WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Catchment Hydrology

Flood Mitigation Adaptation for Painkalac Creek Estuary, Aireys Inlet

Surf Coast Shire

13 June 2025

Document Status

Version	Doc type	Reviewed by	Approved by	Date issued
V01	Report	J Theilemann	J Theilemann	13/06/2025

Project Details

Project Name	Flood Mitigation Adaptation for Painkalac Creek Estuary, Aireys Inlet
Client	Surf Coast Shire
Client Project Manager	Sharna Whitehand
Water Technology Project Manager	William Hibberd
Water Technology Project Director	Toby Devlin
Authors	Michael Clarke
Document Number	25010180_R03V01_CatchmentHydrology.docx

51 Little Fyans Street
Geelong VIC 3220
Telephone (03) 8526 0800
ACN 093 377 283
ABN 60 093 377 283

Certified System Quality ISO 9001 SAI GLOBAL
Certified System Environment ISO 14001 SAI GLOBAL
Certified System OHS ISO 45001 SAI GLOBAL

CONTENTS

APPENDICES

LIST OF FIGURES

LIST OF TABLES

1 INTRODUCTION

This catchment hydrology report is one of multiple key deliverables of the wider project; Flood Mitigation Adaptation for Painkalac Creek Estuary, Aireys Inlet. The objective of the study is to develop an understanding of climate impacts on the flood risk within the Painkalac Estuary and the inundation and erosion hazards on the Aireys Inlet and Fairhaven open coastline.

This report presents the hydrological assessment of the Painkalac Creek catchment. The catchment is located on the Otway coast of Victoria and outlets to the ocean at Aireys Inlet. Catchment hydrology has been assessed in line with Australian Rainfall and Runoff version 4.2. A flood frequency analysis and hydrological modelling has informed the design hydrology. Design hydrographs have been prepared, which have been applied to a separate hydrodynamic model.

The Painkalac Creek catchment contains a significant storage, Painkalac Dam, which was built in 1978 and has a capacity of ~532 ML. The dam was originally constructed to supply water to Aireys Inlet and Fairhaven, however the towns were connected to the greater Geelong water supply system in 2016. The catchment is largely forested with the exception of the cleared floodplain and developed areas of the towns.

The location of the catchment and significant nearby waterways are shown in Figure 1-1 below. Also shown are stream gauges utilised in the study, with the Cumberland River at Lorne adopted as a hydrologically similar catchment for the purpose of extending the flood frequency analysis annual series.

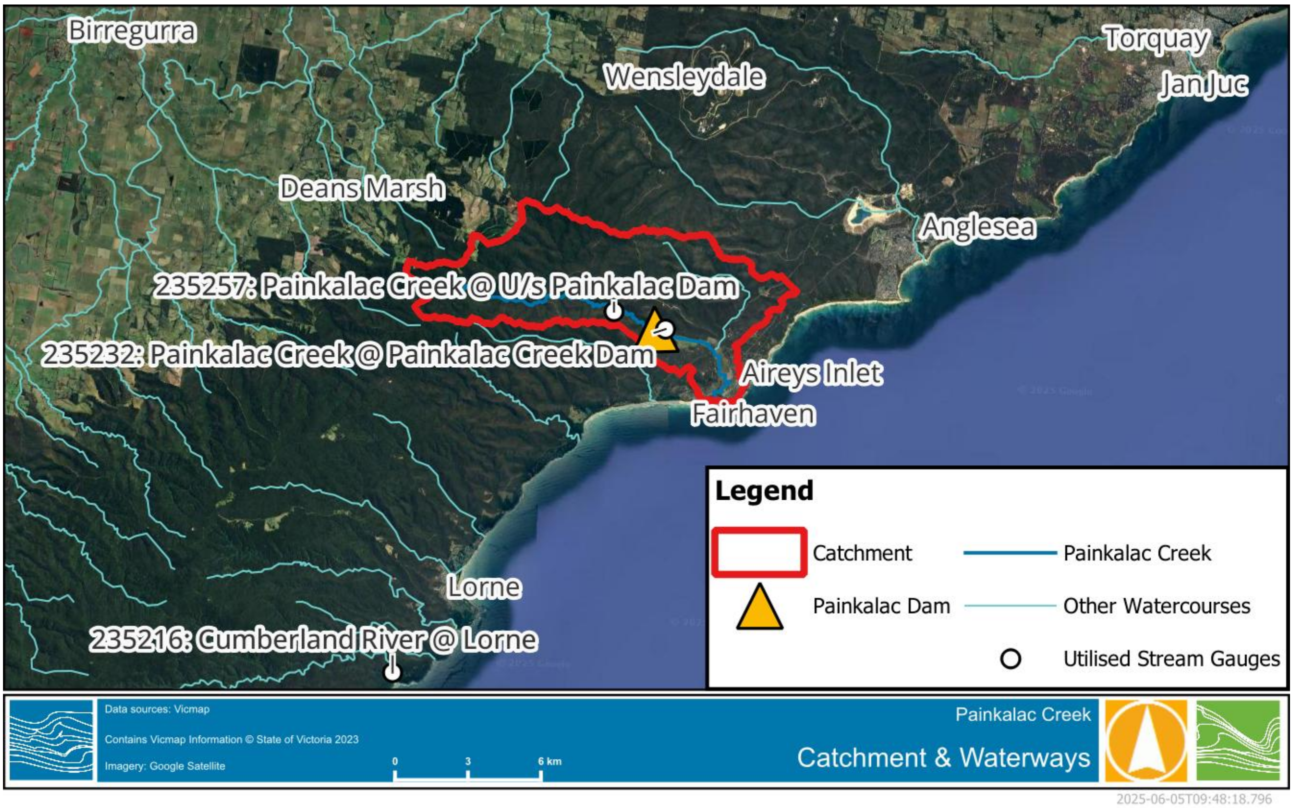

Figure 1-1 Catchment and Waterways

2 HYDROLOGY

2.1 Flood Frequency Analysis

2.1.1 Overview

A flood frequency analysis (FFA) has been performed on the Painkalac Creek @ Painkalac Creek Dam (235232) gauging site. The site has gauge records from 1974 to present, with a gap in the data from 1992 to 1998 inclusive. A total of 45 years of record are suitable for use in the flood frequency analysis. Records at nearby gauges were inspected to determine if a neighbouring catchment could be used to extend this record, with the Cumberland River at Lorne (235216) identified as being potentially suitable.

The extended annual series was analysed using RMC-BestFit version 2.0 (beta). RMC-BestFit applies a Bayesian analysis to fit the data with various distributions and has been shown to produce extremely similar results to FLIKE, another Bayesian analysis software which is recommended in Australian Rainfall and Runoff Book 3 as being suitable for FFA (noting that Book 3 does not preclude the use of other software).

2.1.2 Annual Series

The Cumberland River at Lorne (235216) gauge has a continuous record from 1971 to present with additional daily observations made from 1966 to 1971 and is the only nearby gauge with a longer record than Painkalac Creek @ Painkalac Creek Dam. A comparison was made between the annual maxima for the years where both gauges recorded continuous data, as shown in Figure 2-1 below. While the correlation between the two separate catchments is not perfect it is significant enough to justify using the additional data to extend the annual maxima record, noting that the extended record may have some associated uncertainty.

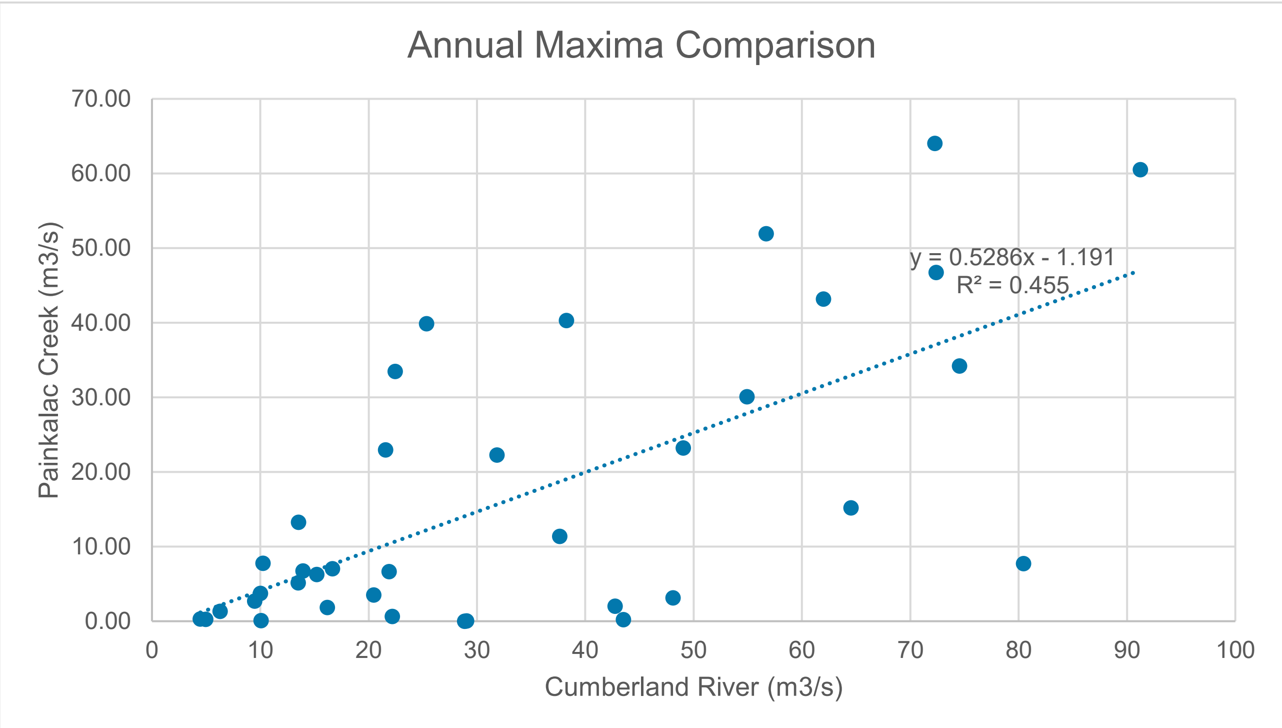

Figure 2-1 Comparison of annual maxima, Painkalac Creek @ Painkalac Creek Dam vs Cumberland River @ Lorne

The extended record allows infilling of the years 1992-1998 inclusive, along with use of data for 1966-1970 inclusive as peaks over threshold given the data for this period is daily observations rather than continuous monitoring.

The annual series is shown graphically in Figure 2-2 and tabulated in Table 2-1 below.

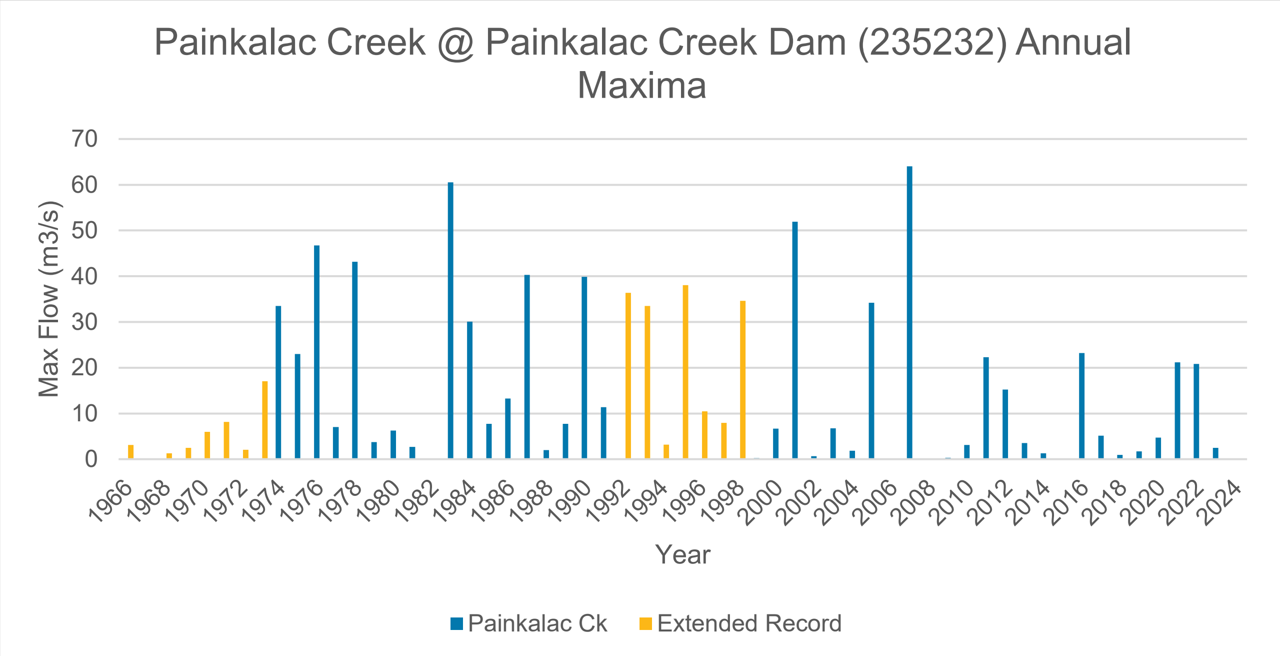

Figure 2-2 Annual flow maxima series at gauge 235232

Table 2-1 Annual max flows at Painkalac Creek @ Painkalac Creek Dam (235232)

Year	Flow (m³/s)	Year	Flow (m³/s)	Year	Flow (m³/s)
1966	3.12*	1986	13.24	2006	0.01
1967	0.03*	1987	40.32	2007	64.04
1968	1.28*	1988	2.01	2008	0.20
1969	2.46*	1989	7.76	2009	0.30
1970	5.95*	1990	39.87	2010	3.14
1971	8.13*	1991	11.38	2011	22.28
1972	2.03	1992	36.37	2012	15.21
1973	17.07	1993	33.48	2013	3.54
1974	33.47	1994	3.19	2014	1.31
1975	22.97	1995	38.08	2015	0.07
1976	46.74	1996	10.43	2016	23.21
1977	7.04	1997	7.96	2017	5.15
1978	43.18	1998	34.59	2018	0.95
1979	3.73	1999	0.24	2019	1.67

Year	Flow (m³/s)	Year	Flow (m³/s)	Year	Flow (m³/s)
1980	6.25	2000	6.67	2020	4.72
1981	2.69	2001	51.95	2021	21.19
1982	0.06	2002	0.66	2022	20.79
1983	60.52	2003	6.72	2023	2.51
1984	30.08	2004	1.82	2024	0.14
1985	7.72	2005	34.20		

* Maximum daily observation – entered as a peak over threshold.

