Estimation of Peak Design Discharges within the Mackay Region

A dissertation submitted by

Hayden Francis Brigg

Towards the degree of

Bachelor of Engineering (Honours) (Civil)

October 2017

Mackay REGIONAL COUNCIL

AECOM

UNIVERSITY OF SOUTHERN QUEENSLAND

University of Southern Queensland

Faculty of Health, Engineering & Sciences

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In fulfilment of the requirements of

Courses ENG4111 and ENG4112 Research Project

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ABSTRACT

Flooding is known to be one of the worst natural disasters and can lead to significant economic damage. The design of flood mitigation measures makes up a significant sector of the civil engineering industry, as well as the assessment of flood risk imposed on existing landscape by new development.

There is currently a variety of methods that are prescribed to the industry as to how to best estimate design floods and their associated peak design discharge. The release of the 2016 revision of the Australian Rainfall and Runoff (ARR) guidelines in late 2016 has introduced new methodologies which may impact infrastructure that has been designed to set flood immunities set out by historical guidelines.

This research project aims to explore methods of calculating design discharge estimates for ungauged catchments, particularly within the Mackay Regional Council boundary. These methods include the Rational Method, at site Flood Frequency Analysis (FFA), the Regional Flood Frequency Estimation (RFFE) Model, rainfall runoff-routing modelling software (WBNM) and hydrodynamic software modelling (TUFLOW). The project also investigates the application of different design rainfall event approaches including the simple and ensemble events as outlined in the ARR 2016 guidelines. Through investigating these various methods and approaches, a comparison of results to existing studies and recorded data was made, with commentary provided on the strengths and shortfalls of each method.

The hydrodynamic (TUFLOW) modelling method was found to deliver what was perceived as the most realistic peak design discharge estimate for sites within the Mackay Region, with other methods having their own limitations for application. The application of the ARR 2016 design rainfall and hydrologic parameters was found to cause a decrease in peak discharges when compared to that of the ARR 1987 counterparts.

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CERTIFICATION OF DISSERTATION

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

Hayden Francis Brigg

Student Number: [redacted]

[redacted]

Signature

12th October 2017

Date

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GLOSSARY

1D	One Dimensional
2D	Two Dimensional
AECOM	AECOM Australia Pty Ltd
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ARR	Australian Rainfall & Runoff
BoM	Bureau of Meteorology
CL	Continuing Loss
DEM	Digital Elevation Model
DNRM	Department of Natural Resources and Mines
FFA	Flood Frequency Analysis
GEV	Generalised Extreme Value
GP	Generalised Pareto
HAT	Highest Astronomical Tide
IFD	Intensity Frequency Duration
IL	Initial Loss
LAT	Lowest Astronomical Tide
LPII	Log Pearson III
LiDAR	Light Detection and Ranging
mAHD	Metres Above Horizontal Datum
MHWN	Mean High Water Neaps
MHWS	Mean High Water Springs
MLWN	Mean Low Water Neaps
MLWS	Mean Low Water Springs
MRC	Mackay Regional Council
ROG	Rain on Grid
ROI	Region of Influence
TUFLOW	Integrated 1D/2D Hydrodynamic Modelling Software
USQ	University of Southern Queensland
WBNM	Watershed Bounded Network Model

CHAPTER 1

1. INTRODUCTION

1.1. Project Background

Flooding is known to be one of the worst natural disasters and can lead to significant economic damage. The design of flood mitigation measures makes up a significant sector of the civil engineering industry, as well as the assessment of flood risk imposed on existing landscape by new development.

Currently, there is a variety of methods that are prescribed to the industry as to how to best estimate design floods and their associated peak design discharge. The release of the 2016 revision of the Australian Rainfall and Runoff (ARR) guidelines in late 2016 has introduced new methodologies which may impact infrastructure that has been designed to set flood immunities set out by historical guidelines.

The Mackay Region is situated around the Pioneer River on the eastern coastline of Queensland, Australia. The Pioneer River catchment encompasses over 1,500km^2 of primarily rural farming land with urbanisation along its downstream reaches. The river is famous for its clear blue water all year round and due to its fast flowing nature reaching from the Eungella Hinterland downstream to the Coral Sea. The region has seen a fair share of flooding, particularly within the past decade.

In 2008 the combination of a monsoonal trough and isolated thunderstorms within the Mackay region led to phenomenally intense rainfalls and severe flash flooding. Levels in the Pioneer River peaked at 9.95 meters ranking in at the 4th highest flood on record (refer Figure 1-1). Official alert stations recorded rainfall calculated to be in excess of the 0.5% AEP (1 in 200 year) event (BoM 2008), however unofficial rain gauges recorded higher intensities in some areas.

Figure 1-1: Pioneer River at the Hospital Bridge 2008 (Daily Mercury, 2008)

Figure 1-2: Pioneer River at River Street from the Forgan Smith Bridge (AAP, 2017)

In March 2017 Severe Tropical Cyclone Debbie made landfall on the Whitsunday Coast north of the Mackay Region and brought significant rainfall to the majority of Queensland (refer Figure 1-3). Extreme weather across four days broke March rainfall records at 62 weather stations across the state. In Sarina, south of Mackay, 1300 mm of rainfall was recorded, a total that was more than four times the region's long-term March average of 300 mm (Bali 2017).

To understand the flood risk and to design appropriately, one of the key steps is the derivation of a design flood. A design flood is estimated as the flow or discharge associated with a specified probability (defined as the Annual Exceedance Probability or AEP).

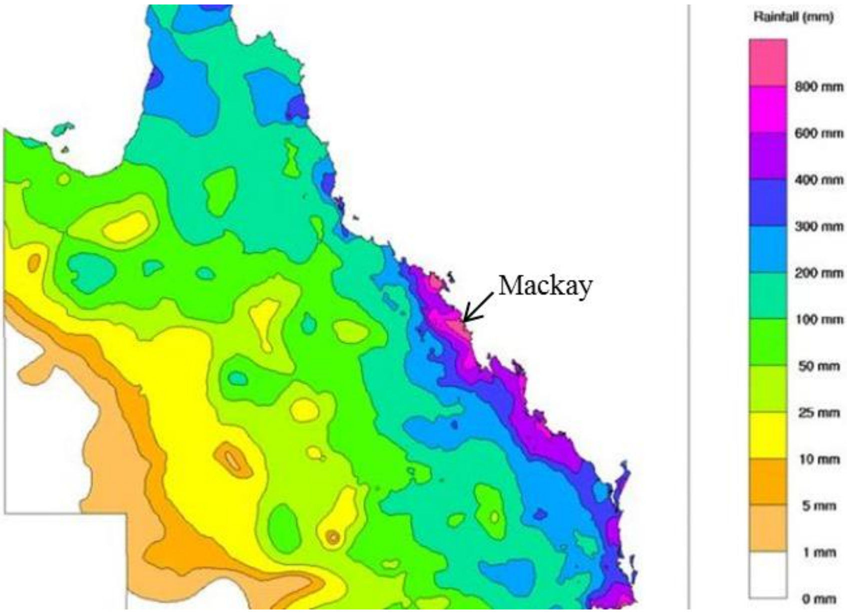

Figure 1-3: Total Rainfall for March associated with TC Debbie (BoM 2017)

The fundamental principal behind the estimation of a design discharge is the conversion of deign rainfall to a flow, this is usually based on a variety of key catchment parameters. There are currently a variety methods that utilise design rainfall inputs and catchment characteristics to compute peak discharges and in some cases flood hydrographs and extents.

A common method of estimating design discharges for a given catchment is to undertake a flood frequency analysis (FFA) on a suitable period of recorded streamflow data. This data is not readily available across all catchments within Australia and there are many catchments with limited or no recorded streamflow data on record, these catchments are often referred to as ‘poorly gauged’ or ‘ungauged catchments’. Within the Central Queensland, both in coastal and inland catchments, there is a small number of streamflow gauges. This in turn deems many catchments within the region as ‘ungauged’, unlike the populous areas of Southeast Queensland, the New South Wales coast and Victoria.

ARR2016 included the release of another method for determining the design discharge for ungauged catchments; the regional flood frequency estimation (RFFE) tool. This method transfers flood characteristics information from a group of gauged catchments (this concept is known as ‘pooling’ or the ‘regionalisation’ of data) to the catchment of interest. Even in cases where there is recorded streamflow data, it is beneficial to combine information in the gauged record with the RFFE information (Rahman et al 2015a)

A regional flood frequency estimation model is formulated around neighbouring data along with simplified assumptions and modelling and is regularly used as a ‘first pass’ estimation or to validate results found through a more detailed modelling approach.

Over time, methods have progressed significantly through the improvement of computing capabilities along with the increase of recorded data. In 1987, the Australian Rainfall and Runoff (ARR) guideline recommended the Rational Method which used linear geographical interpolation of predetermined runoff coefficients. This method was highly recommended for areas in Victoria and along the New South Wales coastline, however the interpolation introduced an assumption to the calculation that was not necessarily satisfied at all locations. As part of the ARR2016 guidelines, a new RFFE model for Australia has been developed that has been aimed at incorporating the latest data and regionalization techniques, to replace the Rational Method in these areas.

Furthermore to the methods outlined above, simulation software is commonly used within the engineering industry to estimate and understand flooding patterns and quantum within catchments. The software can have the capability to estimate simple hydrologic runoff routing or can be more complex in solving hydrodynamic two dimensional calculations. The benefits of these methods are that you get a time series of flows, not only a peak flow.

For larger, catchments runoff routing software is often adopted for hydrologic investigation for flood estimation. This technique involves determining the rainfall-excess and routing it through the catchment storage using flood routing procedures. Computer models are invariably used due to the level of detail necessary in modelling the distributed nature of the catchment storage (Main Roads WA 2014). A runoff routing model is set up by dividing the catchment into smaller sub-catchment areas based on the terrain, watercourse network, land use and rainfall variability. The sub-catchments are connected and runoff within and between each zone is computed.

Two-dimensional (2D) hydrodynamic models are becoming more commonly utilised within the industry for estimating design or historic flooding behaviour. This is due to the improvement in technology and computing power over time. In a 2D model the flow solution is based on the numerical solution of the full 2D depth-averaged equations of motion computed at each active water grid point (Babister & Barton 2012).

1.2. Project Aim

This research project aims to explore methods of calculating design discharge estimates for ungauged catchments, particularly within the Mackay Regional Council boundary. These methods include the Rational Method, at site Flood Frequency Analysis (FFA), the Regional Flood Frequency Estimation (RFFE) Model, rainfall runoff-routing modelling software (WBNM) and hydrodynamic software modelling (TUFLOW). The project also investigates the application of different design rainfall event approaches including the simple and ensemble events as outlined in the ARR2016 guidelines. Through investigating these various methods and approaches, a comparison of results to existing studies and recorded data can be made and commentary provided on the strengths and weaknesses of each method.

1.3. Expected Outcomes and Benefits

This project has been designed to provide a comparison between the current and new peak flow estimation methods as outlined in the 2016 revision of the Australian Rainfall and Runoff Guidelines (Ball et al 2016d). It is expected that the comparison will provide guidance to the local industry as to variances in results and uncertainties of each method, specifically in relation to catchments within the Mackay Region, comparing the outputs for each method at the same location.

It is anticipated that the new methodologies will provide varying results to those found by previous hydrologic and hydraulic investigations within the region. Commentary on the theory behind these variances will help educate and guide decision making for similar studies in the future.

CHAPTER 2

2. LITERATURE REVIEW

This section includes the findings of the literature review that was conducted as part of the study. The research has been summarised into sections directly relating to the estimation of peak flows and the relevant hydrologic and hydraulic phenomenon.

2.1. Hydrologic Processes Contributing to Floods

Within Australia, the main cause of large streamflows that result in flooding is usually rainfall events. Due to the Australian climate, other flooding mechanisms such as the melting of snow and ice are unlikely (Ladson & Nathan 2016).

However, not all rainfall contributes to streamflow and subsequent flooding. When rainfall lands within a catchment the ground conditions at the time of rain will determine whether or not the rainfall will be converted into runoff. Soil infiltration capacity and saturation as well as groundwater levels, vegetation demand and surface storage are just some of the mechanisms that may alter the quantum of flows that will ultimately runoff and become streamflow. In some cases, rainfall may be conveyed through the groundwater table and contribute to flooding by a phenomenon known as baseflow.

The majority of catastrophic flooding within Australia occurs in catchments that are susceptible to high volumes of streamflow with limited rainfall losses. In these scenarios a greater proportion of rainfall is converted to runoff, usually due to the catchment being ‘wet’ meaning that infiltration will be minimal. Some other causes of high streamflow can be short bursts of very intense rainfall, limiting infiltration ability or long periods of low intensity rainfall when evapotranspiration is low, such as in the cooler months.

The following sections describe the hydrologic mechanisms that contribute to flooding. A visual representation of the transformation of rainfall within the catchment as described above has been included in Figure 2-1.

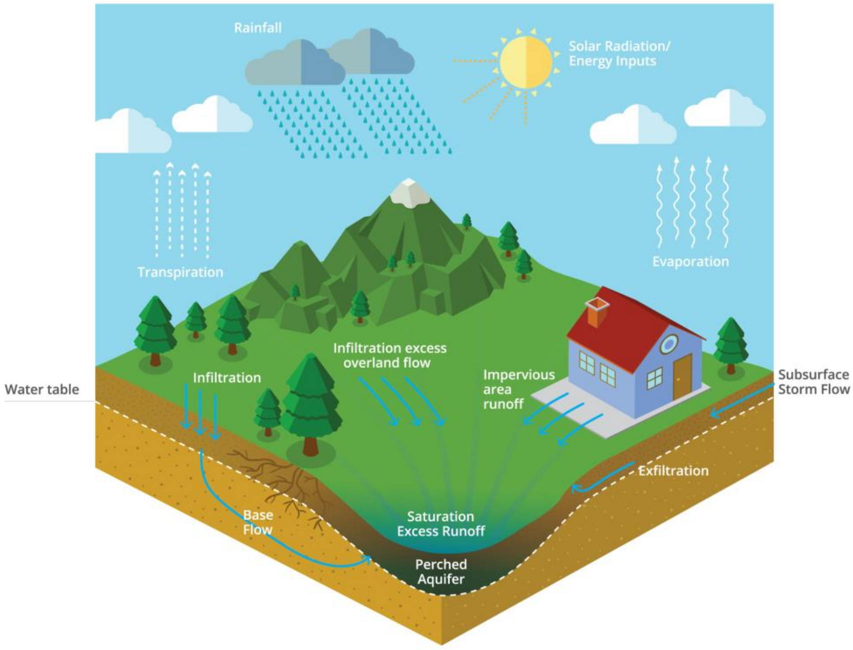

Figure 2-1: Catchment Runoff Generation Processes (Ladson & Nathan 2016)

2.1.1. Runoff Generation

Once rainfall has fallen from the sky there are many different mechanisms that can convert the rainfall into runoff. Figure 2-2 shows the processes that occur once the rainfall has fallen and how they are converted into quickflow, losses, baseflow and routing. These forms of runoff are discussed in the subsequent sections.

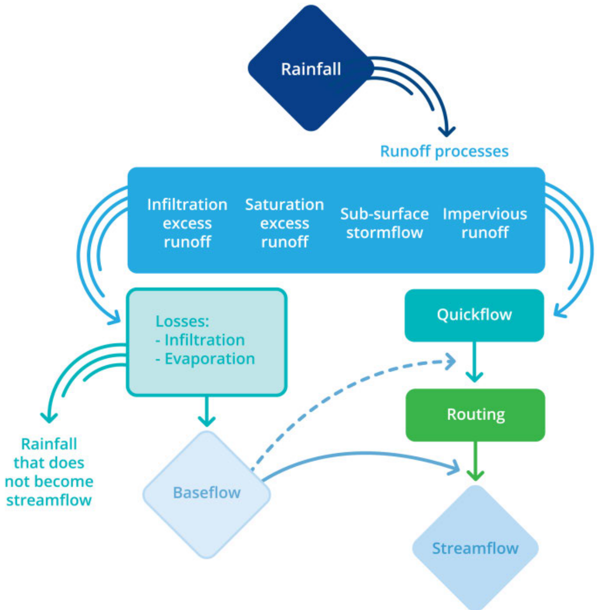

Figure 2-2: Simplified Description of the Processes Converting Rainfall to Runoff and Streamflow (Ladson & Nathan 2016)

2.1.1.1. Quickflow

Excess runoff contributes to the quantum of quickflow within a flood event and is made up of the rainfall that cannot be absorbed through infiltration into the soil. The infiltration rate at which the water enters the soil is dependent on the rate at which the water is supplied to the soil surface and the infiltration capacity or the maximum rate at which the water can enter the soil (Ladson & Nathan 2016). Water will remain on the surface or runoff on sloped terrain when the rainfall rate is greater than the infiltration rate of the soil. This runoff mechanism is also known as Hortonian flow and can cause rapid runoff and flash flooding when intense rainfall is experienced in catchments with favourable conditions.

Saturation excess runoff is very similar to that of infiltration runoff however it occurs when the soil is deemed to be fully saturated and no additional rainfall can permeate the surface. This process is commonly observed in wetlands, stream banks or other permanently saturated regions of the catchment. The excess runoff from this flooding mechanism can be of large quantum in intense rainfall periods and hence makes up the quickflow component of a flood hydrograph.

Subsurface flows can contribute to quickflow and supply runoff through the transfer of water from underground to the surface through pores in saturated soil. This process is common in areas that have steep slopes and conductive soils that create the appropriate conditions. This process is enhanced where there is an impeding soil layer that leads to the formation of perched water tables which cause soils to saturate and become highly conductive (Weiler et al 2005).

