City. Greater Wellington Regional Council (GWRC) has commissioned MWH to complete this study.

1.1 Objective

The aim of this study is to undertake a flood hydrology assessment of the Pinehaven Stream and derive design flood flows for use in a hydraulic model.

Various flood frequency methods are used on the available rainfall and flow data, and a rainfall-runoff model was developed to produce design flood hydrographs at various points in the catchment. The design hydrographs for specific return periods are to be used in subsequent hydraulic modelling of the Pinehaven Stream flood hazard.

1.2 Scope

To develop the required design hydrographs, the following components of work were completed:

i.) Relevant rainfall and flow data was obtained from databases operated by GWRC, Upper Hutt City Council (UHCC) and NIWA.

ii.) An analysis of rainfall records in the area was undertaken to develop an understanding of spatial and temporal rainfall distribution across the catchment and to establish design rainfall for input to the hydrological model. Frequency analysis of extreme rainfall was carried out to derive design rainfall totals of 30 minutes, 1, 2, 3, 4 and 6 hours duration, with Average Recurrence Intervals (ARI) from 2-years to 100-years. The Probable Maximum Precipitation (PMP) is also calculated for the catchment.

iii.) No recorded flow data exists for the Pinehaven Stream so flood frequency estimates are derived using regional methods (McKerchar and Pearson 1989, Pearson 1990, Pearson 1991). The Rational Method is also used as a check/verification on the results obtained.

iv.) A rainfall-runoff model is developed using the Hydstra Modelling software. Currently there is no continuous flow data available to calibrate the rainfall-runoff model, so previous modelling studies and the experience of MWH hydrological experts are drawn upon to determine suitable model parameters. As this investigation began, GWRC hydrology staff were able to obtain a peak flood flow/level measurement during a rainfall event which is used in the calibration process of the rainfall-runoff model.

v.) The design rainfall totals are used as inputs to the model and design flood hydrographs are output at various points in the catchment.

vi.) The design flood peaks estimated from the regional methods and the hydrological model are compared, and final design flood estimates and hydrographs presented.

2 Catchment Description

The Pinehaven Stream is located in Upper Hutt City and is part of the Hulls Creek catchment that flows into the Hutt River between Silverstream and Stokes Valley.

The Pinehaven catchment (Figure 2-1) is bounded by the Mangaroa catchment to the south, Stokes Valley and the Silverstream landfill to the west, and Trentham to the east/north-east. The Pinehaven Stream flows in a northerly direction before joining Hulls Creek near the Silverstream rail station.

The Pinehaven catchment has a total area of approximately 4.7 km^2 and rises to an elevation of 380 m. The stream originates at the boundary with the Mangaroa catchment in the Blue Mountains area with major tributaries flowing down the Pinehaven Rd and Elmslie Rd valleys, before reaching the lower gradient residential areas of Pinehaven and Silverstream.

The analyses presented in this report are based on the catchment area of the Pinehaven Stream to the point opposite Chatsworth Rd. Below Chatsworth Rd the stream flows through a long section of culverts to Hulls Creek. The catchment area above Chatsworth Rd is 4.4km^2.

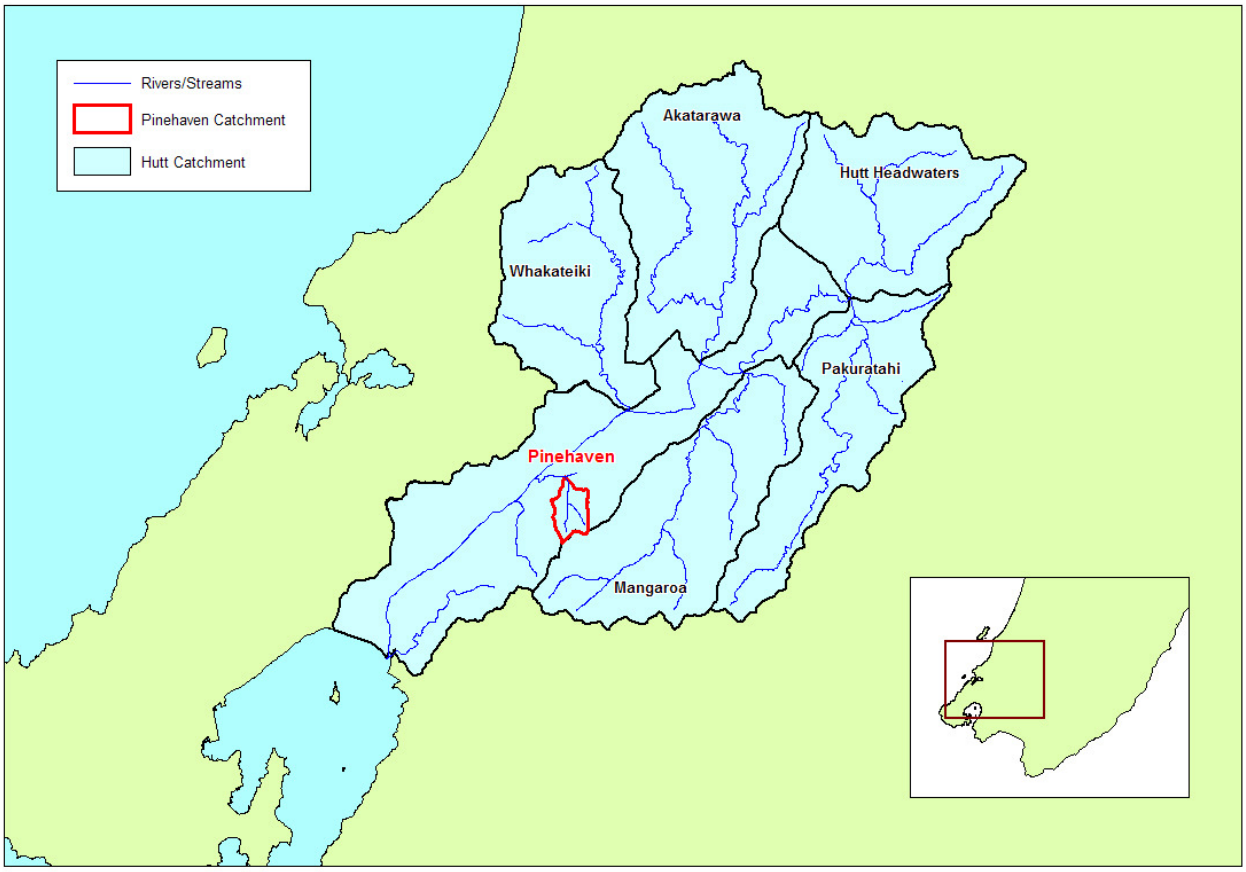

Figure 2-1: Pinehaven Catchment Location

3 Hydrological Data

Rainfall and river flow data used in this report are taken from GWRC, UHCC and NIWA databases.

3.1 Rainfall Data

Table 3-1 details the rainfall data that is available in and around the Pinehaven catchment. The mean annual rainfall for each is also shown. The locations of the raingauges are shown in Figure 4-1.

Table 3-1: Rainfall Stations in and around Pinehaven Catchment

Station	Met No.	Map Reference	Recording Authority	Data Type and Period	Mean Annual Rainfall (mm)
Tasman Vaccine Ltd	E15204	R27:790096	GWRC	Automatic 1980-	1525
Wallaceville	E15102	R27:823061	NIWA	Daily 1939-1985 Automatic 1986-	1300
Pinehaven		R27:785034	UHCC	Daily 1997 Automatic 1998-	1214
Perry St		R27:792050	UHCC	Daily 1997 Automatic 1998-	1240
Heretaunga Dam		R27:807045	UHCC	Daily 1997 Automatic 1998-	1223
Tennyson St	E15104	R27:816073	NIWA	Daily 1954-	1354

The Pinehaven raingauge, operated by UHCC, is ideally located within the study catchment but the length of automatic record is short (approximately nine years) and there are a number of gaps within it.

Wallaceville rainfall data is very similar to the Pinehaven data over the 1998 to 2008 period, and given it's longer length of record and higher data quality it is more useful for rainfall analyses.

The Tasman Vaccine Ltd site is located in the Mangaroa catchment and is more representative of rainfall in the high parts of the Pinehaven catchment.

3.2 Flow Data

There is no recorded flow data within the Pinehaven catchment. GWRC staff carried out a flood gauging measurement of a high flow in the Pinehaven Stream at the Pinehaven Reserve on the 31 July 2008 that is able to be used in the rainfall-runoff model development (Section 6).

The peak flow for the 31 July event was obtained by the 'slope-area' method of estimating flow. The peak water level was marked and surveyed a couple of weeks after the event. The slope-area method allows the flow to be calculated given the stream cross section, slope, peak water level and channel roughness (Manning's n).

GWRC estimate a range of possible peak flows (depending on the channel roughness value used) ranging from 1.8 m^3/s to 5.7 m^3/s. The preferred estimate is 2.5 m^3/s (pers. Comm. J. Marks, GWRC, August 2008).

A more detailed analysis of this flood event is carried out as part of the development of the rainfall-runoff model in Section 6.2.1.

An estimate of peak flow in the Pinehaven Stream resulting from the December 1976 storm exists. Bishop (1997) estimates a flow of 30 m^3/s resulted from an estimated 100 mm of rainfall over three hours, and 180-200 mm over six hours.

It is not known how this peak flow estimate was derived so it must be treated with caution but it was classed as being greater than a 100-year ARI (Bishop 1977).

4 Rainfall Analysis

4.1 Frequency Analysis

Rainfall data from five raingauge sites have been used in frequency analyses to determine high intensity depth-duration frequency totals in and around the Pinehaven catchment. The Event Analysis module within the TIDEDA software was used to carry out the analyses.

Table 4-1 to Table 4-5 detail the depth-duration frequency totals for Tasman Vaccine, Wallaceville, Pinehaven, Perry St and Heretaunga Dam respectively. Results for durations of 30 minutes to 6 hours, and ARI's ranging from 2 to 100-years are presented. The preferred frequency distribution is also shown.

The frequency plots for the analyses and the frequency distributions used are contained in Appendix 1.

The Tasman Vaccine site has the longest available suitable data record (automatic), from 1981 to 2007. Although the Wallaceville site has rainfall data dating back to 1931 it consists of only daily totals until 1985. Therefore, the Wallaceville data used here covers 1986 to 2007. The three UHCC sites – Pinehaven, Perry St and Heretaunga Dam – have suitable data available from October 1998 to the present.

Results from the UHCC raingauges (Table 4-3 to Table 4-5) are similar for durations of 30 minutes to three hours. At the four and six-hour durations the Perry St site displays relatively higher totals for the extreme ARI's.

Table 4-1: Tasman Vaccine Depth Duration Frequency Results (mm) 1981-2007

ARI (Years)	Duration (Hours)					
	0.5	1	2	3	4	6
2	16	22	30	37	42	51
5	22	29	39	47	54	65
10	26	34	44	54	62	75
20	30	39	49	60	70	84
50	35	45	56	69	79	95
100	38	50	61	75	87	104
Distribution	*Gumbel*	*Gumbel*	*Gumbel*	*Gumbel*	*Gumbel*	*Gumbel*

Table 4-2: Wallaceville Depth Duration Frequency Results (mm) 1986-2007

ARI (Years)	Duration (Hours)					
	0.5	1	2	3	4	6
2	12	18	27	33	39	48
5	15	23	33	41	47	60
10	17	26	37	46	53	66
20	18	29	40	50	58	70
50	20	32	44	55	65	76
100	21	34	47	59	69	80
Distribution	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*

Table 4-3: Pinehaven UHCC Depth Duration Frequency Results (mm) 1998-2007

ARI (Years)	Duration (Hours)					
	0.5	1	2	3	4	6
2	12	16	23	30	35	43
5	14	19	29	38	45	53
10	16	23	34	44	51	60
20	18	26	39	50	57	67
50	20	30	46	57	64	76
100	21	33	51	62	70	83
Distribution	*Gumbel*	*PE3*	*PE3*	*PE3*	*Gumbel*	*Gumbel*

Table 4-4: Perry St UHCC Depth Duration Frequency Results (mm) 1998-2007

ARI (Years)	Duration (Hours)					
	0.5	1	2	3	4	6
2	12	17	25	31	35	42
5	14	21	33	41	48	60
10	15	24	38	48	57	72
20	17	26	43	55	65	84
50	18	29	50	64	76	99
100	19	31	55	70	84	111
Distribution	*Gumbel*	*Gumbel*	*Gumbel*	*Gumbel*	*Gumbel*	*Gumbel*

Table 4-5: Heretaunga UHCC Depth Duration Frequency Results (mm) 1998-2007

ARI (Years)	Duration (Hours)					
	0.5	**1**	**2**	**3**	**4**	**6**
2	11	16	25	32	40	49
5	14	20	32	42	52	66
10	15	22	37	48	59	75
20	17	25	41	54	64	82
50	19	28	47	61	71	91
100	20	30	51	67	75	97
Distribution	*Gumbel*	*Gumbel*	*Gumbel*	*Gumbel*	*PE3*	*PE3*

A check of the recorded data shows a high intensity rainfall event on 5 January 2005 that registered at all raingauges but showed much higher totals at the Perry St gauge. Due to the short length of data at the UHCC sites a difference in rainfall totals such as that will cause the frequency analysis results to differ between sites.

The Pinehaven and Wallaceville results are similar. The Tasman Vaccine results are higher and this is to be expected given its location and higher mean annual rainfall.

4.2 Rainfall Distribution

Although the Pinehaven catchment is relatively small there will be a degree of rainfall spatial distribution across it during a high intensity rainfall event.

Mean annual rainfall isohyets are shown for the local area in Figure 4-1. There is a rainfall gradient from the bottom of the Pinehaven catchment and the Wallaceville raingauge up through the catchment to the Tasman Vaccine raingauge. Table 3-1 also shows this with a mean annual rainfall total of 1300 mm at Wallaceville and 1525 mm at Tasman Vaccine.

It is recommended that the Tasman Vaccine results (Table 4-1) be used to represent the rainfall in the upper Pinehaven catchment, and that the Pinehaven/Wallaceville data (

Table 4-2/Table 4-3) is representative of the lower catchment.

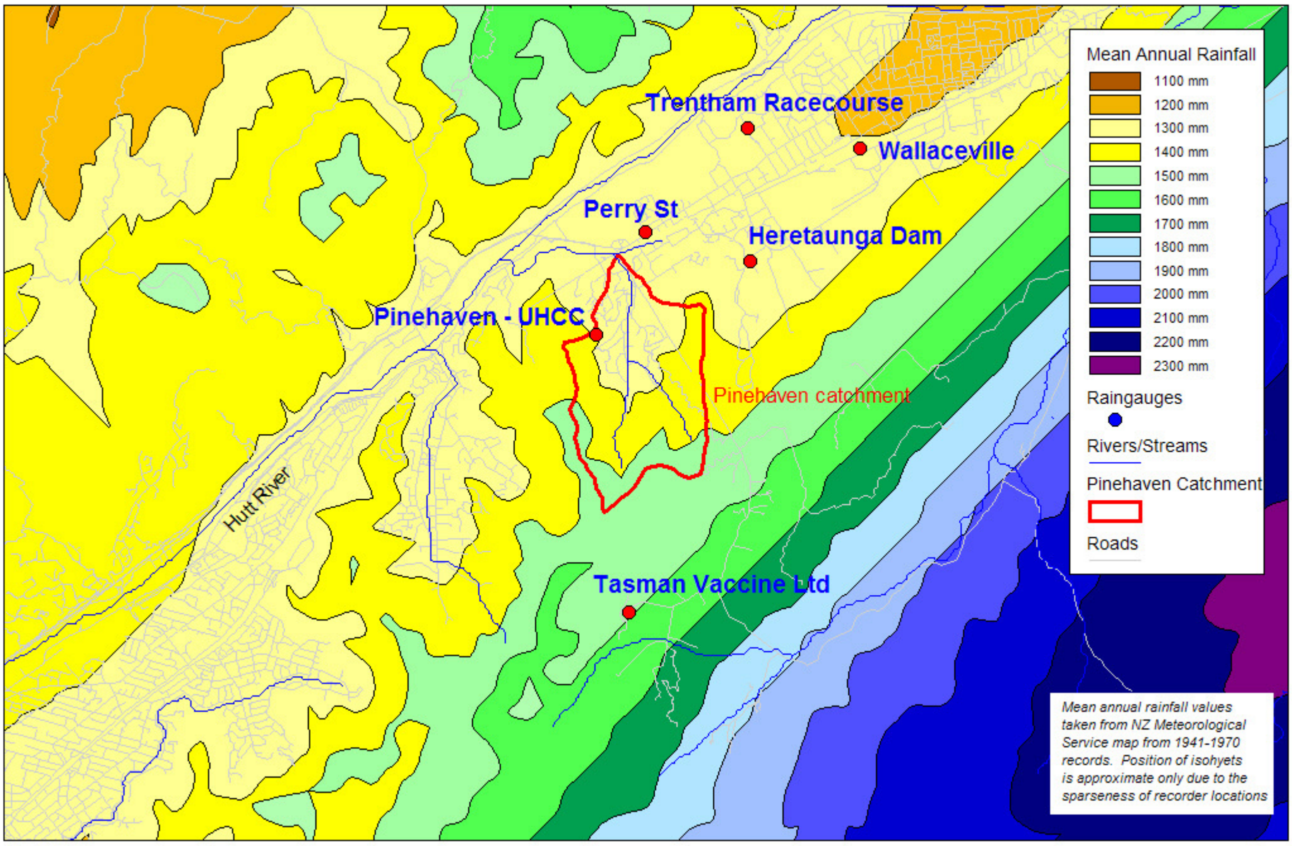

Figure 4-1: Mean Annual Rainfall and Raingauge Locations

4.3 Rainfall Temporal Pattern

Rainfall events tend to exhibit particular temporal characteristics. For example, tropical cyclones tend to distribute rainfall fairly uniformly; severe storms tend to have single peak intensities; while high frequency storms may have several intense rainfall peaks. Thompson and Tomlinson (1992) developed average temporal patterns of rainfall distribution throughout New Zealand based on many recorded storm events which can be used for design rainfalls. An implicit assumption is that design rainfalls of a given return period induce floods of the same frequency and magnitude.

The temporal distribution applied to the design rainfall totals for the Pinehaven catchment is based on the average accumulation during 17 storm events in the Wellington region.

Figure 4-2 details the temporal pattern by the percentage of the rainfall total against the percentage of the rainfall event duration.

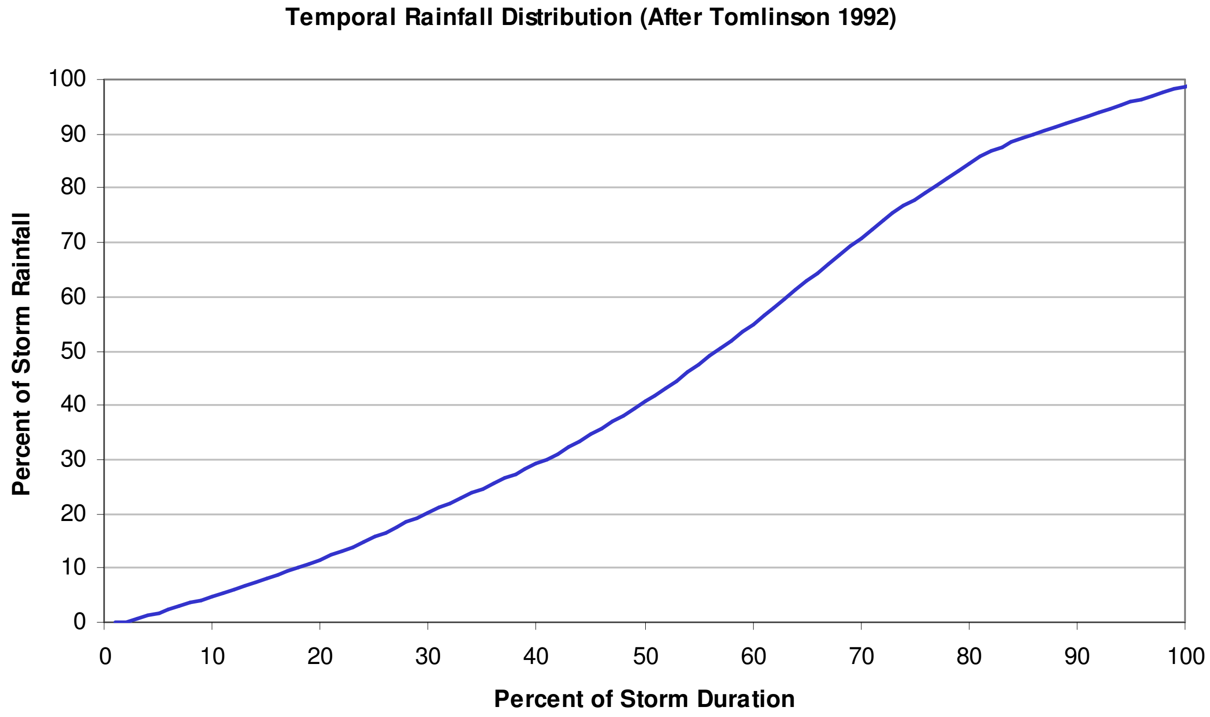

Figure 4-2: Design Rainfall Temporal Distribution

4.4 Probable Maximum Precipitation

Probable Maximum Precipitation (PMP) is defined by the World Meteorological Organisation as the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of year. The PMP is an estimate of an upper physical bound to the precipitation that the atmosphere can produce. An estimate of the PMP for the Pinehaven catchment is required so that the probable maximum flood (PMF) can be modelled.

Two PMP methodologies are defined by Thompson and Tomlinson (1993) to estimate PMP in New Zealand. The method of estimating PMP for small areas (less than 1000 km^2) and short durations up to six hours is used here for the Pinehaven catchment.

The 6:1-hour ratio of 3.5 was chosen as this is close to the ratio of 3.62 observed during the December 1976 storm in the Hutt Valley (Watts, 2005).

The PMP results are summarised in Table 4-6.

Table 4-6: PMP Calculation for Pinehaven Catchment

Step	Description	Value						
1	Catchment Area (km^2)	4.4						
	Maximum Altitude (m)	380						
	Mean Altitude (m)	190						
2	Reference 1-Hour Catchment PMP (mm)	202.3						
3	Adjustment for Location (%)	88%						
4	Adjustment for Altitude (%)	100%						
5	Catchment Average 1-Hour PMP (mm)	177						
		Duration (Hours)						
		0.5	**1**	**2**	**3**	**4**	**5**	**6**
6	Depth-duration adjustments (% of 1 hr)	62	100	162	215	263	307	350
7	Catchment PMP (mm)	**110**	**177**	**287**	**381**	**466**	**543**	**620**

5 Flow Analysis

No recorded flow data exists within the Pinehaven catchment. Therefore flood frequency estimates were derived using regional methods and the rational method.

Advice from GWRC (pers. comm. J. Marks, August 2008) is that a flow recorder has been installed in the Pinehaven Stream near Chatsworth Rd since the commencement of this study. The data is unable to provide any useful benefit for this current investigation but it should be continued to be collected and used for any subsequent 2D hydraulic modelling or used to revise this work in the future.

5.1 Regional Flood Frequency Analysis

Two similar applications of regional flood frequency estimation have been used on the Pinehaven catchment:

- *Flood Frequency in New Zealand* (McKerchar & Pearson, 1989) and its subsequent update for the Hutt catchment in *Hutt River Flood Control Scheme Review – Flood Hydrology* (Pearson, 1990)
- *Regional Flood Frequency Analysis for Small New Zealand Basins* (Pearson, 1991)

PMF events cannot be derived using the regional methods.

The analyses are carried out for the Pinehaven Stream at Chatsworth Rd and the catchment area is 4.4 km^2.

5.1.1 Method 1 - McKerchar and Pearson (1989) & Pearson (1990)

The methodology described in *Flood Frequency in New Zealand* (McKerchar & Pearson, 1989) was used to determine peak flows from the mean annual to the 100-year ARI flood event. The regional flood frequency method is widely used throughout New Zealand when no recorded flow data is available and it is based on regionalised actual recorded flood data.

Whereas McKerchar and Pearson (1989) encompassed the entire country, Pearson (1990) focussed on the Hutt River catchment and further defined the mean annual flood statistic ($Q/A^{0.8}$) and flood frequency factor (q_{100}) contour maps for the area.

Table 5-1 details the results of regional flood frequency analyses using the McKerchar & Pearson (1989) methodology for the Pinehaven catchment and the same methodology with the Pearson (1990) updated flood contour maps.

Table 5-1: Regional Flood Frequency Results (m^3/s), Pinehaven Stream at Chatsworth Rd

ARI (Years)	McKerchar & Pearson (1989)		Pearson (1990)	
	Flow (m^3/s)	Standard Error (%)	Flow (m^3/s)	Standard Error (%)
Mean Annual	9.8	±17	9.8	±17
5	13	±17	13	±17
10	16	±20	15	±19
20	19	±22	17	±22
50	22	±26	20	±25
100	25	±28	22	±28

5.1.2 Method 2 - Pearson (1991)

The methodology described in *Regional Flood Frequency Analysis for Small New Zealand Basins* (Pearson, 1991) was used to determine peak flows for the 5-year to 100-year ARI flood events. This regional flood frequency method, while based on many of the same principles as that in Section 5.1.1, is targeted at small catchments (<10km^2).

Following the prescribed procedures in Pearson (1991), the Pinehaven catchment can be classified into two separate flood frequency groups depending on whether rainfall data from Wallaceville (Group 1) or Tasman Vaccine (Group 3) is used to determine the 5-year ARI 24-hour total. Results from both are presented.

Table 5-2 details the results of regional flood frequency analyses using the Pearson (1991) methodology for the Pinehaven catchment.

Table 5-2: Pearson (1991) Regional Flood Frequency Results (m^3/s), Pinehaven at Chatsworth Rd

ARI (Years)	Flow (m^3/s)			
	Flood Frequency Group 1	Flood Frequency Group 3		Weighted Flood Frequency
Mean Annual	9.8	9.8		9.8
5	14	14		14
10	17	19		18
20	22	23		22
50	27	28		28
100	32	32		32

The procedure to derive weighted flood frequency estimates when two or more groups are relevant (Pearson 1991) is followed and the results are details in the fourth column.

5.2 Rational Method

The rational method is a standard method for calculating the peak runoff rate for a given catchment or parcel of land and is commonly used in New Zealand and around the world. The rational method is based on assumptions that the peak runoff at any design location is a function of the average rainfall intensity during the time of concentration to that location, - the time of concentration being the time required for the runoff from the most remote part of the drainage area to flow to the point under design. The rational method is particularly suited to small catchments.

Table 5-3 details the results of applying the rational method to the Pinehaven Stream catchment at Chatsworth Rd. The time of concentration for all catchments is taken as 60 minutes and a runoff coefficient of 0.5 is used. The runoff coefficient was derived from basic GIS analysis of areas of different runoff characteristics within the catchment and weighting them. The catchment area is 4.4 km^2.

Rainfall intensities from the Tasman Vaccine and Wallaceville raingauges are used. The rainfall data from these two sites has been used to interpolate a set of approximate catchment average design rainfall intensities (intermediate rainfall in Table 5-3) for use in the rational method.

Table 5-3: Rational Method Flood Frequency Results (m^3/s), Pinehaven Stream at Chatsworth Rd

ARI (Years)	Tasman Vaccine Rainfall	Wallaceville Rainfall	Intermediate Rainfall
2	13	11	12
5	18	14	15
10	21	16	18
20	24	18	21
50	28	20	23
100	31	21	24
PMF	108	108	108

6 Rainfall-Runoff Modelling

A rainfall-runoff model was developed for the Pinehaven catchment to produce design hydrographs for specified return period (ARI) events from the design rainfall inputs. The model is an Initial Loss - Continuing Loss type and has been built using Hydstra Modelling software. Hydstra Modelling has been used in many hydrological applications in New Zealand and around the world for rainfall-runoff and design flood modelling. It has been used for a number of GWRC flood modelling and flood design investigations.

6.1 Model Configuration

The Pinehaven catchment was divided into nine sub-catchments (P1 to P9) for the rainfall-runoff modelling (Figure 6-1). The most downstream point of the model is opposite Chatsworth Rd before the stream enters into a long culvert. The catchment area to this point is 4.4 km^2.

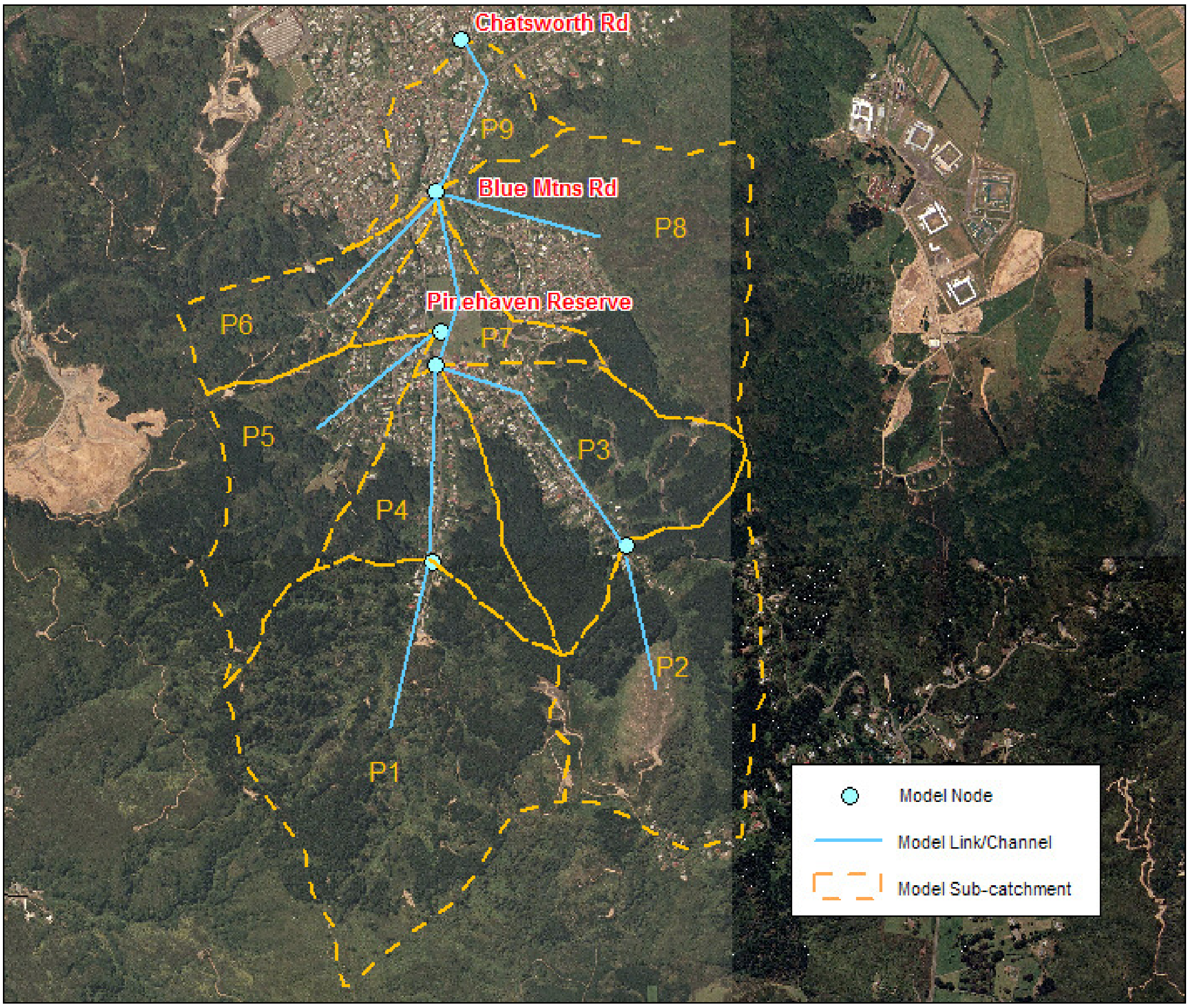

Figure 6-1: Pinehaven Rainfall-Runoff Model Delineation

Sub-catchment delineation and channel lengths were derived from GIS based contour and aerial photograph information, as well as maps presented in the Upper Hutt City Council District Plan (2004).