Note: Orange shading indicates years infilled by correlation with Cumberland River at Lorne (235216).

2.1.3 Distribution and Results

The annual series was analysed in BestFit for a range of distributions. Both the extended series and Painkalac Creek only series were analysed, with the extended series adopted due to it having a narrower high confidence interval at the high end of the scale. No prior distribution information was available to be utilised, however a comparison against the results of the regional flood frequency estimator (RFFE) was made (see section 2.2.4).

The full range of distributions recommended by ARR were tested. The Log Pearson III (LP3) distribution appeared to fit the data well, however the expected flow quantiles stopped increasing after around a 1% AEP event (i.e. a 1-in-10,000 and a 1-in-100 had minimal difference in expected flow). Because of this, other distributions were investigated with the Generalised Pareto and Gumbel distributions appearing to provide reasonable fits.

Rainfall runoff modelling was unable to produce peak flows as predicted by the Generalised Pareto distribution fit in rare events even with zero losses applied, indicating the Generalised Pareto fit was overestimating flows in rare events. The Gumbel distribution flows were within the range of possibility indicated by rainfall runoff modelling, however a number of observed flows fell outside the confidence limits. The range of distributions assessed are shown graphically in Appendix A.

Ultimately, none of the tested distributions was an ideal representation of the annual series. This may be the result of the non-uniform data set utilised, which was influenced by changed catchment conditions (i.e. development of a dam and a change in use of the dam) and an extended record based on a less than perfect correlation. The resultant distributions were compared against rainfall runoff modelling results and the LP3 distribution was adopted with a caveat that it is shown to be less reliable in the less frequent (i.e. 2% AEP and above) section of the curve.

With the LP3 distribution selected as the best fit, the expected parameter quantiles were calculated for the range of AEPs output by BestFit. Given the high uncertainty associated with estimating extreme flooding based on the record available, flows rarer than a 1-in-200 expected probability have not been presented herein. The expected flows, based on the FFA, are shown in Table 2-2 below.

Table 2-2 LP3 FFA Expected Flow at Painkalac Creek @ Painkalac Dam

AEP	Expected Flow (m³/s)	95.0% Confidence Limit	5.0% Confidence Limit
1 in 200	73.38	97.67	61.98
1%	70.16	90.09	59.46
2%	65.38	81.22	55.15

AEP	Expected Flow (m³/s)	95.0% Confidence Limit	5.0% Confidence Limit
5%	55.57	66.87	45.52
10%	44.62	53.03	34.73
20%	30.31	37.18	21.67

The resultant FFA expected flows are lower than the results of the FFA completed by the Corangamite Catchment Management Authority in 2013 and are significantly lower than flows estimated by the regional flood frequency estimator (RFFE)[1].

Table 2-3 Flood Frequency Analysis

AEP	Water Technology (m³/s)	CCMA (2013) (m³/s)	RFFE (m³/s)
1%	70.16	89	139
2%	65.38	78	112
5%	55.57	62	81.5
10%	44.62	50	61.5
20%	30.31	40	44.1

2.2 Hydrologic Modelling

2.2.1 Overview

A hydrologic model of the Painkalac Creek catchment in its entirety through to the ocean outfall at Aireys Inlet was developed. Modelling utilised the RORB rainfall runoff modelling software package to determine flow hydrographs at gauged locations within the catchment and at the upstream extent of the hydraulic model.

RORB is a non-linear rainfall-runoff and streamflow routing model for the calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas, connected by a series of conceptual reaches and storage areas. Observed or design storm rainfall is input to the centroid of each subarea. Initial and continuing losses are then deducted, and the excess runoff is routed through the reach and storage network to produce streamflow hydrographs at selected locations within the model (referred to as “print” locations).

2.2.2 Model Setup

2.2.2.1 Subarea and Reach Delineation

Topographic data which was utilised in the RORB model construction came from the Vicmap 10m resolution Digital Elevation Model (DEM). The DEM has a stated horizontal accuracy of 12.5 metres and a vertical accuracy of 5 metres. While the 10m resolution DEM is not suitable for 2-dimensional hydraulic modelling at the level of resolution and accuracy required for this study, it is suitable for use in subarea and reach delineation for the hydrologic RORB model due to the course nature of the RORB model. In order to make the topographic data “hydrologically correct”, sinks (i.e. local depressions) were filled to allow a continuous flow path to form along the terrain.

Catchment and subarea delineation was produced using the SAGA GIS topographic processing capabilities in QGIS. The overall delineation was deemed accurate and acceptable after comparison against known flow

[1] https://rffe.arr-software.org/

paths and mapped waterway lines. Manual subarea manipulation was undertaken to ensure print nodes were placed at gauge locations where required. Interstation areas (code 7.1 in RORB) were placed at the gauge locations for calibration events. Finally, subareas that were too large were split where required, and subareas that were too small were merged where possible. The model layout is shown in below.

Reach lengths were determined using GIS software, following the hydrologically corrected topography in a continuous flow path to the outlet. Reaches were assigned as Type 1 (Natural) for overland flows and waterways, and Type 4 (Drowned) within storages. Type 2 (Excavated/Unlined) and Type 3 (Lined/Piped) reaches were not used in the final model, however Type 2 was tested for the main waterways and found to route flows too quickly through the model in calibration events. The shapefiles were then imported to ArcRORB where the final data manipulations occurred and the RORB .catg file was produced.

2.2.2.2 Storage

Painkalac Dam was included in the model as a special storage. The dam spillway was modelled based on information available from Barwon Water with the level of the spillway determined from LiDAR in the absence of design plans. The outlet was not included in the modelling as flows from the outlet pipe, with a capacity of around 10 ML/d (0.12 m^3/s) are negligible compared to the spillway. A height-storage relationship was developed based on information provided by Barwon Water for levels below the spillway and extended utilising LiDAR information above the spillway. The height-storage relationship is shown in Table 2-4 below.

The spillway was modelled as a 29.5 metre long Ogee spillway at an elevation of 30.07 mAHD. The dam embankment was not included in the model as no modelled event overtopped the level of the embankment at 34.4 mAHD (measured from LiDAR).

Table 2-4 Painkalac Dam Height-Storage Relationship

Height (mAHD)	Storage (m3)	Height (mAHD)	Storage (m3)	Height (mAHD)	Storage (m3)
23.7	0	26.1	103396.3	28.5	352969.6
23.8	1261.2	26.2	111020	28.6	365282.6
23.9	3159.1	26.3	118962	28.7	378133.2
24	5194.2	26.4	127273.2	28.8	391380.7
24.1	7401.5	26.5	135973	28.9	404800
24.2	9799.3	26.6	145005	29	418400
24.3	12394.7	26.7	154363.2	29.1	432100
24.4	15229.6	26.8	164014.5	29.2	446000
24.5	18313.1	26.9	173925	29.3	460100
24.6	21601.3	27	184048.7	29.4	474500
24.7	25128.3	27.1	194358.5	29.5	489200
24.8	28944.3	27.2	204836.4	29.6	504000
24.9	33056.3	27.3	215481.1	29.7	519000
25	37444.8	27.4	226299.9	29.8	534100
25.1	42092.1	27.5	237256.6	29.9	549200
25.2	47000.8	27.6	248319.9	30.07	575000
25.3	52173.8	27.7	259488.1	31	743183.5
25.4	57624.8	27.8	270765	32	942929.2
25.5	63338.3	27.9	282155.8	33	1159400

Height (mAHD)	Storage (m3)	Height (mAHD)	Storage (m3)	Height (mAHD)	Storage (m3)
25.6	69310.1	28	293658.2	34	1390201
25.7	75563.2	28.1	305272.6	35	1634561
25.8	82100.1	28.2	317000.3	36	1892245
25.9	88930.8	28.3	328852.2	37	2163306
26	96038.9	28.4	340837.9		

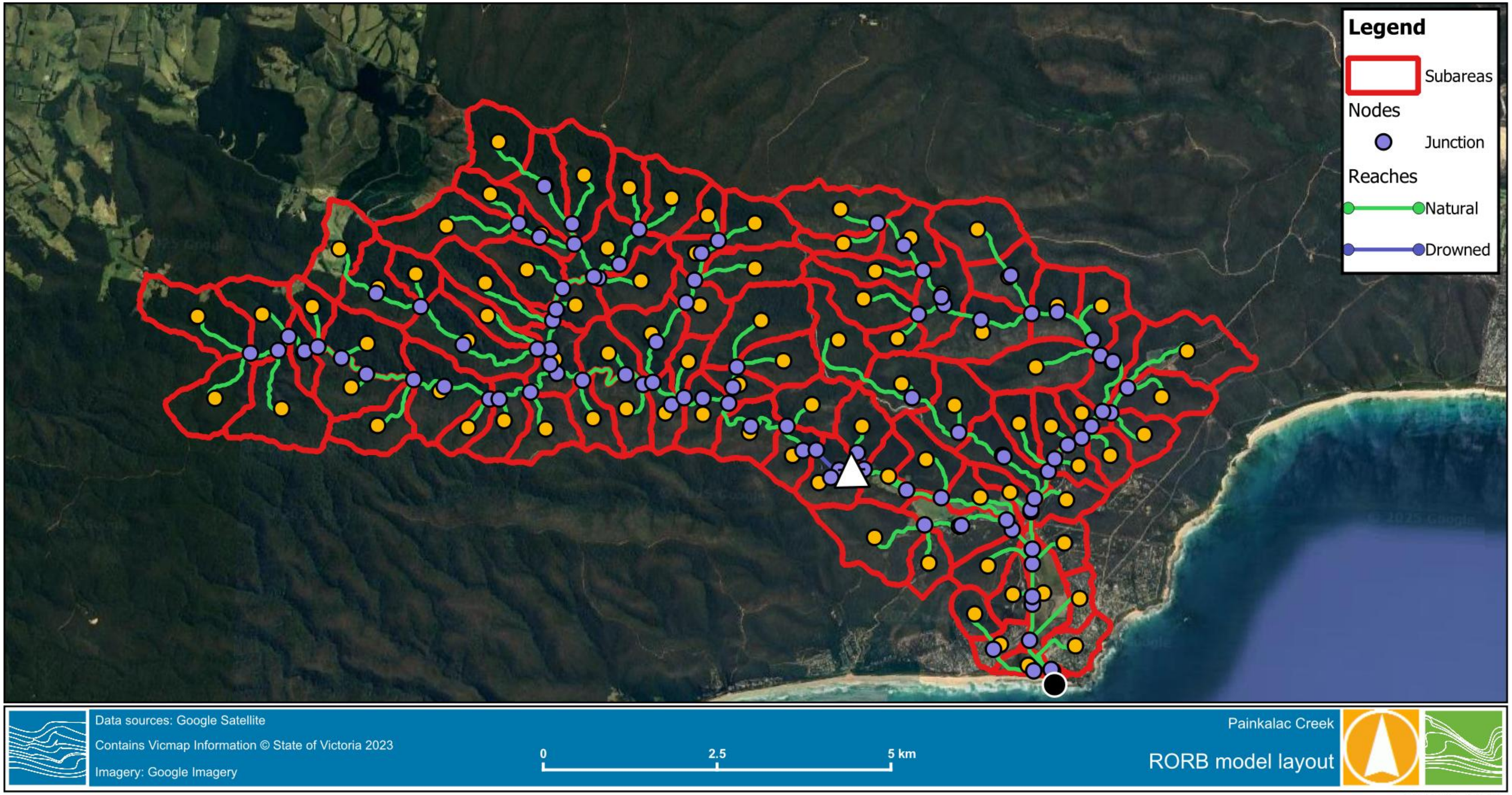

Figure 2-3 RORB Model Layout

2.2.2.3 Fraction Impervious

In line with the recommendations of ARR v4.2, three distinct land areas are considered in the hydrological model: pervious area (PA), effective impervious area (EIA) and indirectly connected area (ICA). EIA comprises areas that are impervious and connected to drainage infrastructure while ICA includes impervious and pervious areas that are connected and interact hydrologically, thus producing runoff between a fully pervious area and EIA.

The vast majority of the catchment is forest, and as such was modelled as entirely pervious. 4 subareas near the outlet cover parts of Anglesea and Airey’s Inlet, and the estuary itself. The estuary was modelled as completely impervious. For the township areas, two distinct land use types were considered: residential and low density residential. Inspections of aerial imagery and council pipe network data informed the final EIA/ICA mix, which is shown in Table 2-5 below. The land use applied to subareas near the outlet is shown in Figure 2-4 below.

Pervious area losses for design modelling were determined by reconciliation of modelled flows with the expected quantiles from FFA as discussed in section 2.2.4. Effective impervious areas adopted an initial loss of 1mm and continuing loss of 0 mm/hr, while indirectly connected areas adopted losses equal to 70% of the pervious losses, as recommended in Book 5 Chapter 3 of ARR.