Rainfall over impervious areas results in the rapid production of quickflow due to the inability to infiltrate. Urbanisation has been found to cause large increases in runoff volumes, flood frequencies and magnitude (Ladson & Nathan 2016). Previous studies have found that the impacts of urbanisation can increase peak flows by up to 10 times in the range of 1 to 4 Exceedances per Year (EY) (Cordery 1976). Runoff in urbans streams was found to respond more rapidly compared to rural catchments (Mein & Goyen 1988).

2.1.1.2. Baseflow

It is common for streamflow to be divided into quickflow and baseflow. The proportion of quickflow is the rapid response runoff from a rainfall event whilst baseflow is contributed to the flood event by stored water being slowly released.

Figure 2-3 shows a diagrammatic representation of quickflow and baseflow. The ARR Guidelines (Ladson & Nathan 2016) described the relationship between quickflow and baseflow as: "initial baseflow represents the contribution from previous events; then as the hydrograph rises, baseflow can be depleted as water enters bank storage or is removed by transmission loss. Later, baseflow can increase as bank storage re-enters the stream, or through other processes such as interflow and discharge from groundwater".

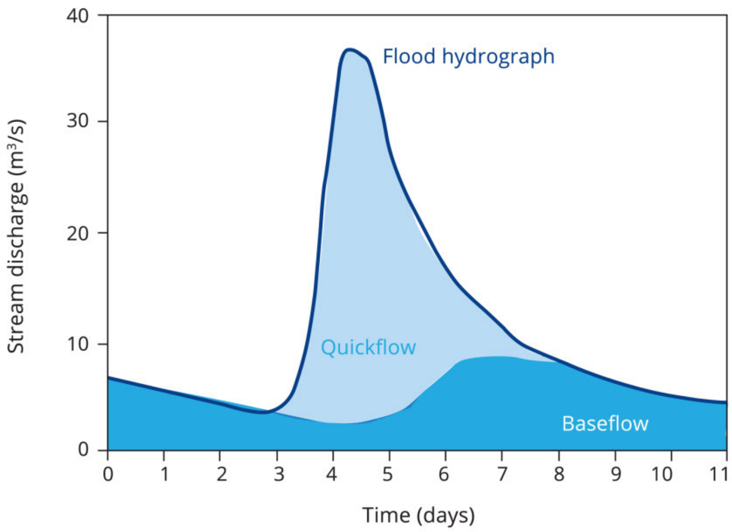

Figure 2-3: Observed Hydrograph - Sum of the Baseflow Hydrograph and the Quickflow (Ladson & Nathan 2016)

2.1.1.3. Losses

Losses refer to of the depth of rainfall that is not converted to quickflow and is primarily converted to soil infiltration. The depth of losses is subtracted from the rainfall depth to calculate the rainfall excess.

2.1.1.4. Routing

Once rain has fallen, excess rainfall not converted to losses is deemed as runoff and is transferred through the catchment via flow paths. This introduces overland flow on slopes, through tributaries and across floodplains as well as flow through natural or artificial storages. "Flow routing is the mathematical description of flow processes that model the attenuation and translation of hydrographs as water moves through this network" (Ladson & Nathan 2016). More detail on runoff routing is provided in 2.6.2.

2.2. Simulation of Design Flood Hydrographs

Some design problems require more than just the peak flows. They require a hydrograph or a time-series of flows. This is very common with drainage volume design problems as the entire flood hydrograph has an influence on the design. These problems are usually driven by the rate of raise of the flood hydrograph, or the volume of water in the hydrograph. To develop a design flood hydrographs, an event simulation process is required.

The simulation of the design flood hydrograph is undertaken using design rainfall data, this allows for the formulation of the shape and volume of the hydrograph. The ARR Guidelines (Ball & Nathan 2016a) published that the rainfall data can be transformed into a selected flood characteristic:

- event-based models transform probabilistic bursts of rainfall to corresponding estimates of floods; and
- continuous simulation models transform a time series of rainfall into probabilistic flood estimates

Following this, hydrodynamic models (refer Section 2.7) can be used to estimate flood behaviour and levels from hydrologic inputs. The guidelines (Ball & Nathan 2016a) also published: "The challenge with these methods is how to achieve probability neutrality, that is how to ensure that the method used to transform rainfalls into design floods is undertaken in a fashion that minimises bias in the resulting exceedance probabilities."

2.2.1. Event Based Approaches

A traditional practice within Australia and also many overseas countries is to use an event based approach for the derivation of design floods from design rainfalls. The ARR Guidelines on event based approaches outlines that the typical hydrological inputs for an event based model include (Nathan & Ling 2016):

- A design storm of preselected AEP and duration: historically it has been most common to only consider the most intense parts of complete storms (‘design burst’), where the average intensity of the burst is determined from rainfall Intensity Frequency Duration (IFD) data. This information is generally available as a point rainfall intensity, and it is necessary to apply an Areal Reduction Factor to correctly represent the areal average rainfall intensity over a catchment;
- An Areal Reduction Factor (ARF) to convert the design point rainfall to a rainfall depth over the entire catchment;
- Temporal patterns to distribute the design rainfall over the duration of the event, and this can include additional rainfalls before the start (and after the end) of the burst to represent complete storms;
- Spatial patterns to represent rainfall variation over a catchment that occurs as the result of factors such as catchment topography and storm movement; and
- Loss parameters that represent soil moisture conditions in the catchment antecedent to the event and the capacity of the soil to absorb rainfall during the event

Following the hydrological inputs, a variety of event based models are available to then convert rainfall into a flood hydrograph. The models simplify the representations of key hydrologic processes within the catchment relevant to the generation of the design hydrograph. These model simplifications were published in the ARR Guidelines (Nathan & Ling 2016) as:

- A loss model is used to estimate the portion of rainfall that is absorbed by the catchment and the portion that appears as direct runoff. This loss is typically attributed to a range of processes, including: interception by vegetation, infiltration into the soil, retention on the surface (depression storage), and transmission loss through the stream bed and banks; and

- A hydrograph formation model or hydrologic routing model (usually based on runoff-routing concepts) is used to transform the patterns of rainfall excess into a design flood hydrograph. This flood hydrograph may include a baseflow component which initially represents the delayed contribution from previous rainfall events, and in the latter stages of the event may represent the contribution from earlier losses.

Utilising a design event approach is the most common approach within Australia which assumes there is a critical rainfall duration for the catchment. This duration is dependent on the catchment characteristics and is determined through trial of a number of rainfall durations. The critical duration will produce the highest flood peak (or volume) for the catchment.

When utilising a design event approach it is important that the inputs defining the event are selected to be probability neutral. This means that the inputs should be such that the 1 in X AEP design rainfall will convert to the corresponding 1 in X AEP flood. This is made difficult due to the realisation that flood response to rainfall is generally non-linear, meaning that the average rainfall or loss conditions are unlikely to produce the same average flood conditions. Determination of the probability neutrality of the inputs depends on the availability of independent flood estimates for comparison.

The ARR Guidelines (Nathan & Ling 2016) report on three approaches to help deal with probability neutrality, these are listed below:

- Simple Event (refer Section 2.2.2), where all hydrologic inputs are represented as single probability neutral estimates from the central range of their distribution;
- Ensemble Event (refer Section 2.2.3), where the dominant factor influencing the transformation is selected from a range of values representing the expected range of behaviour, and all other inputs are treated as fixed; and
- Monte Carlo Event (refer Section 2.2.4), where all key factors influencing the transformation are stochastically sampled from probability distributions or ensembles, preserving any significant correlations between the factors, and probability neutrality is assured (for the given set of inputs) by undertaking statistical analysis of the outputs.

Figure 2-4 illustrates the key differences in each of the approaches listed above. As stated in the ARR Guidelines (Nathan & Ling 2016): “It is worth noting the essential similarities between the three methods. It is seen that these three methods use the same source of design rainfalls and the same conceptual model to convert rainfall into a flood hydrograph. The process involved in calibrating a conceptual model to historic events is common to all three approaches, they differ only in how selected inputs are treated when deriving design floods.”

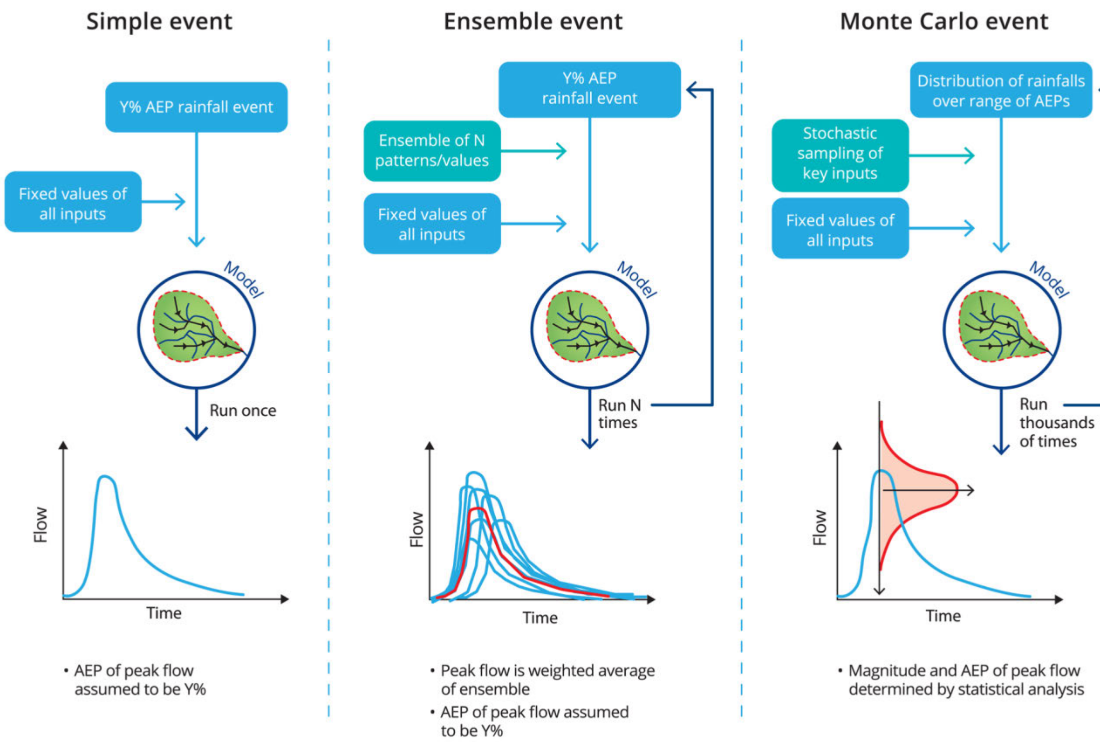

Figure 2-4: Elements of Three Different Approaches to Flooding (Nathan & Ling 2016)

2.2.2. Simple Event

The Simple Event method firstly involves estimating the average intensity or depth of rainfall for a specific AEP. This is completed using design Intensity Frequency Duration (IFD) data. Following this other factors that influence the flood hydrograph are also selected, these include a representative temporal pattern and spatial pattern of the rainfall over the catchment as well as appropriate loss parameters for the catchment conditions.

Temporal rainfall patterns have been determined by applying the Average Variability Method to a sample of historic rainfall samples (Pilgrim et al. 1969). These temporal rainfall patterns were representative of rainfall regions within Australia and published in previous revisions of the ARR Guidelines (Pilgrim et al. 1987). More recent studies have found that there is evidence that patterns of average variability do not ensure probability neutrality (Sih et al. 2008).

The spatial patterns of rainfall have been found to have a lesser influence on the design flood hydrograph than temporal patterns and hence are easier in accommodating probability neutrality. It has been deemed sufficient to adopt spatial rainfall patterns that reflect the systematic variation arising from topographic influences.

Loss models applied within the Simple Event are described in Section 2.6.3. The excess runoff from the loss model is then routed through the catchment parameters to develop the design flood hydrograph. “The hydrograph corresponding to the rainfall burst duration that results in the highest peak (the critical rainfall duration) is taken as the design flood hydrograph, and it is assumed to have the same Annual Exceedance Probability as its causative rainfall” (Nathan & Ling 2016).

It is important to consider that probability neutrality is an assumption that has been untested in regards to the Simple Event. Without calibration of the flood frequency estimates using gauged data from the site there is no way of understanding how the selected inputs may have biased the outcome.

2.2.3. Ensemble Event

The Ensemble Event can be seen as an intermediate step between the Simple Event and the Monte Carlo Event approach. The Ensemble method essentially takes a fixed input that has a significant input on the flood hydrograph and replaces it with a variety or ensemble of values. These values are then all tested to give an array of flood hydrographs of which the design flood hydrograph is taken as the weighted average. The weighting applied to each result is based on the relative likelihood of the selected input occurring.

ARR2016 recommends that an array of temporal patterns should replace the singular pattern when using the Ensemble Event approach. For this type BOM have developed ten temporal patterns for each storm duration. These patterns all have varying shapes and periods of rainfall intensities to represent potential storm patterns that may be experienced within a particular region. The application of design rainfall to each temporal pattern is a similar procedure to that of the simple event, however the process is to be repeated for each temporal pattern.

2.2.4. Monte Carlo Event

The latest ARR guidelines (Nathan & Ling 2016) defines the Monte Carlo methods as “a framework for simulating the natural variability in the key processes that influence flood runoff: all important flood producing factors are treated as stochastic variables, and the less important ones are fixed. The primary advantage of the method is that it allows the exceedance probability of the flood characteristic to be determined without bias (subject to the representativeness of the selected inputs”.

As the Monte Carlo event is data intensive and is currently not commonly accepted within the industry, it has not been used in this investigation.

2.2.5. Continuous Simulation

The ARR Guidelines (Nathan & Ling 2016) define Simultaneous Rainfall Simulation as follows: “The Continuous Simulation method of estimating the design flood is similar in intent to the event-based Monte Carlo approach. Both methods seek to adequately simulate the interactions between flood producing (rainfall and catchment characteristics) variables. The Continuous Simulation method of estimating the design flood involves running a conceptual runoff-routing model for a long period of time such that all important interactions (covering the dry and wet periods) between the storm (intensity, duration, temporal pattern) and the catchment characteristics are adequately sampled to derive the flood frequency distribution. In general, pluviograph data of hourly resolution (or less) is used to drive the runoff-routing models. In most cases the period of record of pluviograph data rarely exceeds 20 years, therefore rainfall data is extended by using stochastic rainfall data generation. The runoff-routing model is calibrated using flow data, where available, and the calibrated model is then used to generate a long series of simulated flow. Finally the simulated flow is then used to extract the Annual Maximum Series and estimate the derived flood frequency curve.”

For similar reasoning as the Monte Carlo approach, the data intensive and time consuming approach of Continuous Simulation will not be perused as a part of this research project.

2.3. At-Site Flood Frequency Analysis

A Flood Frequency Analysis (FFA) is the process where recorded flood data is analysed to estimate the probability model of flood peaks, with this estimation then used to determine peak flows for design events.

2.3.1. The Flood Probability Model

In a FFA it is important to understand the difference in definition of the flow variable:

Q = flow value denoting the flood peak

q = a specific flow realisation (or sample)

The ARR Guidelines (Kuczera & Franks 2016) provide the definition of the flood probability model as: "In its most general form, the flood probability model can be described by its Probability Density Function (pdf) $p\left(s \mid \theta(x)\right)$ where $\theta(x)$ is the vector (or list) of parameters dependent on x, a vector of exogenous or external variables such as climate indexes. The symbol '|' is interpreted as follows: the variable to the left of '|' is a random variable, while the variables to the right of '|' are known values."

The distribution function of Q is defined as the non-exceedance probability $P(Q \leq q)$ and is related to the pdf by:

$$P\left(Q \leq q \mid \theta(x)\right) = \int_{0}^{q} p\left(s \mid \theta(x)\right) ds$$

Equation 2-1

2.3.1.1. Homogeneous Flood Probability Model

The homogeneous flood probability model is the simplest form and arises when θ does not depend on an exogenous vector x. In this case, it can be considered that each flood peak is a random realisation from the same probability model $p(q \mid \theta)$. Flood peaks therefore form a homogeneous time series under this assumption.

2.3.1.2. Non-Homogeneous Flood Probability Model

The non-homogenous flood probability model arises when flood peaks do not form a homogeneous time series and is much more complex than the homogeneous model. The ARR Guidelines (Kuczera & Franks 2016) reports that this may arise when:

- Rainfall and flood mechanisms may be changing over time. For example, long-term climate change due to global warming, land use change and river regulation may render the flood record non-homogeneous.
- Climate may experience pseudo-periodic shifts that persist over periods lasting from several years to several decades. There is growing evidence that parts of Australia are subject to such forcing and that this significantly affects flood risk.

2.3.2. Annual Maximum Series

The Annual Maximum (AM) series is formed by the extraction of the maximum discharge in each year. This can be either a calendar year (as used in the southern states of Australia) or a ‘water year’ as used in the parts of Australia that experience a tropical climate. This data is then used to estimate the probability that the maximum flood discharge recorded for that year exceeds a particular magnitude. In ARR (Kuczera & Franks 2016), this probability is called the Annual Exceedance Probability AEP(w) and is formally defined as:

$$AEP(w) = P(W \le w \mid \theta(x)) = \int_{q}^{\infty} p(s \mid \theta(x))ds$$

Equation 2-2

where w is the maximum flood discharge in a year. Often it is convenient to express the AEP as a percentage X% or alternatively for rare events. as a ratio 1 in Y. For example, the 1% AEP is equivalent to an AEP of 1 in 100 or 0.01 (Kuczera & Franks 2016).

2.3.3. Peak-Over-Threshold (Partial) Series

The Peak-Over-Threshold (Partial) series is formulated by extracting peak discharges from record that independently exceed a peak threshold discharge. Typically the threshold is determined so that the number of peaks extracted is 2 to 3 times the number of years the data is extracted from.