Figure 6-2 shows a schematic of the rainfall-runoff model. Modelled flow hydrographs are output at Pinehaven Reserve, Blue Mountains Rd, and Chatsworth Rd. However, output can be determined from any of the other nodes if required for hydraulic modelling purposes. Sub-catchments P1 and P2, originating on the higher catchment divides, use Tasman Vaccine rainfall as an input. All other sub-catchments use the Wallaceville/Pinehaven rainfall as input.

The catchment areas draining to the Pinehaven Reserve and Blue Mountains Rd sites are 3 km^2 and 4.2 km^2 respectively.

The model is setup to run at 15 minute intervals.

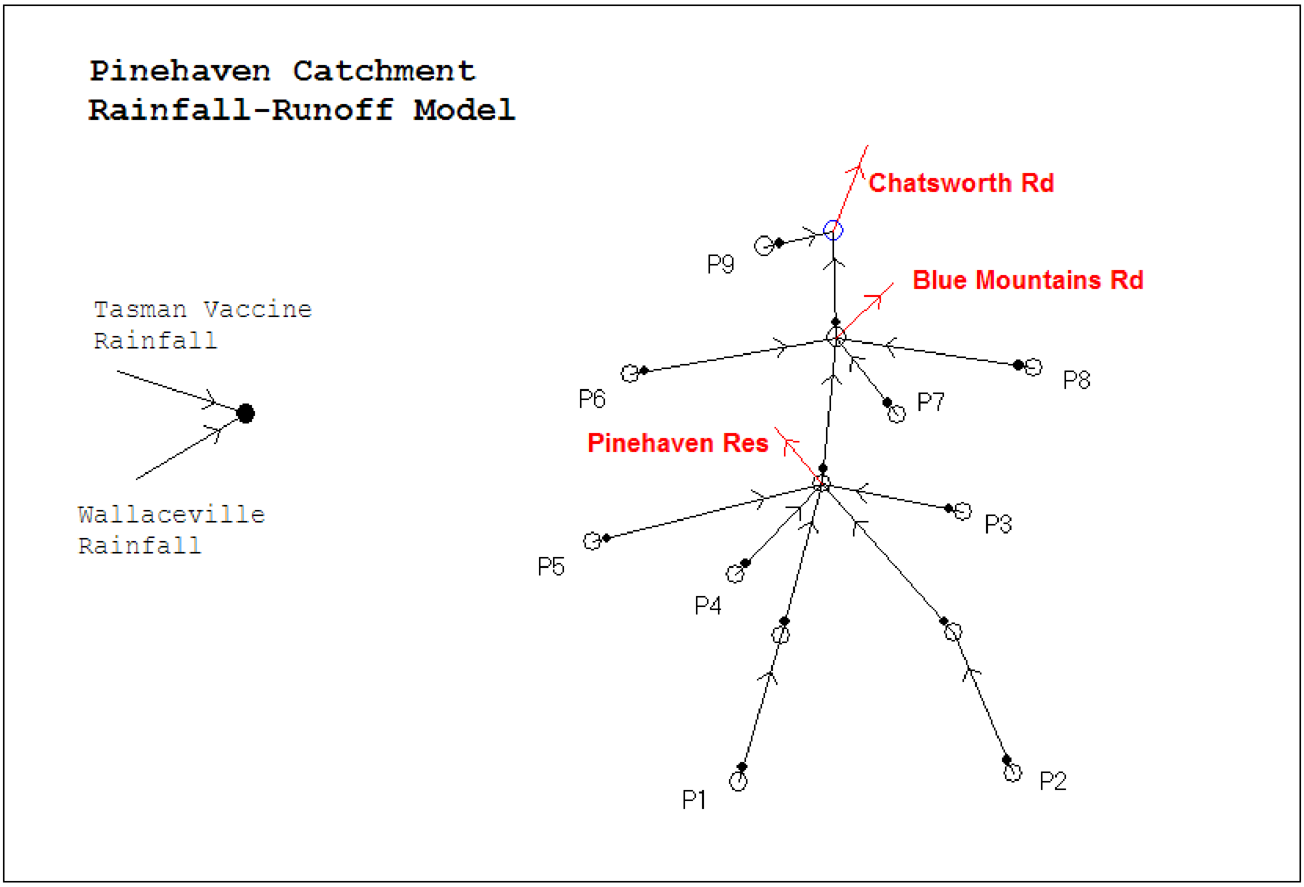

Figure 6-2: Pinehaven Model Schematic

6.2 Model Calibration

At the time of commencing this flood study there was no recorded flood flow data for the Pinehaven catchment. GWRC installed a flow recorder on the Pinehaven Stream at Chatsworth Rd in August 2008 but no suitable data is available for this study.

GWRC hydrology staff pegged out the peak water level of a small flood event that occurred on 31 July 2008. The peak levels, cross sections and slope have been surveyed and used to derive a peak flow estimate (Section 6.2.1) for this event.

This flood estimate is the only calibration data available for the modelling process.

6.2.1 31 July 2008 Flood Estimate

GWRC hydrology staff computed a standard slope-area discharge measurement for the 31 July 2008 event using surveyed peak water levels at the Pinehaven Reserve.

As is often the case with the slope-area method the choice of Manning's n (roughness coefficient) was difficult to accurately assess. Two follow up flow gaugings were carried out at lower flows to assess the actual Manning's n value, however as the flow was contained within the normal stream bed the results are representative of that only and not the banks or berms.

A range of Manning's n values were used that were representative of the channel and berms, or parts thereof. This resulted in a range of peak flow estimates from 1.8 m^3/s (using a Manning's n value of 0.616 - obtained from follow up flow gaugings) to 5.7 m^3/s (using a Manning's n value of 0.02 - taken from literature).

To obtain a reliable peak flow estimate, MWH has used the surveyed cross section and slope data to create a MIKE11 hydraulic model. The Manning's n value is able to be varied across the cross sections within MIKE11 and can therefore replicate the actual channel and berms roughness far more accurately than the slope-area method which can only use a single Manning's n value.

A number of MIKE11 model runs were completed with Manning's n values ranging from 0.06 for the gravel channel (as derived from GWRC follow up flow gaugings) to 0.03 for the grassy berms (derived from literature). A steady-state constant flow was used as the input to the MIKE11 model and numerous iterations carried out by varying this flow input magnitude until the peak modelled water levels approximated those surveyed after the flood event.

The resulting peak flow estimate for the 31 July 2008 event is 2.8 m^3/s.

This is not a major flood event and the rainfall totals suggest it is probably an annual, or possibly more frequent, event. Rainfall totals for the event range from one-third to two-thirds of a 2-year ARI event.

6.2.2 Calibrating Rainfall-Runoff Model to Flood Estimate

The rainfall-runoff model was calibrated using the 31 July 2008 peak flow estimate and recorded rainfall data from the Tasman Vaccine and Wallaceville raingauges for that event.

Table 6-1 details the parameters used in the rainfall-runoff model. The initial loss (IL), continuing loss (CL), α and n parameters are adjusted during the calibration process. These values are "global" and are therefore assumed constant over the catchment.

Table 6-1: Calibration Parameters for Rainfall-Runoff Model

Parameter	Description
Initial Loss (IL) - mm	Amount of water lost before rainfall becomes effective runoff
Continuing Loss (CL) - mm/hr	Continuing loss rate applied to rainfall after IL is satisfied
α	Channel lag parameter for channel routing
n	Non-linearity parameter for channel routing
Area - km^2	Sub-catchment area
L - km	Channel length

The calibration process consisted of using the recorded rainfall data and varying the IL, CL, α and n parameters.

The calibration was carried out by a visual assessment of the magnitude and shape of the hydrograph at the Pinehaven Reserve site as compared to the 31 July 2008 flood estimate. Only the magnitude of the modelled peak can be compared directly. Hydrological practice and judgement was used to ensure a valid hydrograph shape was produced.

Figure 6-3 shows the final calibration hydrograph produced for the 31 July 2008 event from the Hydstra Modelling software. The y-axis flow units are m^3/s and the time interval (x-axis) is 15 minutes. The red line is the estimated 2.8 m^3/s magnitude of the estimated flood peak. Recorded rainfall at Tasman Vaccine and Wallaceville is shown along the bottom axis.

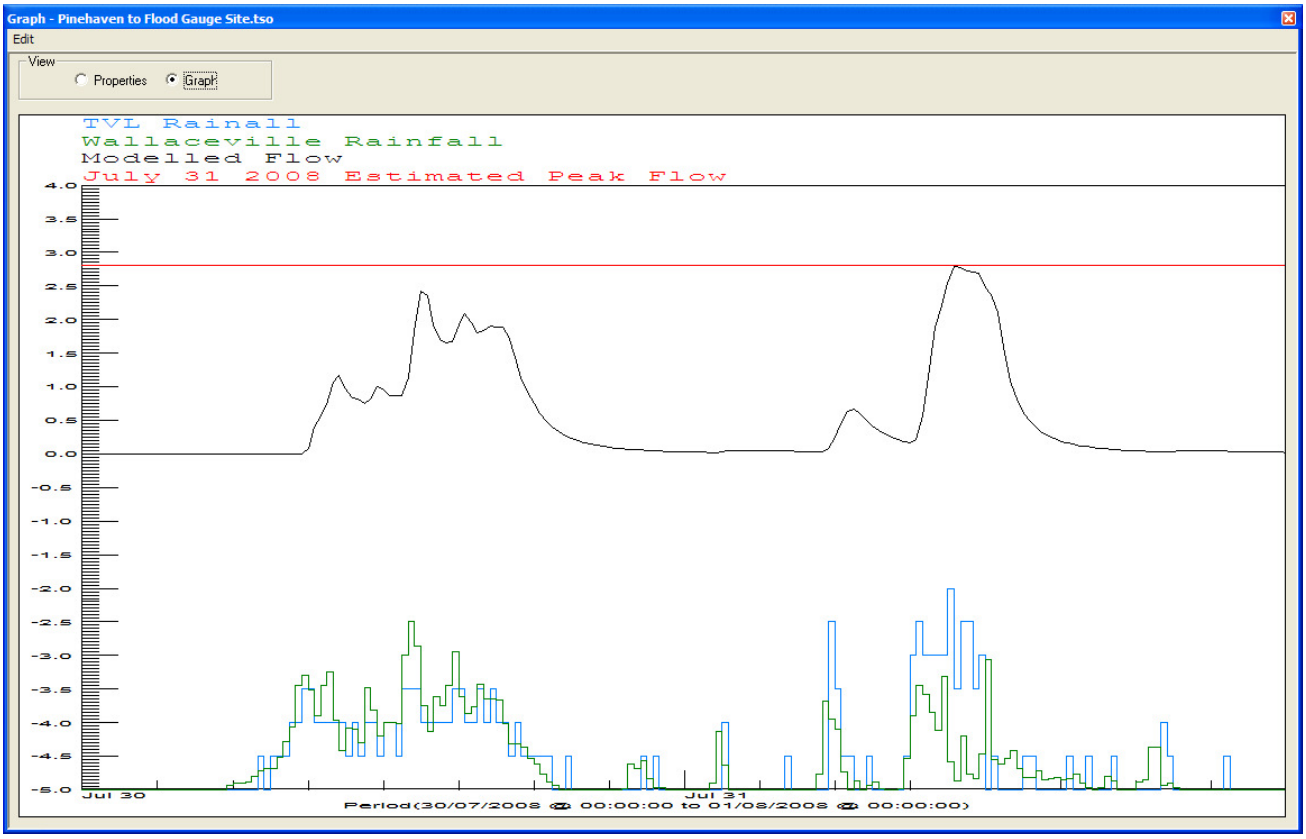

Figure 6-3: 31 July 2008 Calibration Event at Pinehaven Reserve (Hydstra Modelling output)

Table 6-2 details the final model parameters adopted.

Table 6-2: Pinehaven Stream Model Calibration Results

Parameter	Best Fit Value
Initial Loss (IL)	5 mm
Continuing Loss (CL)	2 mm
α	0.9
n	0.72

Comparison was made to other nearby flood modelling results such as the Mangaroa River (Watts, 2005), Waiwhetu Stream (Watts, 2004), Mangaone Stream (MWH, 2002) and Karituwhenua Dams (MWH, 2008) to ensure values for the model parameters were realistic.

The channel lag component (α) is considered reasonable due to the way the Pinehaven Stream flows through many residential property back yards, under bridges and through culverts on its way down catchment.

The calibration flood event of 2.8 m³/s on 31 July is not a major flood event. It is estimated to be an annual event at the most. There are uncertainties in calibrating a rainfall-runoff model to just one single peak value that is not a major flood peak. Ideally a number of recorded flood hydrographs would be available for calibration to provide confidence in the modelled peak flow estimates and hydrographs shapes.

However, it is better to have the one peak flow estimate to calibrate the model to than nothing at all.

6.3 Design Flood Hydrographs

The design rainfall events for the 2 to 100-year ARI and PMP were input to the rainfall-runoff model to produce design flood hydrographs throughout the catchment.

Figure 6-4 shows the 100-year ARI design hydrographs for Pinehaven Reserve and Chatsworth Rd. The one and two-hour events for each are given.

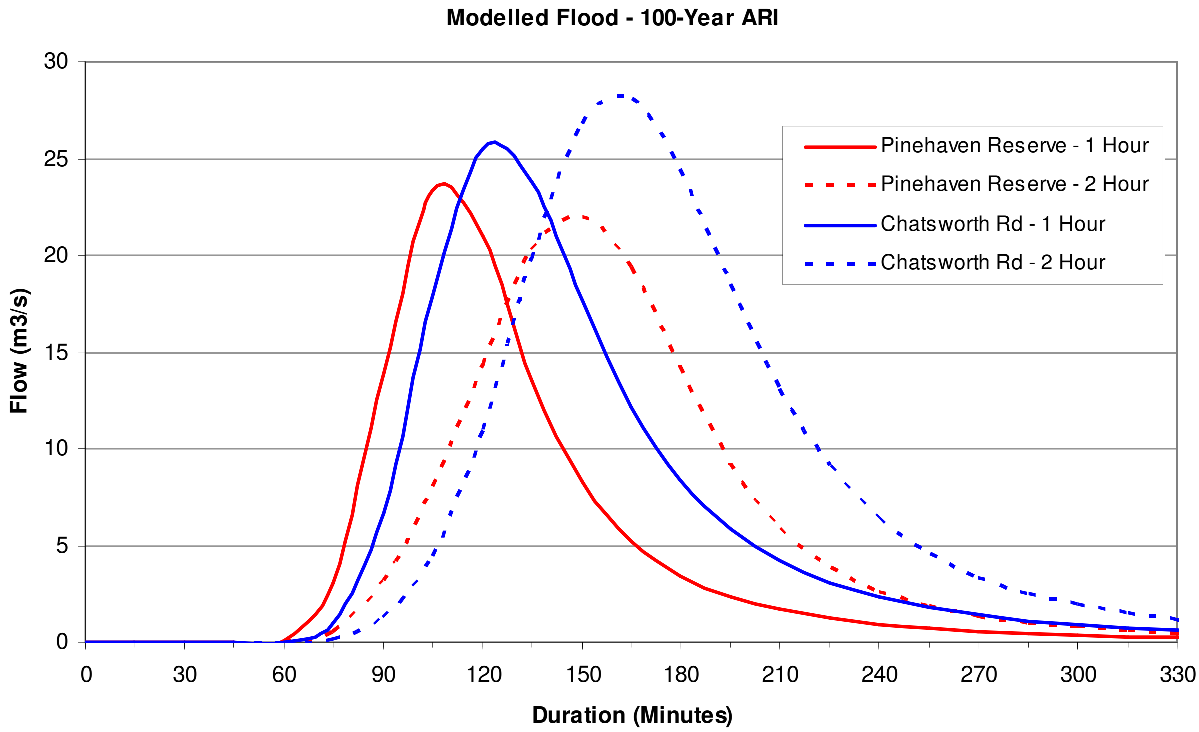

Figure 6-4: Design 100-year ARI Hydrographs

The critical duration 100-year ARI flood event is a one hour duration storm for Pinehaven Reserve and a two hour duration for Chatsworth Rd.

Table 6-3 details the results for all durations and ARI magnitudes for the Pinehaven Stream at Chatsworth Rd. The critical duration event for each ARI is highlighted.

The 100-year ARI modelled design peak flow for the Pinehaven Stream at Chatsworth Rd is 28 m³/s.

Table 6-3: Modelled Design Peak Flows for Pinehaven Stream at Chatsworth Rd (m^3/s)

ARI (Years)	Duration (Hours)					
	0.5	1	2	3	4	6
2	4	7	11	14	14	10
5	6	12	15	15	14	13
10	9	14	18	18	17	15
20	11	19	22	22	21	17
50	13	22	25	25	24	20
100	15	26	28	28	26	21
PMF	124	189	207	199	190	174

6.4 Rainfall-Runoff Model Limitations

The major limitation of the rainfall-runoff modelling process for the Pinehaven Stream is the lack of calibration data. Although a single calibration point was available, it was a relatively minor flood event. The use of the model to simulate extreme flood events will therefore carry relatively high uncertainties. This uncertainty is reduced by comparing modelled output with peak estimates from other methods as summarised in Section 7.

A number of recorded flood hydrographs is preferred for calibration purposes to ensure estimates of peak flows and hydrograph shape are as accurate as possible.

It is recommended that GWRC make use of data from its recently installed flow recorder on the Pinehaven Stream and check/re-calibrate the rainfall-runoff model after a number of years or flood events have been recorded.

7 Summary of Flood Estimates

A summary of the derived flood estimates for the Pinehaven Stream is detailed in Table 7-1.

Table 7-1: Pinehaven Stream at Chatsworth Rd Flood Estimates (m^3/s)

ARI (Years)	Regional Methods			Rational Method	Rainfall-Runoff Model
	McKerchar and Pearson (1989)	Pearson (1990)	Pearson (1991)		
2	9.8	9.8	9.8	12	14
5	13	13	14	15	15
10	16	15	18	18	18
20	19	17	22	21	22
50	22	20	28	23	25
100	25	22	32	24	28
PMF	-	-	-	108	207

The first three result columns detail the estimates derived from the regional flood frequency methods. Of the three methods, Pearson's (1991) method can be considered as the preferred regional method as it is directly applicable to small catchments less than 10 km^2 in area. The Pinehaven catchment is 4.4 km^2 to the Chatsworth Rd site.

The rainfall-runoff model results are similar to Pearson (1991) for all but the 2-year ARI event. It is the extreme ARI events that will be used for design purposes so the slightly conservative model results for lower ARI events are acceptable.

The rainfall-runoff results are also similar to those derived using the rational method.

7.1 Comparison to 1976 Peak Flow Estimate

As described in Section 3.2, an estimate of the flood magnitude in the Pinehaven Stream during the December 1976 extreme storm was presented by Bishop (1997). The peak flow was estimated at 30 m^3/s and this was assigned an ARI of greater than 100-years, matching the estimated ARI of the rainfall event.

Although the exact method of derivation of this estimate is not known it provides a reference point for the results presented here, and is in fact similar to the 100-year ARI estimates as shown in Table 7-1.

8 Recommended Design Flood Estimates

Based on the results obtained in this investigation it is recommended that the rainfall-runoff model results and hydrographs be adopted for design flood purposes.

Table 8-1 presents the recommended design flood estimates for the Pinehaven Stream at Chatsworth Rd.

Table 8-1: Pinehaven Stream Design Flood Estimates

ARI (Years)	Flow (m^3/s)
2	14
5	15
10	18
20	22
50	25
100	28
PMF	207

9 Further Work

Climate change scenarios should be investigated and design rainfall estimates for 2030 and 2080 derived. These can be input to the rainfall-runoff model to provide 'future proofed' hydrographs for the Pinehaven Stream.

The GWRC water level/flow recorder recently installed in the Pinehaven Stream should continue to be operated. While it was too late to be of benefit to this study, the data collected will provide vital information for any subsequent 2D hydraulic modelling.

The recorded flow data will also be useful to review and revise the findings presented here as well as providing further calibration data for the rainfall-runoff model after a period of time or sizable flood events.

It may be of interest to extract rainfall data for the December 1976 storm event and apply these to the rainfall-runoff model and derive a peak flow estimate.

10 References

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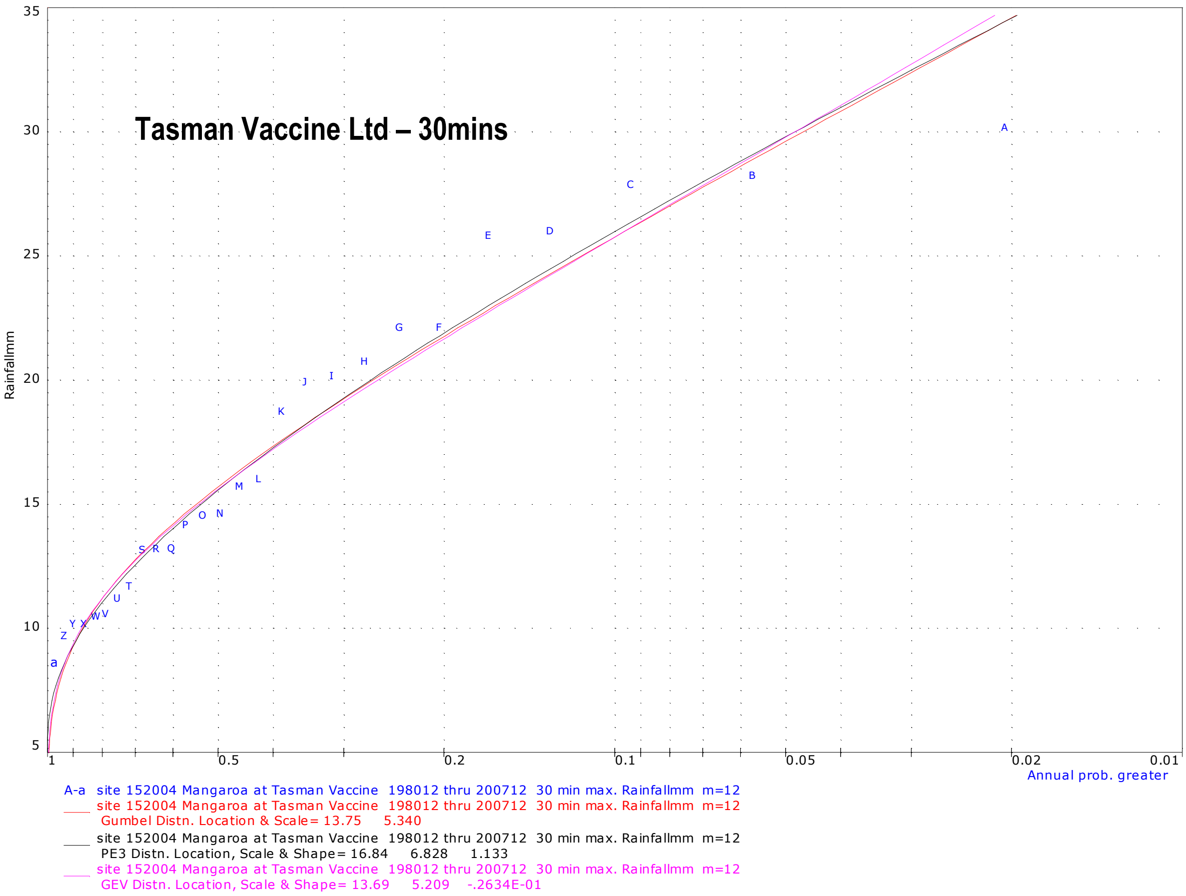
Appendix A Rainfall Frequency Analysis Plots



Tasman Vaccine Ltd – 30mins
Rainfallmm
Annual prob. greater
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PE3 Distn. Location, Scale & Shape= 16.84 6.828 1.133
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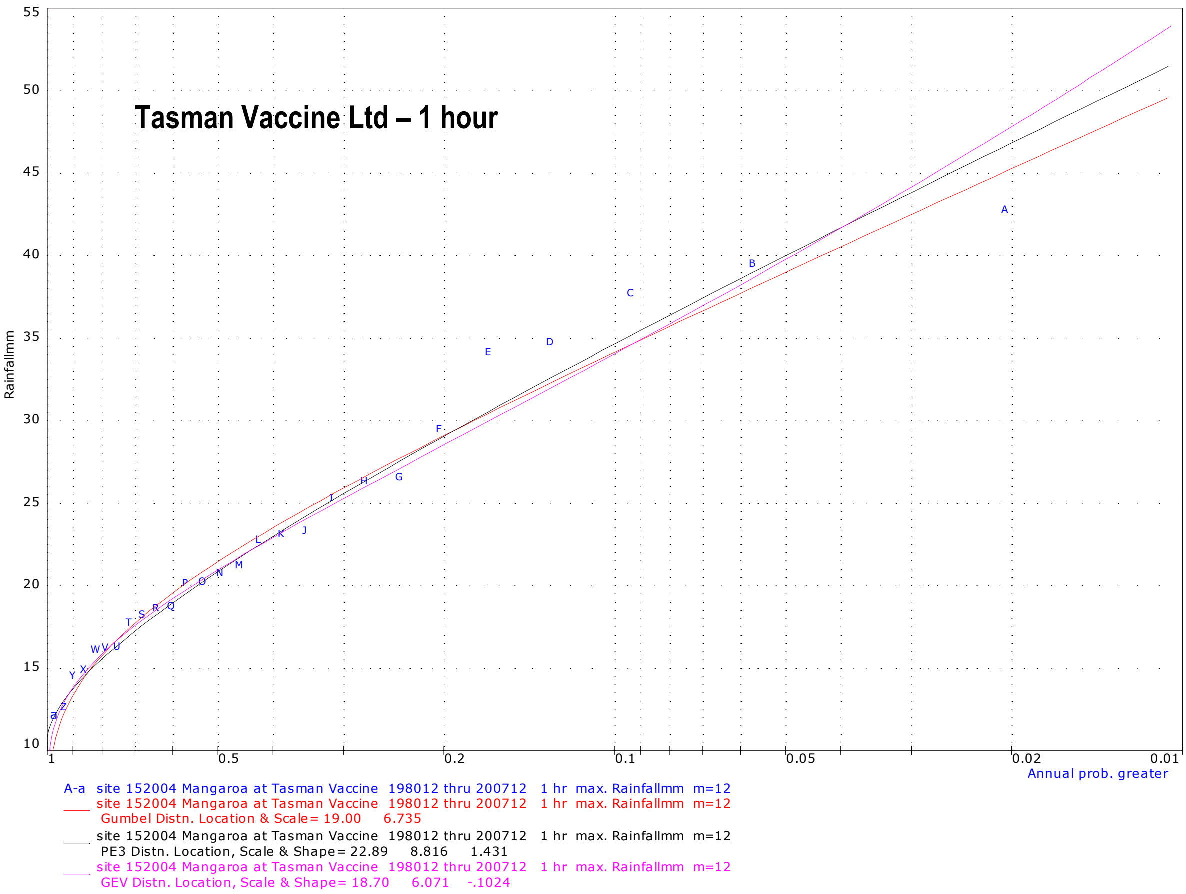