Table 2-5 Adopted Fraction EIA/ICA/PA

Area	Fraction EIA	Fraction ICA	Fraction PA
Estuary	1	0	0
Residential	0.3	0.7	0
Low Density Residential	0	0.2	0.8
Forest (remainder of catchment)	0	0	1

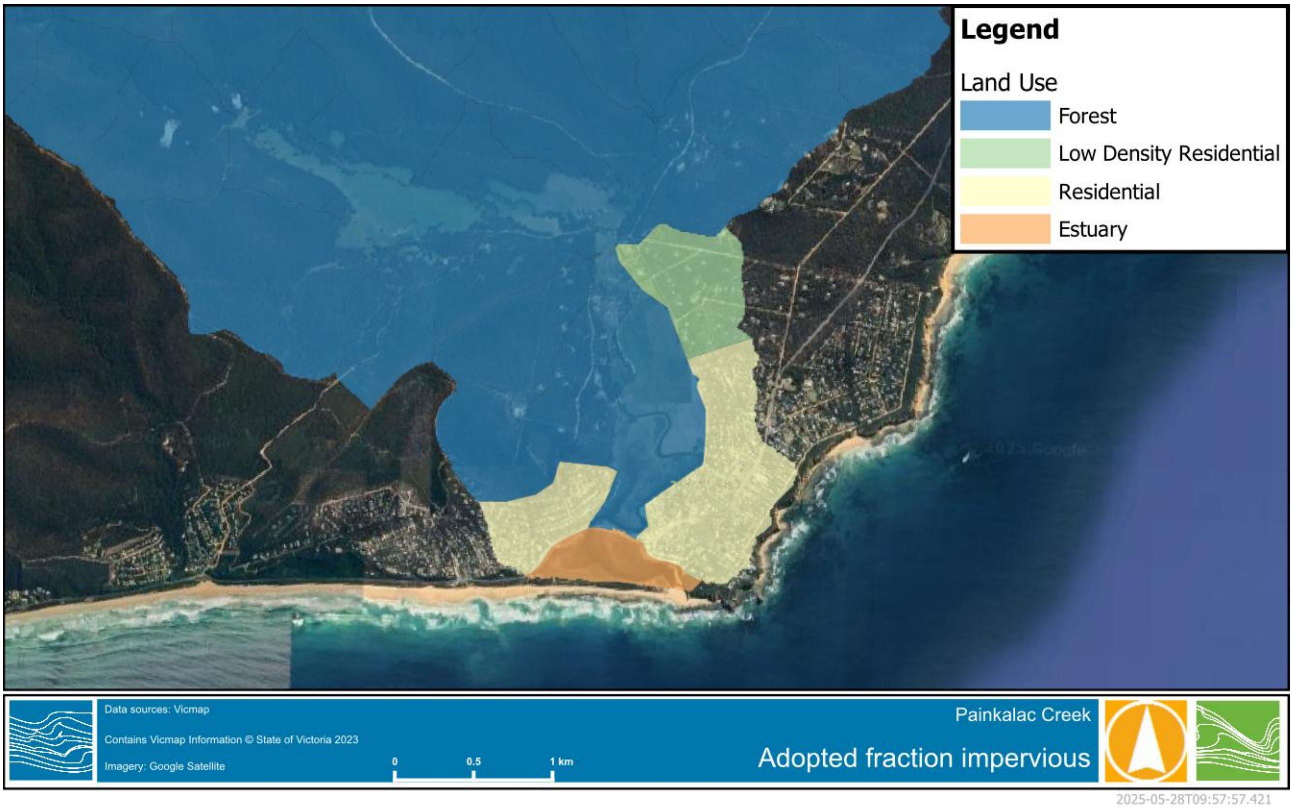

Figure 2-4 Adopted land use and fraction impervious

2.2.3 Model Calibration and Validation

2.2.3.1 Calibration

The RORB model was calibrated at the Painkalac Creek @ Painkalac Dam (235232) gauge to four historic runoff events. Calibration of the model involved varying the routing parameters, Kc and m, and rainfall losses for each event until a good agreement between the modelled and recorded flow was achieved. Current common practice in Victoria is to leave the m parameter at the recommended value of 0.8, which was found to achieve good calibration so was left.

Daily rainfall totals from 5 nearby gauge stations (Table 2-6 and Figure 2-5) were used to develop the spatial patterns of rainfall for each historic event. Available pluviograph data from within and around the catchment informed temporal patterns for each storm.

Table 2-6 details the recorded rainfall, start and stop times of each calibration event. Spatial distributions of rainfall for each event are shown in Figure 2-6, Figure 2-8, Figure 2-10, and Figure 2-12, and the calibration run hydrographs are shown in Figure 2-7, Figure 2-9, Figure 2-11 and Figure 2-13. The November 2007 and September 2016 events were also simulated with the adopted design routing parameters for validation.

Calibration/validation parameters and result statistics for each event are summarised in Table 2-7 and Table 2-8.

Table 2-6 Rainfall recorded near Painkalac Creek catchment for calibration events

Event	Start Time	End Time	Painkalac Creek @ Painkalac Res HG (235264)	Boonah (233813)	Aireys Inlet WRP	Wensleydale (Anglesea Vehicle Proving)	Aireys Inlet (90180)
February 2005	2 February @ 9AM	4 February @ 9AM	111.2	121.8	145.4	126.4	126.4
November 2007	3 November @ 9AM	5 November @ 9AM	53.9	156.6	50.6	115	159.8
January 2011	10 January @ 9AM	14 January @ 9AM	168.2	146.8	152	128.8	140.6
September 2016	9 September @ 9AM	15 September @ 9AM	101.6	104.4	98.6	99.2	106.4