As per the ARR Guidelines (Kuczera & Franks 2016), The data in the POT series can be used to estimate the probability distribution of the time to the next peak discharge that exceeds a particular magnitude:

$$P(\text{Time to next peak exceeding } q \leq t) = 1 - e^{-EY(q)t}$$

Equation 2-3

where t is time expressed in years and EY(q), the number of exceedances per year, is the expected number of times in a year that the peak discharge exceeds q (Kuczera & Franks 2016).

2.3.4. Annual vs Partial Series

The probability definitions of Annual Exceedance Probability (AEP) and Exceedances per Year (EY) are intimately connected. The analysis presented in theory of Section 2.3.2 and 2.3.3 shows that:

$$\begin{aligned} EY(w) &= -\log_e \left[1 - AEP(w)\right] \\ &= -\log_e \left[1 - \frac{1}{Y(w)}\right] \end{aligned}$$

Equation 2-4

where AEP(w) is expressed as the ratio 1 in Y(w). Figure 2-5 shows the relationship between the two. It can be seen that for AEP’s less than 10% the two definitions are the same, however as the AEP increases beyond 10%, EY increases much more rapidly than AEP. “This occurs because in years with a large annual maximum peak, the smaller peaks of that year may exceed the annual maximum peak in other years” (Kuczera & Franks 2016).

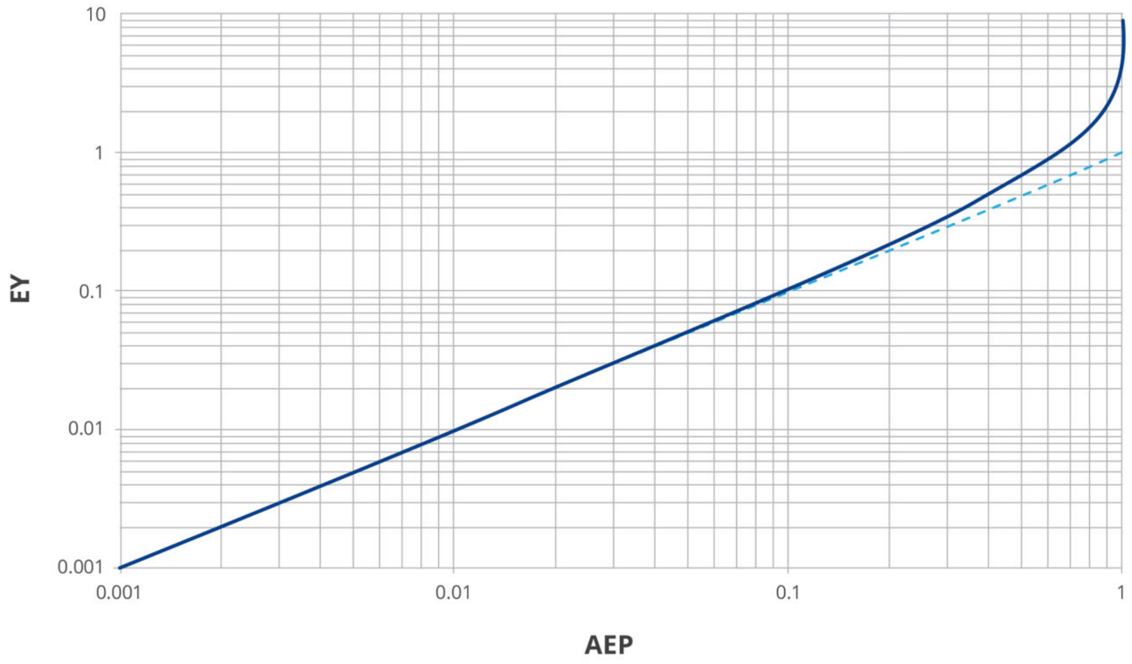

Figure 2-5: Annual Exceedance Probability (AEP) - Exceedances per Year (EY) Relationship (Kuczera & Franks 2016)

The latest revision of the ARR guide (Kuczera & Franks 2016) provides the following guidelines on when Annual Maximum (AM) and Peak-Over-Threshold (POT) (Partial) series approaches should be utilised:

i. AEP of interest less than 10% (i.e. events rarer than 10% AEP)

   AEPs, in this range, are generally required for estimation of a design flood for a structure or works at a particular site. Use of AM series is preferred as it yields virtually identical answers to POT series in most cases, provides a more robust estimate of floods and is easier to extract and define.

ii. EY of interest greater than 0.2 events per year (i.e. events more frequent than 0.2 EY)

   Use of a POT series is generally preferred because all floods are of interest in this range, whether they are the highest in the particular year of record or not. The AM series may omit many floods of interest. The POT series is appropriate for estimating design floods with a relatively high EY in urban stormwater contexts and for diversion works, coffer dams and other temporary structures. However, in practice, flow records are not often available at sites where minor works with a design EY greater than 0.1 events per year is required.

2.3.5. FLIKE Software

Part of the 2016 revision of the ARR Guidelines (Kuczera & Franks 2016) recommends the use of FLIKE (BMT WBM 2015) flood frequency analysis software. The software was developed by Professor George Kucszera and utilises annual maximum stream gauge data to undertake an at site annual flood frequency analysis. The software samples the data using the Bayesian statistical method (refer Section 2.3.6) and then fits the data to a probability model to determine the correlation between discharges and annual exceedance probability.

The probability models that the FLIKE software package incorporates are:

- Log Pearson III
- Log Normal
- Gumbel
- Generalised Extreme Value (GEV)
- Generalised Pareto

Each of the methods utilise complex mathematical and statistical models and are fully described in the 2016 revision of the ARR Guidelines (Kuczera & Franks 2016).

2.3.6. Bayesian Statistical Method

The Bayesian statistical method is built on two factors, the likelihood function and a prior distribution. These factors outline the information about the parameters and describe what is known about the parameters prior to the data observation respectively.

A probability distribution that expresses current knowledge of model parameters is written as $p(\theta)$.

When new data becomes available, information that is contained regarding the model parameters, proportional to the distribution of the observed data, is expressed in a likelihood function $p(y \mid \theta)$.

This information, when combined with the prior, is used to produce the 'posterior distribution', the updated probability distribution on which the Bayesian statistical method is produced. In summary, the posterior is proportional to the likelihood, giving:

$$p(\theta \mid y) = \frac{p(\theta) \times p(y \mid \theta)}{\int_{\theta} p(\theta) \times p(y \mid \theta) d\theta}$$

Equation 2-5

This statistical method is recommended in the ARR Guidelines (Kuczera & Franks 2016) as one the most suitable for flood frequency analysis due to its ability to censor outlying data to improve the fit of probability models. The Bayesian statistical method also has the capacity to use both gauged and censored historical data and has been proven to work for all probability models.

2.4. Regional Flood Methods

The 1987 revision of the ARR Guidelines recommended various design flood estimation techniques for small to medium sized catchments for different regions of Australia (Pilgrim & Doran 1987). Since 1987, the methods in the ARR have not been upgraded although there has been the availability of an additional 20 years of streamflow data and notable development in both at-site and regional flood frequency analyses techniques in Australia and internationally. As part of ARR 2016 Revision Projects, Project 5, Regional Flood Methods for Australia focuses on the development, testing and recommendation of new regional flood estimation methods for Australia by incorporating the latest data and techniques (Rahman et al. 2009).

Through the three stages of the Regional Flood Methods for Australia investigation (ARR Revision Project 5) various methods of flood frequency analysis (FFA) were tested for a variety of catchments in a range of climatic regions across Australia. The new RFFE model was designed to estimate the flood quantity for 6 annual exceedance probabilities (AEPs), being the 50%, 20%, 10%, 5%, 2% and 1% design events at each gauged catchment in Australia (853 catchments in total).

798 of the catchments within the model have been deemed as data-rich, that is data from these catchments is sufficient enough to undertake an annual maximum flood series FFA. Through the research project it was found that a Bayesian generalised least squares (GLS)

regression approach was best suited to develop prediction equations for the parameters/moments of the LP3 distribution (parameter regression technique). These prediction equations require two to three predictor variables (catchment area, design rainfall based off Bureau of Meteorology 2013 design rainfall data at the catchment centroid and the shape factor). These prediction equations largely satisfy the assumptions of the regression analysis (Rahman et al. 2015b).

For the data-poor areas it was determined that a partial series FFA (as average number of events per year = 0.5) would be used to estimated design discharges through a Generalised Pareto distribution and the L moments procedure.

2.4.1. Development of the RFFE Model 2015

The RFFE model has been developed to ensure that design flood discharge estimates are consistent with the gauged records and with results for other ungauged catchments in the region. The technique was developed to be simple, requiring only the readily accessible catchment characteristics with the ability output flood estimates quickly through computer analysis. The section relating to the RFFE model in the latest ARR Guidelines (Rahman et al. 2016) reports that it is recognised that there will be considerable uncertainty in estimates for ungauged catchments because of the limited number of gauged catchments available to develop the method and the wide range of catchment types that exist throughout Australia.

The criteria that were set in the development of the RFFE Model 2015 has been defined as (Rahman et al. 2016):

- National consistency in approach;
- Smooth interfacing at the boundaries between areas;
- Use readily accessible data; and
- Utilise as much of Australia's streamflow database as possible.

2.4.2. Definition of the RFFE Model 2015

The definition of the Regional Flood Frequency Estimation is published as (Rahman et al. 2016): RFFE is a data-driven approach, which attempts to transfer flood characteristics from a group of gauged catchments to ungauged locations of interest (where design floods need to be estimated). A range of different methods are available to extract regional flood information from the pooled data and to transfer the relevant information to an individual ungauged catchment in the region (Sivapalan et al. 2013). All of these RFFE techniques use the results of at-site Flood Frequency Analysis (FFA) as basic data. A RFFE technique essentially consists of two steps:

i. Formation of Regions - which involves identification of the regions for which flood data from the available streamflow gauging stations can be pooled for analysis; and
ii. Development of Regional Estimation Equations - which involves derivation of prediction equations to be used for design flood estimation within a region."

2.4.3. Formation of RFFE Model 2015 Regions

For the methodology adopted for the RFFE Model 2015, the nation has been divided into seven regions. This is made up of five data rich humid coastal regions, each formed using the region of influence (ROI) technique and two arid/semi-arid regions.

For the Tasmanian region, ROI was implemented using 51 stations from Tasmania. All the 558 stations from Victoria (VIC), the Australian Capital Territory (ACT), New South Wales (NSW) and Queensland (QLD) form the East Coast Region. A total of 28 stations from South Australia (SA) form the Humid SA Region. A total of 50 stations from the Northern Territory (NT) and 8 stations from the Kimberley region of Western Australia (WA) are combined to form the Top End NT and Kimberley region. And a total of 103 stations from south-west Western Australia (WA) form the SW WA region.

The remaining arid/semi-arid catchments were then split into two regions. The 11 catchments from the Pilbara area of Western Australia and the remaining 44 catchments form the Pilbara and Arid and Semi-Arid regions respectively. Figure 2-6 shows the delineation of the regions within the RFFE Model 2015.

The boundaries between the two arid/semi-arid (data-poor) and the five data-rich regions in Figure 2-6 are drawn approximately based on the 500 mm mean annual rainfall contour line. To reduce the effects of sharp variation in quantile estimates for the ungauged catchments located close to these regional boundaries, seven fringe zones have been delineated, as shown. For these fringe zones, the flood quantile at an ungauged catchment location is taken as the inverse distance weighted average value of the two nearby regional estimates (Rahman et al. 2015c).

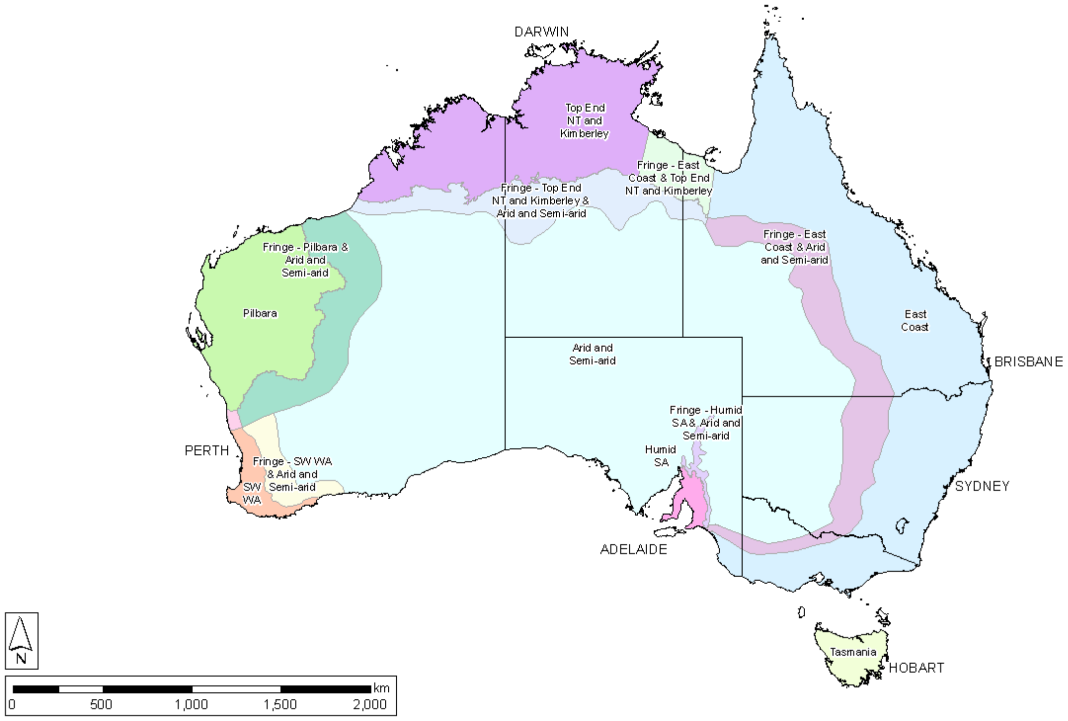

Figure 2-6: Adopted Regions for RFFE Technique in Australia (Rahman et al 2016)

2.4.4. Data Required to Develop the RFFE Model 2015

The reliability, accuracy and success of the RFFE Model 2015 is directly related to the quality and quantity of data available and the capability of the adopted statistical techniques selected to fill gaps in the data across ungauged sites.

The main two types of data that were required for the development and application of the RFFE Model 2015 were:

1) Flood data at gauged sites; and
2) Catchment characteristics relevant to production of floods in both gauged and ungauged catchments.

2.4.4.1. Australian Catchments

A total of 798 catchments have been included in the RFFE Model 2015 from the data-rich areas and also 55 catchments from arid areas (853 total). A summary of the data collected from the selected catchments is given in Table 2-1 with the catchment locations being shown geographically in Figure 2-7 and Figure 2-8.

Table 2-1: Summary of the selected 853 catchments (data-rich and arid areas) (Rahman et al 2015b)

State	No. of Stations	Streamflow record length (years) (range and median)	Catchment size (km2) (range and median)
New South Wales & Australian Capital Territory	176	20-82 (34)	1-1036 (204)
Victoria	186	20-60 (38)	3-997 (209)
South Australia	28	20-63 (37)	0.6-708 (62.6)
Tasmania	51	19-74 (28)	1.3-1900 (158.1)
Queensland	196	20-102 (42)	7-963 (227)
Western Australia	111	20-60 (30)	0.5-1049.8 (49.2)
Northern Territory	50	19-57 (42)	1.4-4325 (178.5)
Sub Total	798	19-102 (37)	0.5-4325 (178.5)
Arid Areas	55	10-46 (27)	0.1-5975 (259)
TOTAL	**853**	**10-102 (36)**	**0.1-5975 (181)**

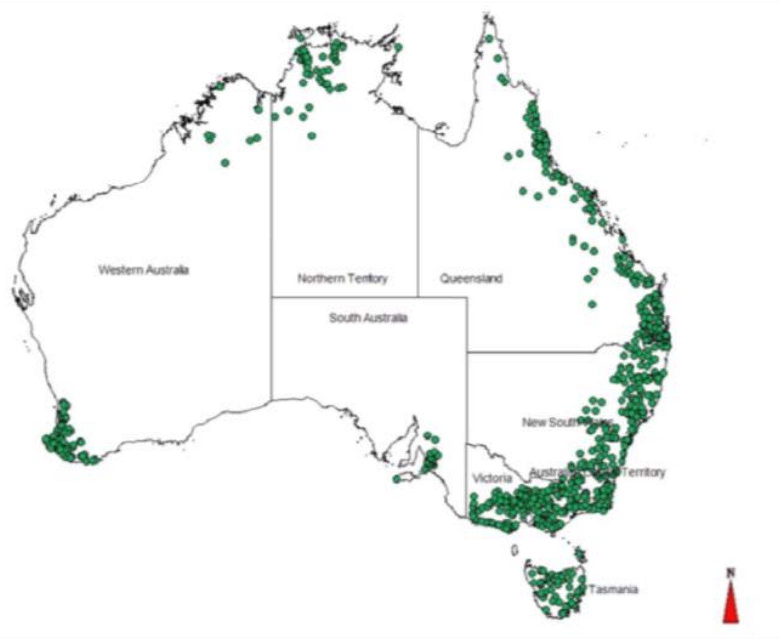

Figure 2-7: Geographical distribution of the selected 798 stations from data-rich areas (Rahman et al 2015b)

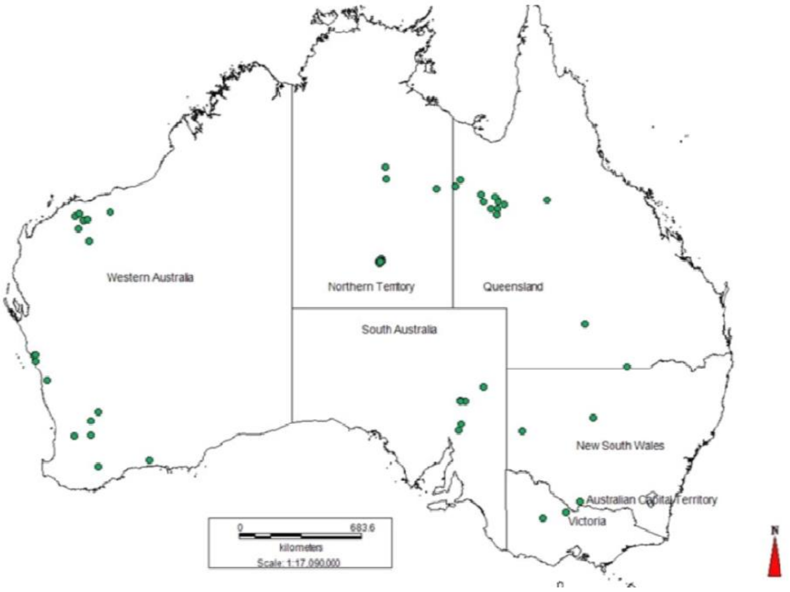

Figure 2-8: Geographical distribution of the selected 55 stations from the arid areas (Rahman et al 2015b)

2.4.4.2. Queensland Catchments

A total of 196 catchments have been included in the RFFE Model from the data-rich areas of Queensland. A summary of the data collected from the selected catchments in Queensland is given in Figure 2-9 and Figure 2-10 with the catchment locations being shown geographically in Figure 2-11.