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Rainfallmm
Annual prob. greater
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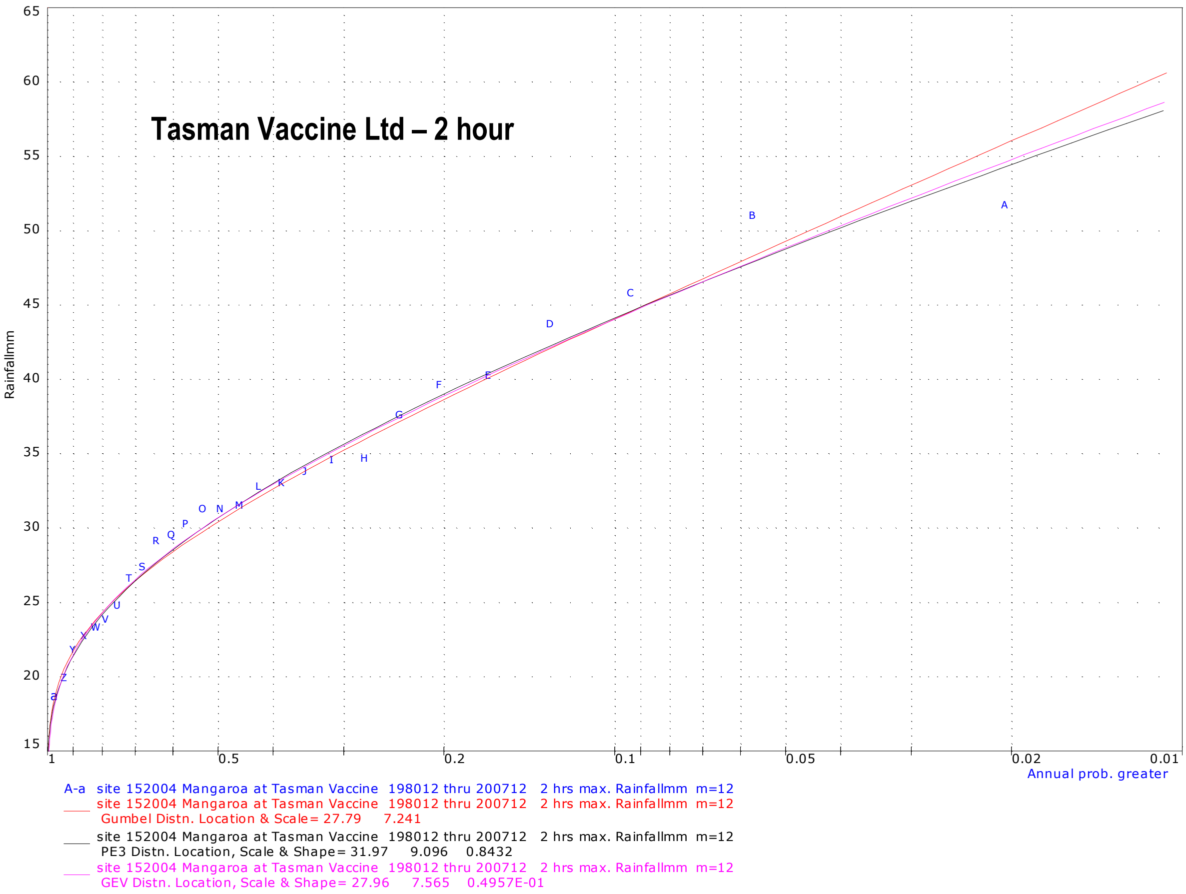
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Rainfallmm
Annual prob. greater
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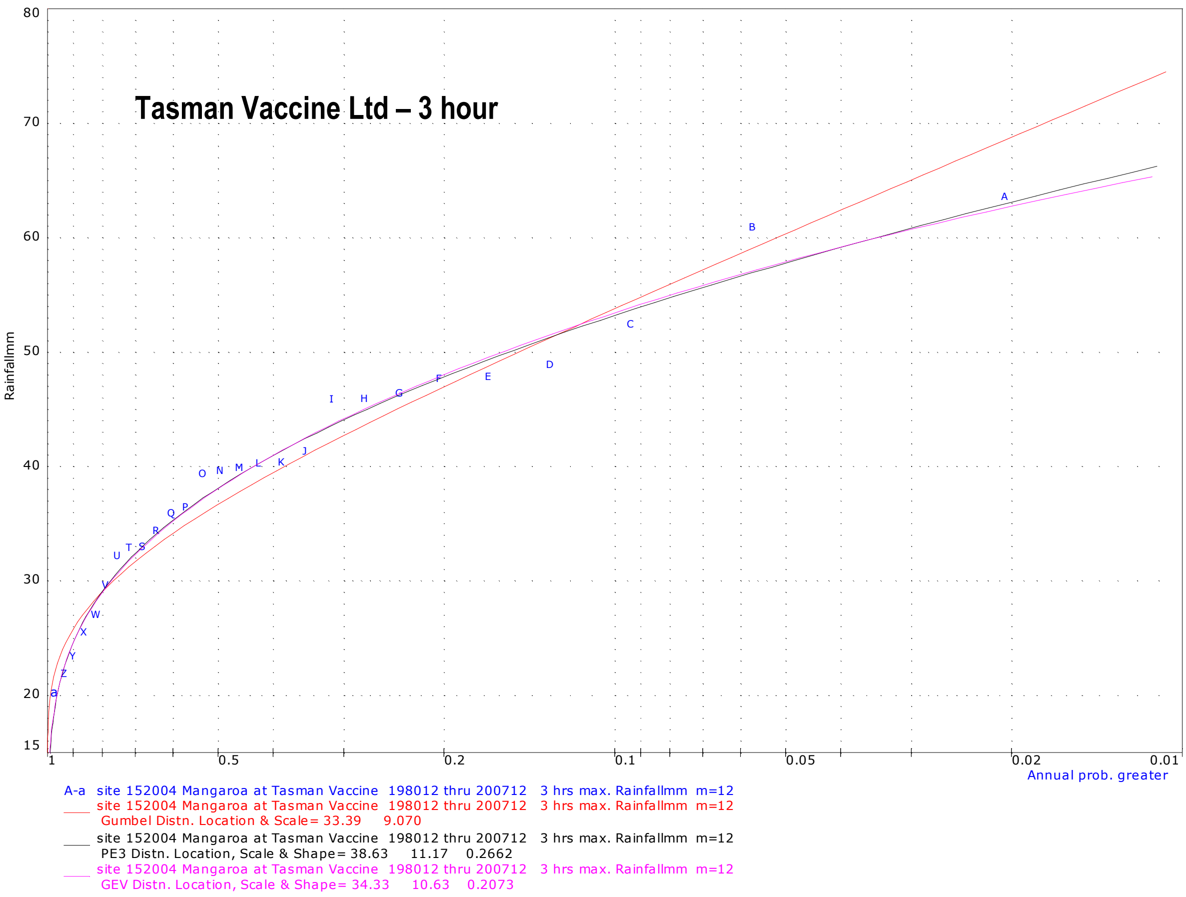
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Rainfallmm
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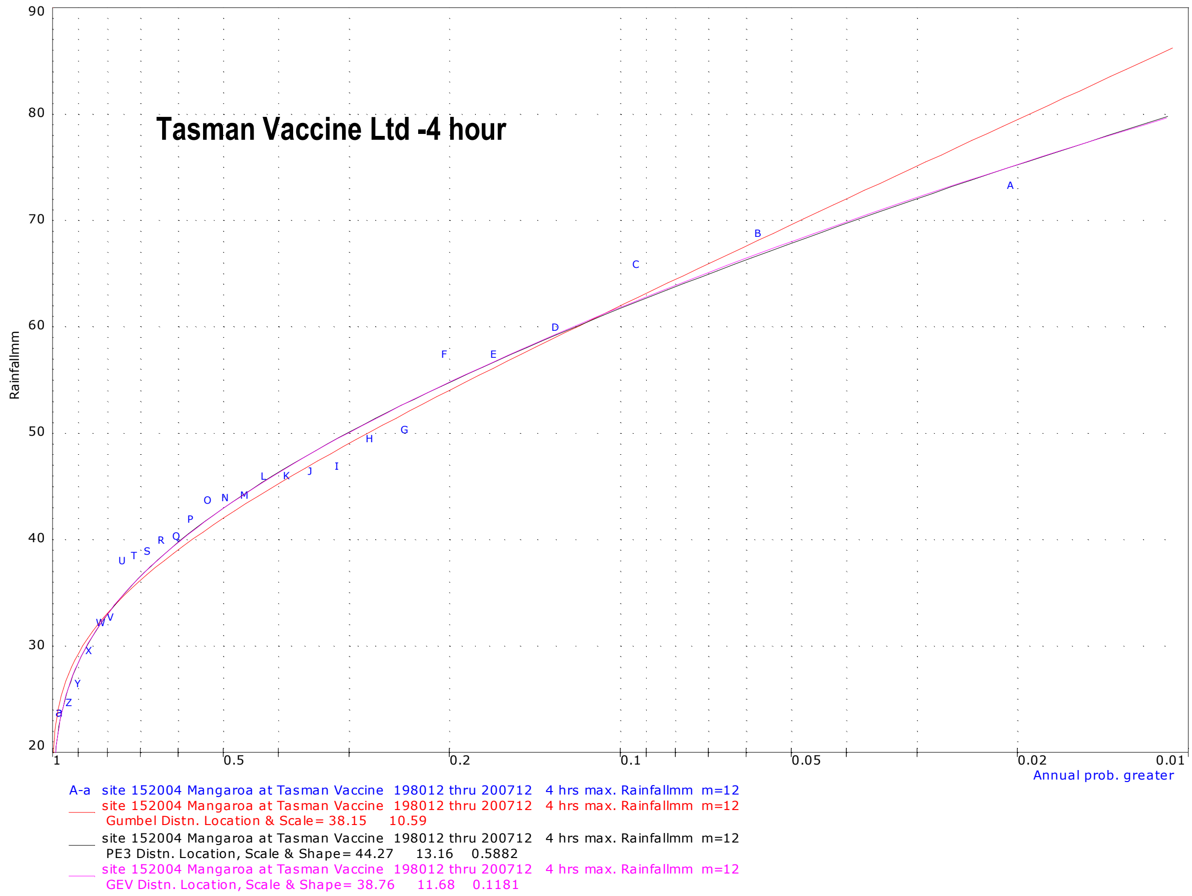
Tasman Vaccine Ltd -4 hour
Rainfallmm
Annual prob. greater
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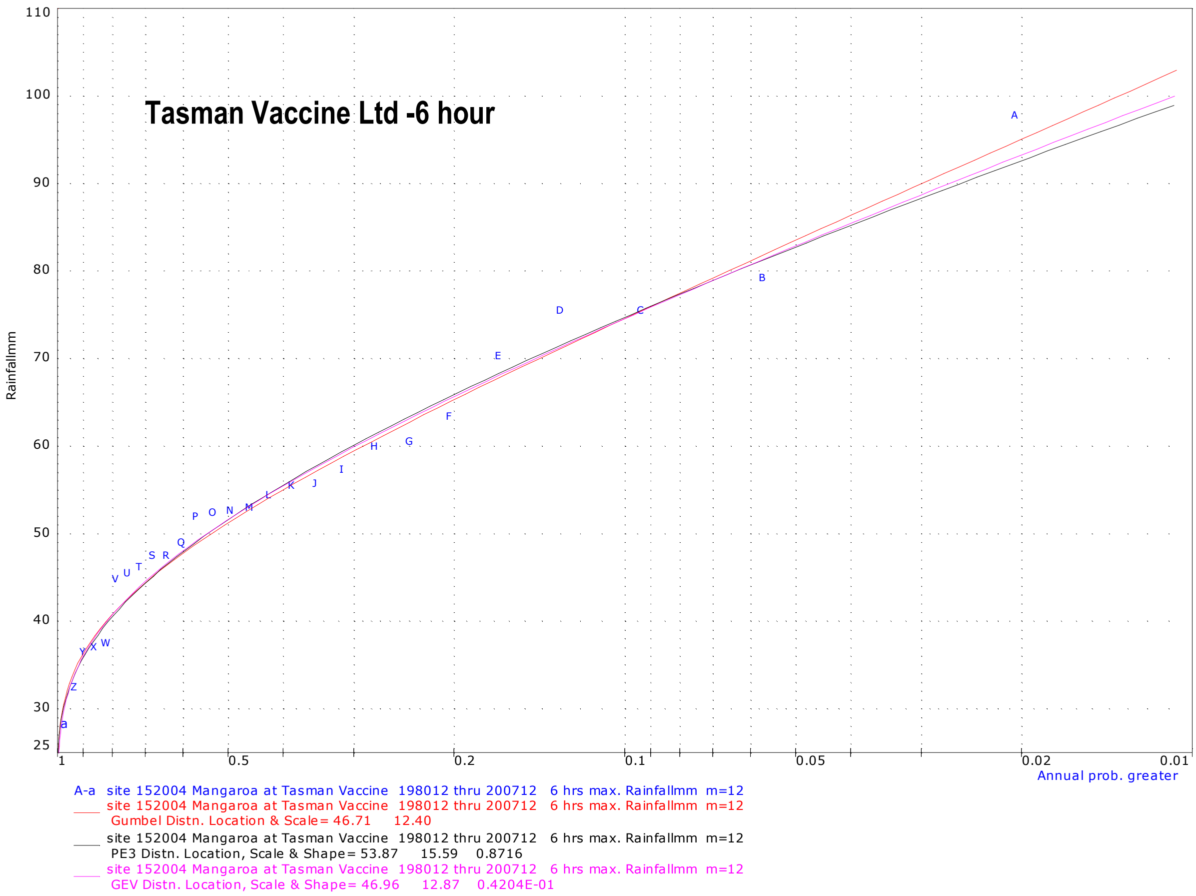
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Rainfallmm
Annual prob. greater
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Gumbel Distn. Location & Scale= 46.71 12.40
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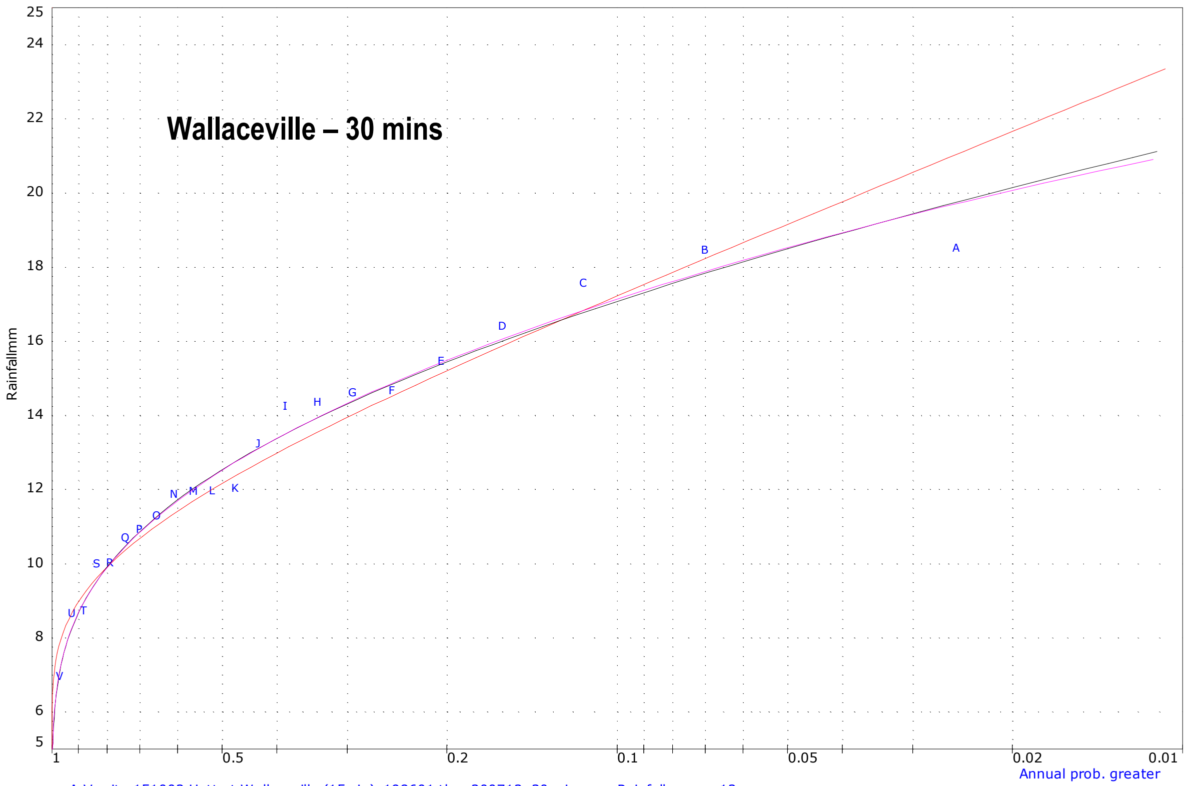

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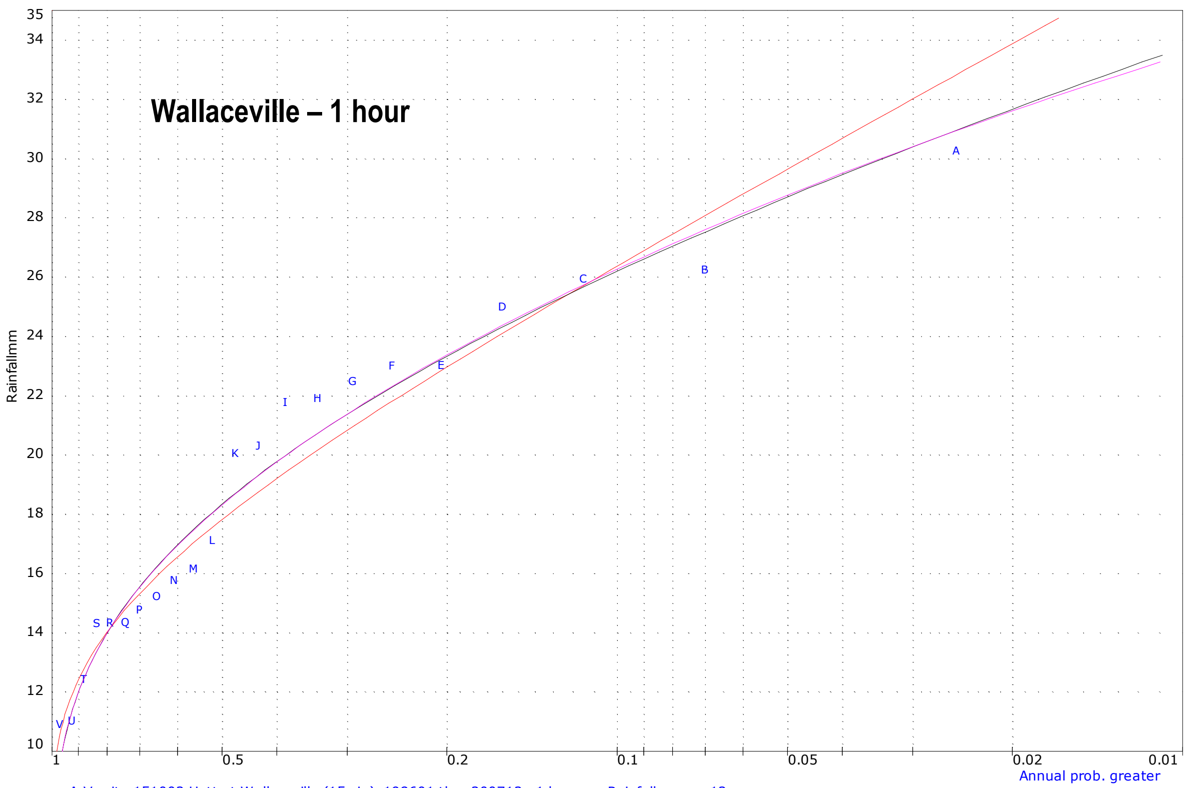

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Gumbel Distn. Location & Scale= 16.16 4.542
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PE3 Distn. Location, Scale & Shape= 18.78 5.618 0.4675
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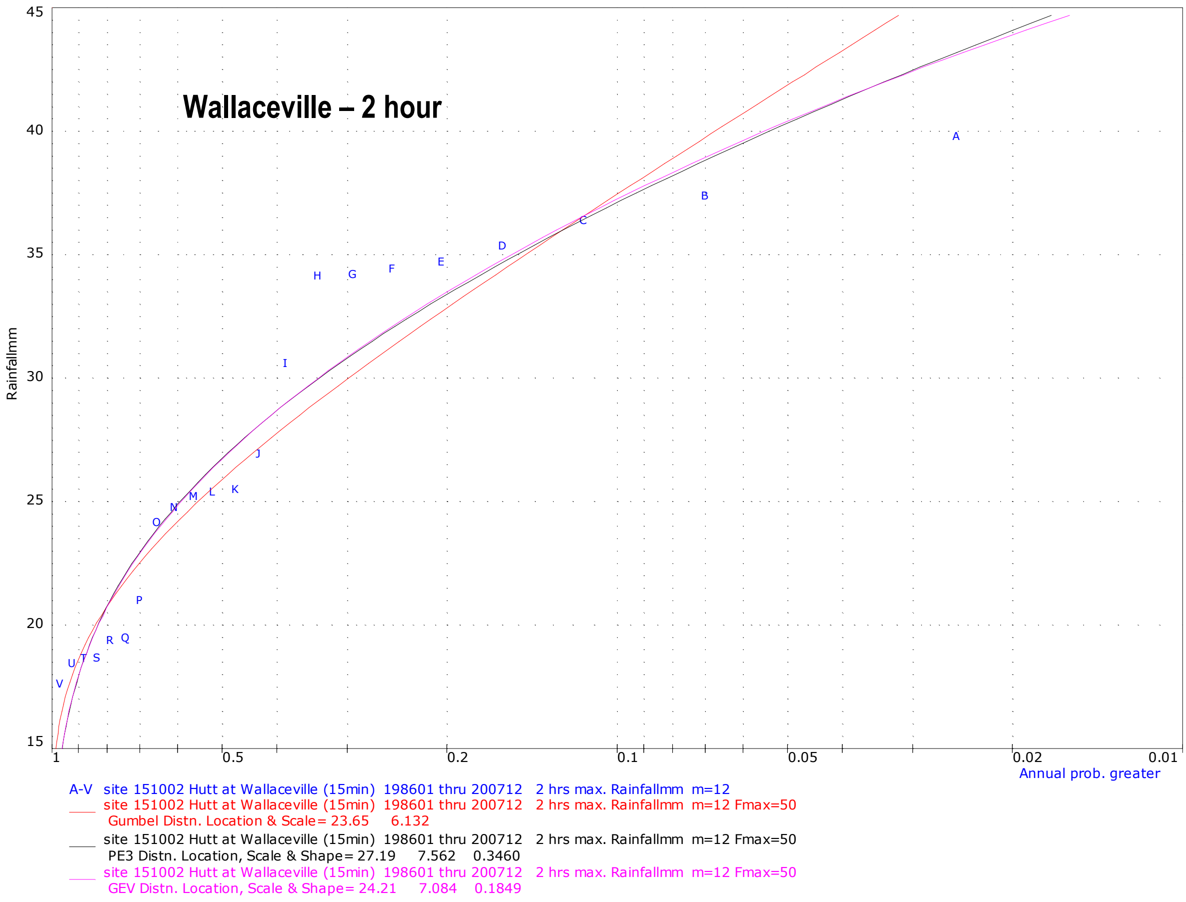

Wallaceville – 2 hour
Rainfallmm
Annual prob. greater
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site 151002 Hutt at Wallaceville (15min) 198601 thru 200712 2 hrs max. Rainfallmm m=12 Fmax=50
Gumbel Distn. Location & Scale= 23.65 6.132
site 151002 Hutt at Wallaceville (15min) 198601 thru 200712 2 hrs max. Rainfallmm m=12 Fmax=50
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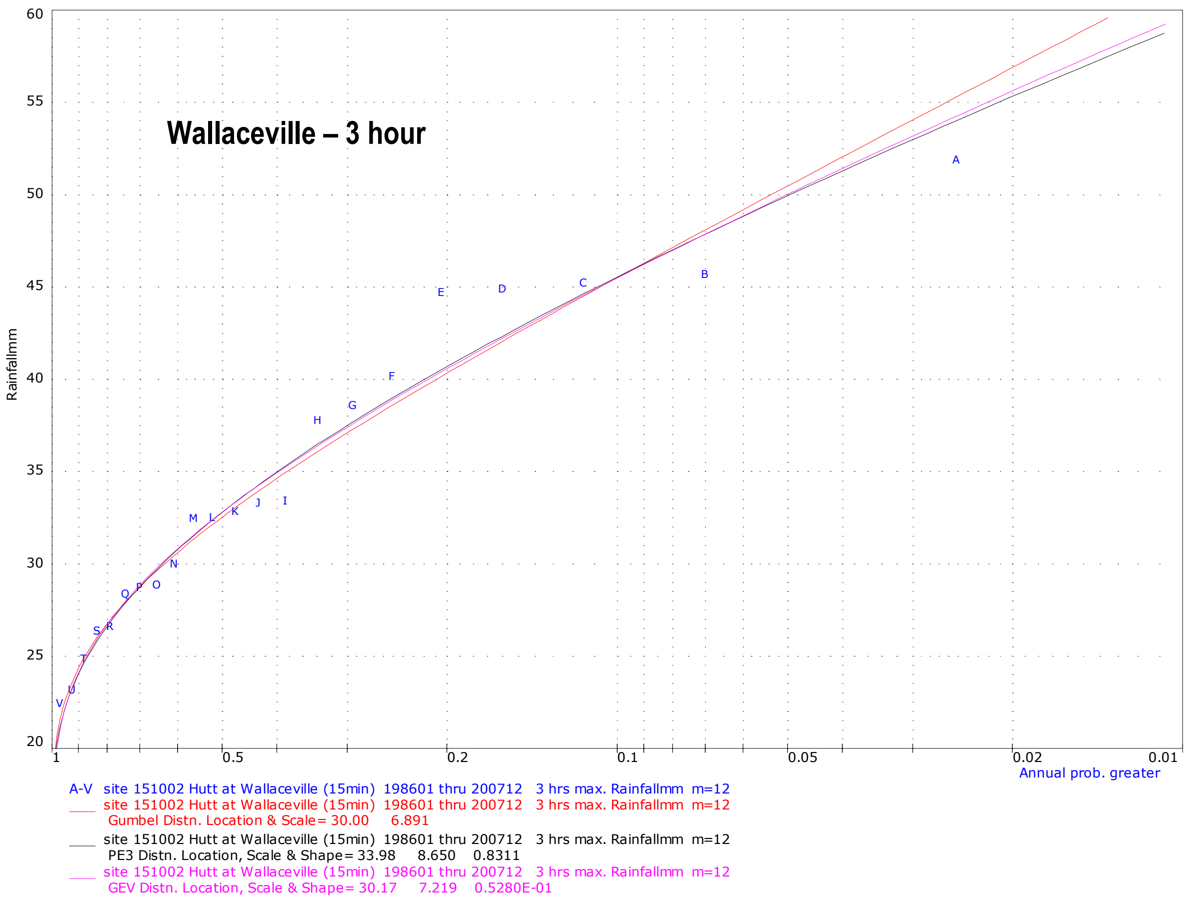

Wallaceville – 3 hour
Rainfallmm
Annual prob. greater
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Gumbel Distn. Location & Scale= 30.00 6.891
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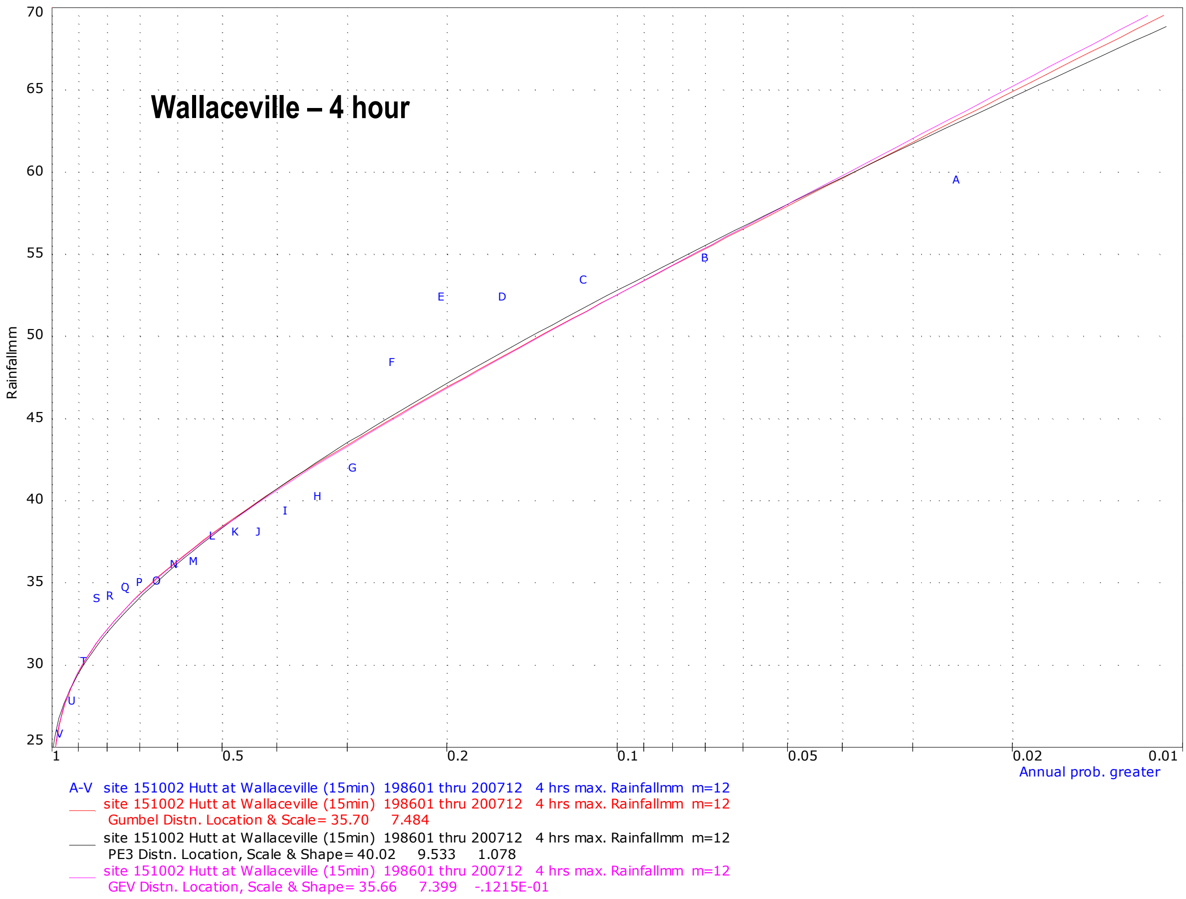
Wallaceville – 4 hour
Rainfallmm
Annual prob. greater
A-V site 151002 Hutt at Wallaceville (15min) 198601 thru 200712 4 hrs max. Rainfallmm m=12
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Gumbel Distn. Location & Scale= 35.70 7.484
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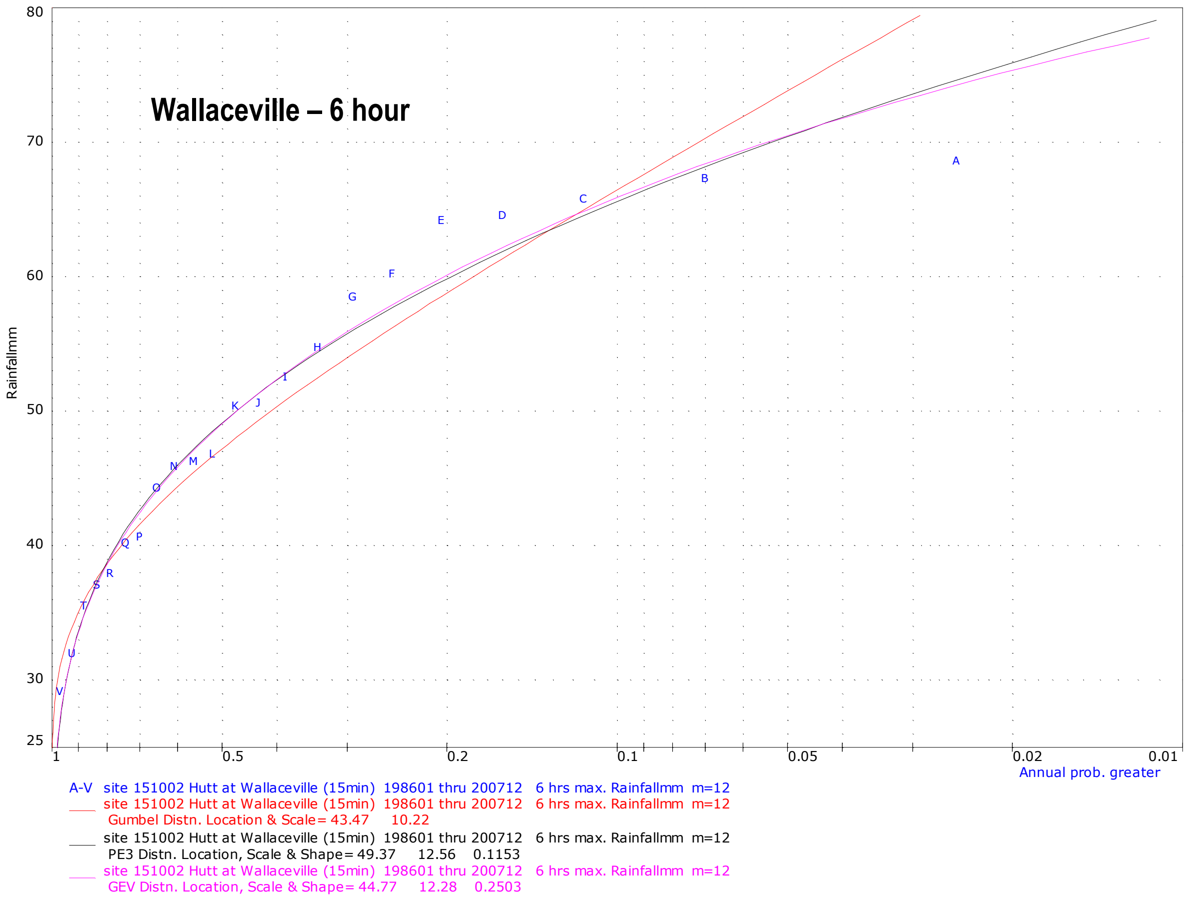
Wallaceville – 6 hour
Rainfallmm
Annual prob. greater
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PE3 Distn. Location, Scale & Shape= 49.37 12.56 0.1153
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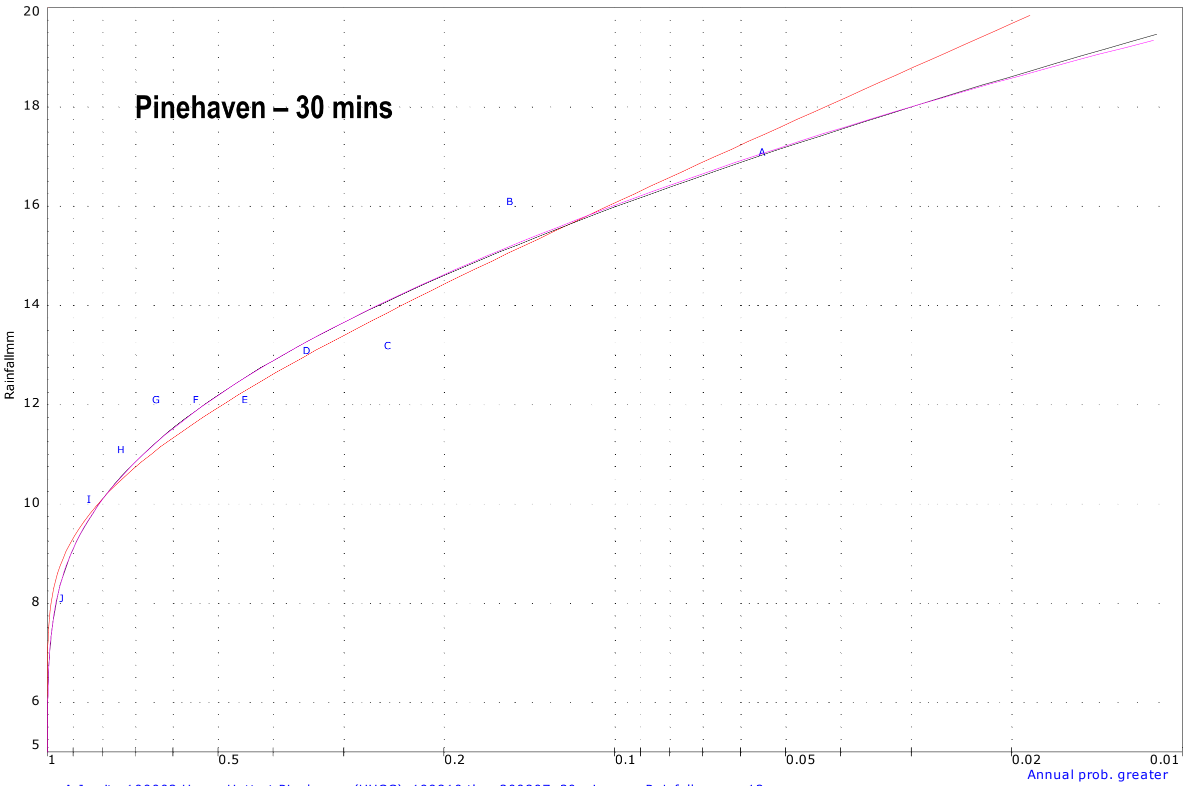