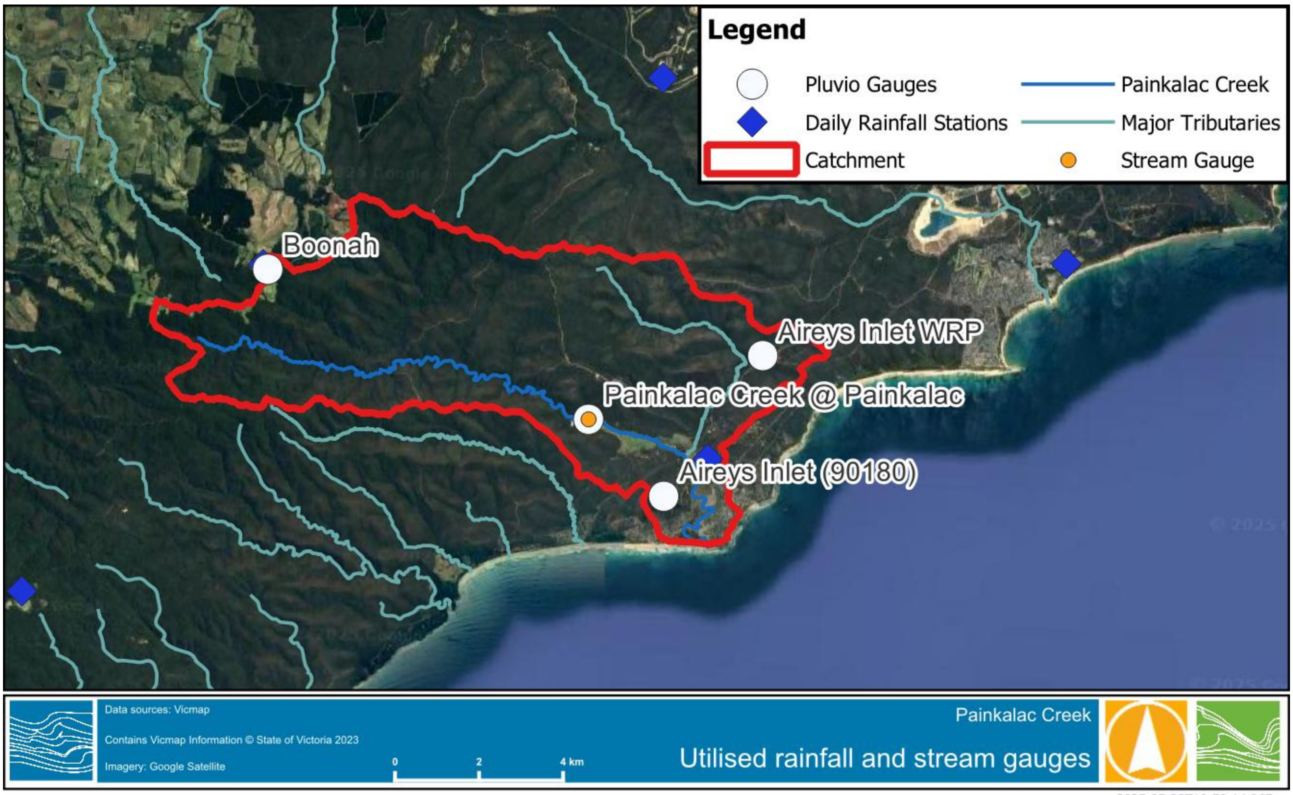

Figure 2-5 Gauges utilised in calibration

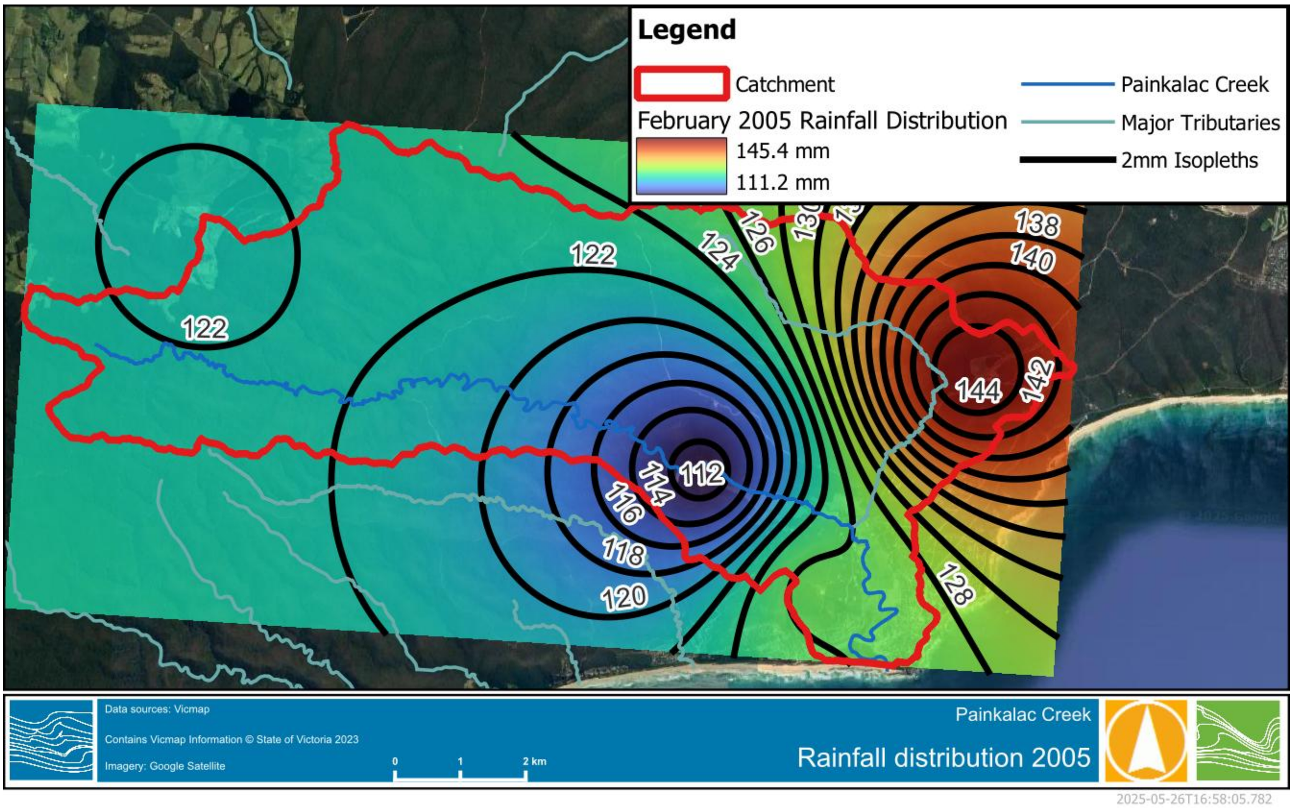

Figure 2-6 February 2005 recorded rainfall

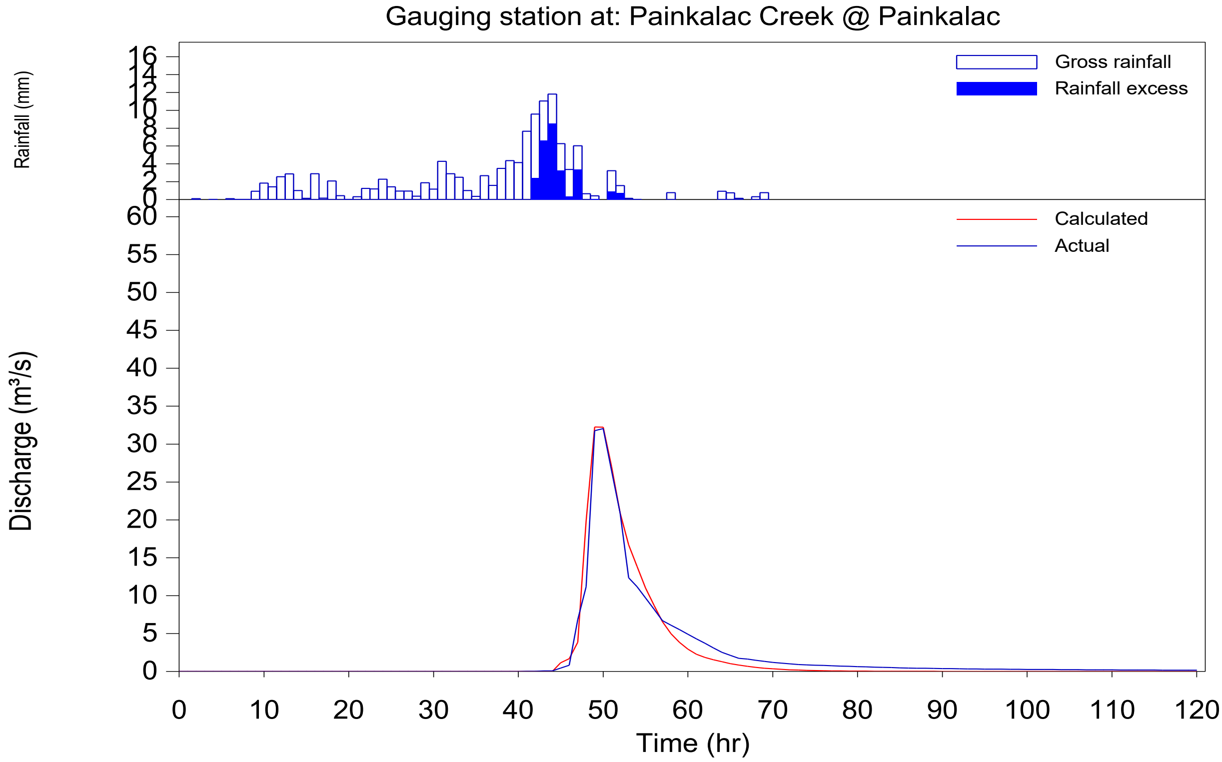

Figure 2-7 February 2005 RORB Calibration – Painkalac Creek @ Painkalac Dam

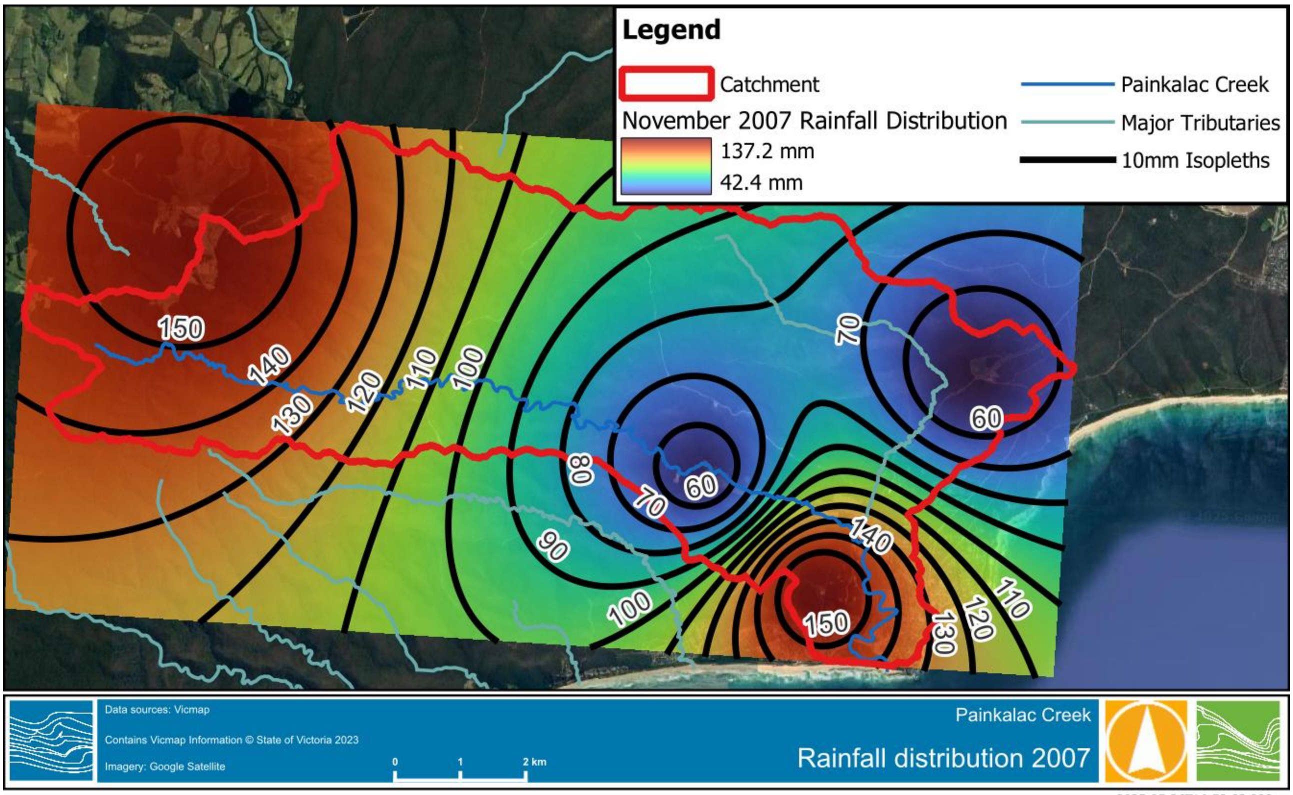

Figure 2-8 **November 2007 recorded rainfall**

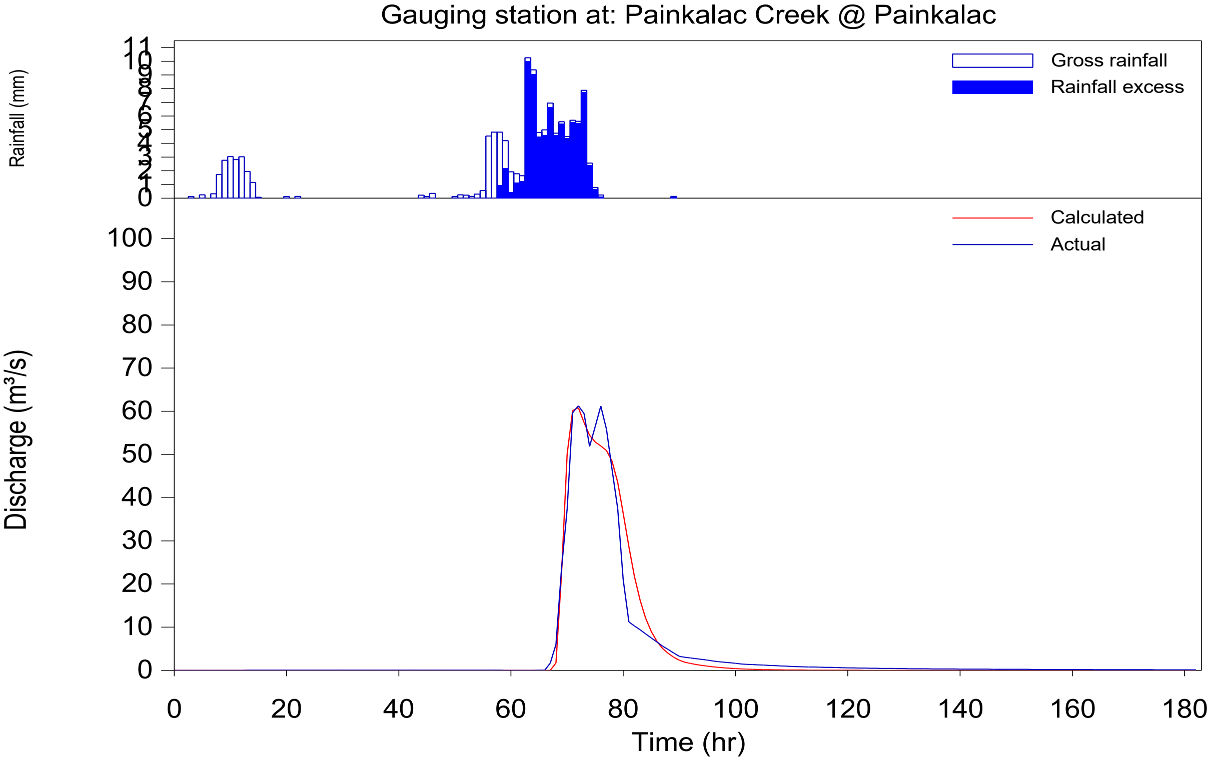

Figure 2-9 **November 2007 RORB Calibration – Painkalac Creek @ Painkalac Dam**

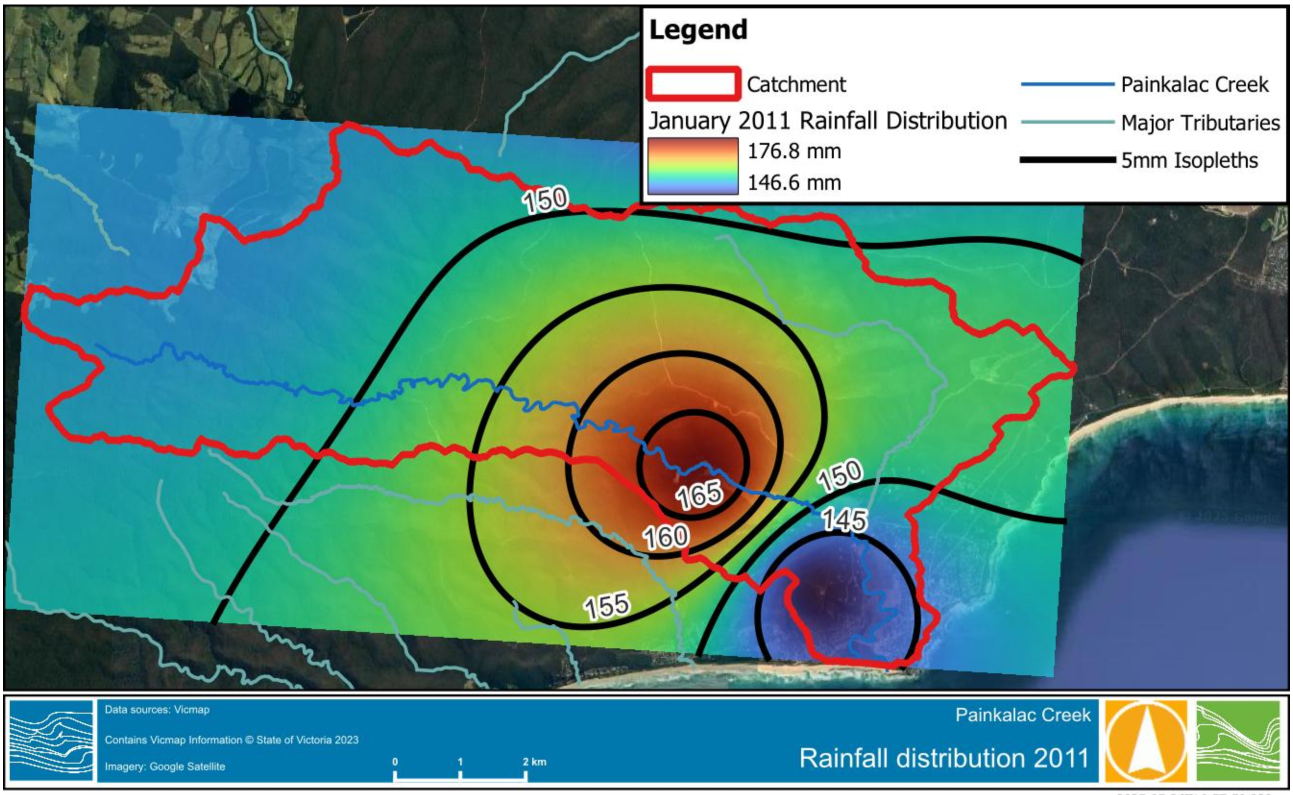

Figure 2-10 January 2011 recorded rainfall

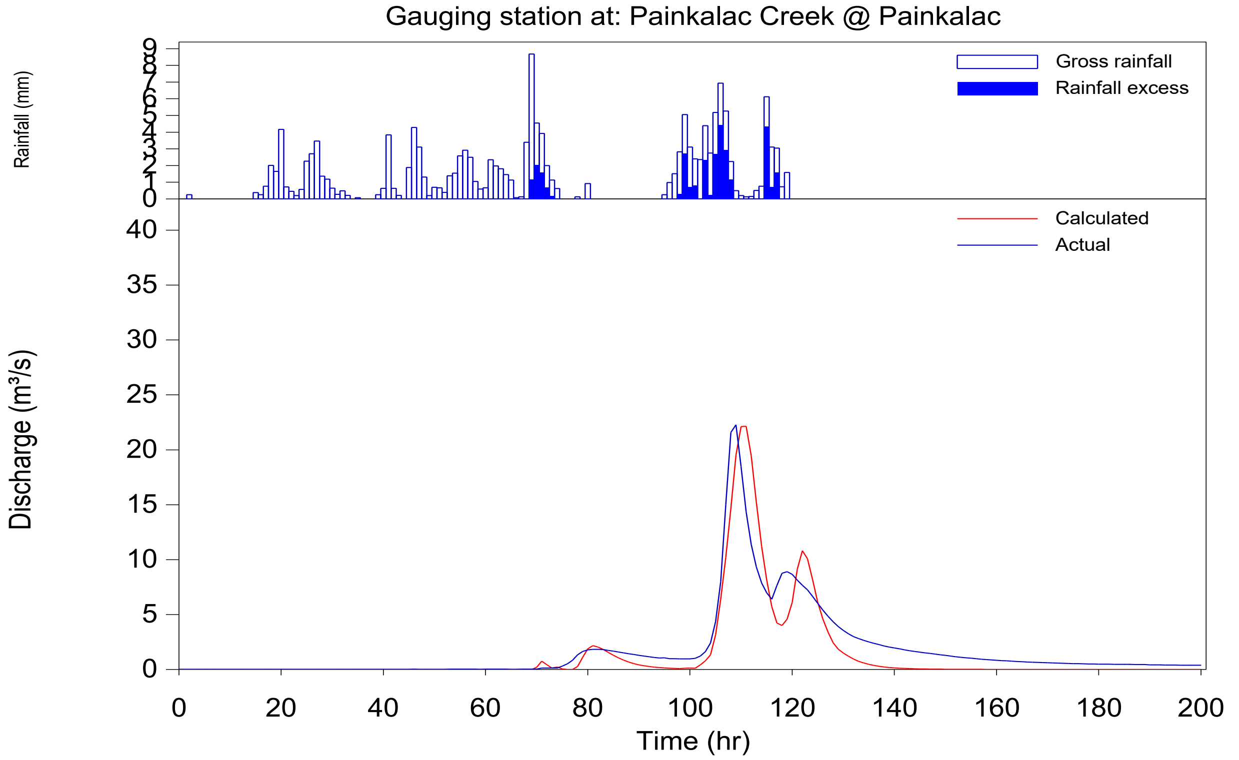

Figure 2-11 January 2011 RORB Calibration – Painkalac Creek @ Painkalac Dam

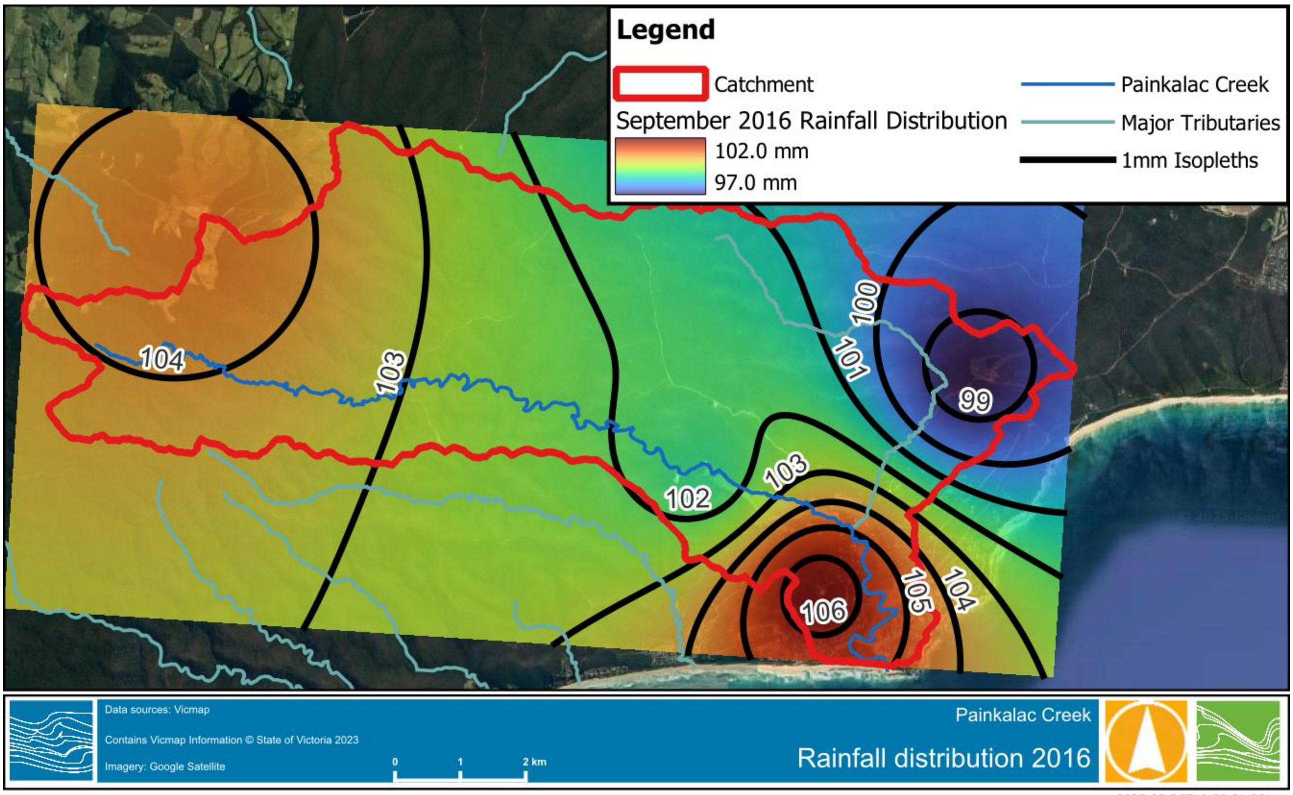

Figure 2-12 September 2016 recorded rainfall

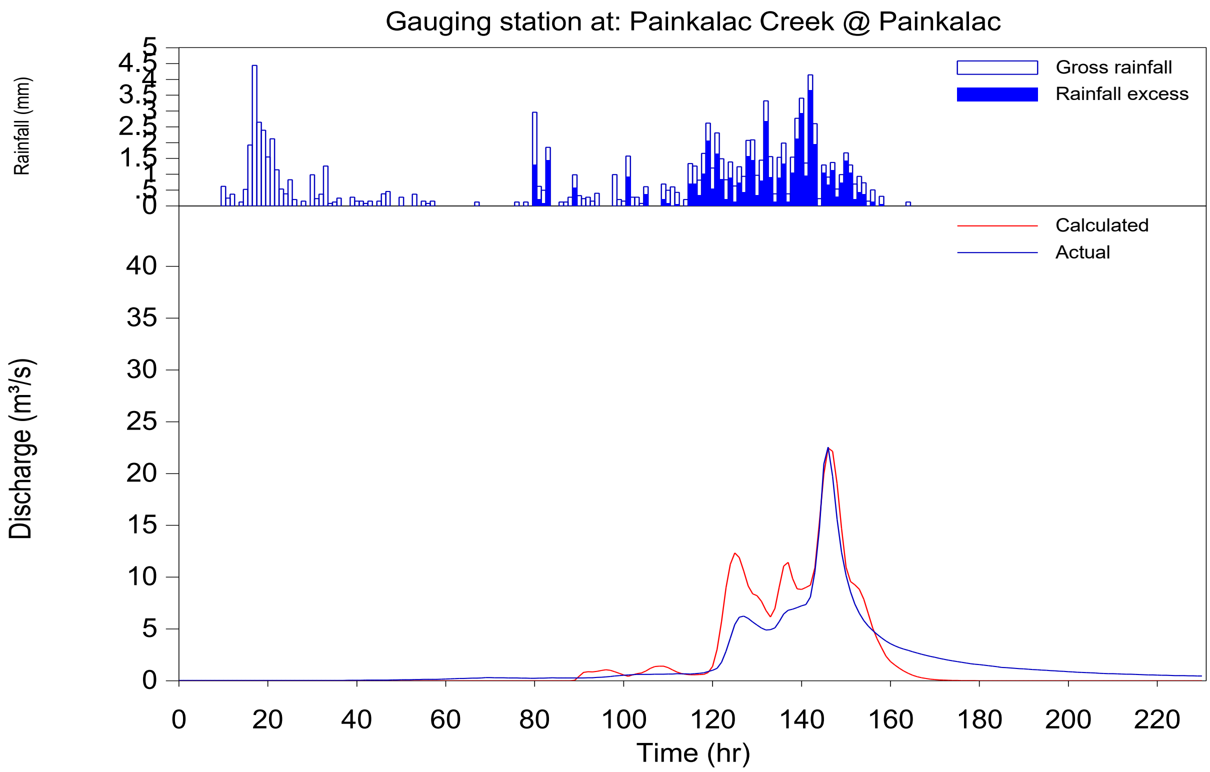

Figure 2-13 September 2016 RORB Calibration – Painkalac Creek @ Painkalac Dam

2.2.3.2 Validation