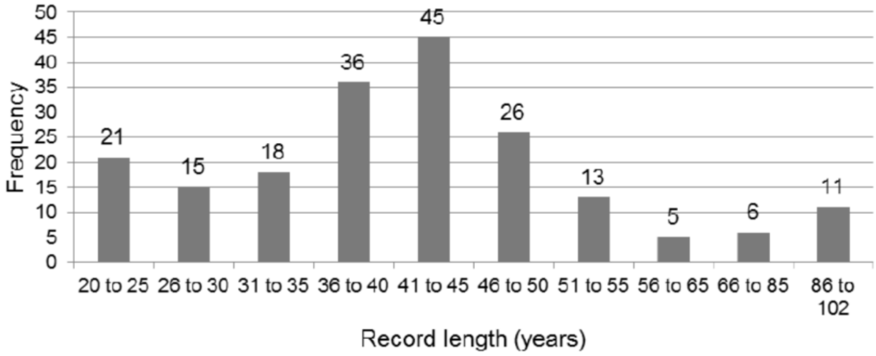

Figure 2-9: Distribution of streamflow record lengths of 196 stations from Queensland (Rahman et al 2015b)

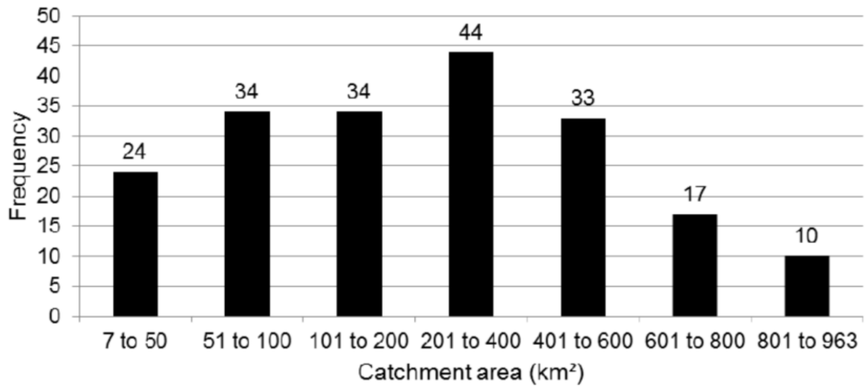

Figure 2-10: Distribution of catchment areas of 196 stations from Queensland (Rahman et al 2015b)

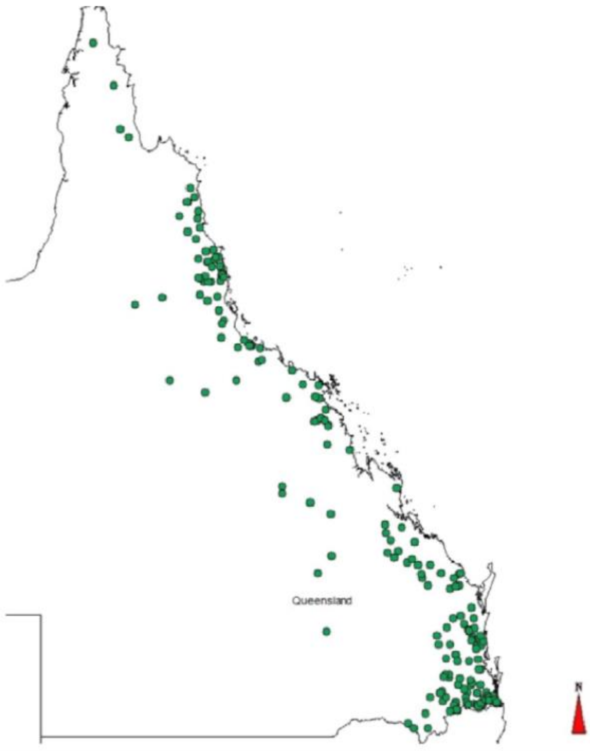

Figure 2-11: Geographical distribution of the selected 196 stations from Queensland (Rahman et al 2015b)

2.4.5. Accuracy of the RFFE Model 2015

The formulation of the RFFE Model 2015 is subject to many uncertainties which are predicted to be substantially greater than that for at-site Flood Frequency Analysis (FFA). The RFFE converts predicted variables to a flood quantile estimate. By using a limited number of predicted variables and the optimisation of transferring these variables spatially, it is believed that the general rainfall-runoff relationship for flood events is capture providing results of acceptable accuracy.

The latest publication on the accuracy of the RFFE Model 2015 as per the ARR Guidelines (Rahman et al. 2016) states that: “Because a RFFE technique typically has limited predictive power, design flood estimates produced by it are likely to have a lower degree of accuracy than those from a well calibrated catchment modelling system. It may be stated that the relative accuracy of regional flood estimates using the RFFE model is likely to be within ±50% of the true value; however, in a limited number of cases the estimation error may exceed the estimation by a factor of two or more. It is unlikely that any RFFE technique would be able to provide flood quantile estimates which are of much greater accuracy given the current availability of streamflow data (in terms of temporal and spatial coverage) and feasibility of the extraction of a greater number of catchment descriptors using simplified methods such as GIS based techniques. Because of the small sample of gauged catchments and limited availability of readily obtainable catchment descriptors, it is not possible to prepare an extremely detailed set of descriptor variables covering all possible conditions, so a sample must be selected that provides a suitable range to represent the critical parameters, but to limit the application of variables that do not contribute significantly to the overall performance of the RFFE technique.”

2.5. The Rational Method

The Rational Method has been published in all three editions of the Australian Rainfall and Runoff (ARR) guidelines (1958, 1977 and 1987) and has been used to calculate design discharges across most of Australia until recent progressions in the Australian RFFE Model. It has been quite common in practice to design small urban drainage networks using peak flows calculated from the Rational Method and the earlier versions of ARR describe the Rational Method as the best known approach to estimating urban stormwater runoff (Coombes 2015).

Although the method has been accepted as reasonable for design discharge estimation, it still has its limitations as published in the Queensland Urban Drainage Manual (QUDM) (Department of Energy and Water Supply 2013). These guidelines were updated whilst this study was being undertaken (IPWEAQ 2017), with similar recommendations about the Rational Method being made.

The Rational Method provides a simple means for the assessment of the peak discharge rate for design storms, but does not provide a reliable basis for the determination of runoff volume, hydrograph shape, or peak discharge rates from historical (real) storms.

Use of the Rational Method is not suitable for the following applications:

- Analysis of historical storms
- Design of detention basins
- Catchments of unusual shape
- Catchments with significant, isolated areas of vastly different hydrologic characteristics, such as a catchment with an upper forested sub-catchment and lower urbanised sub-catchment
- Catchments with significant floodplain storage, detention basins or catchments with wide spread use of on-site detention systems
- Urban catchments with an area greater than 500 hectares
- Catchments with a time of concentration greater than 30 minutes where a high degree of reliability is required in the hydrologic analysis

2.5.1. The Rational Method Formula

The Rational Method equation in its general form as published in the Queensland Urban Drainage Manual (QUDM) (Department of Energy and Water Supply 2013), using the non-standard units of measure for I (m/s) and A (m2) (where Q is in m3/s) is:

$$Q = C \cdot I \cdot A$$

Equation 2-6

For application in the industry, it is quite common to change the key variables to the standard units of measure, Q (m3/s), I (mm/hr) and A (ha), which gives the equation:

$$Q_y = (C_y \cdot {}^{t}I_y \cdot A) / 360$$

Equation 2-7

Where: Q_y = peak flow rate (m^3/s) for annual exceedance probability (AEP) of 1 in ‘y’ years

C_y = coefficient of discharge (dimensionless) for AEP of 1 in ‘y’ years

A = area of catchment (ha)

$^{t}I_{y}$ = average rainfall intensity (mm/h) for a design duration of ‘t’ hours and an AEP of 1 in ‘y’ years

t = the nominal design storm duration as defined by the time of concentration

The value ‘360’ is a conversion factor that is used to suit the units

The total peak flow at any point is not the sum of the calculated sub-area flows contributing at that point, but is dependent on the time of concentration at that point. The actual flow being the product of the sum of the $C \cdot A$ values of the contributing sub-catchments, multiplied by the rainfall intensity appropriate for the time of concentration at that point. The time of concentration is defined as the time for flow to travel from the most remote part of the catchment to the outlet, or the rime taken from the start of rainfall until all of the catchment is simultaneously contributing flow to the outlet (Department of Energy and Water Supply 2013).

2.5.2. The Rational Method in Queensland

The Rational Method formula and explanation previously given were sourced from the Queensland Urban Drainage Manual (QUDM) (Department of Energy and Water Supply 2013) and hence deliver the ideal derivation of the Rational Method for applications in Queensland. The 1987 edition of the Australian Rainfall and Runoff Guide (ARR) published the following statement about the use of the Rational Method and alternate methods for the state of Queensland (Pilgrim 1987):

“No general method based on observed flood data, and which meets the primary criterion is available for Queensland. In the flood estimation procedures used for the design of bridges and culverts by the Main Roads Department, Queensland, account is taken of recorded streamflow in the region, and field and historical data at the site. Also, methods such as unit hydrographs, runoff routing and flood frequency analysis are used in addition to the Rational Method.”

Several methods are available for estimation of design floods on small to medium sized, ungauged rural catchments in Queensland. Some of the more common methods are described briefly below. Without the use of additional data as discussed for the Main Roads Department method, the procedures are basically of an arbitrary nature and are likely to be of lower accuracy than methods developed from observed flood data in other States.

- Main Roads Department Rational Method
  - Time of concentration is first estimated using the modified Friend formula, assuming an estimated peak level of the design flood.
  - Rainfall intensity for duration t_c and the design ARI of 50 years is determined.
  - The runoff coefficient C_{50} is determined.
  - The design discharge for an ARI of 50 years is then calculated by the Rational Method equation.
- Department of Primary Industries Rational Method
  - Time of concentration is estimated from the sum of flow times in overland flow, in contour banks and in channels.
  - Rainfall intensity for the calculated time of concentration and selected design frequency is read from diagrams prepared for each of several districts.
  - Runoff coefficients are estimated from a table depending on the topography, vegetation and soil type.
  - Peak discharge is calculated using the Rational Method equation.

2.6. Hydrologic Modelling

Hydrologic investigation for flood estimation is commonly used for large and/or rural catchments through the application runoff routing software. This technique involves determining the rainfall-excess and routing it through a model of the catchment storage by flood routing procedure.

2.6.1. Overview

Hydrologic modelling is primarily undertaken using runoff routing models. This software estimates the design flood hydrograph by sub-dividing the catchment into a number of sub-catchments in which the runoff generation and flow routing is computed. By analysing flood hydrology through runoff routing techniques, the areal distribution of rainfall, catchment topography and land uses as well as stream characteristics can all be accounted for and modified with proposed development.

Runoff routing models were developed primarily to overcome problems such as the lumping of catchment and rainfall characteristics and the system linear theory, both associated with historical unit-hydrograph calculation methods. Research studies found that catchment flood response is typically non-linear and runoff routing methods help in modelling allowances for the nonlinearity in catchment response (Main Roads WA 2014). Other studies have shown that current runoff routing models used within the industry may require further development to help account for the nonlinearity responses within rivers and extensive floodplains.

There are currently a number of runoff routing software programs that are used within the industry, each requiring different input parameters and data. These include but are not limited to: RORB, RAFTS, WBNM, URBS and ILSAX. The use of a specific modelling program is dependent on the local regulatory authority's requirements and recommendations. For the purposes of this academic project only RORB and WBNM have been considered as runoff routing software options due to limitations and software access permissions.

2.6.1.1. The Flood Hydrograph Estimation Process

The recent revision of the ARR Guidelines (2016) published the following process of how to develop and apply an event-based flood hydrograph estimation model (Ball & Weinmann 2016b):

i. Definition of the problem and the model requirements;
ii. Assessment of data requirements and data availability, data collation and checking;
iii. Study of catchment data and flood information to develop an understanding of the catchment behaviour during floods and to identify important features that need to be represented in the model;
iv. Conceptualised representation of the runoff generation phase (loss model and baseflow model);
v. Conceptualised representation of the flood hydrograph formation phase (the routing elements of the catchment);

vi. Determination of model parameters by calibration to observed events, from experience values in regions with similar flood producing characteristics or from links with measured catchment characteristics;

vii. Validation of the calibrated model to ensure that it is fit for the intended purpose;

viii. Application of the model with design rainfalls, design losses and design baseflows to estimate design flood hydrographs;

ix. Interpretation and presentation of model results, including determination of uncertainty;

x. The modelled design flood hydrographs will generally form the inputs to a hydrodynamic model of the study area.

2.6.2. Basis of Runoff Routing

Runoff routing aims to produce a design flood hydrograph at a given location within a catchment or stream. The hydrograph calculated at this location is determined from the runoff inputs generated by a variety of process in upstream sub-catchments. The primary effect on the downstream hydrograph is governed by the various forms of flood storage within the catchment and losses along the flow route. The ARR2016 Guidelines determined the main elements of a catchment that result in storage or losses to be (Ball et al. 2016c):

- Catchment surfaces (overland flow segments);
- Stream channels;
- Stream banks;
- Floodplains; and
- Drainage channels or pipes.

In nature these forms of storage are usually distributed throughout the catchment, however in runoff routing the different storages can be modelled together as a conceptual storage element rather than separately.

In addition to the distributed forms of storage listed above, detention basins, reservoirs and lakes may also provide storage. These forms of storage can be represented within runoff routing by using a more direct relationship between the inflow and outflow rates.

These forms of storage have two separate effects on the design flood hydrograph. These effects are shown in Figure 2-12 and can be described as (Ball et al. 2016c):

i. Translation of the hydrograph peak and other ordinates in time or, expressed differently, delaying the arrival of the hydrograph peak at a downstream location; and
ii. Attenuation or flattening of the hydrograph as it moves along the stream network; this results in a reduction of the peak flow but also in diffusion (spreading out) of the hydrograph, thus extending its duration.

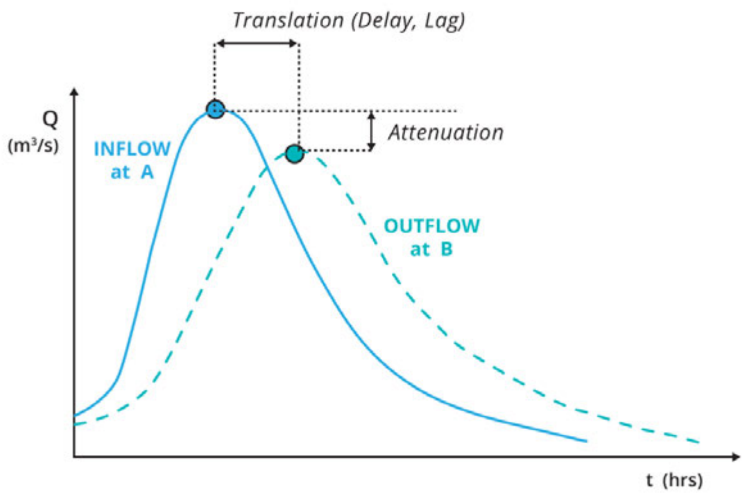

Figure 2-12: Effects of Storage on Transforming Inflow Hydrograph (Ball et al 2016)

The effects of storage can be modelled through the formulation of the continuity equation for a specific catchment element and over a time interval Δt (Ball et al. 2016c):

$$I_v = O_v + \Delta S$$

Equation 2-8

Where: I_v = volume of inflow into the catchment element

O_v = volume of outflow from the element

ΔS = change in storage during the time interval

The inflow volume (Iv) may represent runoff and baseflow inputs or outflow from an upstream element. While ΔS is positive, the inflow volume to the element is greater than the outflow volume and therefore the volume of water in storage within the element will increase over time. Conversely, when ΔS is negative, the outflow volume is greater than the inflow volume and the volume of water in storage in the element will decrease over time (Ball et al. 2016c).

As the principle of mass conservation must be accounted for, the total volumes of inflow and outflow from the storage element must be equal. When losses occur within the storage such as dry banks accounting for infiltration losses, the principle of mass conservation remains with the volume of inflow being equal to the volume of the outflow plus the volume of the transmission loss.