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PE3 Distn. Location, Scale & Shape= 12.41 2.707 0.4650
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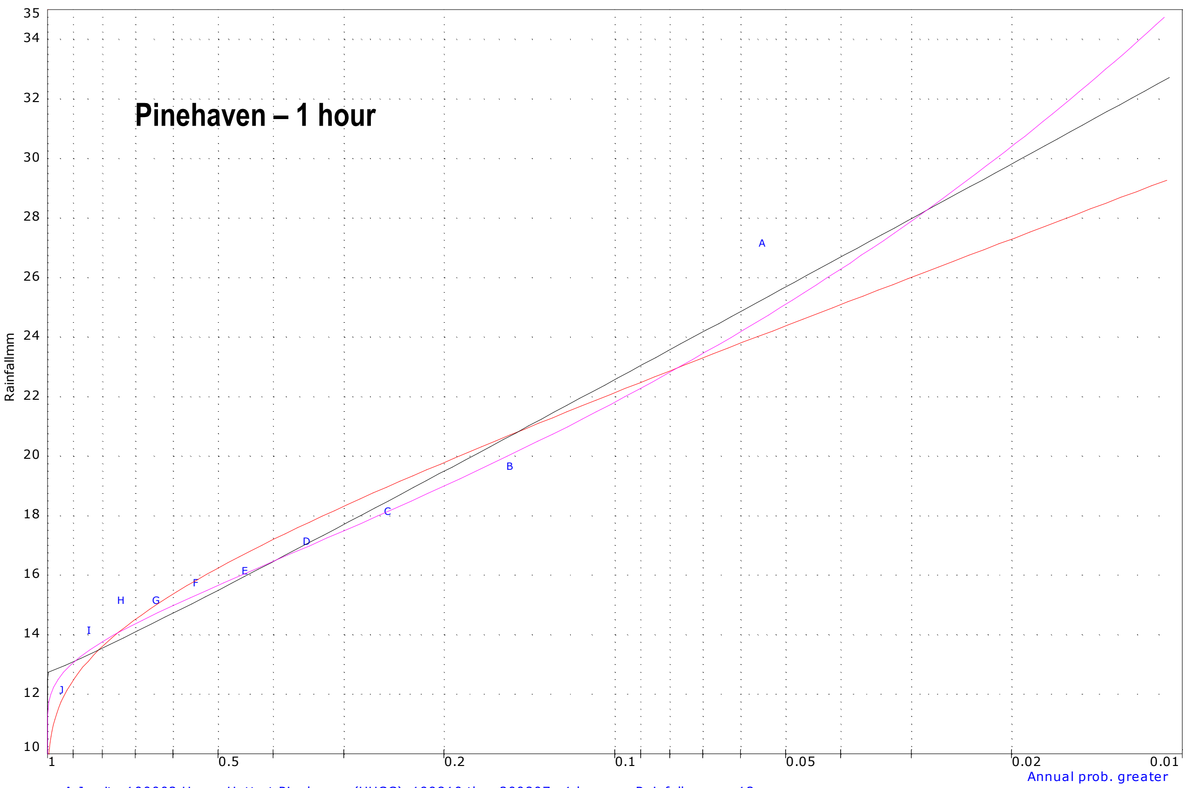

A-J site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 1 hr max. Rainfallmm m=12
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Gumbel Distn. Location & Scale= 15.11 3.125
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PE3 Distn. Location, Scale & Shape= 16.91 4.386 2.108
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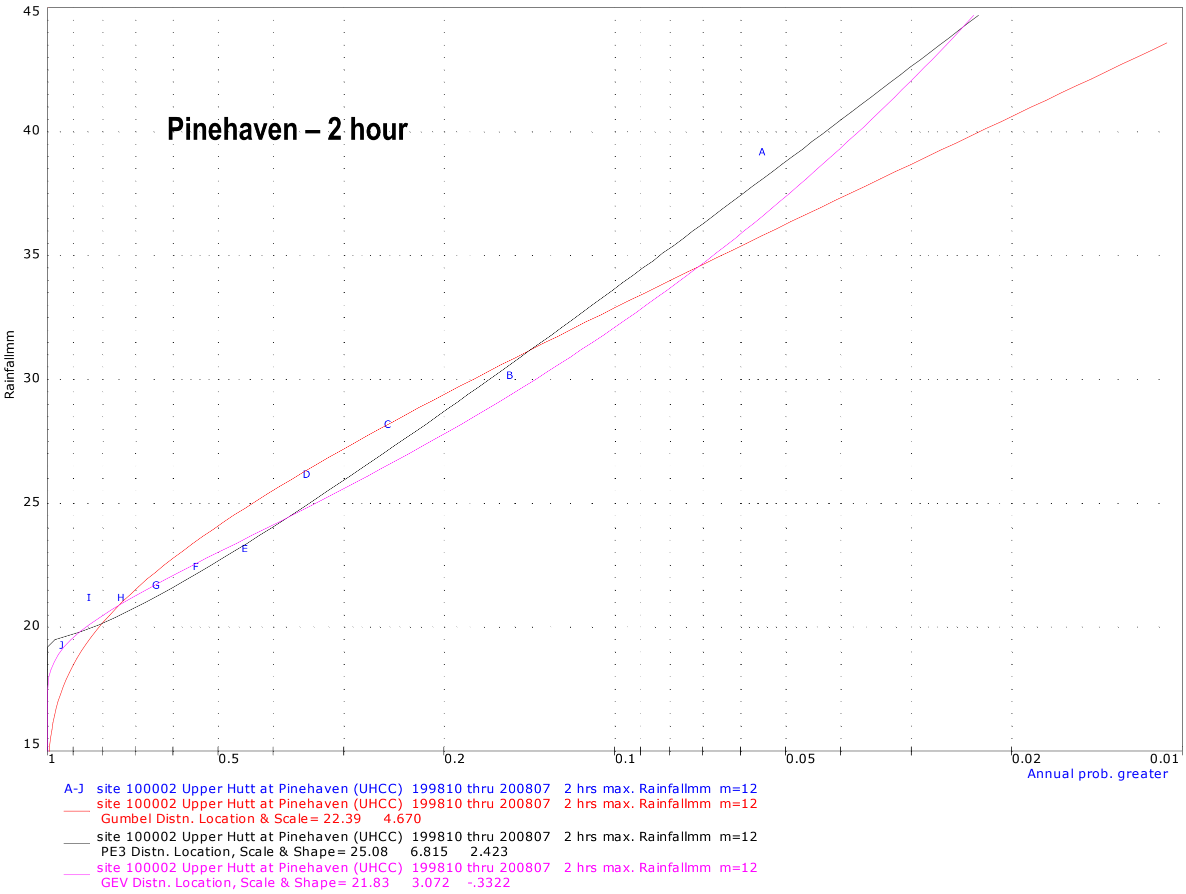
Pinehaven – 2 hour
Rainfallmm
45
40
35
30
25
20
15
1
0.5
0.2
0.1
0.05
0.02
0.01
Annual prob. greater
A
B
C
D
E
F
G
H
I
J
A-J site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 2 hrs max. Rainfallmm m=12
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 2 hrs max. Rainfallmm m=12
Gumbel Distn. Location & Scale= 22.39 4.670
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PE3 Distn. Location, Scale & Shape= 25.08 6.815 2.423
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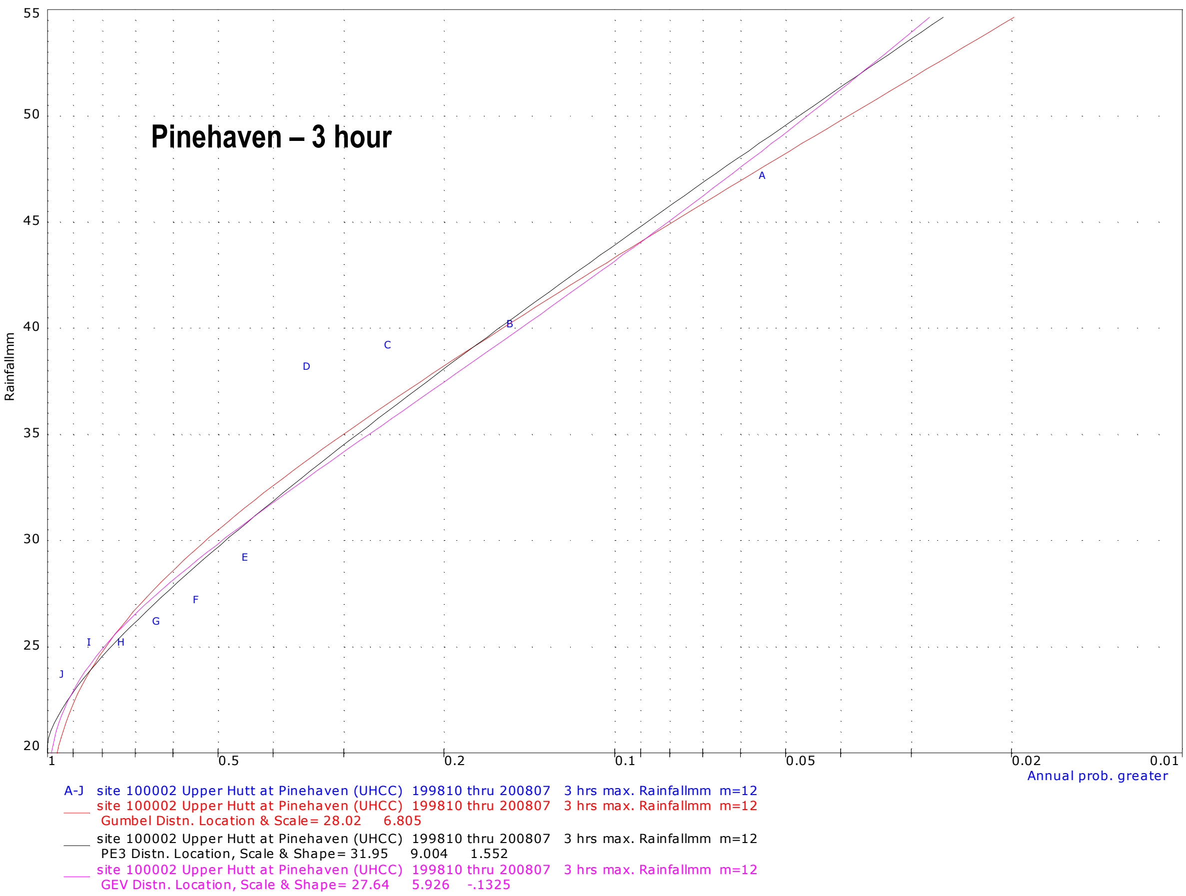
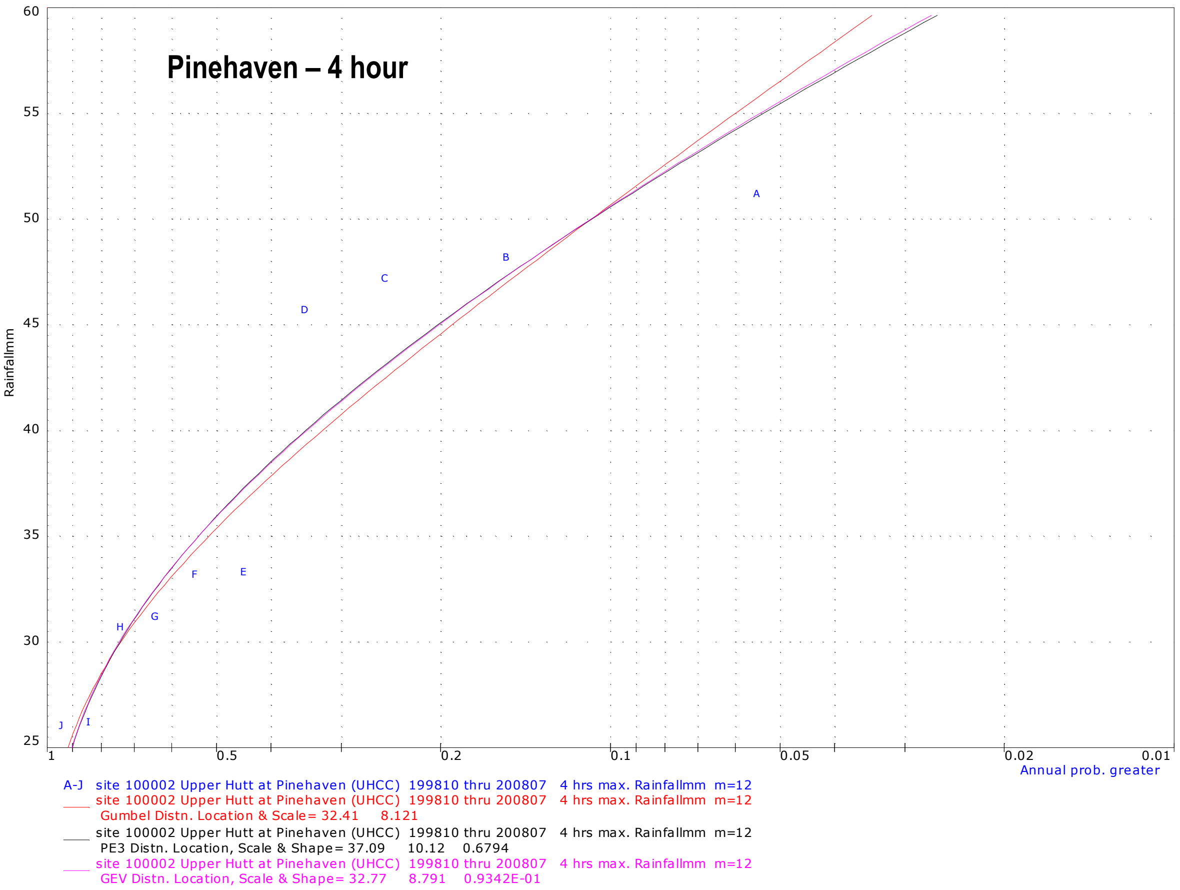
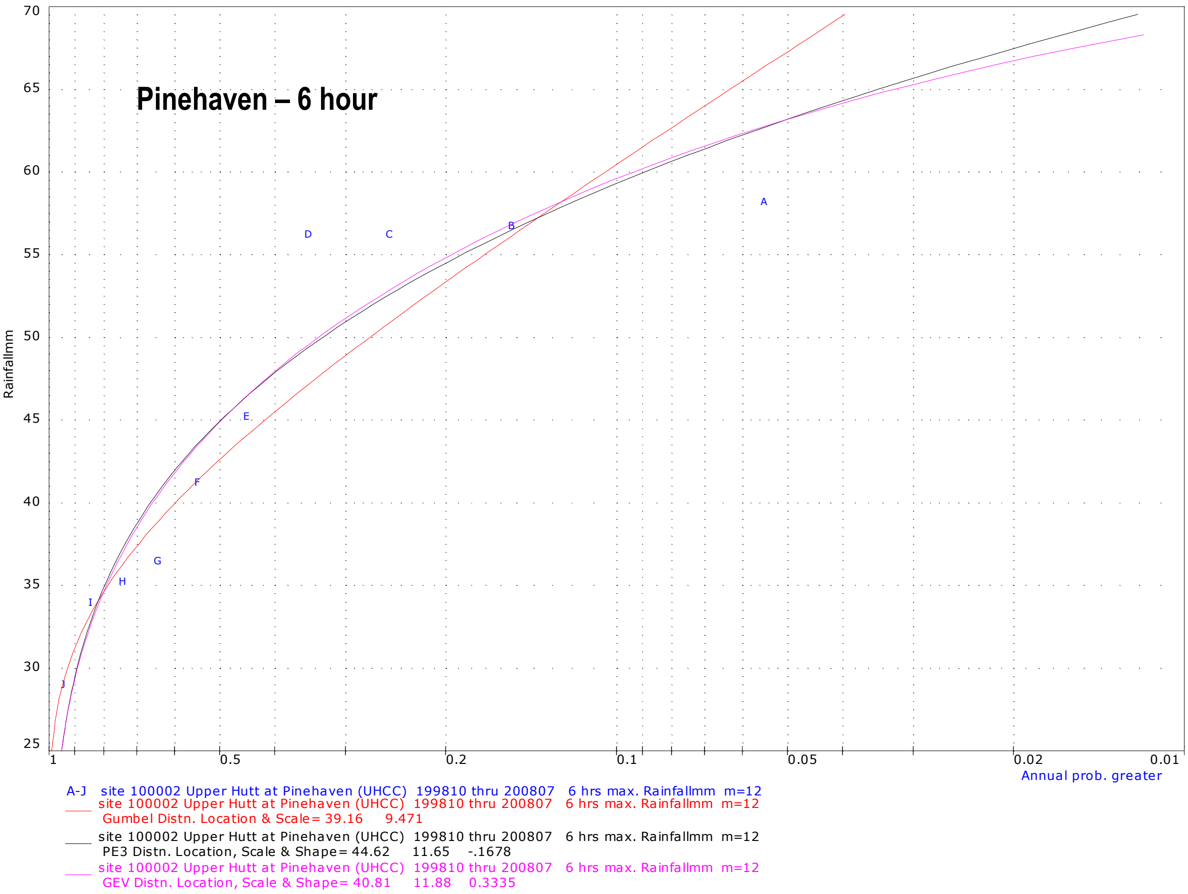

Pinehaven – 3 hour
Rainfallmm
55
50
45
40
35
30
25
20
1
0.5
0.2
0.1
0.05
0.02
0.01
Annual prob. greater
A
B
C
D
E
F
G
H
I
J
A-J site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 3 hrs max. Rainfallmm m=12
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 3 hrs max. Rainfallmm m=12
Gumbel Distn. Location & Scale= 28.02 6.805
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 3 hrs max. Rainfallmm m=12
PE3 Distn. Location, Scale & Shape= 31.95 9.004 1.552
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 3 hrs max. Rainfallmm m=12
GEV Distn. Location, Scale & Shape= 27.64 5.926 -.1325


Pinehaven – 4 hour
Rainfallmm
60
55
50
45
40
35
30
25
1
0.5
0.2
0.1
0.05
0.02
0.01
Annual prob. greater
A
B
C
D
E
F
G
H
I
J
A-J site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 4 hrs max. Rainfallmm m=12
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 4 hrs max. Rainfallmm m=12
Gumbel Distn. Location & Scale= 32.41 8.121
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 4 hrs max. Rainfallmm m=12
PE3 Distn. Location, Scale & Shape= 37.09 10.12 0.6794
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 4 hrs max. Rainfallmm m=12
GEV Distn. Location, Scale & Shape= 32.77 8.791 0.9342E-01


Pinehaven – 6 hour
Rainfallmm
70
65
60
55
50
45
40
35
30
25
1
0.5
0.2
0.1
0.05
0.02
0.01
Annual prob. greater
A
B
C
D
E
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G
H
I
J
A-J site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 6 hrs max. Rainfallmm m=12
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 6 hrs max. Rainfallmm m=12
Gumbel Distn. Location & Scale= 39.16 9.471
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 6 hrs max. Rainfallmm m=12
PE3 Distn. Location, Scale & Shape= 44.62 11.65 -.1678
site 100002 Upper Hutt at Pinehaven (UHCC) 199810 thru 200807 6 hrs max. Rainfallmm m=12
GEV Distn. Location, Scale & Shape= 40.81 11.88 0.3335

Appendix B Revision of Rainfall-Runoff Model and Design Flood Estimates, 25 November 2009

Since the publication of the Pinehaven Stream Flood Hydrology report in November 2008, the rainfall-runoff hydrological model has been updated and subsequently the design peak flows presented have changed.

Continuous flow data is now collected on the Pinehaven Stream and as a result the original rainfall-runoff model is able to be calibrated with greater confidence than previously. In addition, a hydraulic model of the catchment has been developed that presents an accurate picture of catchment runoff characteristics that can be used in the calibration process.

A high flow event in the Pinehaven Stream on 23 July 2009 is the largest event available to date to use for calibration. Rainfall totals for this event indicate it may have reached a 10-year ARI magnitude within the catchment.

The following sections replace the corresponding sections in the 2008 report.

6 Rainfall-Runoff Modelling

A rainfall-runoff model was developed for the Pinehaven catchment to produce design hydrographs for specified return period (ARI) events from the design rainfall inputs. The model is an Initial Loss - Continuing Loss type and has been built using Hydstra Modelling software. Hydstra Modelling has been used in many hydrological applications in New Zealand and around the world for rainfall-runoff and design flood modelling. It has been used for a number of GWRC flood modelling and flood design investigations.

6.1 Model Configuration

The Pinehaven catchment was divided into 15 sub-catchments (A to O) for the rainfall-runoff modelling (Figure 6-1). The catchment area to Chatsworth Rd is 4.4 km^2.

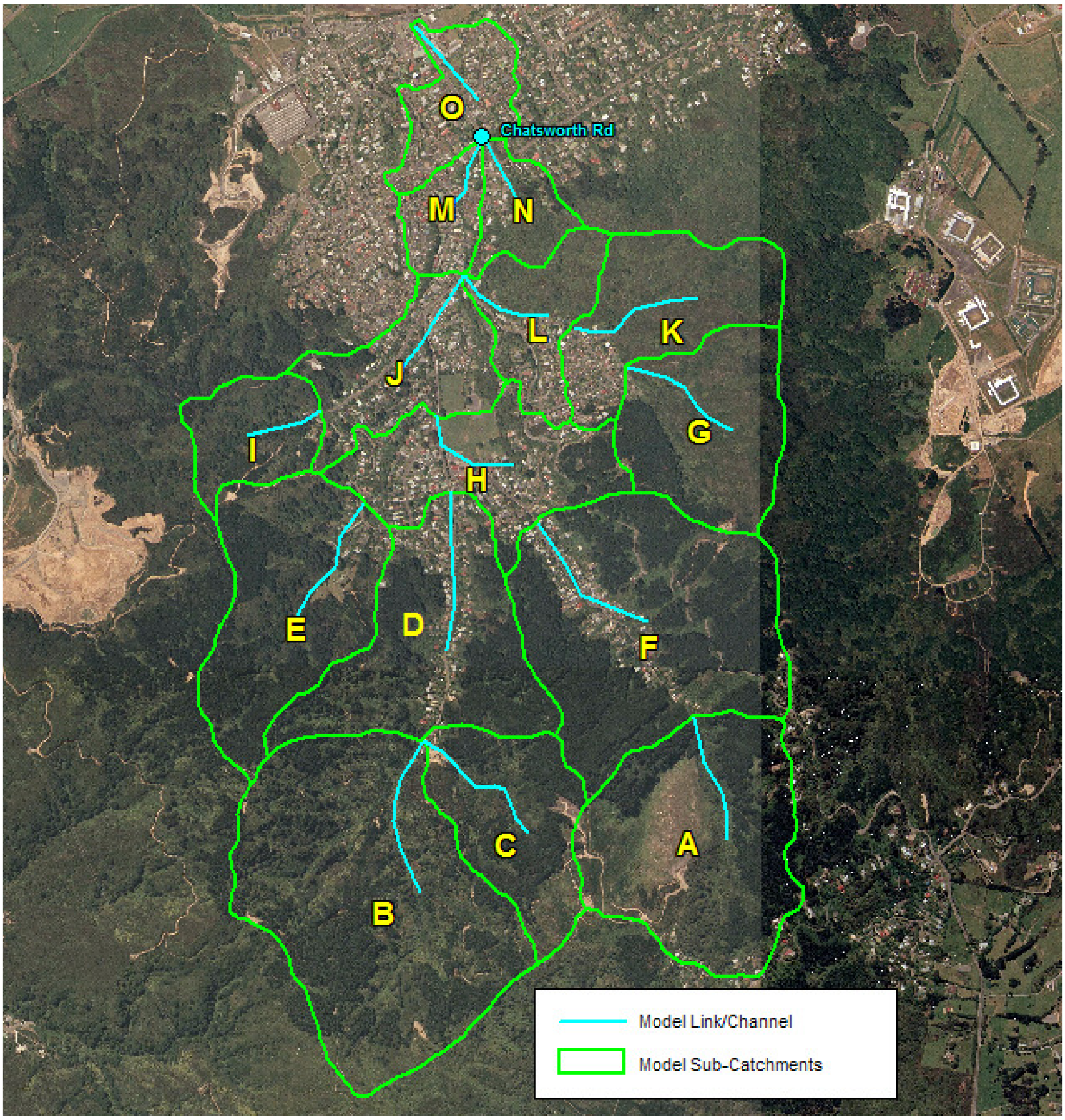

Figure 6-1: Pinehaven Rainfall-Runoff Model Delineation

Sub-catchment delineation and channel lengths were derived from GIS based contour and aerial photograph information, as well as maps presented in the Upper Hutt City Council District Plan (2004).

Figure 6-2 shows a schematic of the rainfall-runoff model. Modelled flow hydrographs are output at Chatsworth Rd. For the subsequent hydraulic modelling process flow hydrographs are output from each sub-catchment.

The model is setup to run with 15 minute intervals.

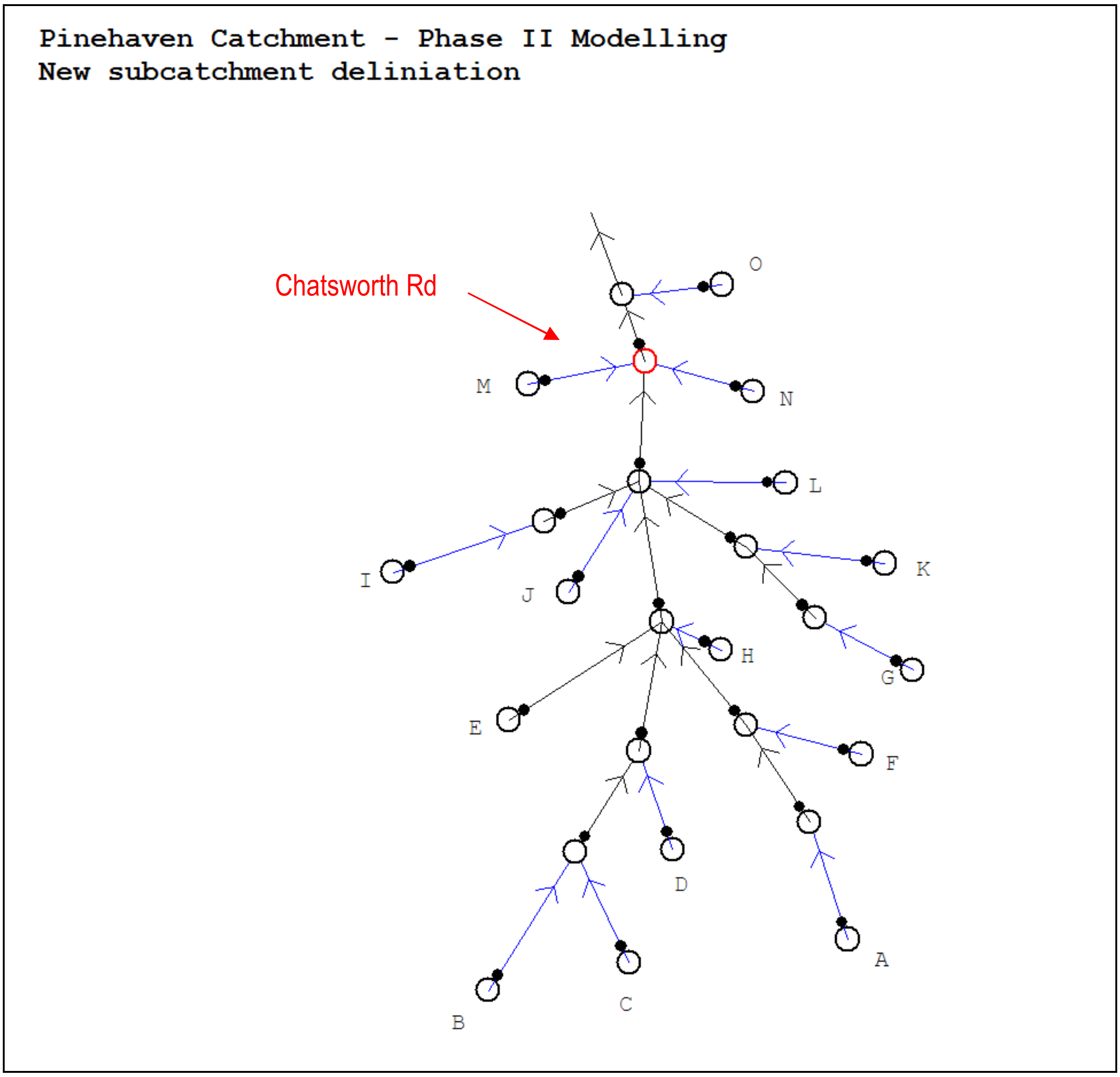

Figure 6-2: Pinehaven Model Schematic

6.2 Model Calibration

At the time of producing this report there was less than one year of recorded flood flow data for the Pinehaven catchment. GWRC installed a flow recorder on the Pinehaven Stream at Chatsworth Rd in August 2008. In this short period of time there has been only one flood event worthy of use for calibration purposes. This event occurred on 23 July 2009.

6.2.1 23 July 2009 Event

Recorded flow data for this flood event have been supplied by GWRC. The peak flow is extimated to be 8.8 m^3/s. It must be noted that due to the short period of record and lack of certainty about the conversion of high measured water levels to flow (rating curve), the 8.8 m^3/s estimate may be revised in the future when new information is available.

6.2.2 Calibrating Rainfall-Runoff Model to Flood Estimate

The rainfall-runoff model was calibrated against the 23 July 2009 event recorded data as well as output from a hydraulic model of the catchment provided by SKM. Recorded rainfall data from the Tasman Vaccine Ltd and Wallaceville raingauges are used as model inputs.

Table 6-1 details the parameters used in the rainfall-runoff model. The initial loss (IL), continuing loss (CL), α and n parameters are adjusted during the calibration process. These values are "global" and are therefore assumed constant over the catchment.

Table 6-1: Calibration Parameters for Rainfall-Runoff Model

Parameter	Description
Initial Loss (IL) - mm	Amount of water lost before rainfall becomes effective runoff
Continuing Loss (CL) - mm/hr	Continuing loss rate applied to rainfall after IL is satisfied
α	Channel lag parameter for channel routing
n	Non-linearity parameter for channel routing
Area - km^2	Sub-catchment area
L - km	Channel length

The calibration process consisted of using the recorded rainfall data as the input and varying the IL, CL, α and n parameters to match the modelled hydrograph to the calibration data.

Table 6-2 details the final model parameters adopted.

Table 6-2: Pinehaven Stream Model Calibration Results

Parameter	Best Fit Value
Initial Loss (IL)	5 mm
Continuing Loss (CL)	2 mm
α	2
n	1.7

There are uncertainties in calibrating a rainfall-runoff model to just a single recorded flood event. Particularly when there is uncertainty associated with the actual flow data due to the short length of record at the site and a lack of other high flow events to confirm the flow rating.

Ideally a number of recorded flood hydrographs would be available for calibration to provide confidence in the modelled peak flow estimates and hydrographs shapes.

However, it is better to have the one peak flow estimate to calibrate the model to than nothing at all.

6.3 Design Flood Hydrographs

The design rainfall events for the 2 to 100-year ARI and PMP were input to the rainfall-runoff model to produce design flood hydrographs throughout the catchment.

Figure 6-4 shows the 100-year ARI design hydrographs for Pinehaven Reserve and Chatsworth Rd. The one and two-hour events for each are given.