Both the November 2007 and September 2016 events resulted in a different routing parameter (Kc) to that adopted for the February 2005 and January 2011 events. To test the appropriateness of adopting Kc = 9.5 for design modelling, the November 2007 and September 2016 events were modelled with Kc = 9.5 to test the validity of the parameter. As expected, the altered Kc changed model behaviour, however the model still produced a good representation of those events at the gauge. The validation event hydrographs are shown in Figure 2-14 and Figure 2-15.

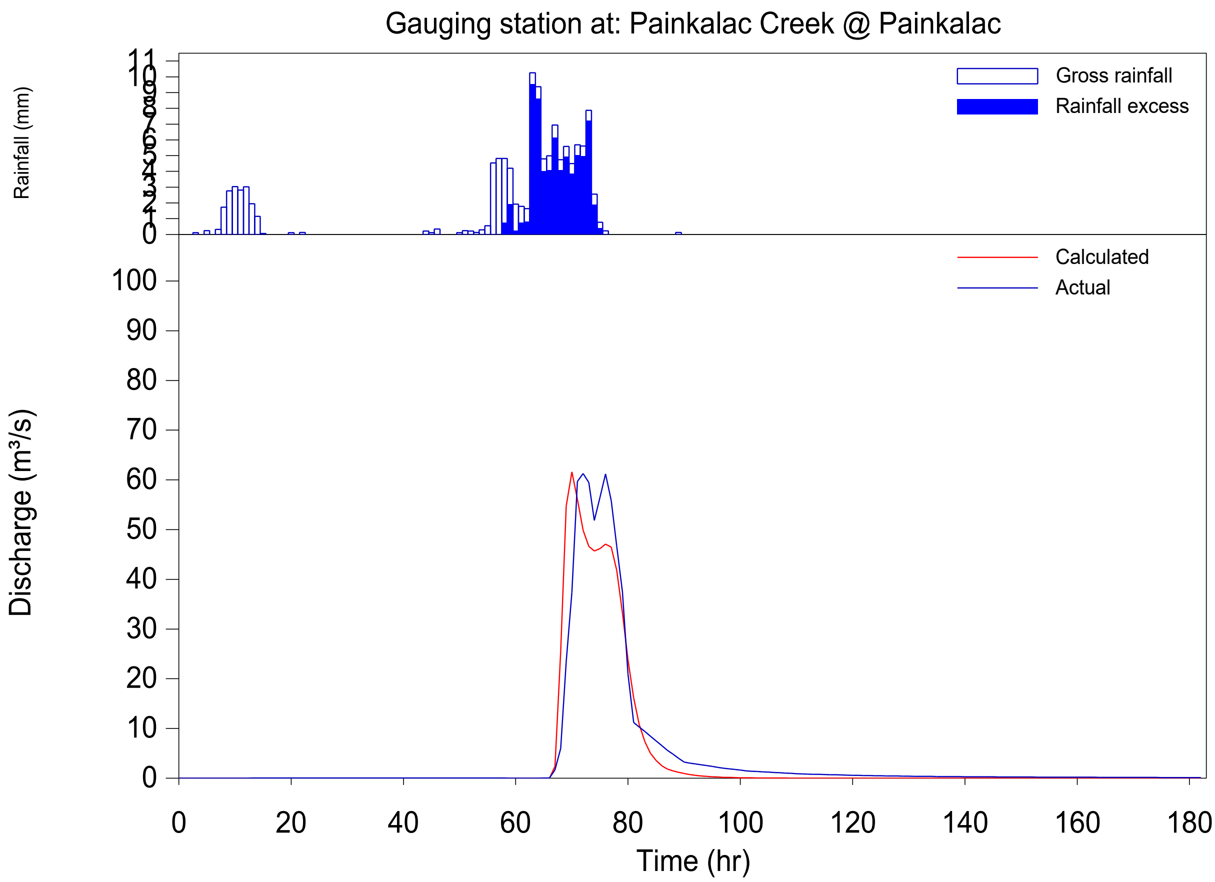

Figure 2-14 November 2007 Parameter Validation – Painkalac Creek @ Painkalac Dam

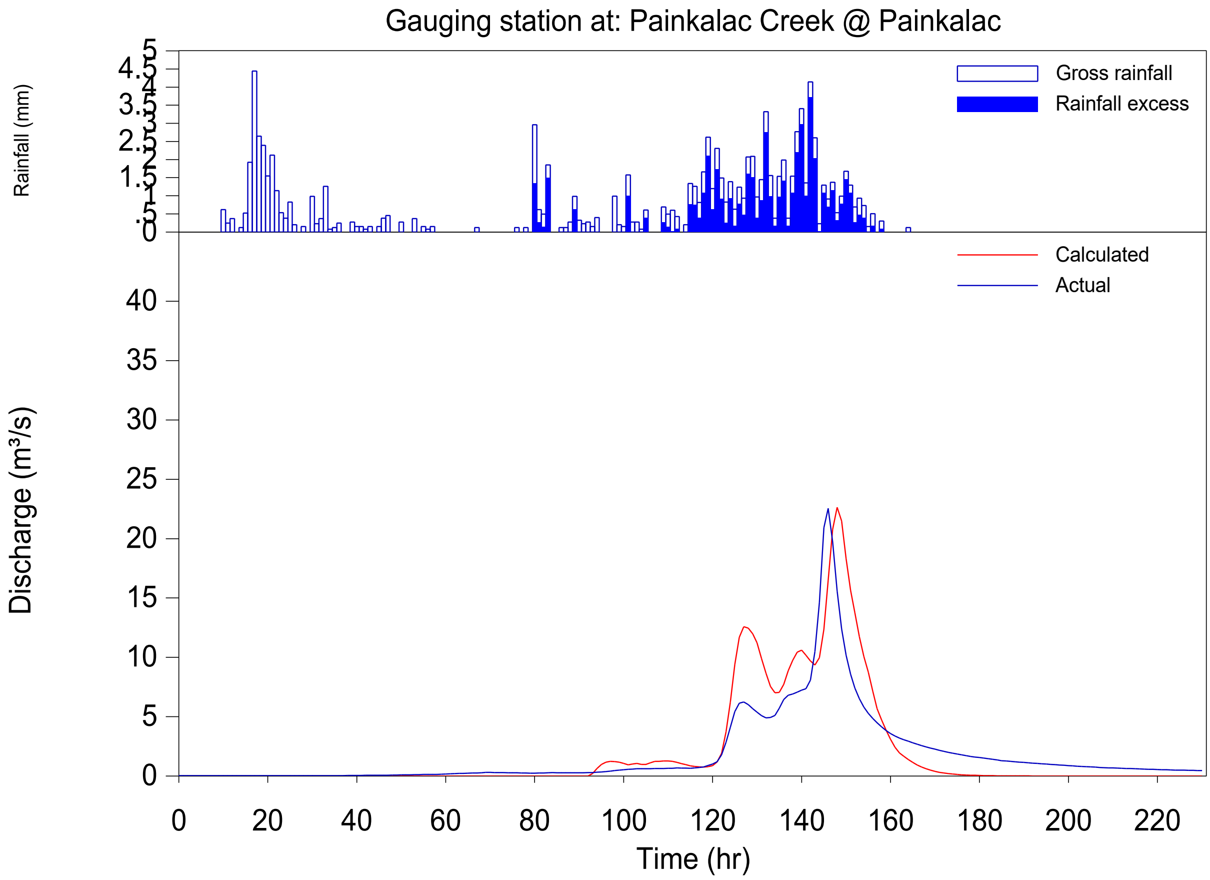

Figure 2-15 September 2016 Parameter Validation – Painkalac Creek @ Painkalac Dam

2.2.3.3 Calibration Parameters and Results Summary

The adopted parameters for each calibration and validation run are shown in Table 2-7. Key model error statistics are summarised in Table 2-8.