Equation 2-8 above applies to forms of 'detention storage' or 'temporary storage' within the catchment, in these storages all water is released within the flood event. However, there may also be 'retention storage' elements where runoff is stored more permanently and released after the flood event such as a reservoir or dam.

2.6.2.1. Hydrograph Translation (Lag)

"The simplest method for routing a hydrograph through a reach is to simply translate all ordinates by a fixed travel time or lag. This method of routing produces pure translation without any attenuation of the hydrograph peak. It is useful for flood routing in systems with little storage (eg. piped drainage systems) or in situations where the timing of the hydrograph peak is of principal interest (eg. flood warning systems)." (Ball et al. 2016c).

The travel time through a pipe segment can be determined by the flow velocity through the pipe. However, the travel time (T) of a flood hydrograph routing through a natural stream or channel of reach length (Δx) is directly related to the kinematic wave speed (ck) through the equation:

$$T = \frac{\Delta x}{c_k}$$

Equation 2-9

Therefore the travel time or lag is directly proportional to the length of the reach.

2.6.2.2. Storage Routing (Attenuation)

“Storage routing methods have been developed as a convenient form of hydrologic routing, to track the movement of a flood wave on its way through a catchment system and to assess the effects of storage on the transformation of an inflow hydrograph to an outflow hydrograph. Storage routing is a lumped approach – it considers only the inputs (inflows) and outputs (outflows) of the system without considering what is happening within the system. Different applications of storage routing principles focus on different types of systems with different forms of storage.” (Ball et al. 2016c).

The continuity equation reflects the Conservation of Mass principle of which the storage routing methods are based:

$$I - O = \frac{dS}{st}$$

Equation 2-10

Where I and O respectively are the average rates of inflow and outflow and dS is the change in storage during the time interval dt. Multiplication of Equation 2-10 by the time interval dt yields the continuity equation expressed in terms of volumes (Ball et al. 2016c):

$$INFLOW - OUTFLOW = CHANGE\ OF\ STORAGE$$

Equation 2-11

As only the change in storage is considered, rather than the total storage volume; this means that the datum used for the storage volume determination is irrelevant as it does not affect the routing calculations.

2.6.2.3. Non-Linear Storage Routing

The hydrograph translation methods described in Section 2.6.2.1 are made on the assumption that storage (S) is directly related to the discharge (Q) in a linear form. As demonstrated from hydraulic analysis of storage elements within catchments, the storage to discharge relationship is typically non-linear. In these cases, the relationship between the storage and discharge of the element can be approximated through a power function similar to:

$$S = kQ^m$$

Equation 2-12

Where k is a dimensionless coefficient and m is a dimensionless constant.

Depending on the storage and discharge characteristics of the element, the exponent m can be smaller or greater than the value of 1.0 (which applies to the linear form of the S-Q relationship). The formulation in Equation 2-12 implies also a lag time K that varies with discharge (Ball et al. 2016c):

$$K = \frac{S}{Q} = kQ^{m-1}$$

Equation 2-13

Some non-linear storage element routing methods may require a more complex and iterative numerical solution such as the Regula Falsi (False Position) method or the Newton-Raphson method.

2.6.3. Losses

Within rainfall routing modelling programs, excess runoff hyetographs from each sub-catchment are calculated through the application of a loss model to the sub-catchment area. The loss model consists of a combination of pervious area losses and impervious area loss modelling. The most common form of loss modelling for pervious areas is the initial/continuing loss model (refer Figure 2-13) whilst the most common loss model for impervious areas is simply a runoff coefficient model (usually with a runoff coefficient of 0.9 (Main Roads WA 2014)). The land use of each sub-catchment area usually defines the proportion of pervious and impervious area.

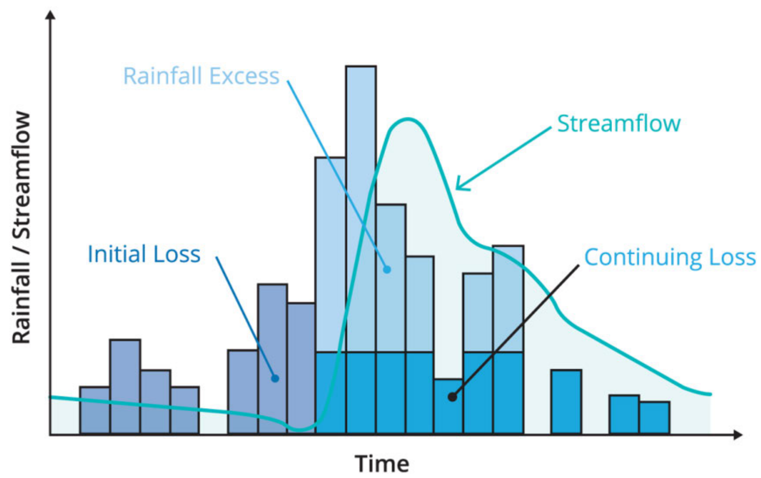

Figure 2-13: Initial Loss / Continuing Loss Model (Hill & Thompson 2016)

The excess runoff is then routed from the centroid of the subject sub-catchment, along a reach to the next downstream sub-catchment node where the hydrograph is combined with:

i. runoff hydrographs from other tributaries and/or
ii. rainfall excess hyetograph from the sub-catchment of the downstream node reach.

This process then continues downstream to the model outlet.

2.6.3.1. Regional Loss Information

The latest revision of the ARR Guidelines (2016) give recommendations for the median initial and continuing loss based on regionalisation.

"The recommended loss values are shown in Figure 2-14 and Figure 2-15 were derived using the prediction equations in the preceding section. For arid areas with mean annual rainfalls less than 350 mm (shown in grey in both figures) there are no recommendations for design loss information because the prediction equations were developed using data from wetter catchments. Recommended loss values can be accessed via the ARR Data Hub." (Hill & Thomson 2016)

It should be noted that the recommended values were derived based upon only 35 catchments and the standard error of the estimates range between 20% and 50%.

Because of the limited number of catchments available, the prediction equations are based upon one or two independent variables. However, it is anticipated that a wide range of characteristics combine to influence the loss values for a particular catchment and therefore judgement is recommended when selecting suitable values for use in design. For example for catchments with very dense vegetation, it would be expected that the loss values would be higher. Similarly, steep catchments with little vegetation would be expected to have lower loss values. Any such adjustment from the regional values should be done giving consideration to the range of loss values obtained in (Hill et al. 2014) and other studies and the implications on the design flood estimates.

"Lastly, it is important to note that the recommended loss values in the figures relate to the median for a particular catchment. It is expected that the loss for any particular event could lie well outside of this range. For many catchments, the storm initial loss for any particular event could range from nearly zero, if the storm occurs on a wet catchment, to more than 100 mm if there is little antecedent rainfall." (Hill & Thomson 2016).

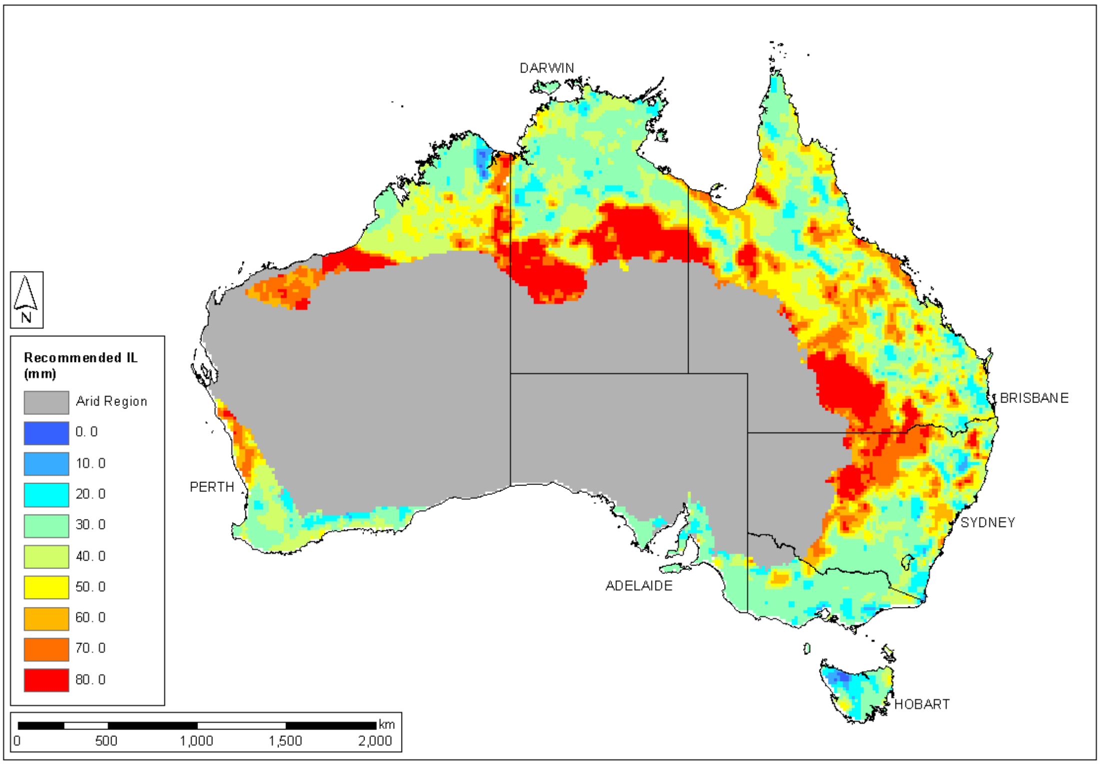

Figure 2-14: Recommended Median Initial Loss (Hill & Thomson 2016)

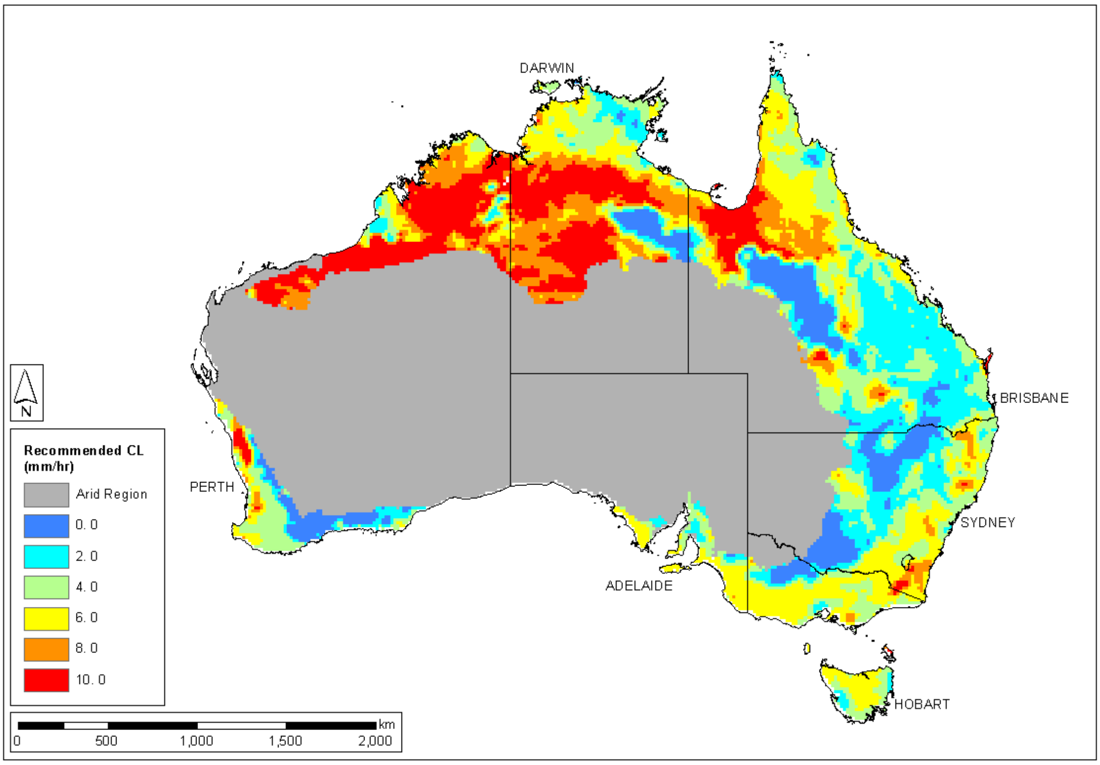

Figure 2-15: Recommended Median Continuing Loss (Hill & Thomson 2016)

2.6.4. Pre-Burst Rainfall

The 1987 revision of the Australian Rainfall & Runoff Guidelines describes a traditional design storm as the complete rainfall event with varying parameters such as initial and continuing losses calibrated to the event. Further investigation into this (Loveridge et al. 2015) found that this can lead to large biases in design flood estimates.

From this, a need to incorporate a pre-burst rainfall depth before the design rainfall event was established as part of the ARR 2016 revision. This was determined to best be achieved through a regionalisation of pre-burst depths that vary across Australia influenced by critical burst severity, critical burst duration and the geographic location with the later of the two being most sensitive to the pre-burst rainfall depth (Loveridge et al. 2015). It has been reported in the latest ARR 2016 guidelines (Babister et al. 2016) that in many parts of Australia the pre-burst rainfall represents a small amount of the event and generally does not contribute to the runoff response (refer Figure 2-16), whereas in some parts pre-burst rainfall can represent a significant part of the rainfall event and runoff response.

Pre-burst rainfall is to be treated in a way dependent on how the estimated magnitude of the pre-burst compares to the catchment losses and whether the depth will have an effect on hydrograph volumes. Where the pre-burst rainfall depth is found to not influence the hydrograph volume it is best to be represented as a reduction of the initial storm losses. Whereas where the pre-burst is found to be influential the initial rainfall depth can be applied to calculations with a typical pre-burst pattern. Figure 2-17 and Figure 2-18 show the median ratio of the pre-burst to burst and the depth of pre-burst in mm for the 6hr duration and probabilities respectively across Australia.

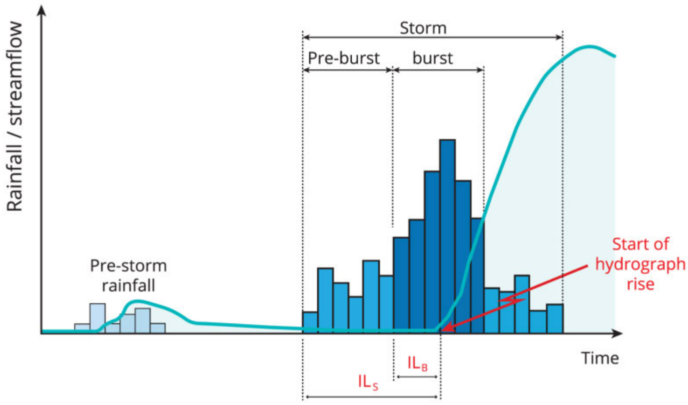

Figure 2-16: Distinction between Storm and Burst Initial Loss (Babister et al. 2016)

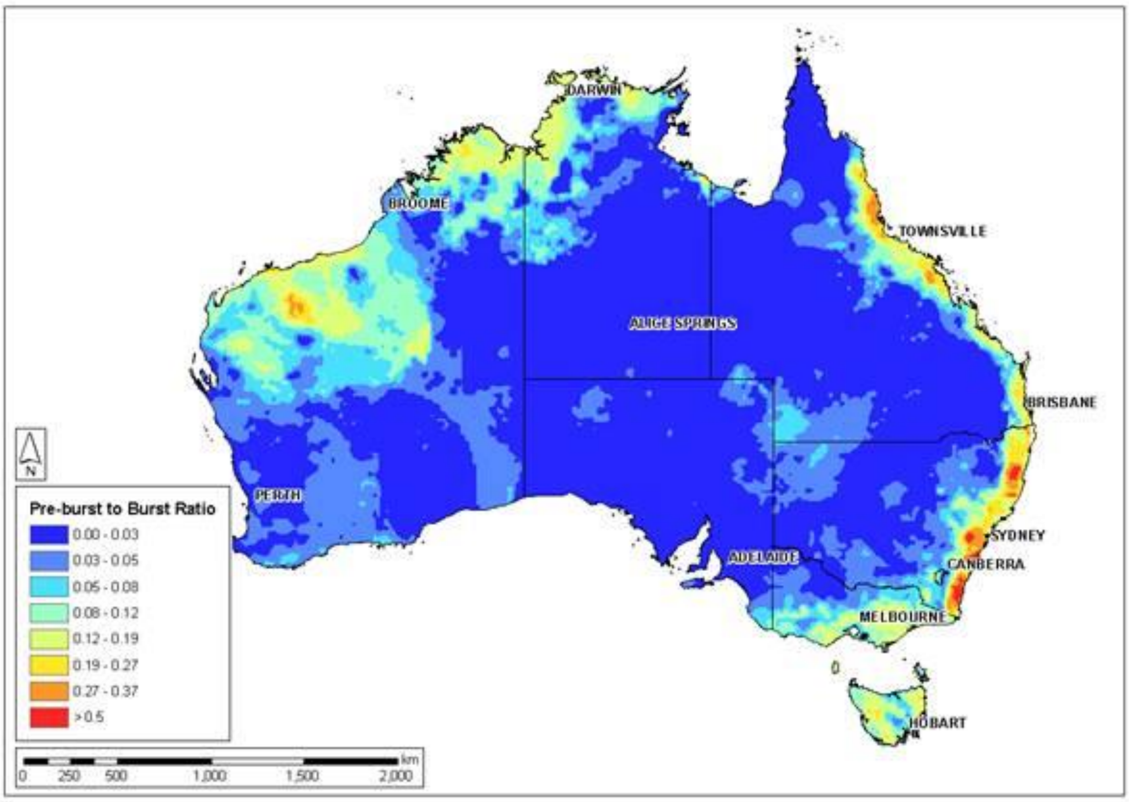

Figure 2-17: Pre-Burst to Burst Ratio (Babister et al 2016)