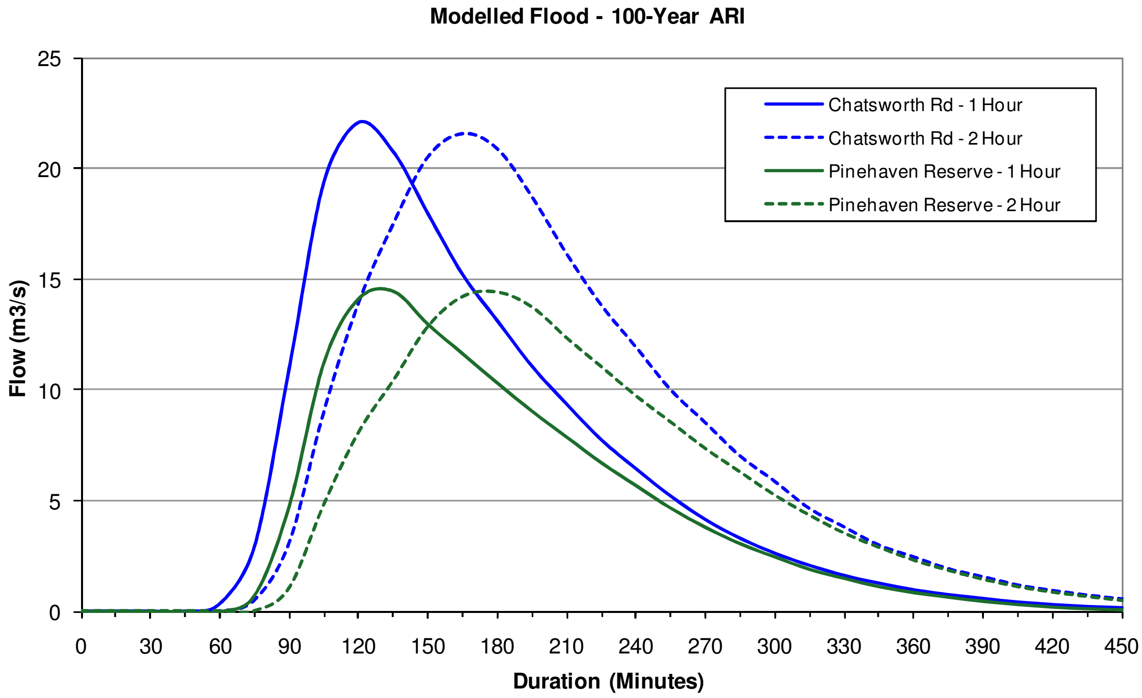

Figure 6-3: Design 100-year ARI Hydrographs

The critical duration 100-year ARI flood event for both locations is a one hour storm. Table 6-3 details the results for all durations and ARI magnitudes for the Pinehaven Stream at Chatsworth Rd. The critical duration event for each ARI is highlighted.

The 100-year ARI modelled design peak flow for the Pinehaven Stream at Chatsworth Rd is 22 m^3/s.

Table 6-3: Modelled Design Peak Flows for Pinehaven Stream at Chatsworth Rd (m^3/s)

ARI (Years)	Duration (Hours)				
	1	**2**	**3**	**4**	**6**
5	14.4	14.7	14.3	13.8	12.4
10	16.2	16.4	16.1	15.6	14.3
20	18.2	18.0	17.8	17.5	16.0
50	20.3	20.0	20.0	19.5	18.0
100	22.0	21.6	21.6	21.2	19.5
PMF	56	69	76	81	86

6.4 Rainfall-Runoff Model Limitations

The major limitation of the rainfall-runoff modelling process for the Pinehaven Stream is the lack of calibration data. Although one calibration event was available, there are uncertainties around the accuracy of the recorded data as the high flow rating is unconfirmed. The use of the model to simulate extreme flood events therefore carries some uncertainty. This uncertainty is reduced by comparing modelled output with peak estimates from other methods as summarised in Section 7.

A number of recorded flood hydrographs is preferred for calibration purposes to ensure estimates of peak flows and hydrograph shape are as accurate as possible.

Another form of calibration was able to be used here by comparing results to those of a hydraulic model of the catchment.

It is recommended that GWRC make use of data from the flow recorder on the Pinehaven Stream and check/re-calibrate the rainfall-runoff model after a number of years when more flood events have been recorded and there is confidence in the accuracy of measurement.

7 Summary of Flood Estimates

A summary of the derived flood estimates for the Pinehaven Stream is detailed in Table 7-1.

Table 7-1: Pinehaven Stream at Chatsworth Rd Flood Estimates (m^3/s)

ARI (Years)	Regional Methods			Rational Method	Rainfall-Runoff Model
	McKerchar and Pearson (1989)	Pearson (1990)	Pearson (1991)		
5	13	13	14	15	15
10	16	15	18	18	16
20	19	17	22	21	18
50	22	20	28	23	20
100	25	22	32	24	22
PMF	-	-	-	108	86

McKerchar and Pearson (1989) and Pearson (1991) are nationwide regional methods and are applicable to areas greater than 10 km^2 and less than 10 km^2 respectively. The Pinehaven catchment is 4.4 km^2 to the Chatsworth Rd site.

Pearson (1990) uses regional data specifically from the Hutt River catchment and surrounding area.

The rainfall-runoff model results are similar to Pearson (1990) which is based on local flood data. The rainfall-runoff results are also similar to those derived using the rational method.

8 Recommended Design Flood Estimates

Based on the results obtained in this investigation it is recommended that the rainfall-runoff model results and hydrographs be adopted for design flood purposes.

Table 8-1 presents the recommended design flood estimates for the Pinehaven Stream at Chatsworth Rd.

Table 8-1: Pinehaven Stream Design Flood Estimates

ARI (Years)	Flow (m^3/s)
5	15
10	16
20	18
50	20
100	22
PMF	86

Appendix B DHI Water and Environment review of the MIKE FLOOD model

DHI WATER & ENVIRONMENT

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Email: qld@dhigroup.com
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Benjamin Fountain
Sinclair Knight Merz
Level 12 Mayfair House
54 The Terrace
Wellington, New Zealand
6011

Init: MFB
Ref: 50658\BFO-1

Date: 1/09/2009

Dear Benjamin,

REVIEW OF MIKE FLOOD MODELLING – PINEHAVEN STREAM PROJECT

In accordance with your request we have reviewed the MIKE FLOOD model developed by Sinclair Knight Merz (SKM) for the purposes of assessing the potential for severe flooding and flood hazard in Pinehaven. This letter summarises our findings at the pre-calibration stage of the model build with brief recommendations where appropriate.

General Overview
The model covers approximately the region between the upper extent of Pinehaven Road and Elmslie Road to Hull Creek. A 5m 2-D MIKE21 bathymetry is used to model the floodplain and a 1-D MIKE11 branch network is used to model sub-grid scale channels and long culverts. There are no model boundaries within the MIKE21 grid and all flow enters and drains from the system via the MIKE11 network and a series of lateral couples. For this review the 10 Year and 100 Year ARI model setups and results were available.

MIKE21 Model

Bathymetry
The selection of a 5m grid size is appropriate considering the scale of features resolved in the MIKE11 model. The modelled area is sufficient as the flood surface does not push up against 'dry land' cells during the largest event simulated (100 Year ARI). Where both left and right lateral couples have been defined for the one branch, cells between the coupled cells have been 'blocked out' with 'dry land' cells to avoid duplication of conveyance in these areas (refer to **Figure 1** in Attachment A). This is considered good modelling practice where the MIKE11 channel exceeds 10m width. No boundaries are specified in the MIKE21 setup file and bathymetry. No obvious interpolation errors or rapidly changing/erroneous bed levels were observed in the grid data.

Timestep & Courant Number
For MIKE FLOOD applications in particular DHI recommends that a Courant number of less than 1 is maintained. With an approximate maximum flood depth of 2m and a timestep of 0.5 seconds the Courant number is approximately 0.5 and within the recommended guideline.

Flood & Drying Depths
A flooding depth of 0.01 m and a drying depth of 0.005 m have been applied. These values are slightly lower than the lowest pair of values generally recommended by DHI for applications where rainfall is not directly applied to the grid as a source. The impact of a very low flooding depth is to artificially increase the speed of the

wetting front across flat areas. We recommend changing the flooding depth to 0.02 m and the drying depth to 0.01 m.

Initial Surface Elevation
The initial surface elevation file specified is appropriate considering the MIKE21 model does not contain inflow or outflow boundaries. No cells are wet at commencement of the simulation, consistent with the relatively steep topography modelled. It may be appropriate to assess any ponded areas at the end of the simulation (areas that do not drain) to determine if these should be filled in the initial condition (only if conservative assessment of lost storage is a project consideration or aids in model calibration).

Eddy Viscosity
A velocity based eddy viscosity value of 1 has been applied globally within the model. This value is within the guidelines recommended by DHI for a grid size of 5 m and timestep of 0.5 seconds. Various empirical relationships exist for estimating appropriate values of eddy viscosity in the absence of observed eddy behaviour. Some of these would yield smaller values of eddy (0.2 to 0.5) based on the relatively shallow flow depths in the model.

Resistance
Four different values of resistance have been defined. These represent road pavement, houses, grass land and forest. Based on visual inspection of aerial photographs the number of regions and the Manning's M values defined for these regions are generally appropriate. However, it should be noted that Manning's M of 6.67 for forest may be found to be too rough during model calibration (refer **Figure 2**).

Results
The MIKE21 model has a one minute save interval and produces a result file of approximately 850mb. Both the save interval and the model result file size are appropriate however a save interval of 30 seconds could be selected and the model result file would still be less than 2GB which is generally targeted as a model result size.

MIKE11 Model

Network
Within the MIKE11 model long pipe sections are represented correctly as cross sections rather than culverts structures. It should be noted that in some cases the closed cross section method for modelling pipes will result in less than adequate head losses as changes in direction and losses at junctions are not properly accounted for. The value for dx Max is currently set at either 20m or 30m depending on the branch in the model, this should be changed to 5m for all branches to suit coupling to the MIKE21 model.

Cross Sections
A number of cross sections within the model have non-monotonically increasing conveyance curves. An example of this is Emislie_Rd CH 453 (refer to **Figure 3**). This results from a discontinuity in the hydraulic radius curve, due to a large increase in wetted perimeter with a small increase in area (water level) and typically occurs where the flow transitions from channel to floodplain. The inflection can be corrected in two ways; first by using the left and right banks markers (markers 4 & 5) at the channel banks forcing the conveyance to be calculated in different zones and second by selecting 'Resistance Radius' over 'Total Area Hydraulic Radius' in the Radius Type drop down box within the Cross Section editor for cross sections where this occurs (refer to **Figure 4** for corrected cross section and conveyance curve). Alternatively, bank markers 1 and 3 could be moved in to the channel bank location to reduce the low flow cross section, resulting in flow being transferred to the MIKE21 grid at lower water levels. This should be done in tandem with assessment of the z values in the MIKE21 bathymetry to which these cross sections will be coupled. Selecting the equidistant level selection method in the processed data cross section editor will also assist in smoothing the conveyance curve.

Boundary Conditions
The MIKE11 boundary conditions were examined and found to be appropriate.

Hydrodynamic Parameters
The Delta value on the Default Parameters tab of the HD11 file is used to control the gravity term in the momentum equation. Delta is a weighting factor between upstream and downstream control of flow momentum. The default values is 0.5 which (centred between upstream and downstream) and values greater than default can be used to dissipate the wave front to produce a more stable model. A value of 0.85 was found to have been applied and is too considered high, a value of 0.7 is generally recommended for upland rivers and should be adopted for this application.

A global Manning's n value 0.035 has been applied and pipe and culvert sections have generally been specified as 0.015 and some small open channels have been specified as 0.2. During the calibration process it may be found that 0.015 is too smooth for aged concrete pipe and culvert sections, whilst 0.2 may be too rough for small channels unless dominated by thick vegetation.

MIKE FLOOD model

Where cross sections are open, the MIKE11 channels are coupled to the MIKE21 grid via lateral couples. Standard coupling options have been applied and the number of coupled cells has been trimmed such that the length of each lateral couple is approximately equal to the length of the MIKE11 branch for each open section. This is considered good modelling practice.

For each of the lateral couples the default options have been applied, that is no exponential smoothing (recommended) and HGH structure type for determining the geometry of the internal weir for each lateral link. HGH takes the highest of either the MIKE21 bathymetry level or the MIKE11 bank marker level. This can be interrogated further by using the "MFLateral" diagnostic in to view both these levels in the MIKE11 cross section editor. This is achieved by the following steps:

- Create an empty text file named MFLateral.txt in the same location as the *.sim11 file;
- Run the MIKE FLOOD simulation for at least 1 timestep to populate the file with data;
- Create an empty *.xns11 file; and
- Import mflateral.txt into the empty *.xns11 file.

For each lateral couple the MIKE11 bank marker level, the MIKE21 bathymetry level and the structure level (or internal weir geometry) is visible (Refer to **Figure 5**). The effect of significant difference between MIKE21 and MIKE11 levels is to control the points along the couple where transfer between the two surfaces is possible. A maximum difference of 0.5m is generally recommended and either selection of a different bank marker location or filling of the bathymetry may be used to achieve this.

The MIKE FLOOD model runs in approximately 4hrs on a standard high performance run computer, this is a good outcome as generally 12hr simulation times are targeted. The model results were reviewed and no evidence of instabilities were found either in the MIKE21 result file or the MIKE11 result file.

Summary
Overall the model has generally been built within the guidelines specified by DHI in training material and during provision of software support to software clients. With the following recommendations the model will be suitable to proceed with calibration and assessment of potential for severe flooding and flood hazard within Pinehaven.

Key Recommendations:

- Change the flooding and drying depths to 0.02 m and 0.01 m respectively;
- Remove the culvert structures that precede pipe cross sections;

- Change the Max dx value to 5m for all branches;
- Rectify non-monotonically increasing conveyance curves via cross section settings;
- Change the Delta value in Hydrodynamic Parameter file from 0.85 to 0.7;
- Review resistance values for pipes and culverts with a Manning's n of 0.015 and open channel cross section with a Manning's n of 0.2; and
- Review the mflateral.txt data to identify coupled locations with large differences between the MIKE21 bathymetry and MIKE11 cross section bank markers.

Please do not hesitate to contact me if you require further clarifications.

Yours sincerely,
DHI Water and Environment Pty Ltd

Mark Britton
MIKE FLOOD Trainer – Australia

Attachment A

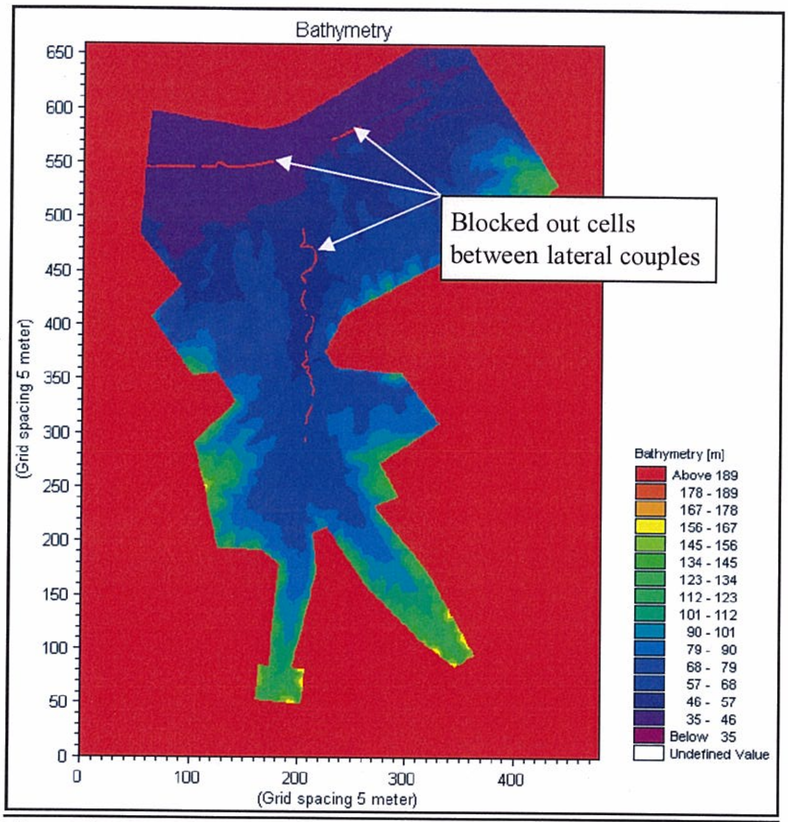

Figure 1 – MIKE21 Bathymetry

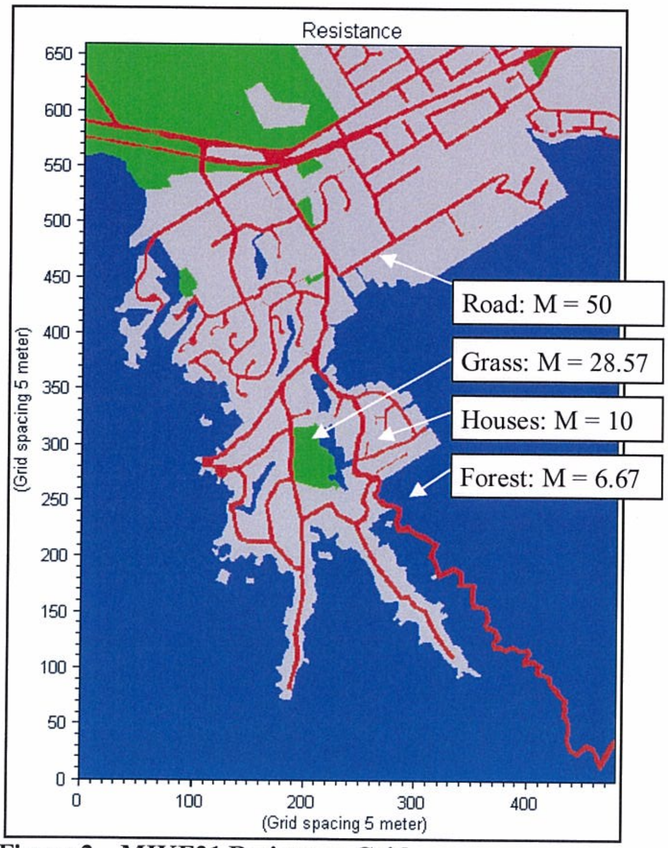

Figure 2 – MIKE21 Resistance Grid

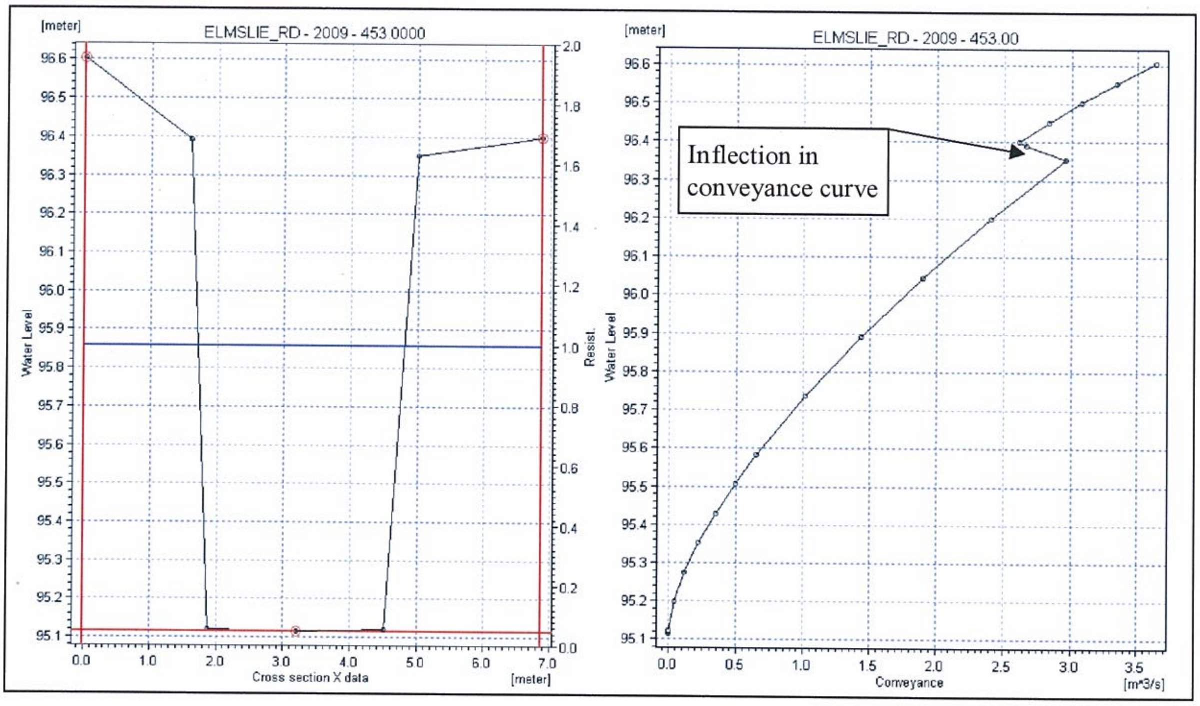

Figure 3 – Elmslie_RD CH 453 – Non Monotomically Increasing Conveyance Curve

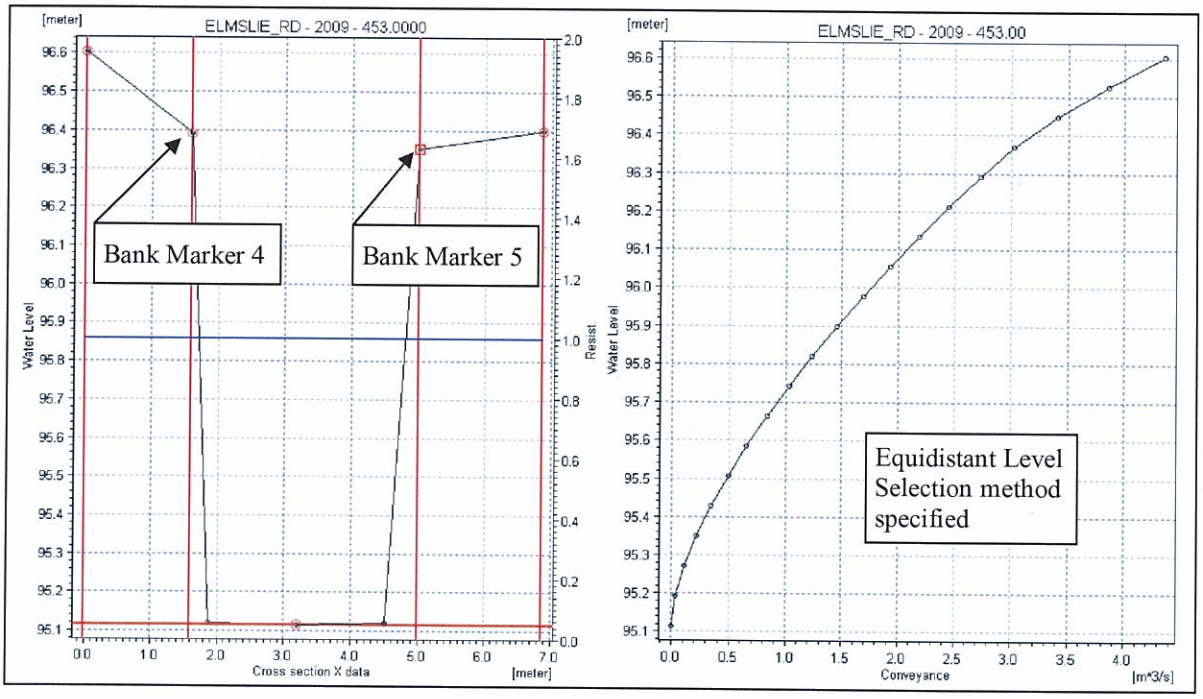

Figure 4 – Elmslie _Rd CH 453 – Rectified Cross Section and Conveyance Curve

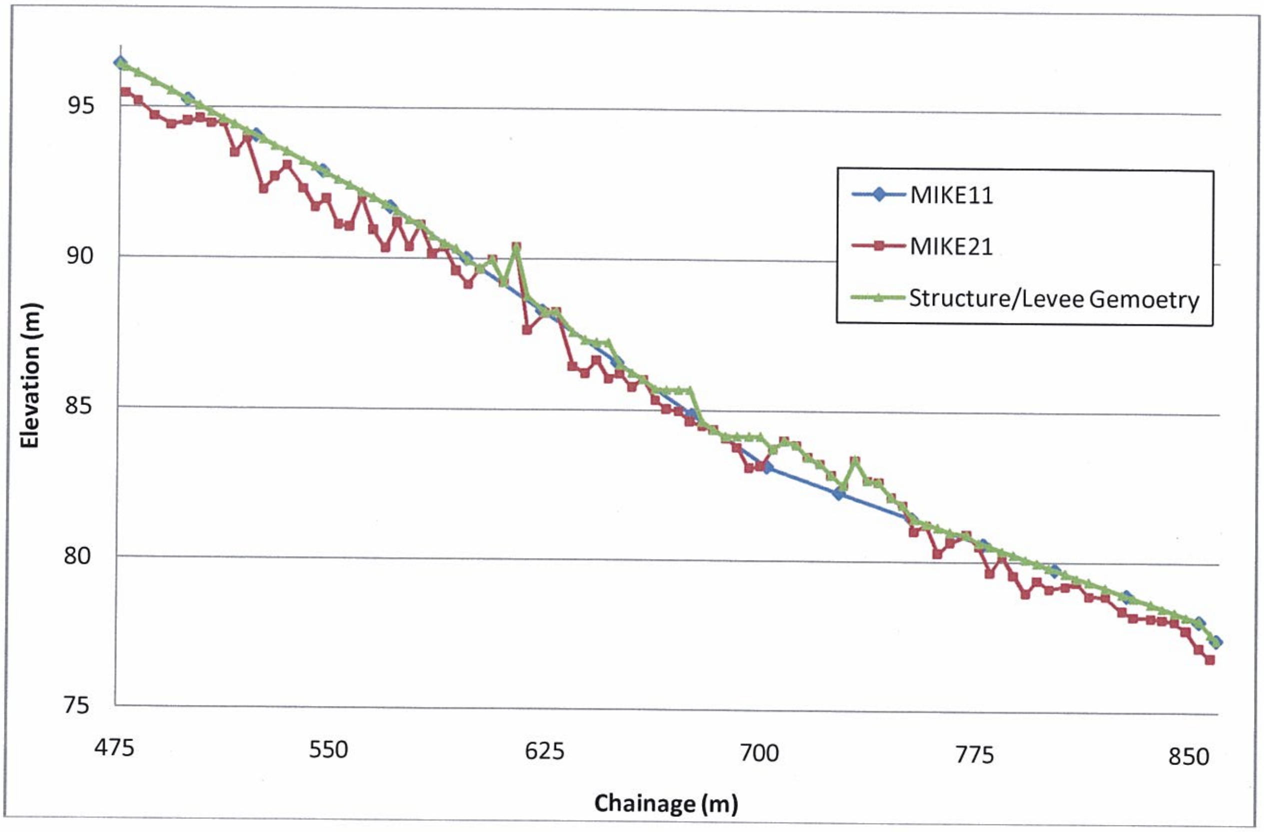

Figure 5 – Chart of Selected Structure/Levee Level verus MIKE11 and MIKE21 model levels for Elmslie_Rd 475 m.

Appendix C Beca Limited Pinehaven Stream – Flood Mapping Audit

www.beca.com

Report

Pinehaven Stream - Flood Mapping Audit

Prepared for Greater Wellington Regional Council

Prepared by Beca Ltd (Beca)

13 July 2015

Revision History

Revision Nº	Prepared By	Description	Date
1	**Michael Law**	Final	13 July 2015
2			
3			
4			
5			

Document Acceptance

Action	Name	Signed	Date
Prepared by	**Michael Law**		13July 2015
Reviewed by	**Graham Levy**		13 July 2015
Approved by	**Graham Levy**		13 July 2015
on behalf of	Beca Ltd		

Executive Summary

Flood hazard maps have been developed for the Pinehaven Stream catchment in the Hutt Valley. The maps are based on the outputs of hydrological and hydraulic modelling carried out from 2008 to 2010, and are being used to inform control of development and flood alleviation options for the catchment.

The scope of the audit described in this report is neatly summarised as follows:

"The audit builds upon previously completed investigations and peer review work and elevates this to an additional level of scrutiny and analysis. These previous investigations and peer reviews found both the hydrology and hydraulic model fit for purpose, however some of the community still had concerns that the scope of the reviews done to date was not extensive enough, and therefore an additional more comprehensive audit has been requested by the Hutt Valley Floodplain Management subcommittee, (the governing body for the development of the floodplain management plan). This audit is to contain a review of the hydrology, hydraulic model and the application of freeboard."[1]

The terms of reference (ToR) for the audit and appointment of the auditor have been subject to community scrutiny. This audit report contains a review of the hydrological and hydraulic modelling, the application of freeboard, and the presentation and interpretation of the flood hazard maps. Meetings have been held with the modellers and with two community groups; Save Our Hills and Pinehaven Progressive Association. The concerns raised, and case studies provided, by the Save Our Hills group are addressed in the audit.

As requested in the RFP ToR, guidance is also provided in the report on how to:

- Set storm water neutrality provisions within district plan.
- Define the impact of intensification of development on the runoff characteristics of the Pinehaven hills.

A review of the hydrological and hydraulic modelling has been carried out as part of this audit, and is described in the ToR as an audit of:

- The type of software and modelling package used for the hydrology and hydraulic model
- The modelling method used and its appropriateness for both hydrology and the hydraulic model
- The use of freeboard and method by which it was applied
- Representation of the flood hazard through the way in which maps are displayed and information provided.

The review found that the hydrological and hydraulic modelling is fit for purpose. The methods and level of detail reflected the catchment information and modelling methods available in 2008-2010.

While there have been advances in modelling methods and available information since 2009 updating and upgrading the models is not recommended by this audit, and doing so would be unlikely to significantly alter the flood extents and depths for the design flood events and scenarios modelled.

The way that the flood extent and hazard maps are presented in published information obscures the components that have been used to derive the extents. Describing the 'flood extent plus freeboard' maps as Flood Hazard Maps does not adequately describe the complexity of information included in the

[1] Paragraph 6 of the ***Request for Proposal - Pinehaven Stream Flood Mapping Audit.*** *WGN_DOCS-#1437397-v3-ToR_Pinehaven_Stream_FMP_Audit.doc*

Maps. These issues lead to confusion and misunderstanding within the community regarding the interpretation and use of the maps. As such, the presentation of flood information in published map form could be modified which may provide greater transparency and understanding.

This may be achieved by distinguishing modelled levels from wider flood sensitive areas, taking freeboard and sensitivity to factors such as debris blockage into account. Currently, this information is available to an individual by request from GWRC; however these additional details are not included in published maps.

Given that the maps are to be used for planning purposes, the inclusion of an allowance for climate change to a suitable horizon is appropriate, as is the inclusion of freeboard. 2090 is suggested as it is one of the time horizons reported in MfE's 2008 guidance. Similarly, the choice of ARI for the map could be altered to reflect local consenting requirements.

The modelling underlying the flood maps is now 6-7 years old. Flood maps are periodically updated in line with council long term plans, or in response to significant new data becoming available after a major storm event, or when major changes occur within the catchment. The community should be made aware of this, and understand that mapped flood extents may be refined in future as a result of programmed revision to flood modelling and mapping.

The issue of including stormwater (or hydrological) neutrality into local planning guidelines is complicated. While general principles regarding matching or lowering peak flows at the outlets from developments are widely adopted, the hydrological effect of potential developments should be considered on a case by case basis, as in some cases downstream flood risk may be reduced if runoff from the development is discharged early to the receiving water course before floodwater from upstream arrives. However, this is unlikely to be the case for the Pinehaven catchment, where runoff attenuation is likely to provide the most benefit to reducing downstream flood risk.

With regard to assessing the hydrological effect of potential future development on the Pinehaven Hills, peak flows in the affected sub-catchments could increase by about 18% (if not attenuated) and flood volumes may increase by about 6%. Further down the catchment the relative percentage increases in peak flow and flood volume will be smaller, as the cumulative catchment area is increased by the inclusion of catchments that have not been subject to future development. Further work will be completed to develop suitable controls for future development within the Pinehaven catchment to support a plan change by UHCC.