Table 2-7 Adopted Calibration/Validation Parameters

Event	Kc	m	Initial Loss (mm)	Continuing Loss (mm/hr)
February 2005 Calibration	9.50	0.8	70.0	3.00
November 2007 Calibration	13.00	0.8	38.0	0.13
November 2007 Validation	9.50	0.8	38.0	0.65
January 2011 Calibration	9.50	0.8	73.0	2.50
September 2016 Calibration	5.80	0.8	30.0	0.65
September 2016 Validation	9.50	0.8	30.0	0.57

Table 2-8 Painkalac Creek RORB Calibration/Validation Statistics

235232 Painkalac Creek at Painkalac Dam	Peak Discharge Absolute Error (m³/s)	Peak Discharge % Error	Time to Peak Absolute Error (hrs)	Time to Peak % Error	Volume Absolute Error (m³)	Volume % Error
February 2005 Calibration	0.2	0.7	-1.0	-2.0	-6.8E+04	-7.9
November 2007 Calibration	-0.4	-0.7	0.0	0.0	6.7E+03	0.3
November 2007 Validation	0.3	0.5	-2.0	-2.8	2.7E+05	-10.5
January 2011 Calibration	0.3	0.5	-2.0	-2.8	-2.7E+05	-10.5
September 2016 Calibration	-0.1	-0.4	0.0	0.0	1.5E+04	1.0
September 2016 Validation	0.1	0.3	2.0	1.4	1.1E+05	7.0

2.2.4 Design Modelling

2.2.4.1 Climate Change Considerations

The adopted methodology described below is based on current guidelines described in the most recent (2024) release of Australian Rainfall and Runoff, version 4.2. Version 4.2 of the guidelines contained a complete update/rewrite of Book 1, Chapter 6: Climate Change Considerations. The update has altered how rainfall and losses are scaled to account for changes to the climate under a range of timeframes and emissions scenarios, referred to as Shared Socioeconomic Pathways (SSPs). Projected temperature changes under the various SSPs are shown in Figure 2-16 below.

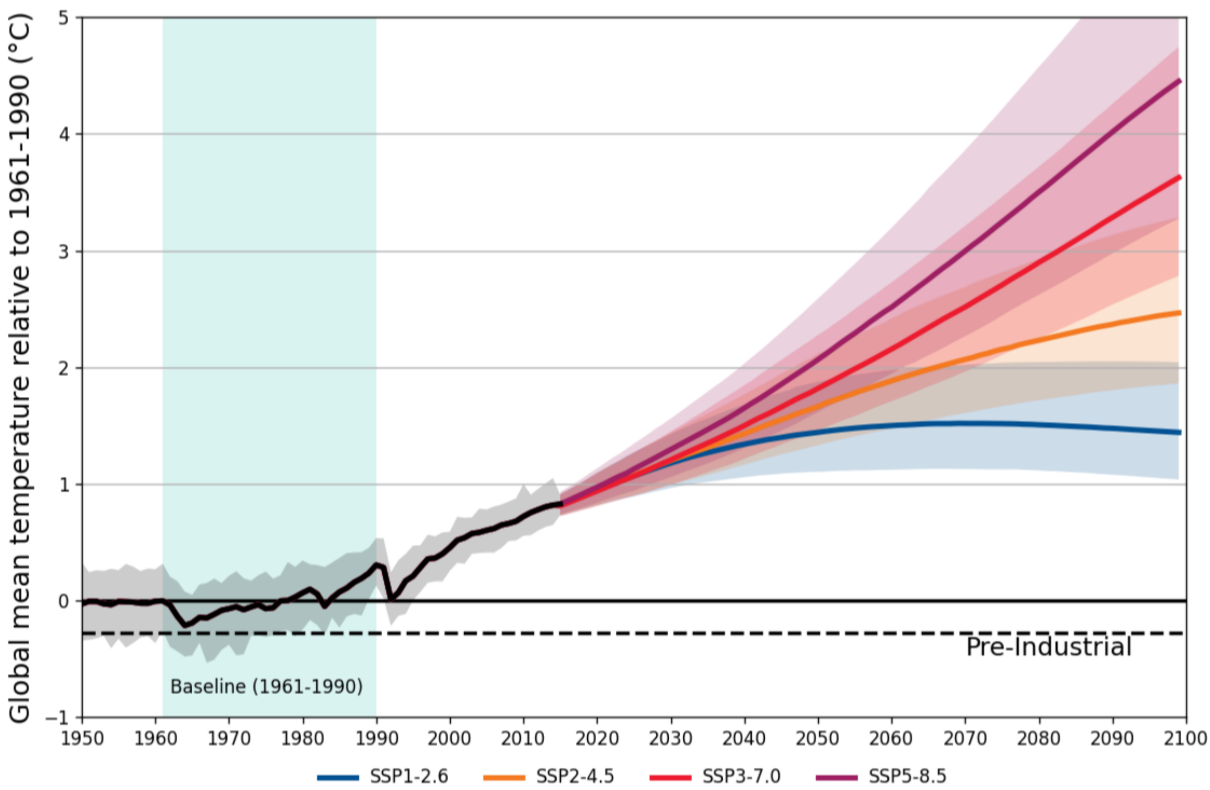

Figure 2-16 Projected temperature increases associated with varying socioeconomic pathways (SSPs), relative to a baseline at 1961-1990

This study has considered five temporal periods of rainfall: baseline, 2030, 2040, 2070 and 2100. "Baseline" refers the baseline period upon which the current Intensity-Frequency-Duration curves are based on, being 1961-1990. Design rainfall for this period comes from the BoM's IFD system and is not scaled. Other periods adopt design rainfall depths which are scaled according to the relevant factor as provided by the ARR datahub, along with losses (as determined by reconciliation of the baseline IFD rainfall runoff modelling with the results from flood frequency analysis) scaled to the relevant factor for that period in line with ARR v4.2. The 2030 event is presented as being the most representative of "present-day" conditions while 2100 has been proposed as a future design and planning scenario, with intermediate timeframes also modelled.

The modelling has adopted Shared Socioeconomic Pathway SSP5-8.5 in consultation with Council and Corangamite CMA.

An ensemble approach was used in this assessment to determine the design flow inputs. The ensemble approach modelled 10 available temporal patterns for each duration recommended in ARR2019. The temporal pattern which determined the median peak flow for each duration was then adopted as the design flow after reconciliation with the FFA via appropriate parameter selection.

2.2.4.2 Design Rainfall (1961-1990 Baseline)

Design rainfall depths were obtained from the Bureau of Meteorology Design Rainfall Data System[2]. Rainfall depths were obtained in ascii grid format to enable spatial variation of rainfall to be considered in line with the recommendations of ARR2019 for catchments exceeding 20km^2. Areal reduction factor (ARF) parameters and temporal patterns were obtained from the ARR Datahub[3].

Temporal patterns for the catchment were adopted from the Southern Slopes (Vic) region. Due to the size of the catchment, areal temporal patterns are recommended for use by ARR2019. Areal temporal patterns are available for storms 12 hours in duration and longer. Given the critical duration was shown to be longer than 12 hours, point temporal patterns were not considered for design rainfall distribution.

The ARF was calculated based on the full catchment area using RORB's internal ARF calculator which adopts the method specified in Book 2, Chapter 4 of ARR2019.

Spatial Variation

Due to the size of the catchment, spatial variation of design rainfall was applied in RORB. GIS tools were used to assign a point rainfall (taken as the average of rainfall grid cells that intersect a subarea) to each subarea. The weighted average rainfall for the catchment and the percentage of the weighted average to be applied to each subarea was then calculated in a spreadsheet for each duration and magnitude of event as per the methodology described in Book 2, Chapter 6 of ARR2019.

Pre-Burst Rainfall

Design rainfall depths obtained from the Bureau of Meteorology represent storm bursts. The application of pre-burst rainfall is intended to represent a complete storm by appending the pre-burst to the start of the burst rainfall. This can be achieved by modelling the complete storm and applying the storm Initial Loss, or lowering the Initial Loss to represent a burst Initial Loss according to the following equation:

$$\mathrm{IL}_b = \mathrm{IL}_s - \text{pre-burst depth}$$

[2] http://www.bom.gov.au/water/designRainfalls/revised-ifd/
[3] https://data.arr-software.org/

For this study, burst Initial Losses were applied by subtracting the median pre-burst depth from the storm Initial Loss and applying the resultant burst Initial Loss to the design burst rainfall.

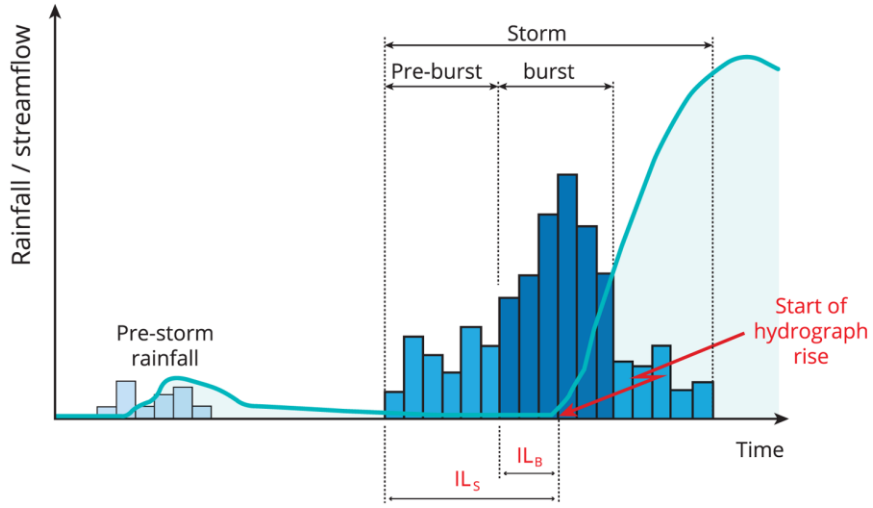

Figure 2-17 Conceptualisation of storm vs burst rainfall and its interaction with Initial Loss[4]

Consideration was given to the Victorian Specific Information of the ARR datahub, which recommends the use of 75th percentile pre-burst depths when applying datahub values for other hydrologic inputs[5]. The median pre-burst depth was selected because the losses were reconciled with FFA results, thus the aim of producing unbiased flows has been achieved.

2.2.4.3 RORB Parameters

Based on the results of the calibration runs, routing parameters shown in Table 2-9 were adopted for design modelling. Table 2-9 also shows the average flow distance of the model and Kc/Dav ratio, which is commonly used to compare similar models and translate routing parameters between areas of a catchment.

Table 2-9 Adopted RORB Routing Parameters

Parameter	Adopted Value
Kc	9.5
Dav (Average flow distance)	10.5 km
Kc/Dav	0.905
m	0.8

Design losses were determined through consideration of nearby studies, calibration event losses, and reconciliation of flows produced by the model with the expected flow quantiles produced by the FFA. Preference was given to adopting a single set of loss parameters rather than varying the losses with AEP to achieve a near perfect reconciliation with FFA as the FFA distribution fit is not perfect itself, and thus is not wholly relied upon.

The datahub recommends an initial loss of 25 mm and a continuing loss of 3.2 mm/hr. Design losses were determined with consideration of the following:

---

[4] Sourced from ARR2019, Book 5 Chapter 3
[5] https://data.arr-software.org/vic_specific

- Previous experience has shown that the latest version of the datahub consistently overestimates continuing losses within southwest Victoria.
- The Birregurra Flood and Drainage Strategy reconciled a RORB model of the upper Barwon River with a FFA of the Barwon River at Ricketts Marsh (233224) gauge with Initial Loss 14.95 mm and Continuing Loss 2.21 mm/hr.
- Calibration event losses varied significantly.
- Flood frequency reconciliation is heavily influenced by the choice of distribution, which directly alters the expected flows for each quantile. There was no clearly superior distribution fit across the full range of magnitudes of interest. The LP3 distribution was adopted with a caveat that it is not reliable in rare (2% and rarer) events.
- The adopted losses provided excellent agreement with the higher confidence section of the FFA curve (20% to 5% AEP).

After some iteration, design initial loss of 30 mm/hr and continuing loss of 2.0 mm/hr produced results in good agreement with the FFA across the range of high confidence. The resultant peak flood flows produced by the RORB model, along with the results of the FFA are shown in Table 2-10 below.

A range of FFA curves, along with RORB results for various loss combinations, are presented in Figure 2-18 below.