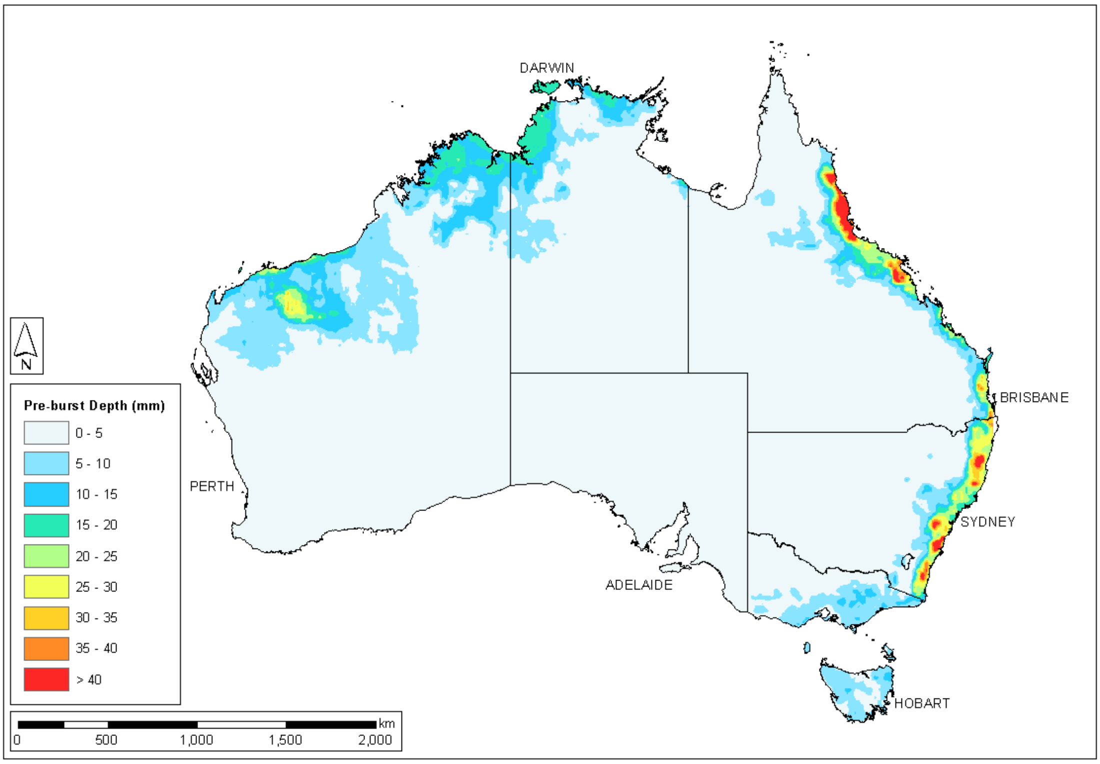

Figure 2-18: Pre-burst Rainfall Depths (Babister et al 2016)

2.6.5. Flow Routing

Once rain has fallen, excess rainfall not converted to losses is deemed as runoff and is transferred through the catchment via flow paths. This allows for the runoff hydrograph to be calculated for each sub-catchment area and transferred downstream eventually to the outlet of the overall catchment.

Figure 2-19 shows the effects of hydrograph translation downstream throughout the catchment whilst Figure 2-20 shows a typical output from RORB a runoff routing program.

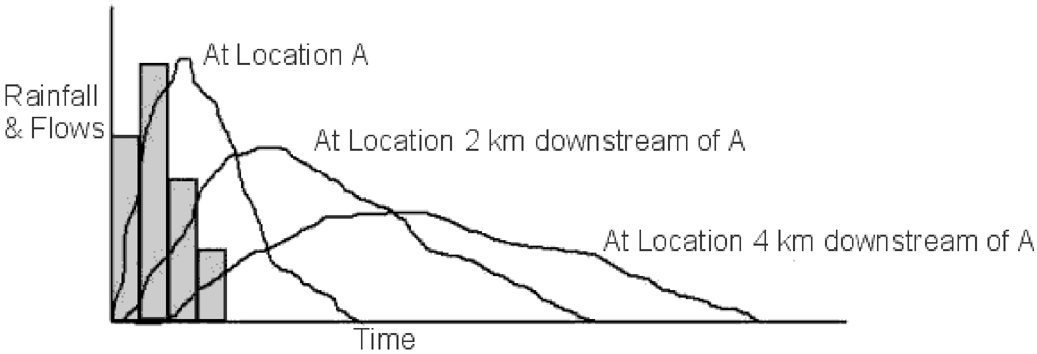

Figure 2-19: Illustration of Storage Effects on Flood Hydrographs (Main Roads WA 2014)

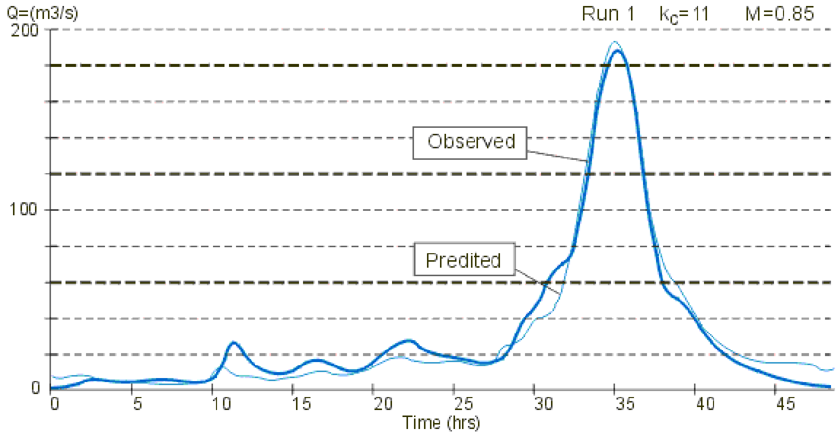

Figure 2-20: Typical Output from RORB Model (Main Roads WA 2014)

2.6.6. Modelling Approaches

As per the model parameters outlined in the previous sections, the typical modelling approach applied within the industry is a node-link type runoff routing model. Figure 2-21 shows the graphical representation of a runoff routing model:

- On the left hand side, the main catchment has been divided into sub-catchments with the flow network represented by a simplified stream and tributary system.
- The figure in the centre shows how the node and links are spatially placed for a RORB model.
- The figure on the right shows the typical node and link configuration for a WBNM model.

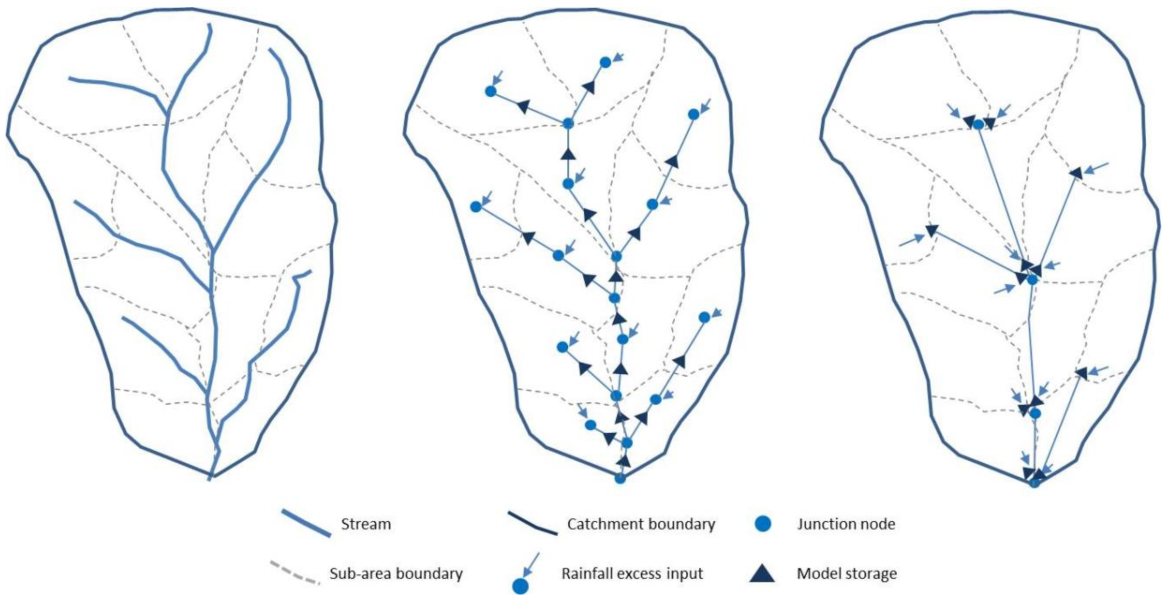

Figure 2-21: Node-Link Type Representation of a Catchment in Runoff Routing Models: Map View and Schematic Representation of Node-Link Network in RORB and WBNM (Ball & Weinmann 2016b)

2.6.6.1. WBNM

The Watershed Bounded Network Model (WBNM) was developed by Boyd, Pilgrim and Cordery (1979) and revised by Boyd, Bates, Pilgrim and Cordery (1987).

The runoff routing platform was included in the 1987 edition of the ARR Guidelines. The model calculates the flood hydrograph resulting from storm rainfall using a runoff routing approach where the catchment is divided into sub catchments using the stream network. Each catchment is allocated a lag time depending on its size, based on studies of the nonlinear variation of lag time on real catchments (Boyd et al. 1996). The model has been developed into a comprehensive computer program that is easy to use and interacts with other software such as GIS tools and MS Excel.

2.6.6.2. RORB

The RORB runoff routing software was developed within the Monash University Water Group of the Department of Civil Engineering by Eric Laurenson and Russell Mein.

RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce the hydrograph. It can also be used to design retarding basins and to route floods through channel networks. The program requires a data file to describe the particular features of the stream network being modelled and is run interactively. It can be used both for the calculation of design hydrographs and for model calibration by fitting to rainfall and runoff data of recorded events (Monash University 2016).

2.7. Hydrodynamic Modelling

Hydrodynamic modelling, commonly in the form of two-dimensional (2D) hydrodynamic computer models are becoming more commonly utilised within the industry as the standard approach for estimating design or historic flooding behaviour especially with work undertaken for Mackay Regional Council (MRC)n particular, the use of TUFLOW software. This is mainly due to the improvement in technology and computing power over time. In a 2D model the flow solution is based on the numerical solution of the full 2D depth-averaged equations of motion computed at each active water grid point (Babister & Barton 2012).

2.7.1. Overview

Numerical hydrodynamic modelling methods such as 2D TUFLOW modelling, aim to provide a realistic representation of flow behaviour in a particular environment. Hydrodynamic modelling allows both the replication of historical flood events and also the ability to predict flooding patterns under different flow conditions or changes in the physical environment such as development. This is commonly used for impact assessments of new or upgraded infrastructure.

Before the development of computing power and modelling software, this type of estimation could only be carried out by creating a scaled hydrodynamic model of the physical environment. This taken a lot of time and involved a lot of costs mainly relying on research institutions. Due to these implications of physical modelling it was usually only undertaken for major infrastructure projects, however now computer modelling is undertaken for both major and minor works.

The rule of thumb is that the more the more realistic the modelling approach, the greater the probability of achieving a successful outcome (Babister & Barton 2012). This means that a very detailed hydrodynamic model that has been developed to represent complex flow patterns has a better probability of achieving a successful outcome compared to that of a simplified calculation based approach. However, just by using sophisticated software will not guarantee an accurate and reliable solution. The ARR revision project investigating the application of 2D hydrodynamic modelling (Babister & Barton 2012) reported that skill of the modeller adapting a generic modelling system to a specific application, and the quality of the data used as model input can be equally important in determining model success.

2.7.2. Development of a Hydrodynamic Model

2D hydrodynamic modelling aims to provide a discretised representation of the physical environment being modelled that essentially mimics the flow behaviour that will be witnessed. This requires a series of steps required in the development of the model, these steps have been reported in the ARR Revision Project 15 (Babister & Barton 2012) and are listed below as well as shown schematically in Figure 2-22.

i. Review and define the physical system (the river and/or floodplain system to be modelled).

ii. Select an appropriate mathematical model (the set of equations used to describe the physical system).

iii. Select a generic numerical model (the modelling software used to solve the equations).

iv. Develop the site-specific numerical model (the generic modelling software combined with site-specific inputs, including topographic data, bed-friction coefficients, flow boundary conditions and other parameters such as pipe or culvert information as appropriate).

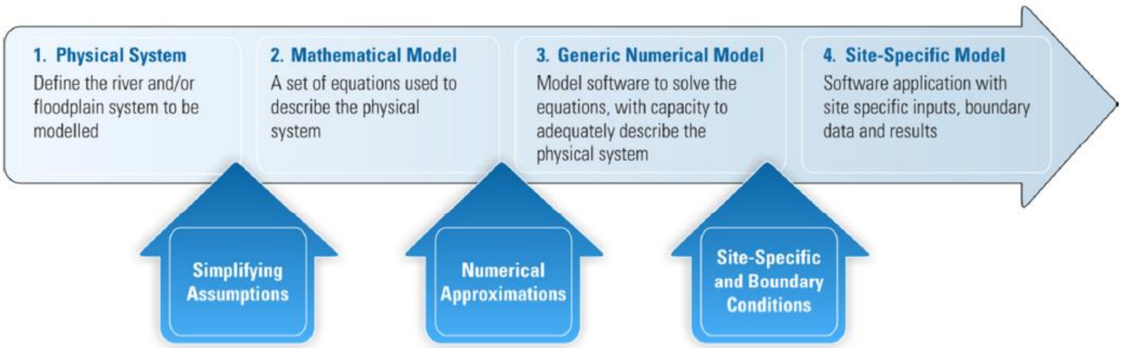

Figure 2-22: Stages in Numerical Hydrodynamic Model Conceptualisation and Development (Babister & Barton 2012)

At each step in this process the modeller will need to be able to apply assumptions, approximations and/or simplifications to accurately replicate the site conditions. These assumptions are unique to the project or environment and require knowledge and experience in the hydrodynamic modelling domain.

2.7.3. Basis of 2D Modelling

The processes undermining hydrodynamic modelling are very complex. The ARR revision project into 2D modelling (Babister & Barton 2012) reported that the significant transitions in open channel flow that can occur will determine the patterns of flood behaviour. Further, the locations of the important rapidly varied flow features can shift during a flood, further complicating the flood modelling process.

Historic uses of scaled physical flood models would deal directly with the behaviours and transactions in open channel flow. However the development of modern computational models now use various forms of the fundamental hydraulic governing equations of fluid flow, each subject to their own previously determined assumptions and parameters. It is very important that throughout the development and modelling process of a 2D hydrodynamic model that the fundamental equations and their relevant assumptions associated with their application be understood by the modeller.

2.7.3.1. 2D Equations of Motion

Fully 2D hydrodynamic models are based on the numerical solution of depth-averaged equations describing the conservation of mass and momentum in two horizontal dimensions x and y (Babister & Barton 2012).

The graphical and computational form of the 2D equations of motion is shown in Figure 2-23. The definition of variables is also provided below.

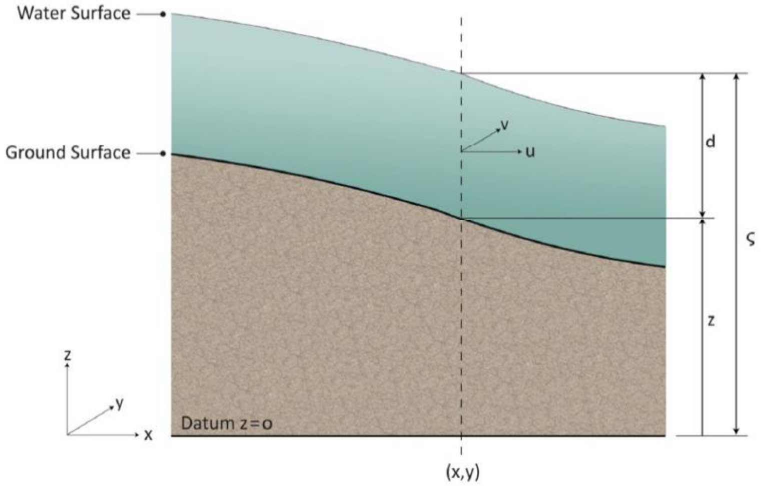

Figure 2-23: Definition of Equation Variables (Babister & Barton 2012)

Where: ζ = water surface elevation relative to a fixed datum (m)

u = depth averaged velocity in the x-direction (m/s)

v = depth averaged velocity in the y-direction (m/s)

These are described as a function of the three main independent variables:

x = horizontal distance in the x-direction (m)

y = horizontal distance in the y-direction (m)

t = time (s)

Additionally, the time varying water depth at any location d(x,y), can be expressed as:

$$d = \zeta - z$$

Where: z = bed surface elevation relative to a fixed datum (m)

2.7.3.2. Mass Equation

For the application of a hydrodynamic model, runoff is considered to be incompressible. From this, the volume of water is used to represent the mass of the water. As per the variables defined in Section 0 the depth averaged conservation of mass (and therefore volume) equation in the two horizontal directions can be defined as:

$$\frac{\partial \zeta}{\partial t}+\frac{\partial (d.u)}{\partial x}+\frac{\partial (d.v)}{\partial y}=0$$

Equation 2-14

Where: $\frac{\partial \zeta}{\partial t}$ = the rate of increase (or decrease) in water level, which for a fixed cell size is representative of the rate of change of volume of water contained in the cell

$\frac{\partial (d.u)}{\partial x}+\frac{\partial (d.v)}{\partial y}$ = the spatial variation in inflow (or outflow) across the cell in the x and y directions.

To summarise the above equation, increases or decreases in volume must be balanced by a net inflow or outflow of water.