Confirming the main conclusion of the audit; **the hydrological and hydraulic modelling underlying GWRC's flood extent and hazard maps is fit for purpose, but the way that flood information is presented in map form could be modified, which may increase the understanding and acceptance of the maps by the community.**

Contents

Appendices

1 Introduction and scope

Flood hazard maps have been developed for the Pinehaven Stream catchment in the Hutt Valley. The maps are based on the outputs of hydrological and hydraulic modelling carried out from 2008 to 2010, and are being used to inform control of development and flood alleviation options for the catchment through local and regional planning guidelines.

The scope of this audit is neatly summarised as follows:

"The audit builds upon previously completed investigations and peer review work and elevates this to an additional level of scrutiny and analysis. These previous investigations and peer reviews found both the hydrology and hydraulic model fit for purpose, however some of the community still had concerns that the scope of the reviews done to date was not extensive enough, and therefore an additional more comprehensive audit has been requested by the Hutt Valley Floodplain Management subcommittee, (the governing body for the development of the floodplain management plan). This audit is to contain a review of the hydrology, hydraulic model and the application of freeboard."[2]

The terms of reference (ToR) for the audit and appointment of the auditor has been subject to community scrutiny. The ToR are listed in Section 7 of the Request For Proposal (RFP), which also provided the prospective auditor with the opportunity to include issues not specifically identified in the RFP. The ToR are provided in Appendix A.

This audit report contains a review of the hydrological and hydraulic modelling, the application of freeboard, and the presentation and interpretation of the flood hazard maps. Meetings have been held with the modellers and with two community groups; Save Our Hills and Pinehaven Progressive Association. The concerns raised, and case studies provided, by the Save Our Hills group are addressed in the audit.

As requested in the RFP ToR, guidance is also provided on how to:

- Set storm water neutrality provisions within the district plan, as laid out in Section 7.4.1 of the RFP.
- Define the impact of intensification of development on the runoff characteristics of the Pinehaven Hills, as laid out in Section 7.4.2 of the RFP.

The RFP requested that the audit be delivered as a single volume Audit Report (this report), containing:

- *"Executive summary including comment about whether the flood maps produced and the process by which these were derived makes them fit for purpose.*
- *A completed checklist with a series of YES/NO questions that answer the key question on a topic by topic basis as to whether that particular aspect of the process used to develop the flood maps is fit for purpose.*
- *A summary explanation of any issue which is deemed as being not fit for purpose and what remedial work would be required to make this fit for purpose and deliver a positive audit result.*
- *Results of the additional investigation requesting guidance on how to set storm water neutrality provisions, and how to define the impact of intensification of development."* (Section 9, RFP)

[2] Section 6 of the ***Request for Proposal - Pinehaven Stream Flood Mapping Audit.*** *WGN_DOCS-#1437397-v3-ToR_Pinehaven_Stream_FMP_Audit.doc*

2 Background information

The following reports were reviewed, along with other information, as part of the audit.

- ***Report on storm of 20 December, 1976***, Wellington Regional Water Board.
- ***Pinehaven Stream Flood Hydrology***, MWH for Greater Wellington Regional Council. 2008, plus 2009 update.
- ***Pinehaven Stream Flood Hazard Assessment: Volumes 1 (modelling report) and 2 (flood and hazard maps)***, SKM (now Jacobs) for Greater Wellington Regional Council and Upper Hutt City Council. 25 May 2010.
- ***Pinehaven Stream Floodplain Management Plan***, Greater Wellington Regional Council. 13 October 2014.
- ***Review comments on Pinehaven hydrology report***, Greater Wellington Regional Council memo. 24 October 2014.
- ***Terms of Reference for Audit Pinehaven Stream Flood Maps***, Save Our Hills presentation to GWRC Hutt Valley Floodplain Management subcommittee. 24 February 2015.

It is assumed that the reader of this audit report has a general knowledge of the Pinehaven catchment. However, if needed, a good description of the Pinehaven catchment, Pinehaven Stream, and Flood History is provided in Section 2 of the Volume 1 of SKM's *Pinehaven Stream Flood Hazard Assessment*.

The 1976 flood is used as the reference storm event for the Pinehaven catchment. It resulted in significant flooding within the catchment and had an average recurrence interval (ARI) of about 100 years. As such, it is the event against which modelled flood extents are compared. A contributing factor to the impact of the flood was surges in flow caused by the breaking of debris dams in the upper catchment. Following the 1976 flood, a bypass culvert was constructed in the lower part of the catchment to increase conveyance of floodwater to Hulls Creek.

Unfortunately, stream flows and water levels were not recorded in the catchment prior to MWH's hydrological modelling in 2008, which meant that the derived flow hydrographs in their report were derived from general hydrological methods rather by calibration against observed events. Temporary flow and water level measurement was installed for a period during 2008 and 2009, during which a small flood event was recorded on 23 July 2009. This event was used to calibrate the hydrological modelling in the 2009 update to the report, but it is noted that the July 2009 event had an ARI of about 10 years; significantly lower than the 1976 event.

GWRC reviewed MWH's hydrology and did not find any major issues, although they acknowledged the absence of data against which to calibrate the modelling.

MWH's derived flow hydrographs were used in the coupled 1D/2D hydraulic modelling of the Pinehaven catchment by SKM (now Jacobs) in 2009. The hydraulic modelling was reviewed by DHI (suppliers of the MIKE FLOOD software used) at the time, and found to have been built within DHI's model build guidelines. 5-year to 100-year ARI scenarios were modelled. Community consultation was carried out to provide feedback and comments on the draft outputs of the modelling. Scenarios incorporating combinations of climate change, culvert blockage, and increased land development were modelled to assess the sensitivity of the model results and inform the choice of a suitable allowance for 'freeboard' above modelled flood levels.

The key outputs from SKM's modelling were three sets of maps, as presented in Volume 2 of the *Pinehaven Stream Flood Hazard Assessment.* The maps are:

- Base scenario Q10[3] flood inundation
- Q100 design scenario including partial blockages, freeboard and predicted impacts of climate change
- Flood hazard zone and erosion set back line.

The flood maps are reproduced in GWRC's *Pinehaven Stream Floodplain Management Plan*, in which options for flood alleviation and mitigation are described, and put in their consenting and legal context.

3 Meetings

As part of this audit, the auditor (Mike Law) carried out the following meetings and visits:

- 1 April 2015 — Briefing by Alistair Allan (GWRC's Project manager), and site visit to the Pinehaven catchment.
- 15 April 2015 — Meetings with:
  - Stephen Pattinson and Darryl Longstaffe, representing the Save Our Hills group
  - David Brown and Chris Coslett, representing the Pinehaven Progressive Association
  - Kristin Stokes (MWH).
- 7 May 2015 — Meetings with:
  - Ben Fountain (SKM [now Jacobs])
  - Mike Harkness (GWRC), author of the 2008 MWH hydrology report
  - Alistair Allan (GWRC).

4 Model Review and Checklist

As noted in Section 2, the hydrological modelling has been reviewed by GWRC and the hydraulic modelling reviewed by DHI. Both reviews found the modelling to be acceptable. An additional review of the hydrological and hydraulic modelling has been carried out as part of this audit as required by the terms of reference, and is described below.

The general scope for the model review is described in the terms of reference as an audit of:

- The type of software and modelling package used for the hydrology and hydraulic model
- The modelling method used and its appropriateness for both hydrology and the hydraulic model
- The use of freeboard and the method by which it was applied
- Representation of the flood hazard through the way in which maps are displayed and information provided.

Elements of the modelling have been reviewed (Sections 4.1 and 4.2) and rated using a 0-3 scoring system (described in Table 4.1), which flags up issues that will affect model use. This provides more definition than the simpler Yes/No categorisation specified in the terms of reference.

[3] Q10 = 10-year ARI

Table 4.1 – Model review rating scheme

Description	Audit rating	Fit for use
No issue: The element or parameter being reviewed is modelled acceptably	0	Yes
Minor issue: There is an issue, but it is unlikely to significantly affect model results	1	
Major issue: Failure to resolve the issue compromises the model and should be rectified, but may be resolved by explanation or acceptance of model limitations.	2	?
Fatal flaw: Failure to resolve this issue severely compromises the model, and should be rectified before the model is accepted.	3	No

4.1 Hydrological modelling

The hydrological modelling was carried out by MWH in 2008. The modelling was updated in 2009 following calibration against a small flood event that had been recorded by temporary flow monitoring equipment.

Table 4.1 – Hydrological modelling

Item	Comment	Audit rating	Fit for Use
Software	The hydrological modelling was undertaken using Hydstra software. Hydstra is a standard software package that incorporates a catchment runoff model, and is appropriate for this level of analysis.	0	Yes
Rainfall data	There are rain gauges in (or close to) the lower lying parts of the Pinehaven Stream catchment, and one gauge that is representative of the hills of the upper catchment. The modelling of extreme rainfall depths and profiles is well described in the MWH report and is considered appropriate. As rainfall records lengthen over time and more severe storm events are included in the record, it is worth undertaking occasional reviews of the design rainfall depths and profiles as this will increase the reliability of the modelling in predicting more extreme storms.	1	Yes
Critical storm duration	The critical storm duration for the Pinehaven Stream catchment is 2-3 hours. The critical duration will be less for smaller sub-catchments than for the whole Pinehaven Stream catchment. The temporal pattern used by MWH was based on analysis of 17 Wellington storms. The resulting storm approximated to a triangular profile, with peak rainfall occurring about 70% through the storm. Use of a nested storm profile might improve peak flow calculation for the upper catchments, but is unlikely to have a significant effect on flood extents.	1	Yes
Catchment definition	Catchment and sub-catchment definition is acceptable. The number of sub-catchments was adjusted to meet the hydraulic modelling network	0	Yes
Hydrological method and calibration	Regional flood frequency methods were used to estimate peak flows. These were then used to check the results of rainfall-runoff modelling. Initial and (constant) Continuing losses were used to calculate the effective rainfall, and coefficients used to route flows through the catchment.	0	Yes
Measured flood flows and calibration	The modelled flows were calibrated against the relatively small flood events of 31 July 2008 (Mean Annual Flood) and 23 July 2009 (10-year ARI). Ideally, the model should be calibrated against a larger flood event. In the absence of recorded water level and flow data for the catchment, calibration against the hydrological response of a monitored catchment with similar hydrological characteristics would increase confidence in the modelled flow hydrographs.	1	Yes

Item	Comment	Audit rating	Fit for Use
Calculated flows	The calculated peak flows have been cross-referenced against regional methods for estimating peak flows, and similar results found. It is six years since the hydrological modelling was carried out, and consideration should be given to reviewing the hydrology as a longer period of rainfall data becomes available, as predictions for the effects of climate change evolve, and as the understanding of the hydrological response of the Pinehaven Stream (and similar catchments) improve.	**1**	**Yes**
Climate change	Climate change was not included in the MWH hydrological modelling, but was recommended to be included in further work. Note comments regarding climate change in the review of the hydraulic modelling, below.	**1**	**Yes**

The conclusion of the review of the hydrological modelling is that the derived peak flows and hydrographs are fit for use in the subsequent hydraulic modelling in 2009/2010.

It is six years since the hydrological modelling was undertaken. Flood maps are periodically updated in line with council long term plans, or in response to significant new data becoming available. At such time, the hydrology should be updated to account for longer rainfall records and more storm events. More robust hydrology could be provided by calibration against recorded flow data, especially for a large flood event. In the absence of recorded data, calibration against the hydrological response of a similar catchment should be considered when the hydrology is reviewed. Updating the hydrology is unlikely to make significant changes to the flood maps at the catchment scale, although there may be refinements at the property level.

4.2 Hydraulic modelling

Hydraulic modelling of the Pinehaven Stream catchment was carried out in 2009/2010 by SKM[4], utilising the outputs of MWH's hydrological modelling.

DHI carried out a review of the model in 2009, and confirmed that the build was in line with DHI's own guidelines and training. The DHI model review concentrated on model build parameters, such as Timestep, Flood & Drying Depths, and Hydrodynamic factors. While these have been considered for this audit, more emphasis has been placed on inputs to the model, model extents, and whether the model provides an appropriate representation of flood depths and extents in the Pinehaven catchment.

Table 4.1 – Hydraulic modelling

Item	Comment	Audit rating	Fit for Use
Software	The hydraulic modelling was carried out using DHI's MIKE FLOOD software package to build a coupled 1D/2D model. The stream channels and culverted bypass were modelled in 1D using MIKE 11 and the floodplain in 2D using MIKE 21. DHI's MIKE software is widely used worldwide and is suitable for modelling Pinehaven Stream.	**0**	**Yes**

[4] SKM merged with Jacobs in late 2013, and now operate under the Jacobs name, but for the purposes of this report the name SKM will be retained.

Item	Comment	Audit rating	Fit for Use
Model Extent	The upstream limits of the model are the points where tributary streams enter the built environment. Upstream of these points the streams flow through dense and steep bush and forestry. The downstream boundary of the model is the outfall to Hulls Creek. The model extents are appropriate.	**0**	**Yes**
Floodplain cross-sections and/or 2D extent	Gridded LiDAR data was used to construct the 2D model bathymetry. The LIDAR data was collected on 4 June 2009, and so was current at the time of model build. The use of LiDAR data is generally appropriate. LiDAR is widely used when constructing flood models, but can be less reliable in dense vegetation and for small channels, where topographical survey is required. The 2D model bathymetry had a grid spacing of 5 m. While this gives reasonable definition in generally flat areas, it is relatively coarse for defining flood extents in steeper terrain and detailed overland flow paths where smaller obstructions (such as road curbs) may have a significant effect. Ideally a smaller grid size would be used, but a smaller grid would have significantly increased the time taken to run the model when it was built in 2009. As such, a 5 m grid spacing would have been appropriate at that time, and is still commonly used due to run time constraints. However, DHI's MIKE software now has the ability to use a flexible mesh approach to model bathymetry, which coupled with advances in computing power since 2009 could be used in future to improve the definition of flood extents and overland flow paths.	**1**	**Yes**
Cross-sections	Cross-sections of the stream channel were surveyed for the modelling by Landlink Ltd in June 2009. An appropriate number of cross-section were surveyed around larger structures (such as road crossings), but there are longer than ideal distances between surveyed sections where the streams flow through or behind private properties. Access can be an issue in these circumstances, so the gaps are understandable. They are unlikely to have a significant effect on modelled flood outlines, but do represent a less than optimal situation, especially in channels where cross-sections change over short distances and where there are multiple obstructions, culverts and bridges (see comment below). This may affect flood outlines at the individual property scale.	**1**	**Yes**
Flood plain obstructions	Roughness factors (Manning's 'M') are applied to the 2D model bathymetry surface to represent how easily water can flow across the surface. Smoother surfaces such as roads have a higher M value (lower roughness) than dense bush. The M values used in the MIKE 21 model are appropriate. A Manning's 'M' value of 10 has been used to represent the developed parts of the catchment. This indicates a rough surface, which would be expected with buildings, fences and vegetation providing barriers to flow. It does not appear as though individual buildings and structures on the floodplain have been blocked out or given very high roughness values. This may be due to the use of the fairly coarse 5 m grid for the 2D surface and the computing processing available in 2009. If the model were being built in 2015, it could be expected that buildings would be treated differently than the land around them.	**1**	**Yes**

Item	Comment	Audit rating	Fit for Use
Stream channel roughness coefficients	Within the stream channels a default Manning's 'n' value of 0.035 was applied to represent channel roughness[5]. This is appropriate for reasonably straight and uniform natural channels such as those in the middle and lower reaches of Pinehaven Stream. In the upper reaches higher (0.200) roughness values were used, which reflects the smaller, more vegetated channels, but also the means by which culverts and bridges were accounted for in these reaches (see comment below).	**0**	**Yes**
Structures - Weirs, bridges and culverts	Road bridges, larger culverts and bypass channels have been included in the MIKE 11 1D model. Smaller bridges and crossings, especially in the upper Pinehaven Road and Elmslie Road tributaries, have not been included in the model. Rather, their effect on water levels has been represented by the use of a higher channel roughness coefficient. Reasons for not including the smaller channels in the model include difficulty gaining access to survey the crossings, and increased model complexity that can lead to model stability issues, especially in steep channels. The use of increased roughness to represent small bridges and crossings is reasonable, so long as it is realised that the definition of water levels and flood extents at the property scale will be reduced in these areas. Long culverts are modelled as closed cross-sections, rather than as culverts. Manhole losses are not included when this modelling approach is taken.	**1**	**Yes**
Boundary conditions	The upstream boundary conditions for the model are the flow hydrographs derived by MWH. The hydrographs for each of the 15 sub-catchments modelled were applied at the top of the tributaries, or as lateral inflows along the stream channels. The downstream model boundary is the water level in Hulls Creek. Water levels are not recorded, and so a constant water level was defined by SKM with due regard to anecdotal evidence of water levels in the Creek during the 1976 event and subsequent remediation works, including upstream storage in Hulls Creek that attenuates flow. Sensitivity checks were carried out on the downstream boundary. The boundary conditions are considered acceptable.	**0**	**Yes**
Design events and climate change	The MIKE FLOOD model has been run for the: ▪ 5, 10, 20, 50 and 100-year ARI storm events without an allowance for climate change. ▪ 23 July 2009 storm event for calibration ▪ PMF (Probable maximum flood) ▪ 100-year ARI storm events with an allowance (16% increase in rainfall) for climate change. In addition scenarios including full or partial blockage of culverts and/or increased development of the catchment were modelled. See below.	**0**	**Yes**

[5] Note that Manning's 'n' is the inverse of Manning's 'M'. SKM used 'M' for the floodplains and 'n' for the cross-sections.

Item	Comment	Audit rating	Fit for Use
Blockage	The probability and consequence of culverts, bridges and channels being fully or partial blocked during floods by water borne debris is a reality, especially in heavily vegetated (including forestry) catchments with lots of culverts and bridge, such as Pinehaven. Model runs were carried out that included partial or full blockage of 12 culverts in the catchment. This is a reasonable approach for assessing the sensitivity of the catchment to blockage given the uncertainty surrounding the timing, location and extent of blockage that may occur during an event.	**0**	**Yes**
Future development	The upper parts of the Pinehaven catchment are bush and forestry. Sub-division development has been mooted for these areas and it could be expected that there would be some infill development in the lower parts of the catchment. While not pre-judging the outcome of any application to develop within the catchment, it is prudent to assess the effects of possible future development when undertaking flood mapping and hazards studies. To that end, SKM ran the model with reworked hydrographs to represent the additional impervious area associated with the development of 1665 lots of 750 m^2 in the upper parts of the catchment. This is probably an over-estimate of the number of lots that could be developed, and as such represents an upper bound on the effect of development on catchment flows and flood extents. Given that the upper catchment is steep, natural runoff could be expected to be quite high and so the relative effect of development would not be great. Were development to occur, mitigation measures would almost certainly be required to attenuate flows and at least reduce peak flows to existing conditions. As noted in Section 8 below, including future development increases modelled peak flows by 18% in sub-catchment B and 13% in sub-catchment E. However, there is no post-development increase in flood volumes. This is unexpected given the increase in impermeable area. MWH were unable to provide an explanation for the lack of increase in flood volume, and so the future development runs of SKM's flood model are potentially compromised in this regard.	**2**	**No**

The conclusion of the review of the hydraulic modelling is that the model is fit for use for producing the flood extent and hazard maps for current development, but that better definition of flood depths, extents and overland flow paths could be provided if the model were to be updated to account for current computer processing power and advances in modelling software. Specifically:

- The use of a finer grid or flexible mesh to construct the 2D model bathymetry would provide better definition of flood extents and overland flow paths.
- Blocking out buildings within the 2D model bathymetry would improve definition of overland flow paths and should be considered if the models are to re-run.
- Review (and update, if necessary) future development hydrology for use in model runs assessing the impact of potential development in the catchment.

Given access issues and the high cost of survey, it is probably impractical to include additional channel cross-sections or model all of the minor bridges and culverts across the stream channel at a catchment level.

4.3 Flood hazard mapping

Figures 4.1, 4.2, and 4.3 show examples of the three sets of flood extent and hazard maps produced by SKM Jacobs as outputs from the hydraulic modelling. The three maps are:

- ***Base scenario Q10 flood inundation:*** 10-year ARI flood depth and extent.
- ***Q100 design scenario including partial blockages, freeboard and predicted impacts of climate change***: 100-year ARI flood depth and extent. This scenario does not include future development.
- ***Flood Hazard Zone and erosion set back line:*** Flood Hazard Zone extent defined by the '*Q100 design scenario including partial blockages, freeboard and predicted impacts of climate change'* extent. Erosion hazard zones and setback shown along channels.

The extent of the Flood Hazard Zone is the same as the extent of the Q100 design scenario including partial blockage, freeboard and predicted impacts of climate change. The Q100 flood depth and extent map includes an allowance for 'freeboard'. Freeboard is an additional depth added to modelled water levels, and is an allowance for:

- Uncertainty in the modelling process or parameters, such as limited survey, lack of recorded flow data, and assumptions regarding stream and floodplain roughness, and antecedent conditions.
- The residual risk of flooding from extreme events (i.e. those greater than the design event), although this is not an element included in freeboard applied to GWRC Flood Hazard Maps.
- Local wave action and obstructions.

Figure 4.1 – Base scenario Q10 flood inundation

Figure 4.2 – Q100 design scenario including partial blockages, freeboard and predicted impacts of climate change

Figure 4.3 – Flood hazard zone and erosion set back line

Application of freeboard extends the potential floodplain beyond the modelled flood extent, and is used in development control to flag that flooding is an issue to be considered at the site and to assist in the setting of levels for floors and vulnerable services. The application of freeboard is one of the issues reviewed in Table 4.4 as part of the audit of the flood and hazard maps.

Elements of the Flood maps have been reviewed and rated using a 0-3 scoring system (described in Table 4.1), which flags up issues that will affect the understanding and interpretation of the maps. As with the review of the hydrology and hydraulic modelling this provides more definition than the simpler Yes/No categorisation specified in the terms of reference.

Table 4.3 – Model review rating scheme

Description	Audit rating	Fit for use
No issue: The element or parameter being reviewed is represented acceptably	0	Yes
Minor issue: There is an issue, but it is unlikely to significantly affect use of the maps	1	
Major issue: Failure to resolve the issue compromises the maps and should be rectified, but may be resolved by explanation or acceptance of map limitations.	2	?
Fatal flaw: Failure to resolve this issue severely compromises the understanding and interpretation of the maps, and should be rectified before the maps are accepted.	3	No

Table 4.4 – Flood hazard mapping

Item	Comment	Audit rating	Fit for Use
Application of freeboard	For the Pinehaven Flood Hazard Map, freeboard is added to the modelled flood levels that already incorporate the effects of partial or total culvert blockage. This is a conservative approach (as the effects of culvert blockage can be incorporated in the freeboard), but reflects the importance given by the Council to debris and blockage in the catchment, as occurred during the reference 1976 flood.	1	Yes
Freeboard height	A freeboard of 0.3 m above the 100-year ARI flood level has been used for the majority of the Pinehaven catchment, with the exception of the reach between Pinehaven Reserve and the bypass channel at Whitemans Road where the freeboard of 0.5 m is allowed. Both of these freeboard depths are in the range used elsewhere in New Zealand, and are considered appropriate.	0	Yes
Scenarios	Two scenarios have been modelled and three maps produced. There is a significant difference between the inputs to the 'Baseline' 10-year ARI flood map and the 100-year ARI map that includes climate change, culvert blockage and freeboard. Such a change makes it impossible to assess the effects of each input that has changed. More clarity would have been provided if a Baseline 100-year ARI map had been produced, then separate maps showing the 100-year map incorporating climate change and blockage; individually and in combination. Finally, the Flood Hazard Map including freeboard would be presented. The effects of uncontrolled runoff from future development in the catchment are not incorporated in the three published flood hazard maps. This means that the issues raised above regarding future development hydrology are not an issue with regard to the published flood hazard maps.	1	Yes

Item	Comment	Audit rating	Fit for Use
Presentation of flood extent maps	The two flood maps show both flood depth and extent. There is an advantage in this in terms allowing users to assess the severity of the flooding. However, different scales are used for the depth of flooding on the Q10 and Q100 flood maps. On the Q100 map the increments of flood depth shown on the maps are 500 mm. There can be a big difference in the consequences of flooding between a flood depth of 50 mm and one of 450 mm, especially in areas where freeboard is set at 300 mm. Reducing the flood depth increments to those on the Q10 flood map would improve the usefulness of the Q100 maps. Where GIS viewers are available, it can be helpful to view flood extents for different flood events at the same time. This isn't an option with the hard copy maps. Due to the concerns raised in Table 4.2 regarding the stream cross-section spacing, modelling of small crossings, and the size of the grid for the 2D model bathymetry, the flood extents shown on the maps may not be detailed enough to define flood levels at, or across, individual properties, especially in the steeper upper reaches of the modelled area. In these areas, a degree of caution and judgement will be required in the use of the maps.	**2**	**Yes**
Presentation of flood hazard maps	As noted above, the current Flood Hazard Map extents equate to the flood extent for the 100-year ARI storm including climate change, blockage and freeboard. Using a single shading for the whole flood hazard extent does not give a full understanding of the hazard in each location. GWRC advise the public to contact the Council for more complete and detailed information and advice. Flood hazard maps often show the flood hazard calculated as a product of the flood depth and water velocity. This is useful from a hazard assessment perspective to understand potential danger to people, and can be readily calculated from the outputs of 2D hydraulic models. However, such a map would not show any hazard in the buffer zone between the modelled flood extent and the extent including freeboard. This suggests that using the term 'hazard' in the title of the maps may be inappropriate and that an alternative name should be used for these maps, as they are used to indicate areas where the risk of flooding should be taken into account. Some alternatives are noted in Section 6.1. The terminology currently used may be one of the reasons why some sections of the local community are struggling to accept the current maps. This may be especially the case where a property owner perceives that their property is not at risk of flooding, and that inclusion within the mapped flood hazard extent could adversely affect the value or development potential of the property.	**2**	**?**

While there is logic to the information contained within the current flood extent and hazard maps, it is not immediately obvious what information was used to generate them. This is demonstrated in the failure of some sections of the local community to understand and/or accept the concepts of the maps, their use and limitations.

Flood maps are limited by the quality of the information used to derive them. As has been noted above, the definition of flood depths and extents in the Pinehaven catchment is restricted by the grid spacing of the model bathymetry and the number of stream cross-sections. This means that in some areas (such as the upper reaches of Pinehaven Road and Elmslie Road) where there is a shallow overland flow path along a road that is raised above the stream channel and streamside properties, the plotted flood extent may be too wide and may be wrongly interpreted as implying deep flooding of properties if it is assumed that flood levels will be the same from one side of the flood extent to the other.

5 Community concerns

The Save Our Hills (SOH) group, which was formed in 2014, has expressed strongly held concerns about what they perceive as serious discrepancies between the published maps and calculations. These concerns were presented to the Hutt Valley Flood Management Sub-Committee (HVFMS) on 27 February 2015, and were discussed with the auditor during a meeting on 15 April 2015 at Beca's office in Wellington. SOH were represented at the meeting by Stephen Pattinson and Darryl Longstaffe.

5.1 SOH case studies and flood hazard mapping

The main concern expressed by SOH was that the flood and hazard extents shown on the published map appeared too great for the modelled flows. Cross-sections were provided for case studies at the following four locations within the catchment:

- Top of Pinehaven Road
- 27 Elmslie Road
- Dunns Street
- Pinehaven Reserve

Figures B.1 and B.2 in Appendix B show the information provided by SOH for 27 Elmslie Road. At this location, the Flood Hazard Map extent is approximately 70 m wide, stretching from edge of Elmslie Road to approximately 15 m beyond the stream channel on the true right (east) bank.

Elmslie Road runs along a relatively narrow valley containing a tributary of Pinehaven Stream. At No. 27, the edge of the road is approximately 50 m from the stream, with the property between the road and the stream. The ground cross-sections produced by SOH and contained within the 5 m gridded model bathymetry show that ground level around the house to be about 1.0 m to 1.5 m below road level, while SOH's cross-section indicate that the bed of the stream is a further 2 m lower.

SOH assumed that the water surface across the cross-section was flat, and then calculated the cross-sectional area (not including an allowance for freeboard) to be about 15 m^2. SOH assumed an average flow velocity of 1 m/s, giving a flow of 15 m^3/s. This is approximately three times the 100-year ARI flow for that part of the catchment. The opinion of SOH is that the discrepancy between the expected modelled flow and the flow that they calculated casts doubt on the validity of the Flood Hazard Map extents. This issue was discussed by the auditor with Ben Fountain of SKM/Jacobs, who was the project manager for the flood modelling and preparation of the flood and hazard maps.

One critical area of difference between SOH's understanding of flood/hazard extents relates to the water surface across the cross-section. As noted above, SOH assumed that the water surface is flat, while SKM have provided cross-sections that indicate that the water level varies across the cross-section. Figure B.3 in Appendix B shows modelled water levels for SKMs sections in the vicinity of 27 Elmslie Road. The water levels were extracted from the 2D surface model results. They indicate shallow flooding (as low as 10 mm) at the margins of the floodplain, and lower water levels in the main channel. This may be due to overland flow paths on the floodplain carrying flow that has broken out from the stream channel upstream.

We note that the varying water level surface is confirmed by the Q100 flood map (Figure 5.1), which shows that flood depth across the site is less than 500 mm. If the water surface were flat across the cross-section, flood depth of more than 500 m could be expected close to the stream channel due to the slope of the ground. With the water level surface dipping towards the stream, the active cross-section area will be lower than the 15 m^2 that was calculated by SOH, and hence the calculated flow will also be lower.

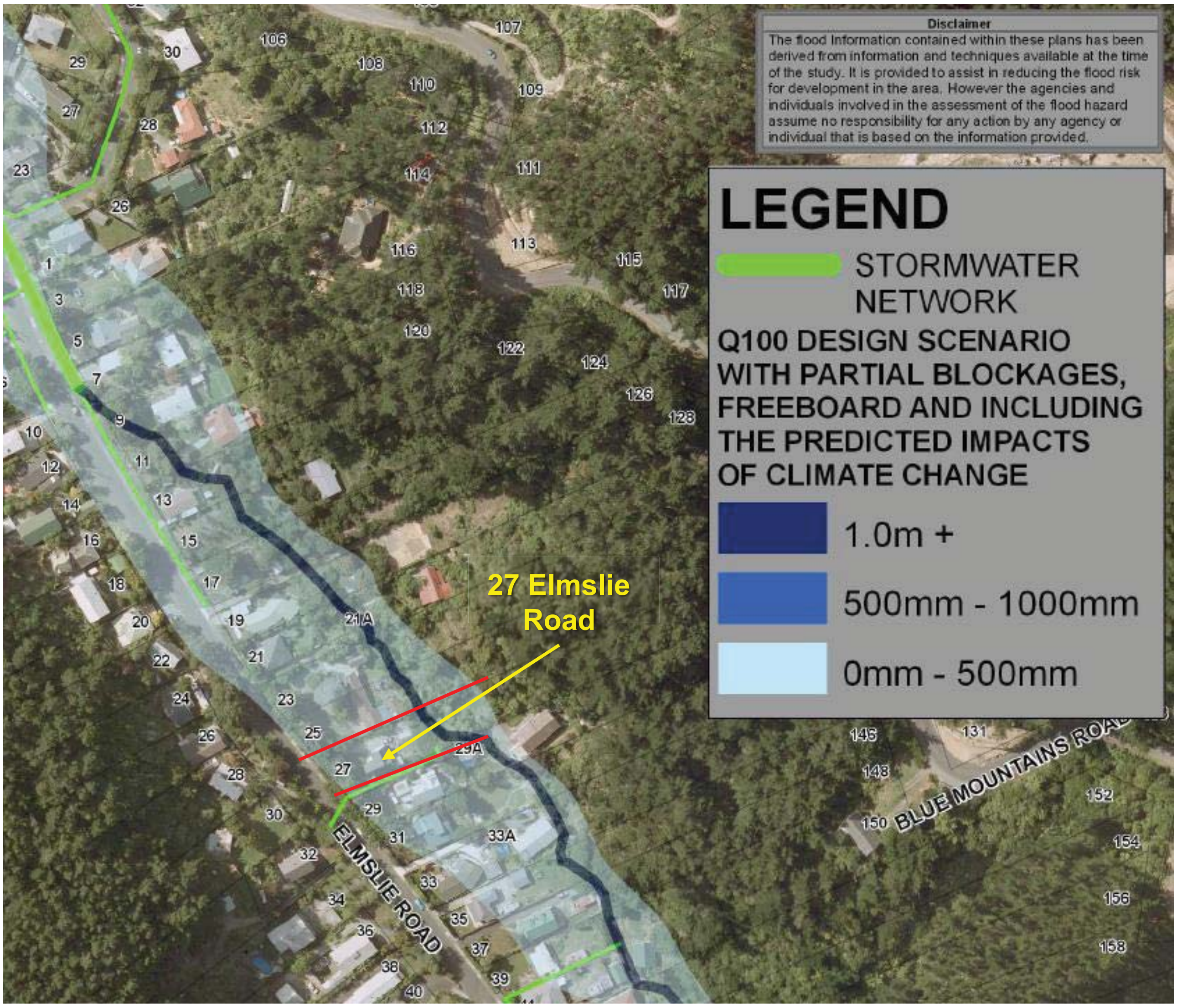

Figure 5.1 – Q100 (including culvert blockage, climate change and freeboard) flood extent at 27 Elmslie Road

Figure 5.2 represents some of the elements that could contribute to the definition of the flood extent; illustrating why:

- The flood level may be different on opposite sides of a valley when 2D modelling is used.
- Why it may be inappropriate to assume that the water surface is flat across the flood extent.