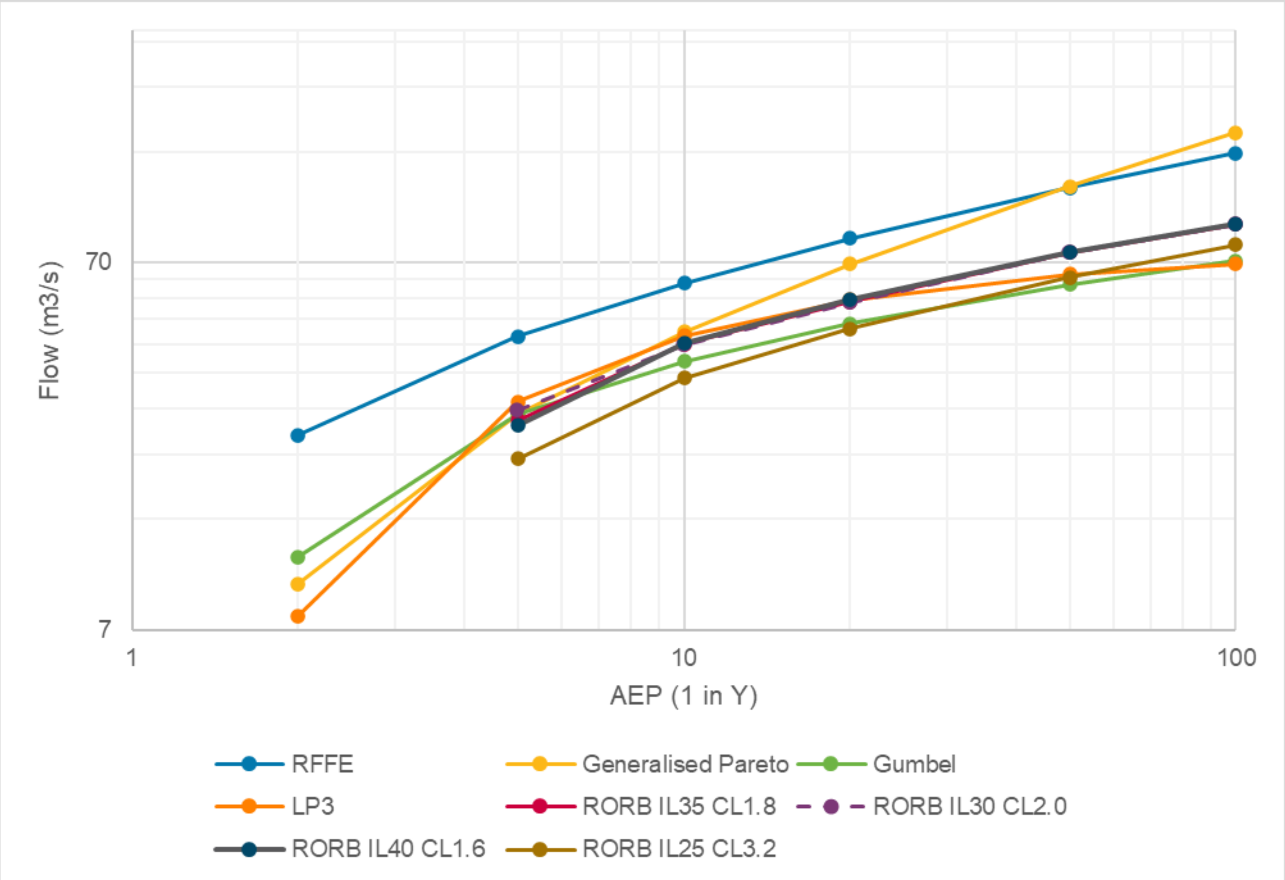

Figure 2-18 Estimated flood flow comparisons – FFA and RORB

Table 2-10 Design RORB (baseline) and FFA Peak Flow Estimates – Painkalac Creek @ Painkalac Dam

AEP	RORB Design Flow (m³/s)	FFA Expected Flow (m³/s)	FFA 5% Confidence Limit (m³/s)	FFA 95% Confidence Limit (m³/s)
20%	27.74	30.31	37.18	21.67
10%	41.54	44.62	53.03	34.73
5%	54.53	55.57	66.87	45.52
2%	74.46	65.38	81.22	55.15
1%	88.83	70.16	90.09	59.46

The adopted RORB modelling parameters and inputs (baseline climate scenario) are summarised in Table 2-11 below.

Table 2-11 RORB modelling parameters (baseline climate scenario)

Kc	Dav (km)	Kc/Dav	m	IL (mm)	CL (mm/hr)
9.50	10.5	0.90	0.8	30.0	2.0

The parameters in Table 2-11 above were applied to the entire model and flows for the range of design events were extracted from Painkalac Creek at Old Coach Road and Distrillery Creek at Old Coach Road, both of which form the upper extent of the hydrodynamic model. Rainfall on areas downstream of these locations was routed to the estuary in RORB and added directly to the estuary in the hydrodynamic model.

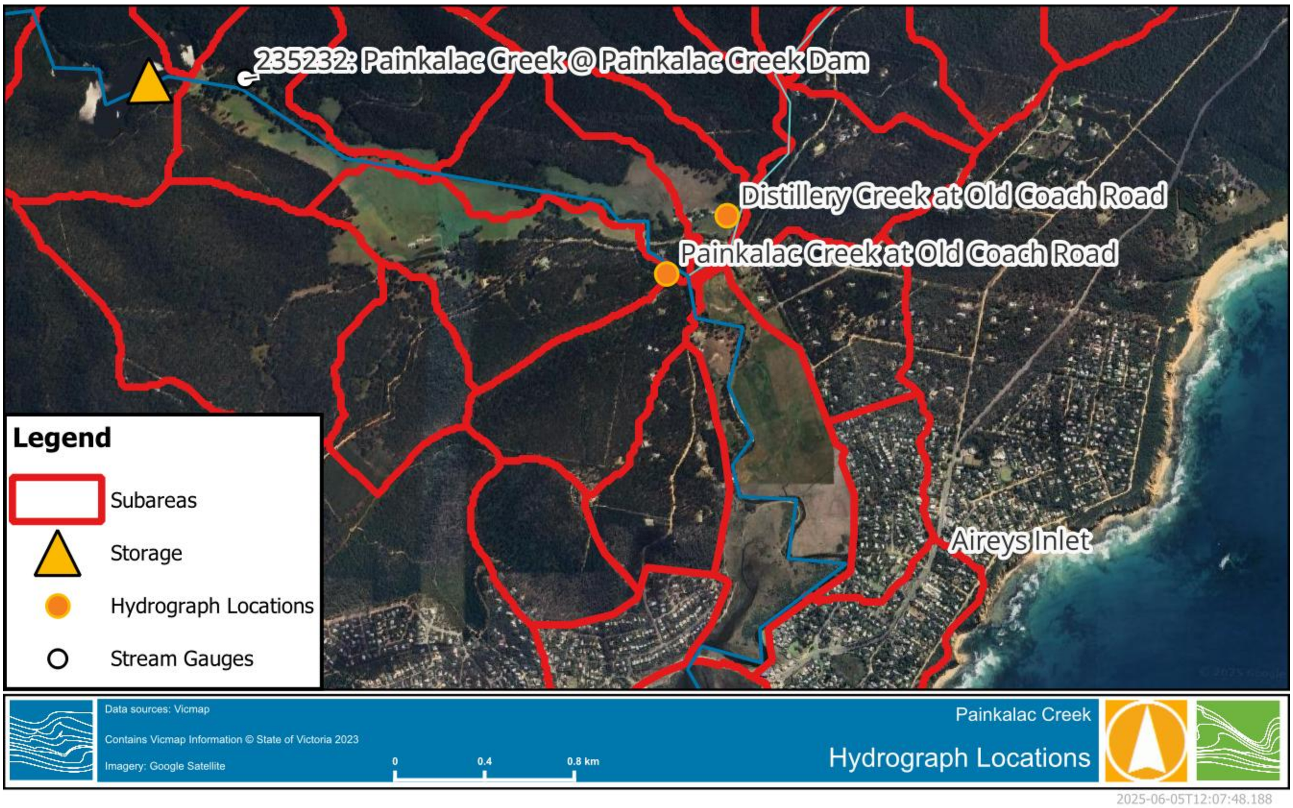

Figure 2-19 Hydrograph Locations

The design peak flow at the hydrograph locations under the baseline climate scenario are shown in Table 2-12 below, along with a comparison of design peak flows produced herein and those of the 2013 CCMA study.

Table 2-12 Design inflow hydrographs and comparison to 2013 CCMA study design flows

AEP	Painkalac Creek at Old Coach Road		Distillery Creek at Old Coach Road	
	WT (2025)	CCMA (2013)	WT (2025)	CCMA (2013)
20%	26.9	42	10.5	18
10%	40.7	52	19.5	25
5%	55.8	65	25.2	29
2%	79.4	80	36.2	35
1%	94.2	89	42.9	39
0.5%	110.4	104	50.7	50

2.2.4.4 Climate Change Design Flows

Baseline rainfall depths and design losses were scaled to account for climate change in accordance with ARR Book 1 Chapter 6. Scaling factors for the adopted climate scenario (SSP5-8.5) and timeframes are shown in Table 2-13 and Table 2-14 below.

Table 2-13 Rainfall Scaling Factors (SSP5-8.5)

Year	<1 hour	1.5 Hours	2 Hours	3 Hours	4.5 Hours	6 Hours	9 Hours	12 Hours	18 Hours	>24 Hours
2030	1.2	1.18	1.17	1.16	1.14	1.13	1.13	1.12	1.11	1.11
2040	1.26	1.24	1.22	1.2	1.18	1.17	1.16	1.15	1.14	1.14
2070	1.52	1.47	1.43	1.4	1.36	1.34	1.31	1.29	1.27	1.26
2100	1.86	1.77	1.71	1.64	1.58	1.54	1.5	1.47	1.43	1.41

Table 2-14 Loss Scaling Factors (SSP5-8.5)

Timeframe	Initial Loss Scaling Factor	Continuing Loss Scaling Factor
2030	1.05	1.11
2040	1.07	1.14
2070	1.12	1.28
2100	1.19	1.44

Peak flow rates at the Painkalac Creek at Old Coach Road inflow location under the modelled climate scenarios are shown in Table 2-15 below.

Table 2-15 Design flows at Painkalac Creek at Old Coach Road under modelled timeframes (SSP5-8.5)

AEP	2030 (m^3/s)	2040 (m^3/s)	2070 (m^3/s)	2100 (m^3/s)
20%	33.0	34.2	41.5	52.2
10%	47.3	49.0	57.6	69.1
5%	63.8	65.7	76.8	92.1
2%	91.4	94.1	106.8	123.5
1%	108.7	111.7	125.9	146.9
0.5%	129.0	132.6	150.9	178.8

3 SUMMARY

Flood frequency analysis and rainfall runoff modelling within the RORB software package have informed design hydrology for the Painkalac Creek catchment. The calibrated hydrologic model has been utilised to simulate design storms with consideration of climate change for numerous climate timeframes in line with the recommendations of the latest Australian Rainfall and Runoff guidelines. Design event hydrographs were extracted for application to a hydrodynamic flood model.

The hydrologic model was validated against the FFA and compared to previous modelling undertaken by CCMA in 2013. The FFA itself is limited by the quality of data upon which it is based, however flows extracted from the model are fit for the purpose of defining the best estimates of flooding and resultant impacts as expected by events of varying magnitude.