2.7.3.3. Momentum Equation

Similar to the conservation of mass within a hydrodynamic model, momentum is also conserved in both the x and y directions and can be expressed through the following formula for the respective directions:

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial \zeta}{\partial x}=0$$
$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial y}+u\frac{\partial v}{\partial x}+g\frac{\partial \zeta}{\partial y}=0$$

Equation 2-15

Where: g = the acceleration due to gravity (m/s^2)

$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}$ = the partial differential form of the flow acceleration du/dt in the x-direction (similar for y)

$g\frac{\partial \zeta}{\partial x}$ = the hydrostatic pressure gradient in the x-direction (similar for y)

Equation 2-15 shows effectively an impulse/momentum equation, where the flow acceleration or rate of increase (or decrease) in momentum is balanced by the impulse of the hydrostatic pressure gradient (Babister & Barton 2012).

2.7.3.4. Assumptions

The derivation of the fundamental equations listed above have been based upon the following assumptions reported in the ARR revision project into 2D modelling (Babister & Barton 2012):

- The flow is incompressible
- The pressure is hydrostatic (i.e. vertical accelerations can be neglected and the local pressure is dependent only on the local depth).
- The flow can be described by continuous (differentiable) functions of ζ, u and v (that is, it does not include step changes in ζ, u and v).
- The flow is two-dimensional (that is, the effects of vertical variations in the flow velocity can be neglected).
- The flow is nearly horizontal (that is, the average channel bed slope is small).
- The effects of bed friction can be included through resistance laws (e.g., Manning's equation) that have been derived for steady flow conditions.

2.7.3.5. Model Application of Equations

In summarising the equations and assumptions listed above, the application of these in the TUFLOW hydrodynamic modelling platform is similar to that shown in Equation 2-16.

The more complex equation includes the attrition of Coriolis Force, bed resistance, viscosity, atmospheric pressure and external forces. For the purposes of this study these more advanced variables have not been investigated.

$$\underbrace{\left[\frac{\partial v}{\partial t}\right]}_{\text{How Velocity changes over time}}+\underbrace{\left[u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}\right]}_{\text{Inertia}}+\underbrace{\left[c_f u\right]}_{\text{Coriolis Force}}+\underbrace{\left[g\frac{\partial \zeta}{\partial y}\right]}_{\text{Gravity}}+\underbrace{\left[gv\left(\frac{n^2}{H^{4/3}}+\frac{f_l}{2g\Delta y}\right)\sqrt{u^2+v^2}\right]}_{\text{Bed Resistance}}-\underbrace{\left[\mu\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)\right]}_{\text{Viscosity (Turbulence)}}+\underbrace{\left[\frac{1}{\rho}\frac{\partial \rho}{\partial y}\right]}_{\text{Atmospheric Pressure}}=\underbrace{\left[F_y\right]}_{\text{External Forces (Wind, Waves etc.)}}$$

Equation 2-16

2.7.3.6. Solution of the Equations

The TUFLOW hydrodynamic modelling platform solves the equations above on a fixed square grid finite difference method. The ARR Revision Project 15 reported the following summary on finite difference solutions (Babister & Barton 2012):

“In the finite difference technique each discretised volume is treated as a unique control volume (cell) represented by volume-averaged values of the conserved variables. The finite difference methods are most intuitively thought of as control-volume methods, due to their basis in the conservative-integral form of the shallow water equations. The rate of change of conserved variables is derived by integrating the cell-interface fluxes. Various implicit and explicit integration methods can be used to advance the solution in time. Being based on the conservative integral form of the shallow water wave equations, finite difference schemes are generally better able to handle shocks (hydraulic jumps and bores) and may therefore potentially perform better in mixed regime flow situations.”

Within TUFLOW, the equations are solved on a fixed computational grid similar to that shown in Figure 2-24. In this grid, the water level ζ, the water depth d and the bed level z is specified at the corner intersections of each x and y grid line. The velocities in the x-direction (u) and in the y-direction (v) are specified at the mid-points along each grid line. This allows the locations of the variables to differ in space and time given the discrete number of grid sizes jΔx and kΔy along with the time steps nΔt.

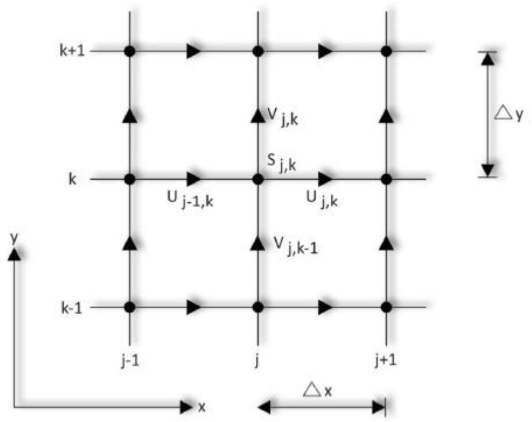

Figure 2-24: Example of a Computational Grid (Babister & Barton 2012)

2.7.4. Data Requirements

2D hydrodynamic modelling software, in particular the TUFLOW software platform uses quite complex and sophisticated methods for data interpolation and extrapolation, however the accuracy and reliability of the results from the modelling is directly dependent on the data used for the modelling exercise.

For example, if hydrodynamic modelling was to be undertaken to assess upstream afflux to an accuracy of ±0.1m, then the input data used for the investigation should be at least of the same accuracy but preferably better than the required output. It is advisable that the best data available at the time be used in all instances and the accuracy of the data sets is assessed prior to commencement of the work.

The ARR revision project into 2D modelling (Babister & Barton 2012) reported that the required data for hydrodynamic modelling can be classified into the following purposes:

- Model development
- Model calibration/verification
- Model application/presentation

The model development phase involves collating the data that will depict the flow patterns around the site of interest. A thorough understanding of the flooding behaviour in the study area can help influence selection and assessment of data for accuracy in this phase. Some of the datasets that may be sourced include but are not limited to:

- Local and floodplain topographic features
- Drainage infrastructure data

- Land use and hydraulic roughness information
- Downstream controls on flood behaviour

Following the development of the model the validity of its performance needs to be assessed. This phase typically involves benchmarking model results with observed flood results or information. Input data in this phase could include:

- Observed/estimated flow rate and volume
- Historical flood levels and extents
- Anecdotal information from local stakeholders describing flooding

2.7.5. Model Schematisation

In 2D hydrodynamic modelling, the schematisation of the model is the process of creating the conceptual representation of the physical system. This is the process where the existing conditions are categorised into a series of discrete elements. The ARR investigation (Babister & Barton 2012) into 2D model schematisation stated that: "The physical system being modelled may be schematised in many different ways depending on the selection of model elements within the modelling tool and the choices made by the modeller. The accuracy, reliability and usefulness of the model are significantly influenced by the skill of the modeller in completing this process."

The primary considerations of the existing catchment conditions that are necessary in the schematisation process are reported in the ARR report (Babister & Barton 2012) and are as follows:

- Type of model to apply;
- Model extent;
- Mesh or grid resolutions and orientation;
- Simulation timesteps;
- Specification of specific hydraulic features and controls; and
- Types, location and design of boundary conditions

2.7.6. Direct Rainfall Modelling

The TUFLOW modelling platform, like many other emerging software packages, offers the option for direct rainfall modelling or Rain On Grid (ROG) modelling. Direct rainfall modelling is where design rainfall depths can be applied directly onto the Digital Elevation Model (DEM) without the need for a separate hydrological model to determine peak inflows to the subject area.

This technique is still relatively new, having only entered into the mainstream commercial 2D modelling software packages over the last 10 years however it is increasing in popularity due to time savings and simplicity of the modelling.

Only a limited amount of research has been undertaken into this area with most of the existing studies comparing the outputs from the hydrological models with the direct rainfall method. There has been little research into how the results from direct rainfall modelling compare with that of gauged catchments. This is mainly due to the fact that there are a limited number of gauged catchments in Australia.

The accuracy of direct rainfall modelling has been deemed as ‘difficult to determine’ when compared to traditional hydrologic routing models (Babister & Barton 2012). Research previously conducted into the comparison of the two modelling methods found that there is as much difference in discharge time series between two different traditional hydrological models, as there is between direct rainfall and traditional hydrological models (Rehman et al. 2007).

2.7.7. Model Calibration and Sensitivity

Calibration of hydrodynamic models is a critical stage of the models development to ensure that the simulation is representative of the catchment conditions and capable of reproducing flood behaviour within acceptable parameter bounds. This process is primarily undertaken by comparing model results and outputs to historical floods. In the case where historical flooding information is not available the model must still be calibrated to some other source of investigation, usually though a desktop analysis.

The review into 2D hydrodynamic modelling undertaken by the ARR revision team reported that (Babister & Barton 2012): “Regardless of hydrodynamic model type or complexity, the calibration process is critical to ensure the model is capable of adequately representing the physical system and, in doing so, producing reliable results. While 2D hydrodynamic models provide a superior numerical solution, accurate results are not guaranteed. Calibration is just as important for 2D model applications as it is for simpler models."

2.8. Previous Studies

Key previous hydraulic investigations that have been undertaken both for Mackay Regional Council (MRC) and the Queensland Reconstruction Authority (QRA) have been sourced and reviewed as part of this investigation.

A summary of the studies is provided in the subsequent sections.

2.8.1. Pioneer River Flood Study (WRM 2011)

The purpose of this study was to develop hydrologic and hydrodynamic modelling tools to determine the flood risk throughout the study area from the Pioneer River (from Mirani to Mackay CBD) to assist MRC in land use planning and development assessment. Design flood discharges, flood levels and flood extents have been determined for a range of events from the 5 year to the 500 year Average Recurrence Interval (ARI) events. The study also assesses the impact of climate change based on Queensland Government (2010) recommendations (WRM Water & Environment 2011).

2.8.2. Bakers Creek/Walkerston Flood Study (WRM 2013)

The purpose of this study was to develop hydrologic and hydrodynamic modelling tools to determine the flood risk along Bakers Creek through the township of Walkerston that will assist MRC in land use planning and development assessment. Design flood discharges, flood levels and flood extents were determined for the 5 year, 50 year, 100 year, 200 year and 500 year Average Recurrence Interval (ARI) events, and the Probable Maximum Flood (PMF). The potential impacts of climate change on flooding along Bakers Creek based on Queensland Government (2010) recommendations have also been assessed (WRM Water & Environment 2013a).

2.8.3. Finch Hatton Flood Hazard Mapping Study (WRM 2013)

The township of Finch Hatton is located on the southern floodplain of Cattle Creek. Cattle Creek is a major tributary of the Pioneer River. The modelling found that the lower areas of Finch Hatton including several houses are susceptible to inundation from Cattle Creek flooding for the 2% AEP flood event. The majority of the impacted properties are located in a significant hazard zone for this event. The flood protection levee is overtopped by over 0.7m to the south of Mackay Eungella Road for this event. Larger flood events overtop the levee and inundate properties to a greater depth (WRM Water & Environment 2013b).

CHAPTER 3

3. METHODOLOGY

This research project seeks to analyse techniques of peak flow estimation specific to catchments within the Mackay Region. This will result in a comparative benchmark for the methods currently utilised within the industry and outline the strengths and weaknesses of the different approaches directly related to the accuracy of results.

The study will use recorded data from official weather stations within the Mackay Region, this data has been provided by Mackay Regional Council under a data use agreement outlining that the inputs/results should not be used or interpreted for any other purposes.

3.1. Catchment Selection and Data Sources

As this study is focused around the Mackay Region, the Pioneer and Plane drainage basins were central to the investigation. These basins are formed around the major watercourses of the Pioneer River which discharges to the north east of the Mackay CBD and Plane Creek discharging to the Coral Sea east of Sarina. The boundary of each basin as well as streams of interest in the study is shown in Figure 3-1.

3.1.1. Catchment Delineation

The Pioneer River was found to contain most of the stream gauges and alert stations for the Mackay Region and hence was chosen as the main watercourse of this investigation, Bakers Creek (south of the Pioneer River) was also included to diversify the approach. These catchments were selected based on the availability of data at the time of the study and do not necessarily represent all catchment configurations across the wider Mackay region. Subcatchments that make up the wider Pioneer River catchment were selected based on from the Australian Hydrological Geospatial Fabric (Geofabric) (BoM, 2017). This methodology is similar to that currently undertaken within the industry and also aligns with previous investigations undertaken in the area as part of the Pioneer River Flood Study (WRM Water & Environment 2011). The derived greater Pioneer River catchment is shown in Figure 3-2.