As well as inundation of the floodplain adjacent to the stream channel, the flood extent may be extended laterally by hillslope runoff towards the channel, secondary overland flow paths (such as roads), and water spilling between secondary flow paths and the stream channel.

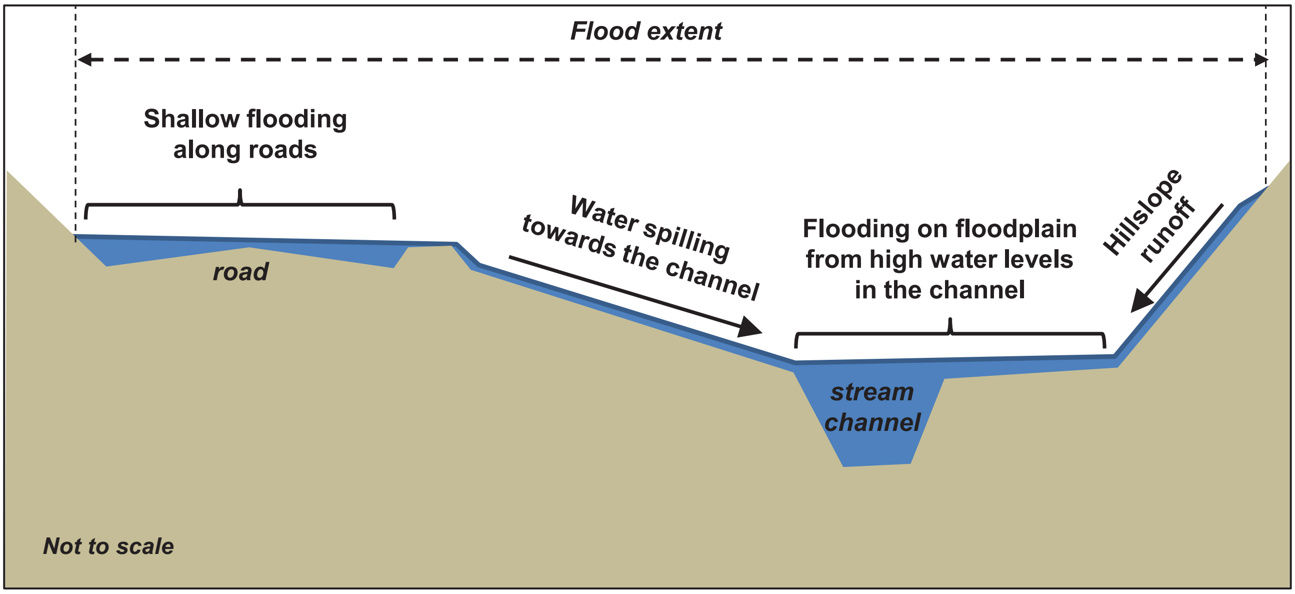

Figure 5.2 Flood extent elements

Similar situations to that at Elmslie Road occur at the other three locations identified by SOH's case studies. In general the flood hazard extents shown on the maps are accurate, in terms of representing the Q100 (incorporating blockage and climate change) flood extent plus freeboard. However, they are open to misinterpretation. Alternative approaches to providing flood extent and hazard information in maps are discussed in Section 6.

5.2 Other issues raised by SOH

While the perceived discrepancy regarding the mapped flood hazard extents is the main issue for SOH, the group members also expressed concern regarding:

- Including culvert blockage in the Q100 map used for defining the flood hazard extent.
- Future development:
  - Was it included in the published flood hazard extent map
  - The small difference between existing and future development flood extents.
- Whether the flood maps are too conservative.
- The terms of reference for the audit in relation to:
  - Stormwater neutrality
  - The impact of future development.

5.2.1 Blockage

SOH are of the opinion that blockage should not be included in baseline modelling used to define floodplain extents. At a separate meeting, representatives of Pinehaven Progressive Association indicated that they were content for blockage to be included in the published flood maps.

As noted in the review of the hydraulic modelling (Section 4.2), the probability and consequence of culverts, bridges and channels being fully or partial blocked during floods by water borne debris is a reality, but there is uncertainty surrounding the timing, location and extent of blockage that may occur during an event. Channel blockage and subsequent breaking of the debris dams caused surges of floodwater during the 1976 flood event.

As such we conclude it should be accounted for in flood hazard mapping. Model runs could be carried out to assess the sensitivity of the modelled flood depth and extents to blockage, and either an allowance for potential blockage included in freeboard, or alternatively a suitable blockage scenario used to better inform the flood extant mapping (which may differ in the flood plain as a result of blockage) but with less freeboard allowed. Separating blockage out from the baseline modelling reduces the opportunity for uncertainty regarding the flood maps.

5.2.2 Future development

The results of the future development scenario modelled by SKM are not included in the Q100 flood map that includes blockage, climate change and freeboard, and which is used for defining the flood hazard extent.

SOH noted the small differences between the existing and future development flood extents for the 100-year ARI including climate change event, as shown in Figure 19 of Volume 1 of SKM's *Pinehaven Stream Flood Hazard Assessment* report, and which is reproduced below as Figure 5.3. In the upper parts of the catchment, existing and future development flood extents are very similar, but larger differences are evident towards the bottom of the catchment.

The edges of the flood extents in the upper catchment are generally steeper than in the lower catchment. As such an increase in flood level due to increased future development runoff will not result in a large increase in the lateral extent of the floodplain. The small scale of Figure 19 in the SKM report makes it difficult to see small increases in floodplain extent.

In the lower reaches, the land is flatter and (due to the most of the flow being culverted to Hulls Creek) the relative difference in existing and future development overland flows means that much of the increased flow spreads across the wider flood plain and there is consequently a greater increase in flood extent, which is visible on SKM's Figure 19.

As described in more detail in Section 8, while SKM's modelling of future development resulted in an increase in modelled peak flows, there was not the expected increase in flood volume. SKM used hydrology provided by MWH. However, MWH have not provided an explanation as to why there is no increase in future development flood volumes. Therefore, SOH's concerns are upheld that the effects of future development on flood extent are not modelled correctly. However, because the peak flow has been increased, and because there is freeboard incorporated into the results, the flood maps are unlikely to be materially affected by this apparent anomaly.

5.2.3 Are the mapped flood and hazard extent conservative?

Based on the model review (Section 4) and consideration of SOH's case studies (Section 5.1), the published flood maps represent the modelled situation appropriately, although there are legitimate concerns over:

- Whether blockage should be included as a separate item to freeboard,
- The level of definition provided by the 5 m grid spacing for the 2D model bathymetry,
- The spacing between surveyed cross-sections, and
- The representation of minor stream crossings.

Figure 5.3 – Q100CC existing and future development flood extents

5.2.4 Audit terms of reference

When the draft terms of reference (ToR) for this audit were discussed at the HVFMS in February 2015, SOH were concerned that the ToR did not:

- *Make existing baselines for assessing stormwater neutrality explicit*
- *Investigate GWRC's assertion that 1,665 new houses on 4 hill sub-catchments will have only a "minor" impact on the catchment, nor address how future developments will be assessed for stormwater neutrality*

These items were included in the final ToR, and are considered in sections 7 and 8 of this report.

6 Presentation of flood hazard information

It is assumed that a map is the best method for displaying flood risk or hazard information for an area. That being the case, it is essential that an effective means of communicating the information is used. As has been noted above, the way that information is presented on the Pinehaven Flood Hazard Map has led to misinterpretation of the flood risk and hazard in the catchment.

Below, alternative ways of presenting flood risk/hazards maps are considered, and a recommendation made as to how flood hazards maps for Wellington Region may be presented in future.

6.1 Alternative practice for flood hazard mapping

When considering how flood risk or hazard is best represented it is worth considering how this is achieved in other jurisdictions, and draw on best practice when proposing a way forward. While one of the recommendations of this audit is that GWRC undertake a thorough review of best practice in New Zealand and overseas, three examples are provide below; Auckland Council, Christchurch City Council, and the Environment Agency in England.

In the two New Zealand cases, the difference between modelled flood extents and the extents incorporating freeboard are differentiated. In England a different approach is taken, with two flood zones being used. This is not a comprehensive review of alternative practice, but a snapshot of some alternatives.

6.1.1 Christchurch

Flood maps for Christchurch are found on the city's public GIS website[6]. The user is able to choose which map layers are displayed, and these include 50-year and 200-year flood extents and corresponding 'Control' areas. The layers are defined[7] as follows:

- **Flood extent (50 year)** – estimated water level in a rainfall event with an average return interval of 50 years or a likelihood of 1/50 (=2%) in any one year.
- **Flood extent (200 year)** – estimated water level in a rainfall event with an average return interval of 200 years or a likelihood of 1/200 (=0.5%) in any one year. This return interval is used in the City Plan Flood Management Areas (FMA) to provide extra protection to areas which are otherwise vulnerable. The

[6] http://maps.cera.govt.nz/advanced-viewer/?Viewer=Ccc-Floor-Levels

[7] http://www.ccc.govt.nz/homeliving/goaheadbuildingplannings00/buildingandplanningprojects-s02/property-s02s0305/floorlevels-s02s0305-08.aspx

viewer shows these areas within the FMA only as they are not used for setting floor levels beyond the FMA.

- **Floor Level Control Areas** – include the 50 year and 200 year flood extents plus the area encompassed by an increase in water level of 400 mm (representing the 400 mm freeboard to floor levels that Council applies in these areas).
- **Flood Management Areas**[8] - were identified in a City Plan change before the Canterbury earthquakes and are areas that are prone to flooding as a result of major tidal or rainfall events and are vulnerable to the effects of rising sea levels.

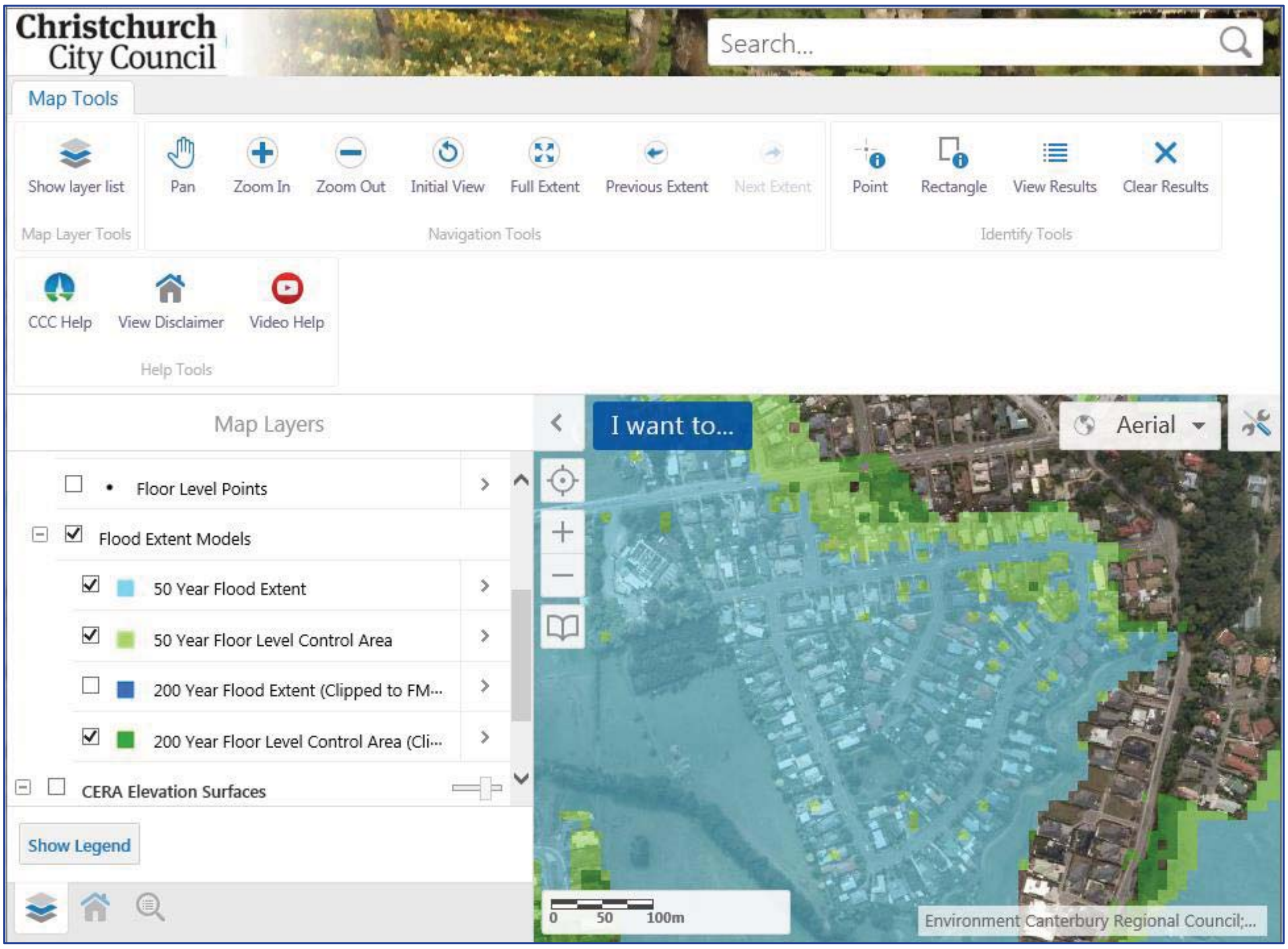

Figure 6.1 – CCC flood map

Though the ARIs of the design events and the amount of freeboard are different, Christchurch's flood control areas are the equivalent of Wellington Regions flood hazard areas. Both represent those areas beyond the design flood event extent in which measures are appropriate to mitigation against the design event and the residual flood risk associated with extreme flood events, unforeseen blockages, and other factors that could

8 http://www.ccc.govt.nz/thecouncil/policiesreportsstrategies/districtplanning/cityplan/proposedvariations/operativevariation48.aspx

increase flood levels beyond modelled levels. However, Christchurch use best estimate of modelled flood levels then explicitly specify the freeboard to be added for setting the finished floor levels in these areas.

6.1.2 Auckland

Figure 6.2 shows an extract from Auckland Council's GIS Viewer, which includes layers for floodplains, flood prone areas, and flood sensitive areas, which are defined as:

- **Floodplains** are areas predicted to be covered by flood water as result of a rainstorm event of a scale that occurs on average once every hundred years. These areas have been produced from hydraulic modelling. The floodplain contains the most up to date information for each of the 23 Stormwater Catchments in the Auckland region. Summary data for each catchment is attributed against each floodplain.
- **Flood prone areas** are topographical depressions. The areas occur naturally, or are created by dammed gullies created by man-made features such as roads and railway embankments. The flood prone extent is the area water will pond up to in a 1% AEP extreme rainfall event assuming the outlet to the topographical depression is blocked.
- **Flood Sensitive Areas** are areas adjacent to the 100yr ARI floodplain that are within 0.5 m of the predicted 100yr ARI flood level. These mapped areas are to ensure the appropriate planning rules are considered for properties developing adjacent to the floodplain

The map also shows overland flow paths, with the line style reflecting the size of surface catchment draining to that area. The overland flow path does not necessarily indicate that flood will occur along its length, as the stormwater network will convey water.

For Auckland, the Flood Sensitive Areas are the equivalent of Wellington Region's flood hazard areas and Christchurch's flood control areas.

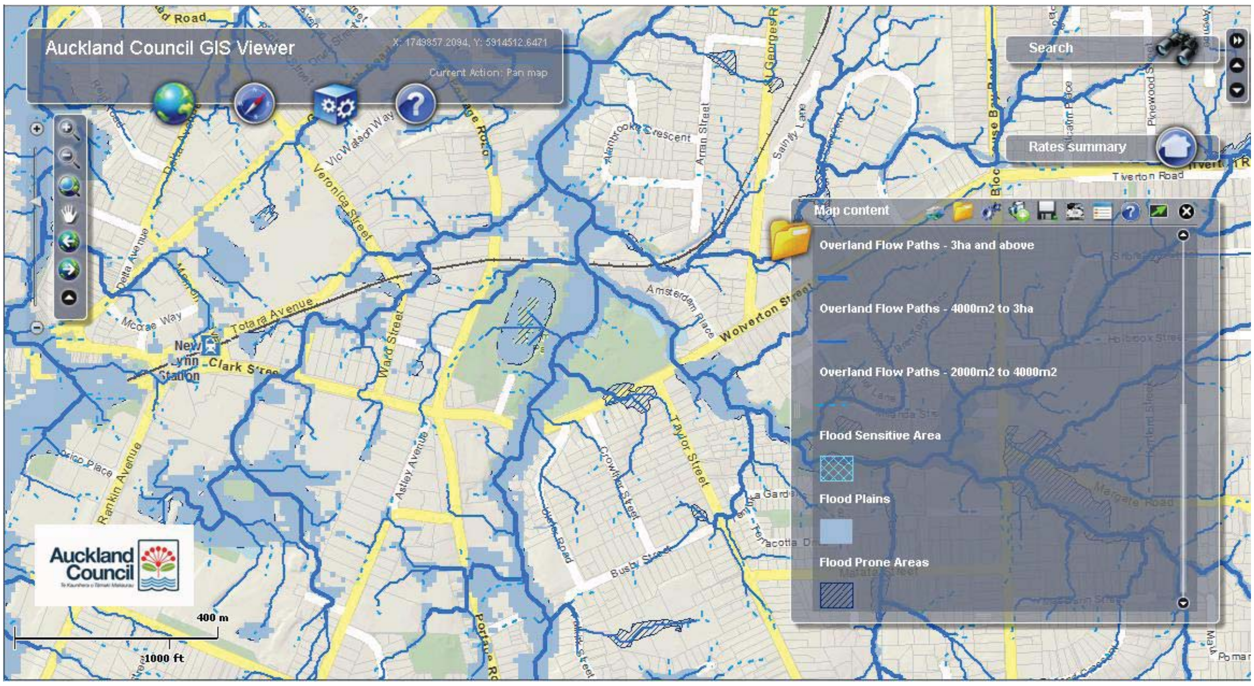

Figure 6.2 – Auckland Council GIS Viewer flood layers

6.1.3 Environment Agency in England

Rather than each council having a separate approach to defining and presenting flood risk/hazard information, the information is providing a uniform manner across England. The information is provided by the Environment Agency, and forms part of the planning process. Figure 6.3 shows the ***Flood Map for Planning***[9][10] for York in the North of England.

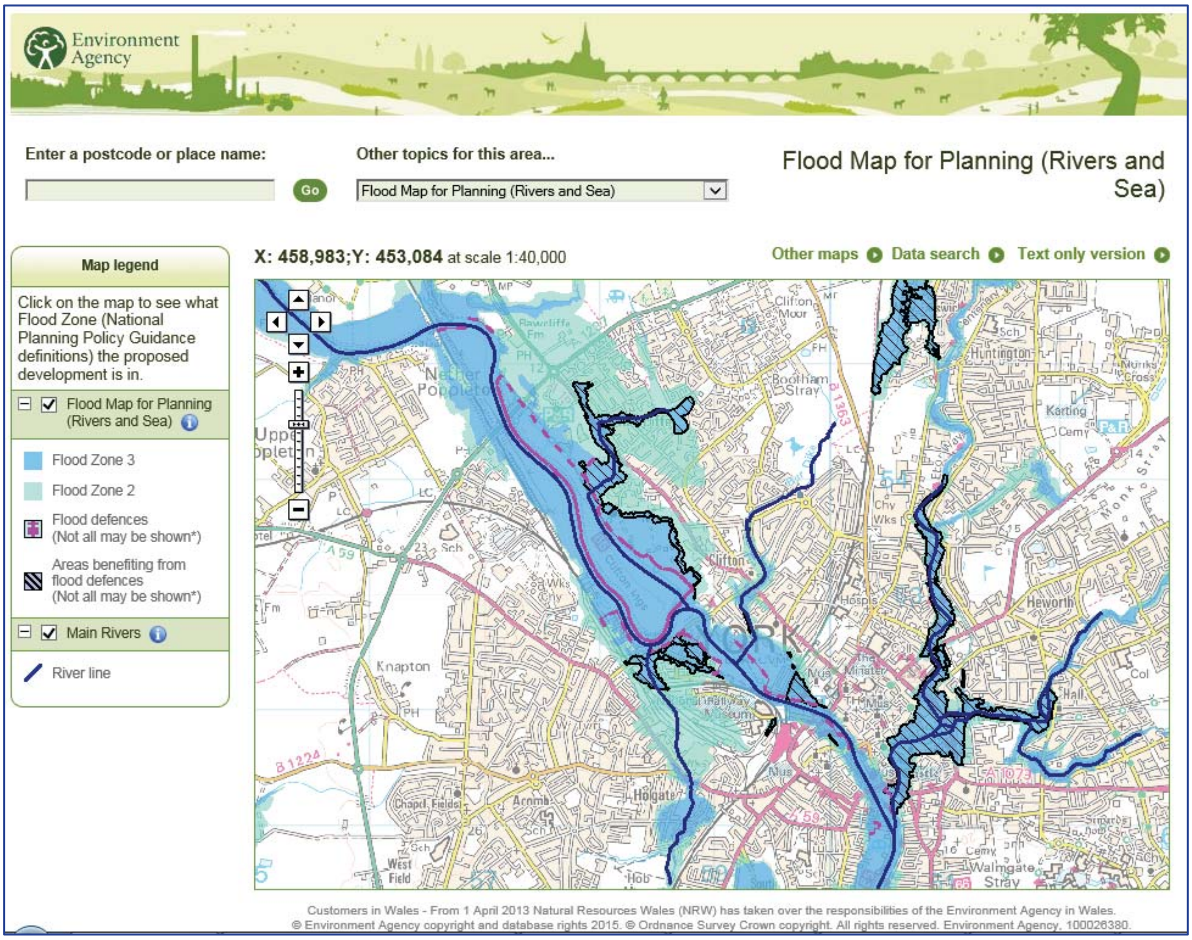

Figure 6.3 – Environment Agency Flood Map for Planning

There are two different kinds of area shown on the Flood Map for Planning:

- **Dark blue** ■ (Flood Zone 3) shows the area that could be affected by flooding, either from rivers or the sea, if there were no flood defences. This area could be flooded:
  - from the sea by a flood that has a 0.5 per cent (1 in 200) or greater chance of happening each year;
  - or from a river by a flood that has a 1 per cent (1 in 100) or greater chance of happening each year.
- **Light blue** □ (Flood Zone 2) shows the additional extent of an extreme flood from rivers or the sea. These outlying areas are likely to be affected by a major flood, with up to a 0.1 per cent (1 in 1000) chance of occurring each year.

[9] http://apps.environment-agency.gov.uk/wiyby/37837.aspx

[10] http://planningguidance.planningportal.gov.uk/blog/guidance/flood-risk-and-coastal-change/

Where there is no blue shading, this shows the area where flooding from rivers and the sea is very unlikely. There is less than a 0.1 per cent (1 in 1000) chance of flooding occurring each year. The majority of England and Wales falls within this area. For planning and development purposes, this is the Flood Zone 1.

With regard to development control, the flood zones are used in conjunction with the *Sequential Test* to steer vulnerable development away from areas of high flood risk. Where there is no option but to develop in flood zone, an *Exception Test* is applied to minimise the risk and consequences of flooding through the adoption of mitigation measures.

The flood extents do not include freeboard. Rather, an allowance is made for freeboard when setting floor levels and flood sensitive infrastructure through the development control and planning process.

6.2 Suggestions for future flood hazard mapping for Pinehaven

6.2.1 Map format

The common theme of the Christchurch and Auckland flood maps is that users can clearly differentiate between the modelled flood extents (or floodplain) and the areas included when freeboard is applied, and in which flood risk should also be considered and mitigated against. This indicates that Auckland and Christchurch recognise that users, including the local community, are able to understand the difference between modelled flood extents and the ‘buffer’ zones represented by the Flood Control Areas (Christchurch) and Flood Sensitive Areas (Auckland).

With the areas differentiated, users can see how the flood maps are drawn up, which will increase understanding of the maps’ purpose. With this approach, Council can still define the flood sensitive margins as requiring consideration from a flooding point of view.

For example, describing the area covered by freeboard beyond the modelled flood extent as a Flood Sensitive Area may be more transparent and more appropriate than GWRC’s use of all-encompassing Flood Hazard Areas. Changing the name would allow GWRC to provide true flood hazard maps, based on the combination of water depth and flow velocity at any location. These flood hazard maps can be particularly informative in areas where flood extents are large, but there is also deep or fast flowing water in defined flow paths or depressions. Figure 6.4 shows how flood hazard is defined in Hamilton, while Figure 6.5 shows an example of a flood hazard map from the UK based on similar principles.

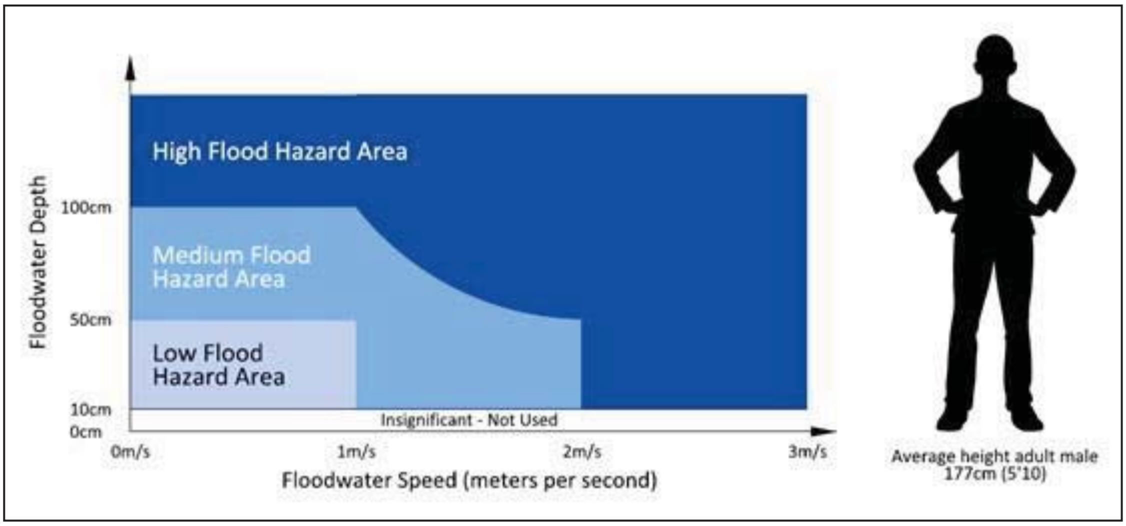

Figure 6.4 – Definition of flood hazard[11]

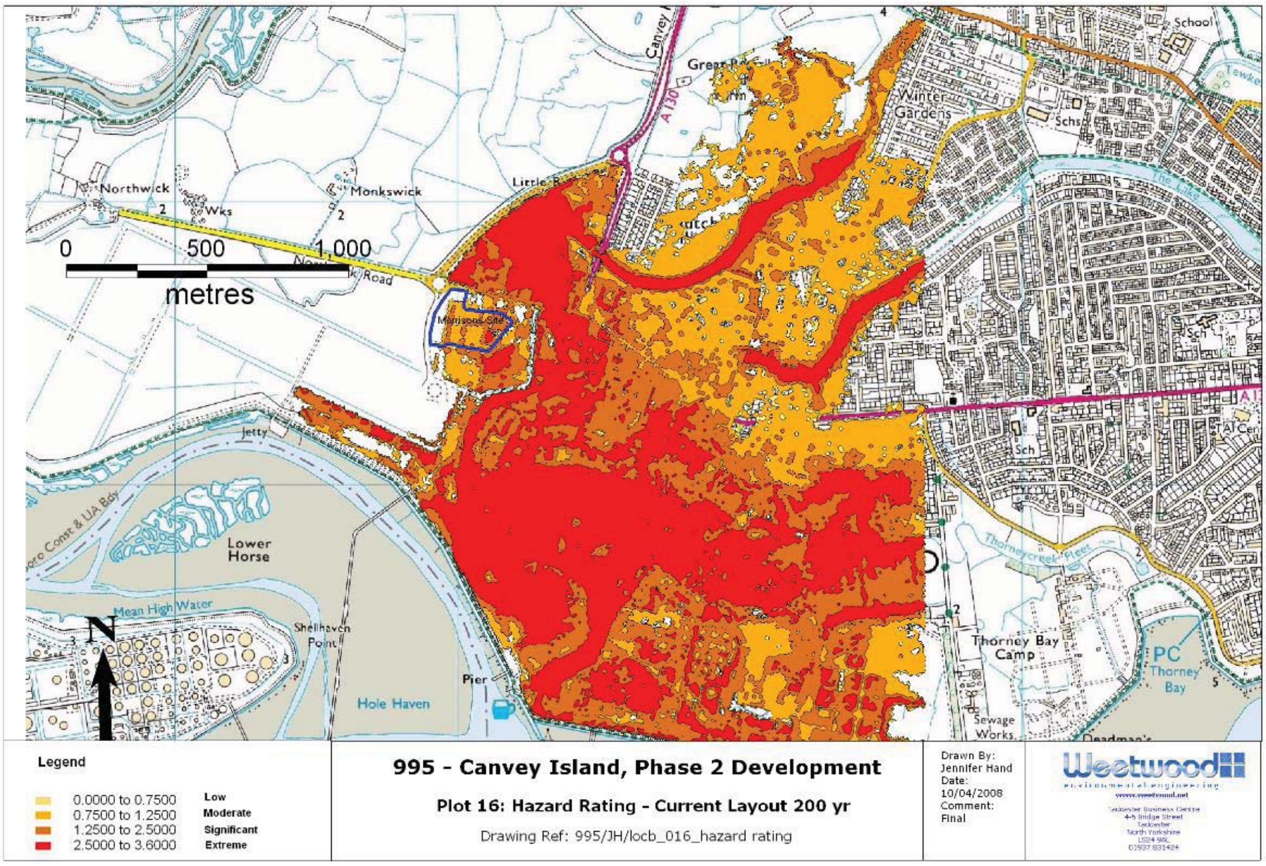

Figure 6.5 – Flood Hazard Map

It is recommended that for Pinehaven, flood maps should show the following map layers, which would provide greater clarity:

- **Flood extent (10-year ARI):** Model extent of flooding in a rainfall event with an average return interval of 10 years, and incorporating climate change to 2090, as already provided.
- **Flood extent (100-year ARI):** Model extent of flooding in a rainfall event with an average return interval of 100 years, and incorporating climate change to 2090.
- **Flood Hazard Map (100-year ARI):** Areas within the 100-year ARI flood extent are defined by flood risk ranging from Low to High, based on an assessment of flood depth and flow velocity. The information required to generate these maps can be extracted from the existing flood models.
- **Flood Sensitive Areas:** Model extent of flooding in a rainfall event with an average return interval of 100 years, and incorporating:
  - Climate change to 2090
  - The application of freeboard to the modelled flood extents. Freeboard will include the potential effect of channel/culvert blockage by debris.