APPENDIX A
FLOOD FREQUENCY ANALYSIS FITS

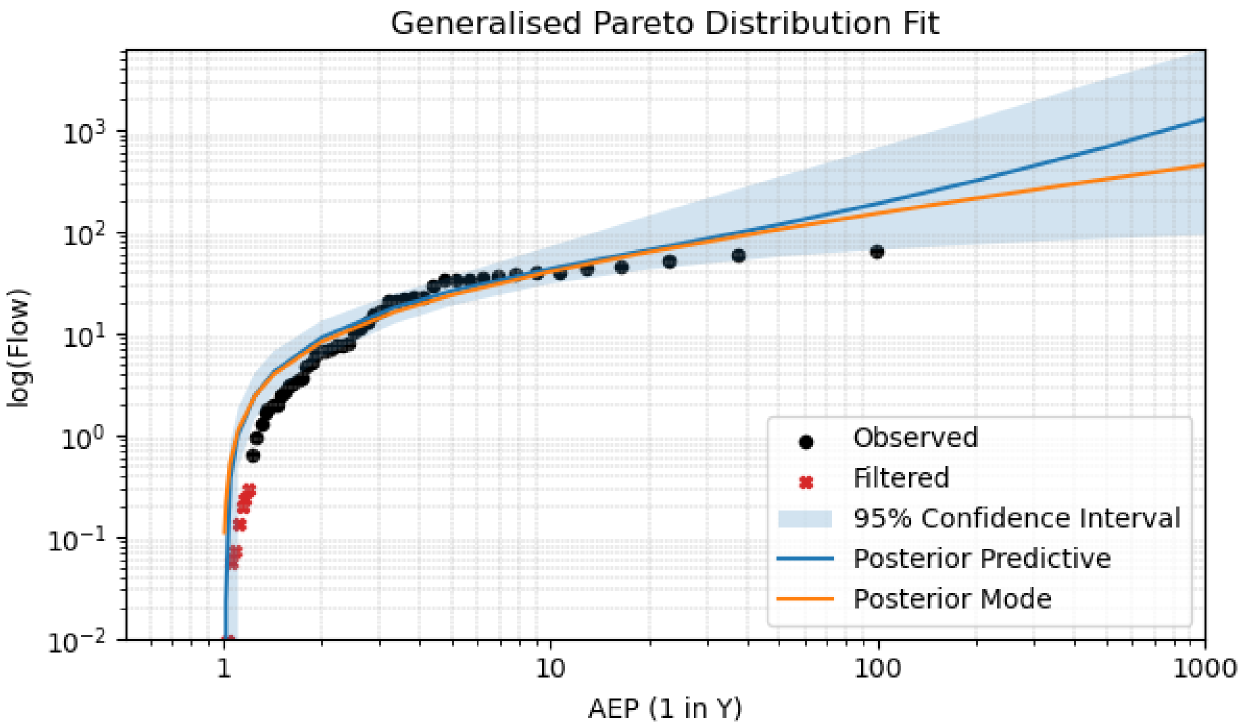

Figure A-1 FFA Generalised Pareto Distribution

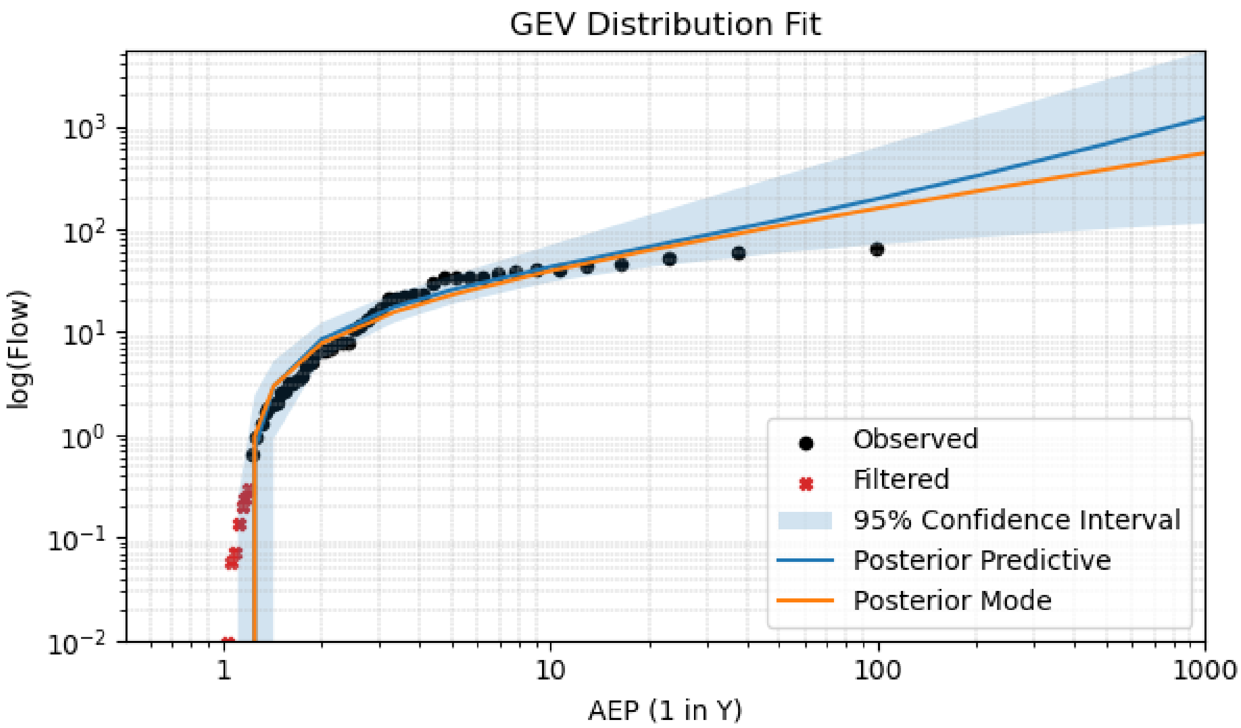

Figure A-2 FFA Generalised Extreme Value Distribution

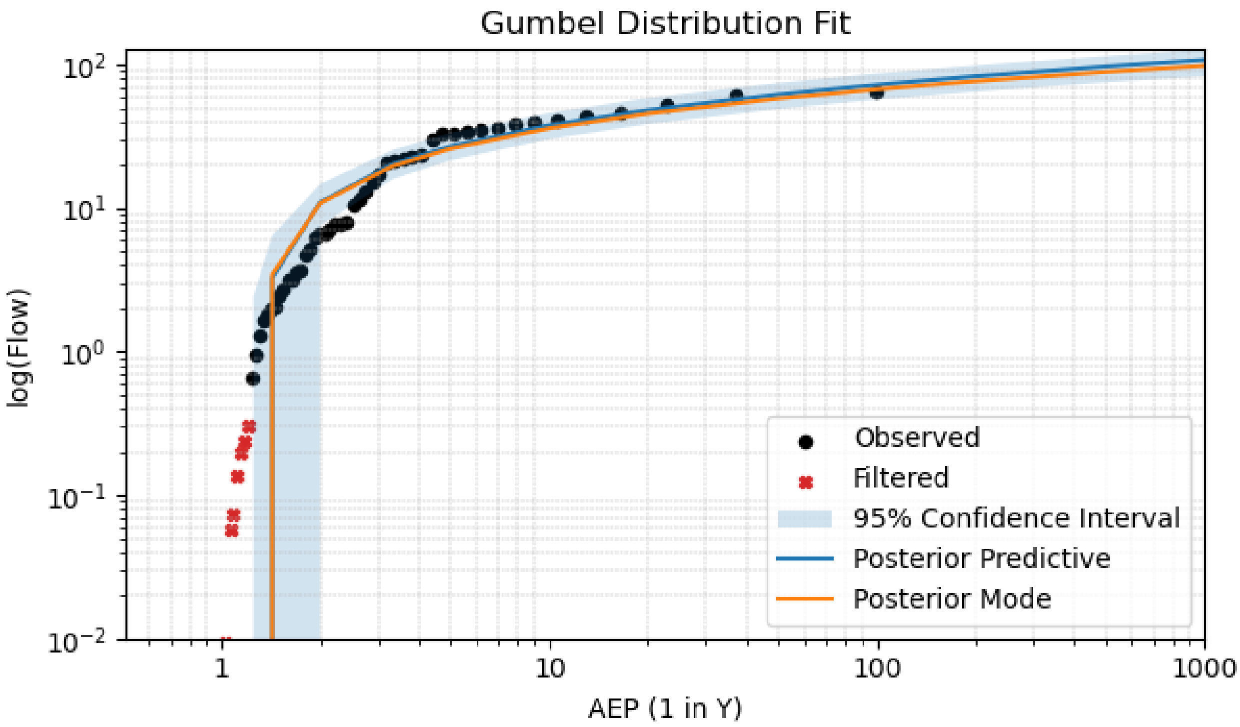

Figure A-3 FFA Gumbel Distribution

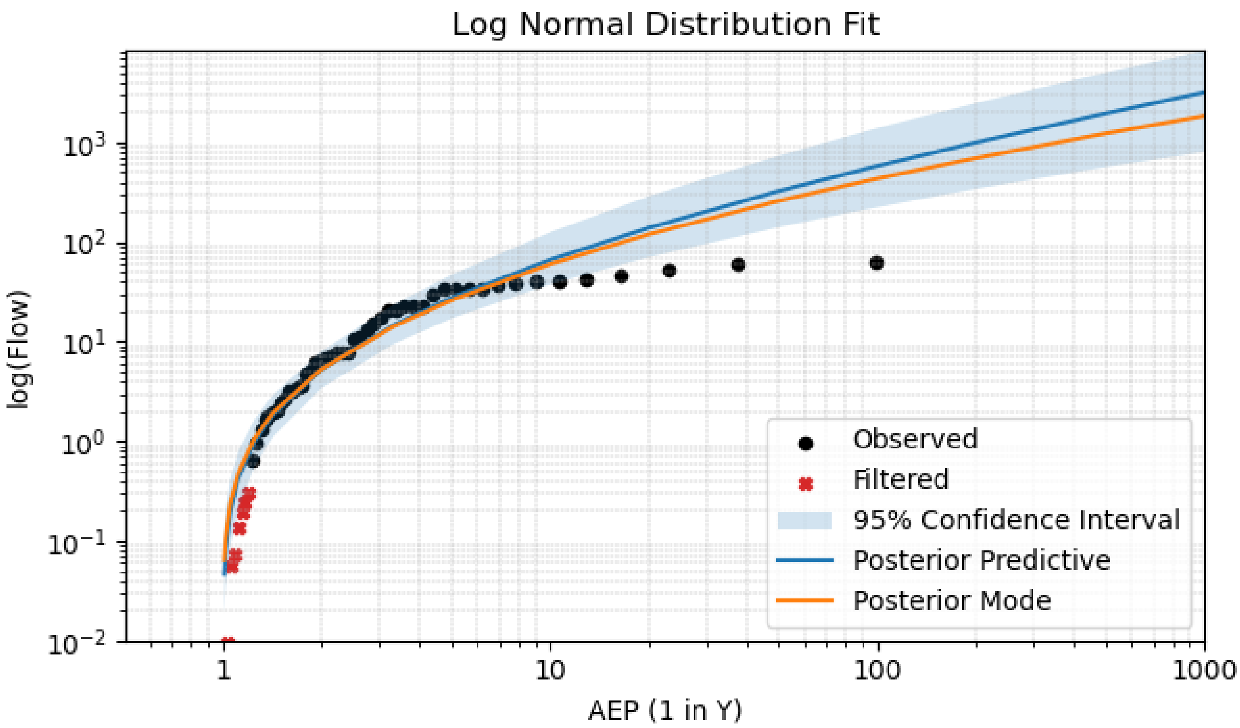

Figure A-4 FFA Log Normal Distribution

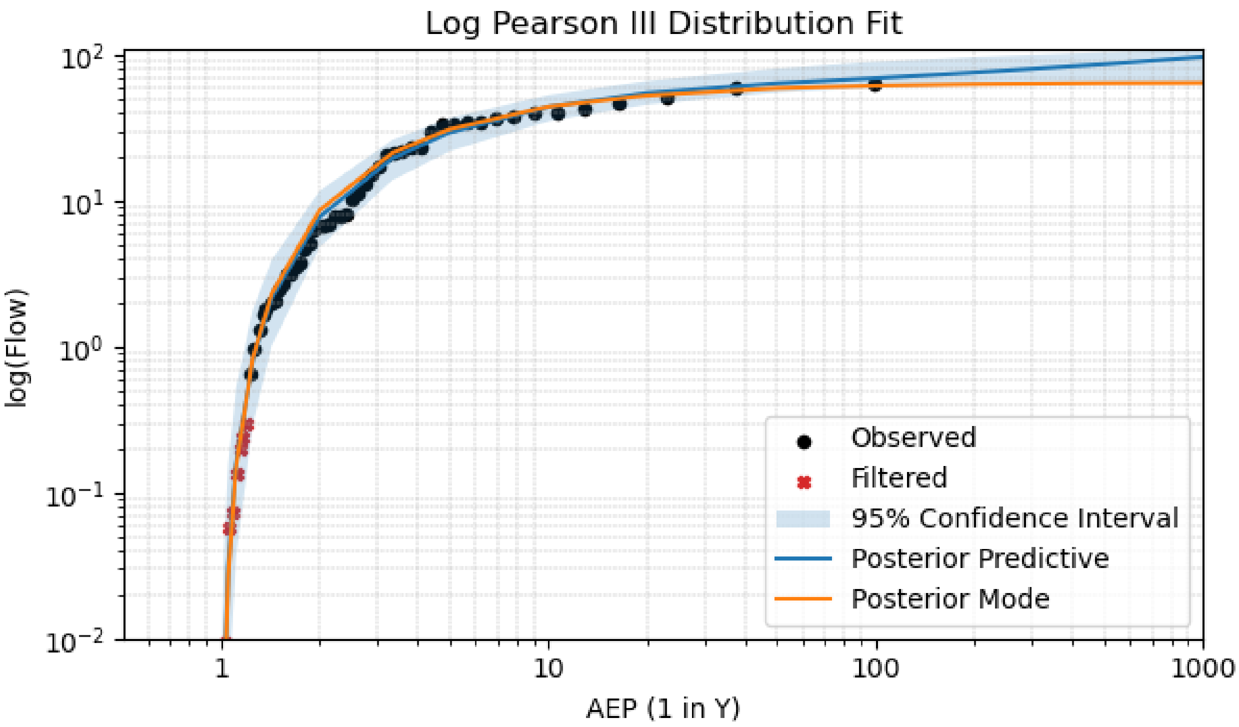

Figure A-5 FFA Log Pearson III Distribution

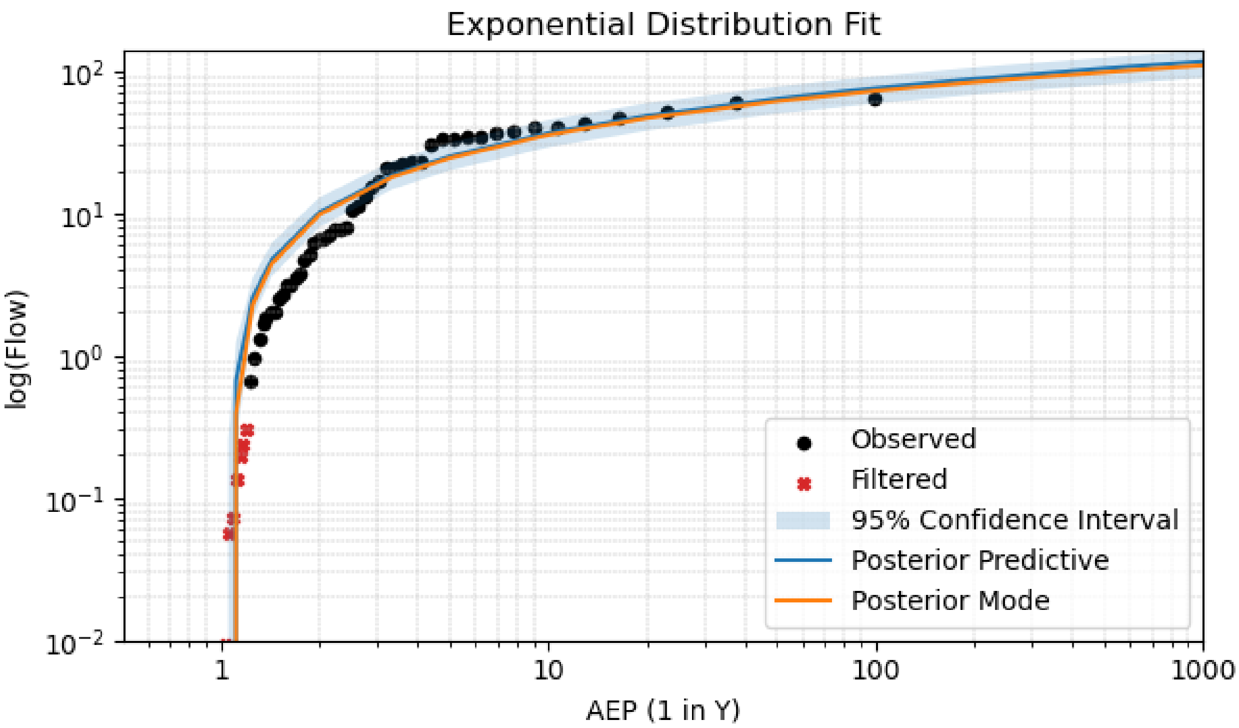

Figure A-6 FFA Exponential Distribution

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Melbourne
15 Business Park Drive
Notting Hill VIC 3168

Sydney
Suite 3, Level 1, 20 Wentworth Street
Parramatta NSW 2150

Brisbane
Level 5, 43 Peel Street
South Brisbane QLD 4101

Adelaide
1/198 Greenhill Road
Eastwood SA 5063

Perth
Level 1, 21 Adelaide Street
Fremantle WA 6160

New Zealand
7/3 Empire Street
Cambridge New Zealand 3434

Wangaratta
First Floor, 40 Rowan Street
Wangaratta VIC 3677

Geelong
51 Little Fyans Street
Geelong VIC 3220

Wimmera
597 Joel South Road
Stawell VIC 3380

Gold Coast
Suite 37, Level 4, 194 Varsity Parade
Varsity Lakes QLD 4227

Darwin
5/5 Goyder Road
Parap NT 0820

Sunshine Coast
Office #4 of the Regatta 1 Business Centre
2 Innovation Parkway
Birtinya QLD 4575

1300 198 413

watertech.com.au