It is noted that different depths of freeboard are applied across the Pinehaven catchment. This is an acceptable approach, allowing freeboard to vary with location and risk profile.

[11] http://www.hamilton.govt.nz/our-council/council-publications/districtplans/flood/Pages/Flood-FAQ.aspx

6.2.2 Accessing flood information

It should be easy for the community to find flood map information. Internet searches for “Flood Map” for Wellington, Auckland and Christchurch yield varying levels of success in finding flood maps. Some links lead to web pages stating that flood mapping has been carried out, but all too often there is no link to a map viewer. In other cases, PDF versions of maps are provided at a scale that does not allow close examination of specific addresses or location.

An internet search for “Pinehaven Flood Hazard Map” yields links to PDFs of Flood Hazard Information Sheets for the catchment, which include the maps. However, large scale copies of the flood extent and hazard maps are not readily found.

7 Storm water neutrality provisions

As part of the implementation of the floodplain management plan, Upper Hutt City Council will set storm water neutrality controls through the District Plan. The council is seeking independent guidance about how these should be established and how these should be measured.

The main purpose of hydraulic neutrality is to not increase the risk of flooding elsewhere in the catchment.

Volume One of the Proposed Kāpiti Coast District Plan defines Hydraulic Neutrality as “a nil increase in the peak stormwater runoff discharged from new subdivision, new buildings and/or new land use activities undertaken on the site.”[12]

Peak flow is just one measure of the changes in hydrological response due to development. Increases in peak flow are caused by a combination of a reduction in permeable area and quicker runoff from smoother post-development channels and overland flow paths. The decrease in permeable area is also likely to result in an increase in flood volume.

Therefore to be truly neutral, the post-development runoff should match pre-development runoff peak flows, runoff volume and timing of runoff at the outlet from the development area. In practice, this is difficult to achieve, hence the focus on limiting peak flow to no more than pre-development peak flows. Peak flows can be reduced by providing storage within the development to attenuate the flow hydrograph.

The effect of attenuation is to release storm runoff later than would have occurred without storage in the expectation that flood levels throughout the receiving catchment will be receding when the water is released and so peak flood levels are not increased. This approach works where flood volume is not the critical factor in determining flood levels and where attenuated flows do not coincide with peak flows arriving from other parts of the receiving catchment that have longer times of concentration. Further, the increase volume of runoff means that there is increased likelihood of high runoff from sub-catchments coinciding. It may be that for developments close to bottom of catchments, it is better to have little attenuation and allow discharge of peak flows early in the event so that they have passed out of the catchment before peak flows arrive from the upper parts of the catchment.

Therefore, developments need to be considered on a case by case basis, and appropriate conditions for managing hydrological neutrality applied. Initial guidelines for comment would include:

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[12] http://www.kapiticoast.govt.nz/contentassets/68a0006af1314ac3b1f1570d37a2763c/chapter01-introduction-and-interpretation.pdf

- Conditions for post-development flow peaks and volumes should be applied at the outlet from the development site, as hydrological conditions elsewhere in the catchment are beyond the control of the developer.
- Generally, in urban and growth areas post-development peak flows should be no more than 80% of pre-development peak flows. Providing a 20% reduction in peak flows is used in other Council areas, such as Kāpiti Coast, and provides the opportunity for betterment or off-setting any negative effects of increased flow volumes. The exception to implementing this rule would be where it can be demonstrated through modelling that quick release of runoff from the site is beneficial to reducing flood risk elsewhere in the catchment.
- The developer should undertake an assessment of the receiving catchment to determine whether flood volumes are a significant factor in determining peak flood levels and extents. Where that is the case, modelling should be carried out to demonstrate to the Council's satisfaction that the risk of flooding is not increased elsewhere in the catchment. However, caution should be used when modelling individual developments, as this does not adequately address the cumulative effects of multiple developments where zoning or growth strategies allow such developments.

The Council should be clear as to the event or range of events for which hydrological neutrality should be achieved. While proposed stormwater management structure may attenuate post-development peak flows to 80% of pre-development peak flows in the 10-year ARI storm (for example), consideration should also be given to the effects in other ARI events. Will the developer be required to match 80% flows in those events, as well? It may not be possible to provide hydrological neutrality for all events.

8 Defining the impact on runoff of development

As part of the flood hazard study carried out by SKM, a future case scenario was modelled to determine the impact of a future development scenario for the Pinehaven Hills. In undertaking this modelling, assumptions were made about the runoff changes that would occur as a result of future development, based on:

- 1665 lots
- Average lot size of 750 m^2
- 40% increase in impermeable area across the affected sub-catchments

Figure 8.1 shows the change in flood hydrographs for existing development (*E4_Q100CC_2hr_HB.bnd11*) and future development (*E4_Q100CC_FP_2hr_HB.bnd11)* for sub-catchment B, which is in the southwest of the catchment and drains to the top of Pinehaven Road. Future development increases the peak flow by 18% (from 3.07 m^3/s to 3.64 m^3/s), and the flow recession is steeper than for the existing land use. However, the flood volume does not increase. This is unexpected, as increasing the impervious area of sub-catchment by 40% to reflect the development would be expected to reduce rainfall losses and increase runoff volume. Similar results were found for sub-catchment E, which drains to Wyndham Road.

Assuming a 100-year ARI plus climate change rainfall depth of 87.1 mm for the 3-hour storm, an Initial Loss of 5 mm, Ongoing Loss of 2 mm/hr, and 40% impermeable area for the affected post-development sub-catchments, then the effective rainfall depths would be;

- 76.7 mm (88%) for existing land use
- 80.8 mm (93%) for post-development land use

The difference between existing and post-development flood volumes would be expected to be to a similar ratio. The existing ground cover of bush and pine forest on sloping catchments generated relatively high runoff, when compared to natural vegetation on flatter ground. This is reflected in the 88% effective rainfall for the existing situation and only 5.6% increase in effective rainfall post-development.

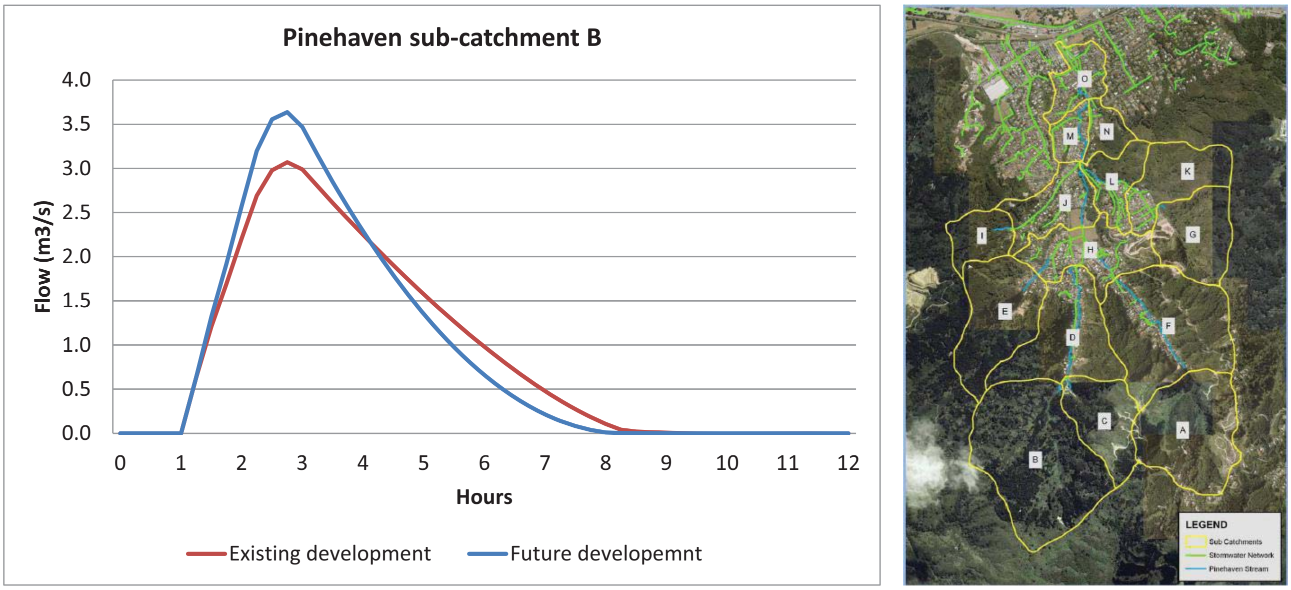


Figure 8.1 – Existing and maximum probable development hydrographs

Lower density development would have a smaller effect on peak flows and flood volumes. With reference to Section 7, it is highly unlikely that post-development runoff from developments on the Pinehaven Hills would be consented to discharge to the streams without attenuation to at least match, or reduce, peak flows.

The greatest effect on un-attenuated flood flows as a result of development on the Pinehaven Hills will be seen in the upper catchments, as this is where there is the greatest relative change in modelled impervious area. Further down the catchment, the relative change in impervious areas reduces and so the difference in modelled flows will be less.

The issue of no increase in post-development flood volume was raised with MWH, but they have not been able to provide an explanation as to why there is not an increase in flood volume. While this does not affect the validity of flood extents defined for current development, it does invalidate the post-development flood extents and reduces community confidence in the flood mapping process.

9 Conclusions and Recommendations

9.1 Conclusions

The hydrological and hydraulic modelling used to derive the flood hazard maps is fit for purpose. The methods and level of detail reflected the catchment information and modelling methods available in 2008-2010.

However, the way that the flood extent and hazard maps are presented obscures the components that have been used to derive the extents. Describing the ‘flood extent plus freeboard’ maps as Flood Hazard Maps does not adequately describe the Maps. These issues lead to confusion and misunderstanding within the community regarding the interpretation and use of the maps. As such, the presentation of flood information in map form should be modified.

9.2 Recommendations

9.2.1 Hydrological and hydraulic modelling

The modelling underlying the flood maps is 6-7 years old. The flood maps will be updated as new information becomes available or changes in the catchment occur and in conjunction with District Plan and Regional Planning Review work programmes. The community should be made aware of this, and understand that mapped flood extents may be refined as a result of revised modelling and mapping in the future.

Better definition of flood depths, extents and overland flow paths could be provided if the modelling were updated to account for current computer processing power and advances in modelling software. Specifically:

- Review hydrological modelling:
  - To account for longer rainfall records
  - Ensure that ‘future development’ hydrology is correctly modelled
  - With reference to flood flows for hydrologically similar catchments to validate design flood hydrographs for the ungauged Pinehaven catchment.
- The use of a finer grid or flexible mesh to construct the 2D model bathymetry would provide better definition of flood extents and overland flow paths.
- Blocking out buildings with the 2D model bathymetry would improve definition of overland flow paths and should be considered if the models are to re-run

While advances in modelling methods and available information since 2009 could be used to improve aspects of the modelling, it is unlikely that updating and upgrading the models would significantly alter the flood extents and depths for the current design flood events and scenarios modelled.

9.2.2 Presentation of flood maps

A limited review of how flood extents and risk are mapped elsewhere indicates alternative approaches that GWRC could utilise to improve the understanding and acceptance of the Pinehaven flood mapping and the modelling that underlies the maps. For example, using the same modelling results, the flood maps could show the following map layers:

- **Flood extent (10-year ARI):** Model extent of flooding in a rainfall event with an average return interval of 10 years, and incorporating climate change to 2090.
- **Flood extent (100-year ARI):** Model extent of flooding in a rainfall event with an average return interval of 100 years, and incorporating climate change to 2090.
- **Flood Hazard Map (100-year ARI):** Areas within the 100-year ARI flood extent are defined by flood risk ranging from Low to High, based on an assessment of flood depth and flow velocity.
- **Flood Sensitive Areas:** Model extent of flooding in a rainfall event with an average return interval of 100 years, and incorporating:
  - Climate change to 2090
  - The application of freeboard to the modelled flood extents. Freeboard will include the potential effect of channel/culvert blockage by debris.

Given that the maps are to be used for planning purposes, the inclusion of an allowance for climate change to a suitable horizon is appropriate, and this has been included.

However, it is recommended that GWRC undertake a review of best practice flood mapping in New Zealand and overseas so as to ensure that flood risk and hazard information is communicated clearly to the community, and is able to still be applied in a robust manner.

9.2.3 Stormwater neutrality

The issue of including stormwater (or hydrological) neutrality into local planning guidelines is complicated. Developments should be considered on a case by case basis, and appropriate conditions for managing hydrological neutrality applied. General guidelines for comment would include:

- Conditions for post-development flow peaks and volumes should be applied at the outlet from the development site.
- Post-development peak flows in urban or urbanising areas should be no more than 80% of pre-development peak flows so as to provide opportunities for betterment or off-setting any negative effects of increased flow volumes and consequent greater coincidence of subcatchment peaks.
- The developer should demonstrate to the Council's satisfaction that the risk of flooding is not increased elsewhere in the catchment.
- Caution should be used when modelling individual developments, as this does not adequately address the cumulative effects of multiple developments where zoning or growth strategies allow such developments.

Confirming the main conclusion of the audit; the hydrological and hydraulic modelling underlying GWRC's flood extent and hazard maps is fit for purpose, but the way that flood information is presented in map form could be modified, which may increase the understanding and acceptance of the maps by the community.

Appendix A – Terms of Reference

Section 7 of the ***Request for Proposal - Pinehaven Stream Flood Mapping Audit.*** *WGN_DOCS-#1437397-v3-ToR_Pinehaven_Stream_FMP_Audit.doc* Request For Proposal (RFP)

7. Terms of reference for audit

The audit will comment on the appropriateness and fitness for purpose of the following criteria. We invite additional suggestions for assessment criteria as part of the proposal.

7.1 General

The following are *general* assessment items to be included in the audit;

- The type of software and modelling package used for the hydrology and hydraulic model
- The modelling method used and its appropriateness for both hydrology and the hydraulic model
- The use of freeboard and method by which it was applied
- Representation of the flood hazard through way in which maps are displayed and information provided

7.2 Numbers

The assessment of the *numbers* used to create the flood model shall include;

- Rainfall data
- Measured flood flows
- Cross section surveys
- Lidar surveys

7.3 Assumptions

The assessment of *assumptions* used to create the flood maps include;

- Run-off coefficients
- Predicted flood flows
- Roughness coefficients of the channel
- How the buildings and structures on the floodplain are treated through use of roughness coefficients
- Treatment of bridges, culverts and pipe crossings
- Use of freeboard to define flood hazard
- How the freeboard has been applied to the model and suitability of the freeboard values used

7.4 Additional Work

In addition to the key audit tasks above, it has been agreed with the community that the following additional investigations would be carried out by the appointed auditor.

7.4.1 Guidance on how to set storm water neutrality provisions within district plan

As part of the floodplain management plan implementation, Upper Hutt City Council will set storm water neutrality controls through the District Plan. The council is seeking independent guidance about how these should be established and how these should be measured.

This independent guidance will be considered when developing the plan change that will incorporate these controls.

Key information sought is;

- How to establish a base line against which any development proposal will be measured in a District Plan context
- What are appropriate levels at which to set controls

7.4.2 Guidance on how to define the impact of intensification of development on the run off characteristics of the Pinehaven hills

As part of the flood hazard study carried out by SKM, a future case scenario was carried out to determine the impact of a worst case development scenario for the Pinehaven Hills. This made some assumptions about the run off changes that would occur as a result of this development.

We would like a comment on assumptions about the impact of intensification of development within the Pinehaven catchment and how this would affect the run-off characteristics of the current usage if it was changed from pine forest into a partly developed or intensively developed area.

Key information sought is;

- What impact a high intensity development may have on run-off from the Pinehaven hills area
- What impact a medium intensity development may have on run-off from the Pinehaven hills area
- What impact a low intensity development may have on run-off from the Pinehaven hills area

Appendix B – Large Figures

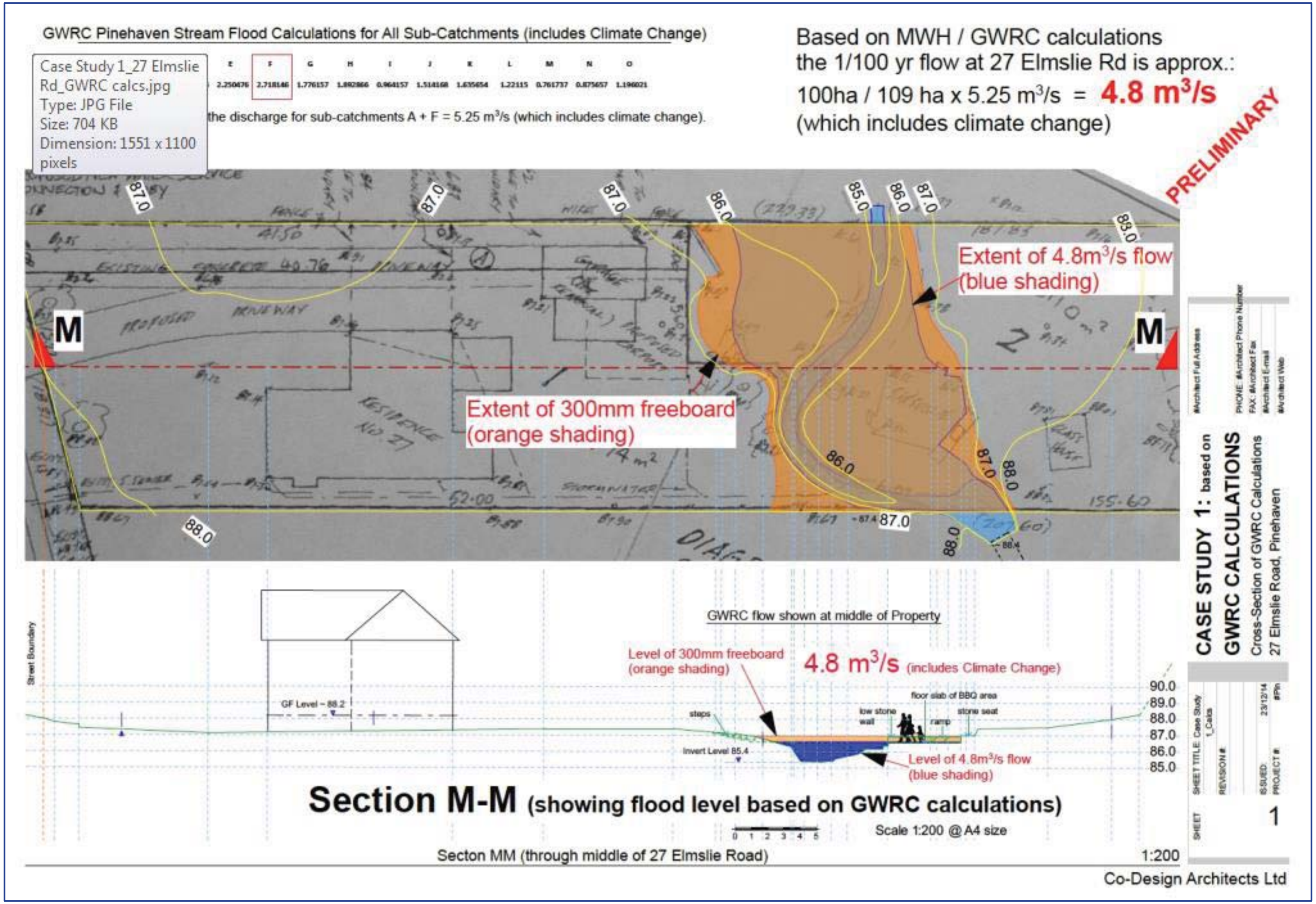


Figure B.1 – SOH slide 1 for 27 Elmslie Road

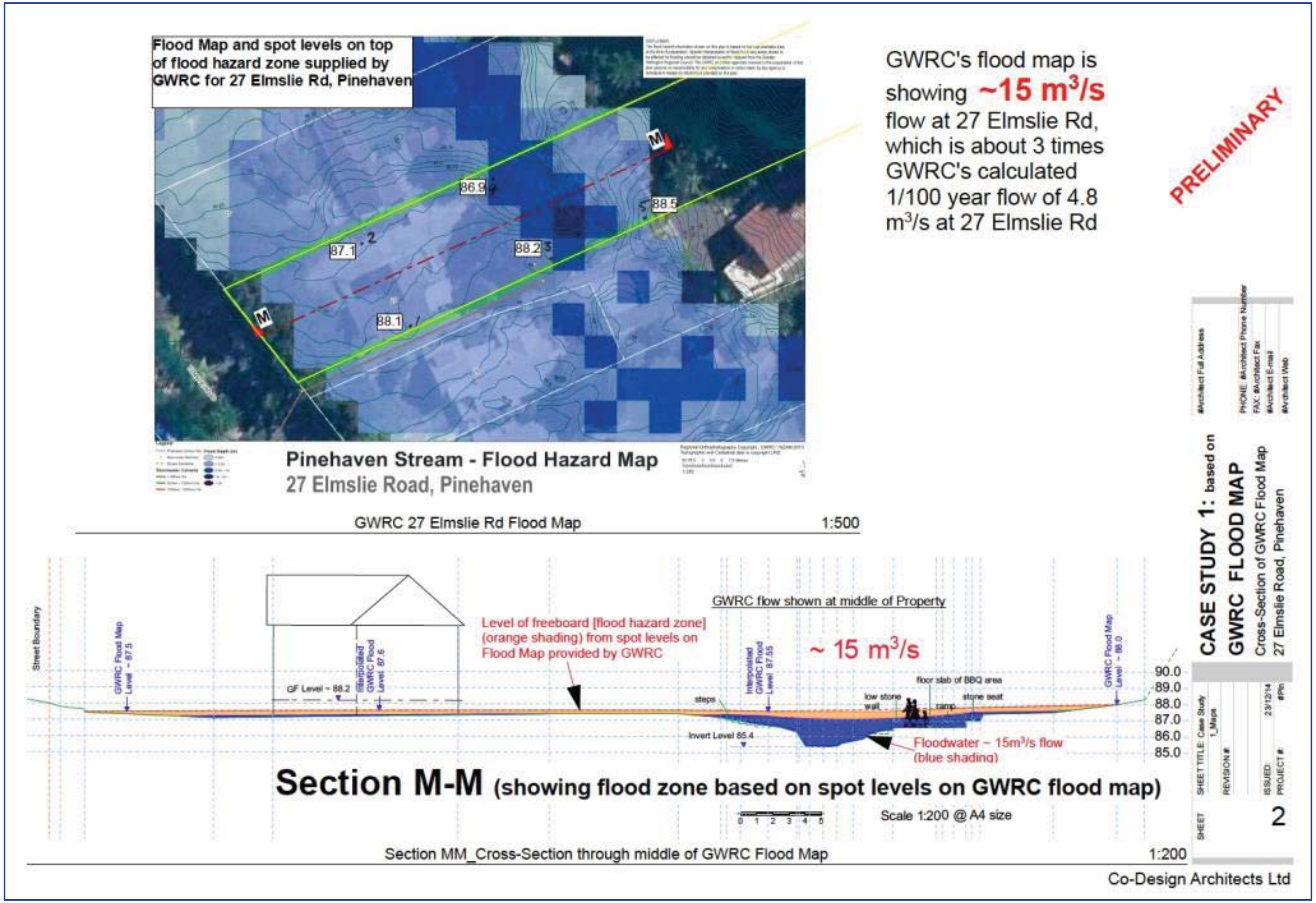


Figure B.2 – SOH slide 2 for Elmslie Road

COMBINED DATA — Nulls are yellow

Terrain — Chainage 638 (Parallel to the upstream boundary)

X	Y	Water Level, Q100CC
-30.65	88.61	
-25.65	87.66	87.66
-20.65	87.83	87.83
-15.65	87.34	87.45
-10.65	87.81	87.81
-5.65	87.77	87.82
-0.65	87.82	87.91
4.35	87.48	87.53
5.96	86.99	87.53
8.00	86.57	87.53
8.42	86.30	87.53
9.23	86.39	87.53
10.07	86.90	87.53
10.82	87.13	87.53
11.23	87.47	87.53
11.23	87.73	87.79
16.23	88.26	88.27
21.23	89.20	
26.23	90.20	
31.23	91.45	
36.23	93.10	

Terrain — Chainage 659

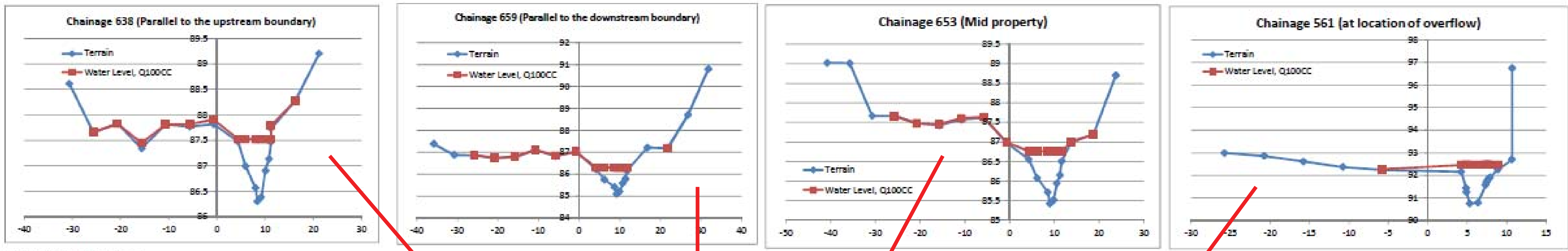
X	Y	Water Level, Q100CC
-35.75	87.39	
-30.75	86.88	
-25.75	86.86	86.87
-20.75	86.73	86.74
-15.75	86.79	86.81
-10.75	87.10	87.11
-5.75	86.82	86.84
-0.75	87.02	87.03
4.25	86.22	86.30
6.24	85.76	86.30
8.76	85.41	86.30
9.28	85.11	86.30
10.00	85.20	86.30
10.77	85.60	86.30
11.45	85.79	86.30
11.81	86.16	86.30
16.81	87.21	
21.81	87.19	87.19
26.81	88.71	
31.81	90.80	

Terrain — Chainage 653

X	Y	Water Level, Q100CC
-40.73	89.02	
-35.73	89.01	
-30.73	87.67	
-25.73	87.66	87.66
-20.73	87.47	87.48
-15.73	87.43	87.45
-10.73	87.57	87.61
-5.73	87.61	87.64
-0.73	86.98	86.99
4.27	86.56	86.76
6.16	86.08	86.76
8.56	85.72	86.76
9.05	85.43	86.76
9.80	85.52	86.76
10.58	85.95	86.76
11.28	86.15	86.76
11.66	86.51	86.76
13.78	87.00	87.00
18.78	87.19	87.19
23.78	88.71	

Terrain — Chainage 561

X	Y	Water Level, Q100CC
-25.76	92.99	
-20.76	92.86	
-15.76	92.61	
-10.76	92.37	
-5.76	92.23	92.27
4.24	92.16	92.46
4.84	91.44	92.46
4.90	91.25	92.46
5.32	90.75	92.46
6.35	90.79	92.46
7.32	91.57	92.46
7.42	91.68	92.46
7.51	91.75	92.46
7.67	91.84	92.46
7.91	91.92	92.46
8.89	92.26	92.46
10.67	92.70	
10.70	96.75	
14.91	101.96	



Peak Flows at 27 Elmslie Road
100 Year Rainfall with the predicted impacts of climate change
In channel 3.4m3/s
On Floodplain 1.0m3/s

Crossection Averaged Peak Velocities at 27 Elmslie Road
100 Year Rainfall with the predicted impacts of climate change
0.9m/s

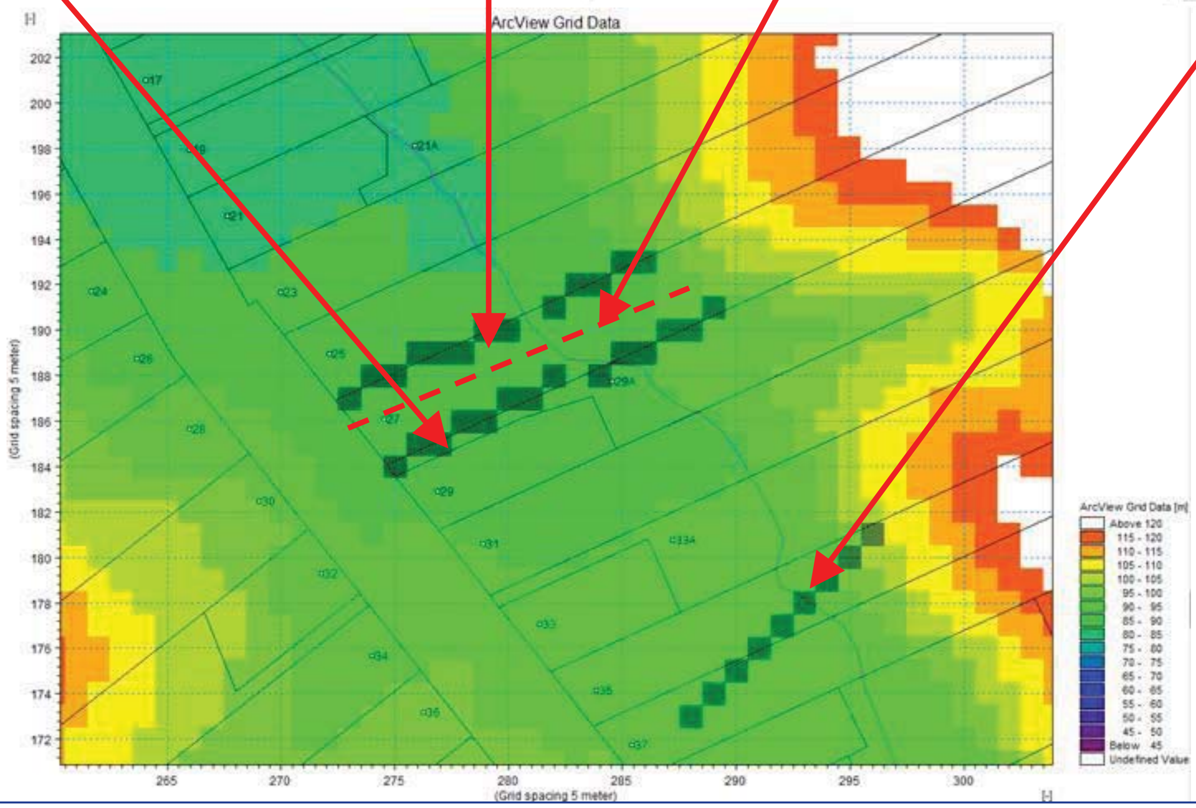


Figure B.3 – SKM Jacobs example cross-sections at 27 Elmslie Road

Appendix D Information relating to speakers at the hearing

WELLINGTON WATER LIMITED SPEAKERS' INFORMATION				
Speaker[8]	Technical expertise (if any)	Topic(s) on which they will be presenting	Providing evidence prior to the hearing?	Length of time (including anticipated questioning)
Nicky McIndoe	N/A	Legal submissions	No	1 hour 30 minutes
Ben Fountain	N/A	Project need	Yes	20 minutes
Eric Skowron	Engineering	Project overview	Yes	20 minutes
Peter Kinley	Flood modelling	Flood model design and flooding effects	Yes	40 minutes
Tim Haylock	Civil engineering	Construction methodology and effects	Yes	20 minutes
Claire Conwell	Water quality	Erosion and sediment control and water quality effects	Yes	20 minutes
Adam Forbes	Ecology	Terrestrial ecology effects	Yes	20 minutes
Alex James	Ecology	Aquatic and riparian ecology effects	Yes	20 minutes
David Compton-Moen	Landscape architecture and visual impact	Landscape and visual effects	Yes	20 minutes
Helen Anderson	Planning	Planning	Yes	30 minutes

[8] The order of speakers presented is the proposed order of appearance.