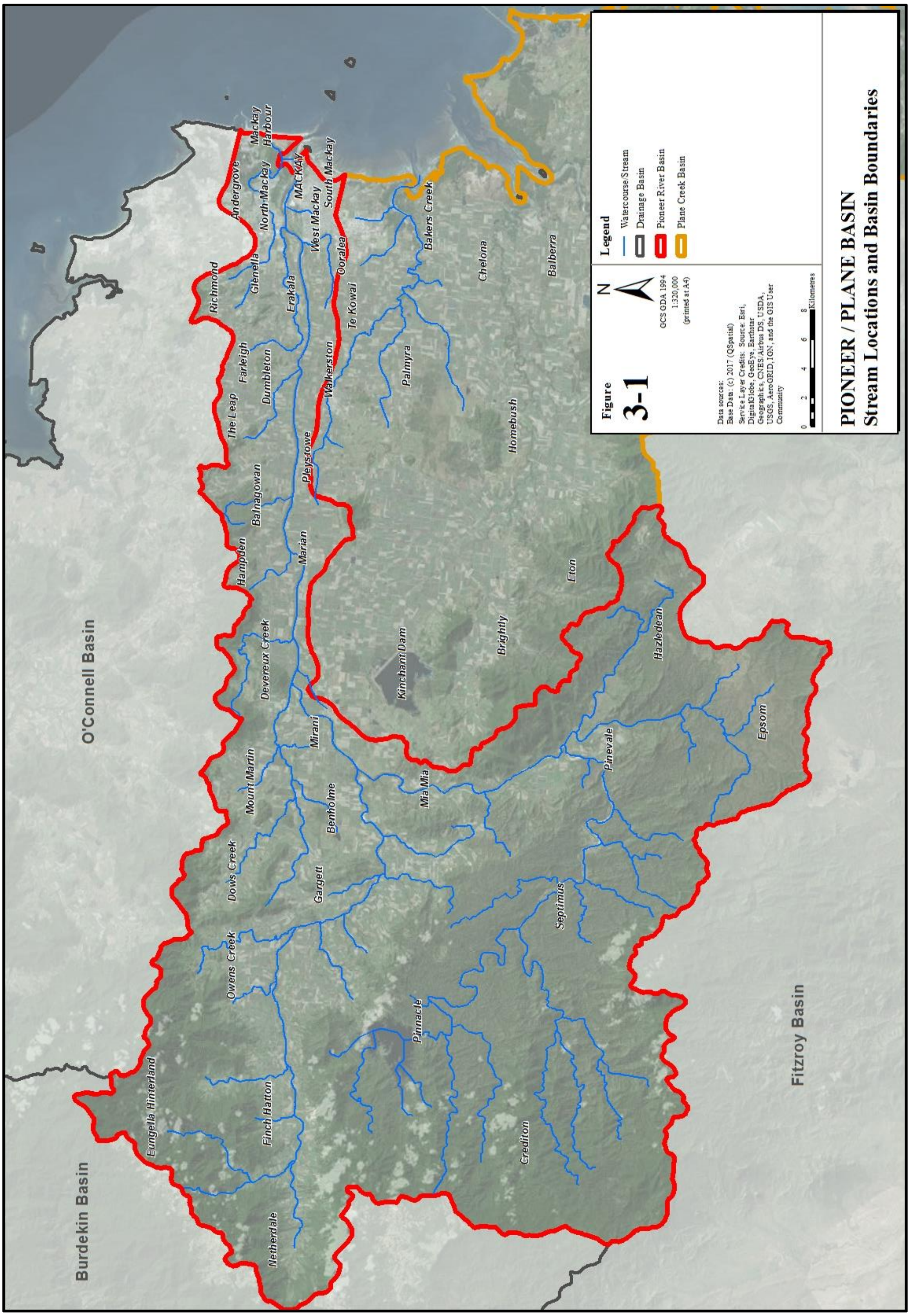

Figure 3-1: Pioneer and Plane Basin Locality

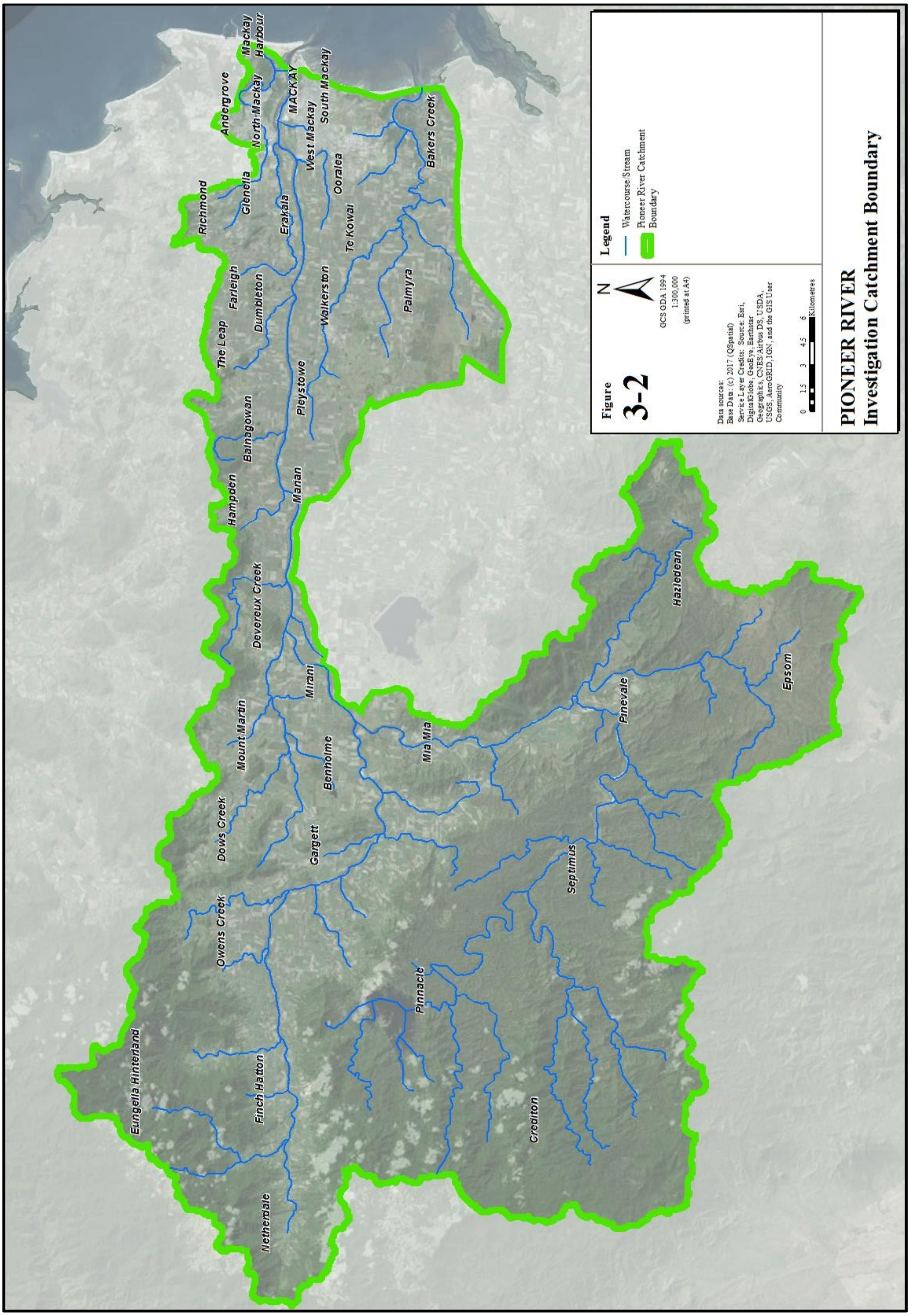

Figure 3-2: Pioneer River Catchment Boundary

y

3.1.2. DNRM Stream Gauge Stations

A number of stream gauges were utilised from the Department of Natural Resources and Mines (DNRM) Water Monitoring Information Portal (WMIP). These gauges record continuous stream height and discharge over time and are available for public use through the DNRM site. A summary of the sites and the data available is given in Table 3-1. The geographical location of the sites, both in operation and closed, is shown in Figure 3-3. A detailed summary of each site is contained in Appendix J.

Table 3-1: Available DNRM Stream Gauge Data

Site Number	Start of Record	Maximum Gauge (m^3/s)
125009A Cattle Creek at Highams Bridge	19/06/2002	910.05
125004B Cattle Creek at Gargett	03/07/1986	2495.26
125005A Blacks Creek at Whitefords	12/12/1973	3450.61
125002C Pioneer River at Sarichs	17/02/1958	5074.75
125007A Pioneer River at Mirani Weir Tailwater	09/11/1977	6415.75
125016A Pioneer River at Dumbleton Weir Tailwater	22/12/2005	3834.02

3.1.3. BoM Alert Stations

A select number of alert stations from the Bureau of Meteorology (BoM) that measure rainfall and stream height over time were also made available by Council as part of the investigation. These levels were used to calibrate the models created as part of this study. Details of the alert stations that were accessible for this investigation are given in Table 3-2 with a map of stations shown in Figure 3-4.

Table 3-2: Available BoM Alert Station Data

Site Number
533060 Hospital Bridge Alert
533061 Gooseponds Alert
033303 Mackay Alert
533063 Bakers Creek Alert

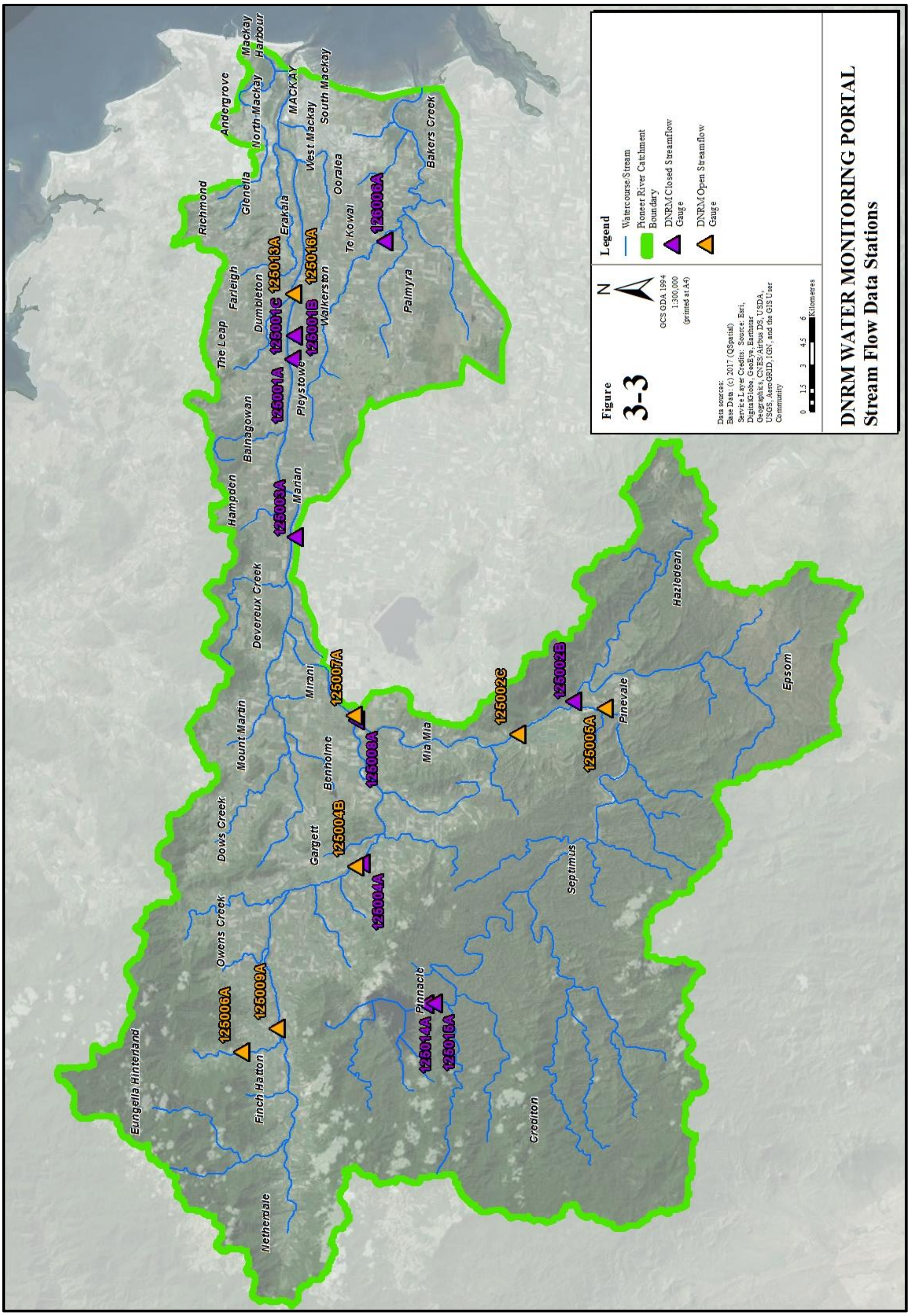

Figure 3-3: DNRM Streamflow Data Stations

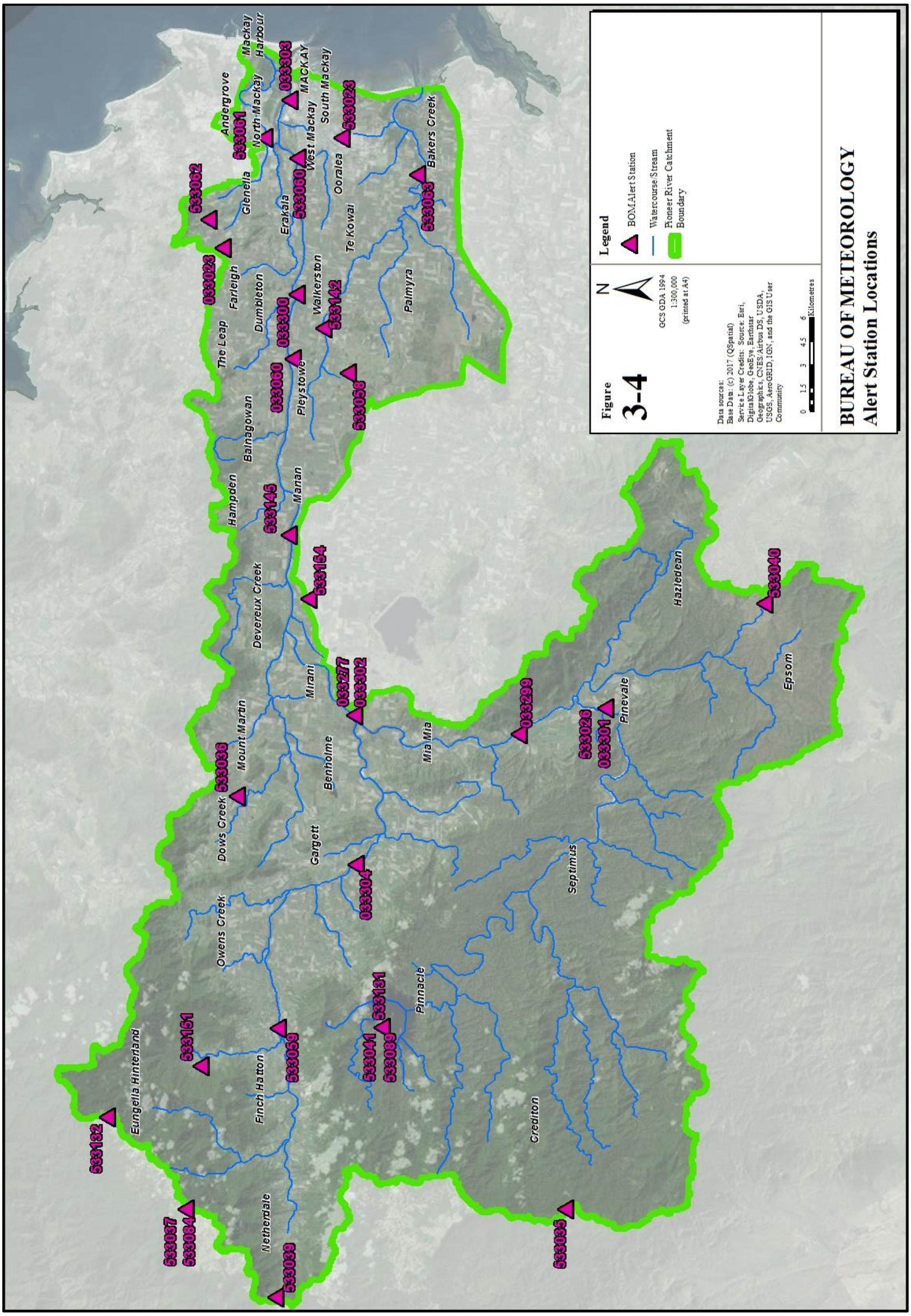

Figure 3-4: BOM Alert Stations

3.2. Selection of Historic Rainfall Calibration Events

The Mackay Region is located within the tropical region of Queensland and hence is subject to experiencing significant rainfall events associated with weather phenomena particularly in the warmer months ranging from November through to April.

Two significant rainfall events that have caused the Pioneer River to peak above its moderate flood level have been found to have significant amounts of data on record available for use as part of this investigation. Monsoonal troughs swept through the region in both 2008 and 2007, the significance and cause of each event is outlined in the sections below. The rainfall, river level and stream discharge data sourced from these events will be used as part of the model calibration (refer Section 3.6.1) and will be key in the flood frequency analysis (refer Section 3.3).

3.2.1. 2008 Rainfall Event

In an active phase of the monsoon trough, a low, originating in the Gulf of Carpentaria, traced a path southeast across southern Cape York Peninsula to intensify over land to the near west of Proserpine. The system brought with it localised severe winds to Airlie Beach and heavy rainfall to areas between Townsville and Rockhampton. The low quickly moved out to sea allowing a strong high-pressure ridge to develop along the Queensland coast south of the monsoon trough. This brought about stable conditions to most parts of the state except the far north, where the monsoon trough remained active and about the Central Coast on the leading edge of the strengthening ridge. Conditions in Mackay deteriorated early on the 15th of February producing phenomenally intense rainfalls in the area and severe flash flooding (BoM 2008)

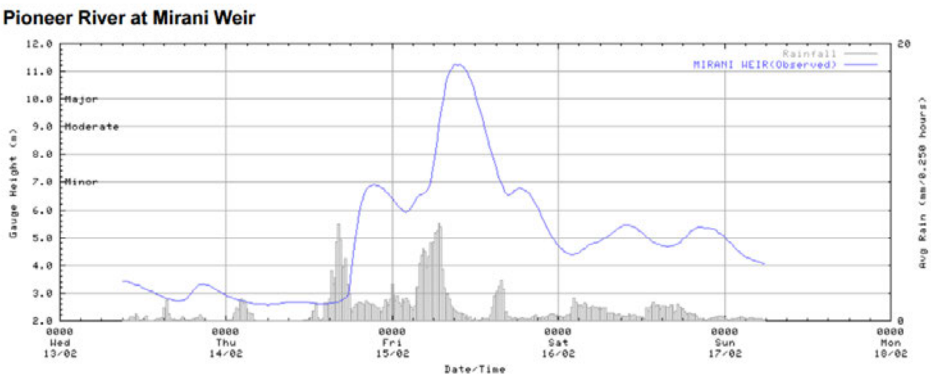

Figure 3-5: Pioneer River Height and Rainfall at Mirani Weir- 2008 Event (BoM, 2008)

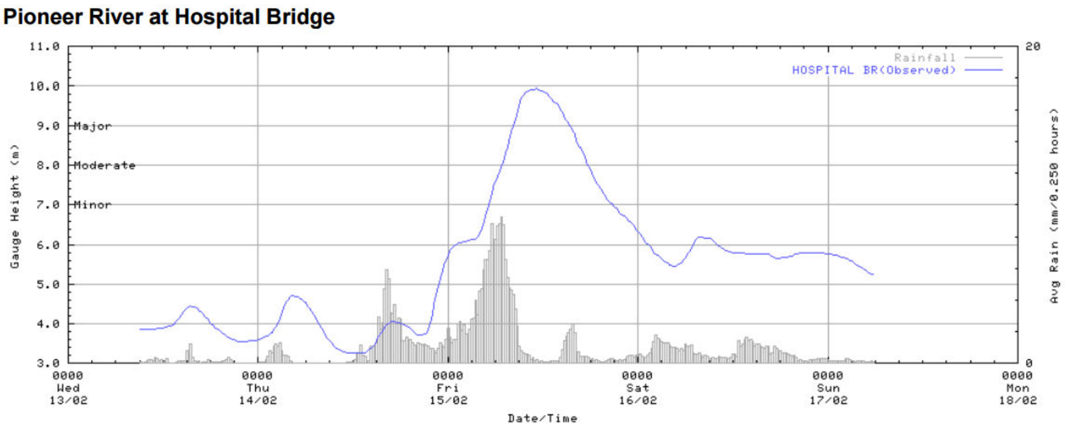

Figure 3-6: Pioneer River Height and Rainfall at Hospital Bridge - 2008 Event (BoM, 2008)

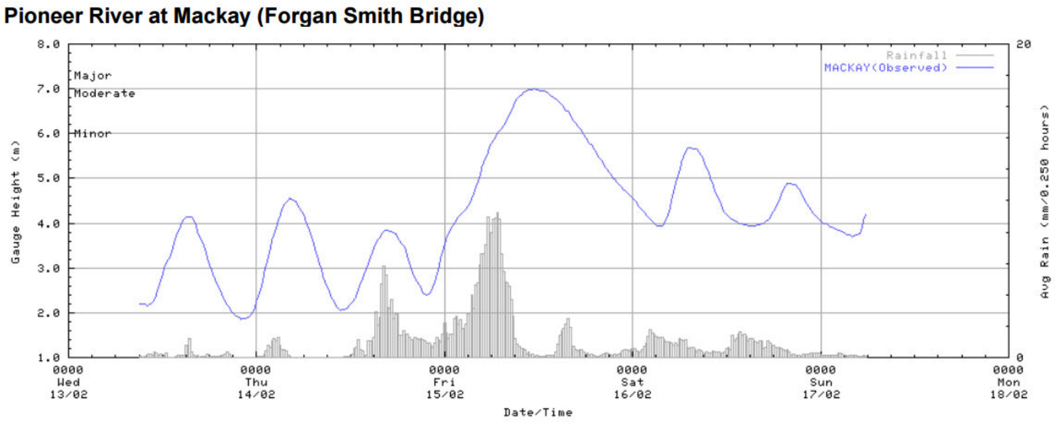

Figure 3-7: Pioneer River Height and Rainfall at Forgan Smith Bridge- 2008 Event (BoM, 2008)

3.2.2. 2007 Rainfall Event

In late January 2007 an active monsoon trough developed over the far north Queensland coastline delivering heavy rainfall over the northern and central coastlines as well as inland regions over the 1st and 2nd of February. The heaviest falls associated with the rainfall event were measured from north of Mt Isa through to Mackay.

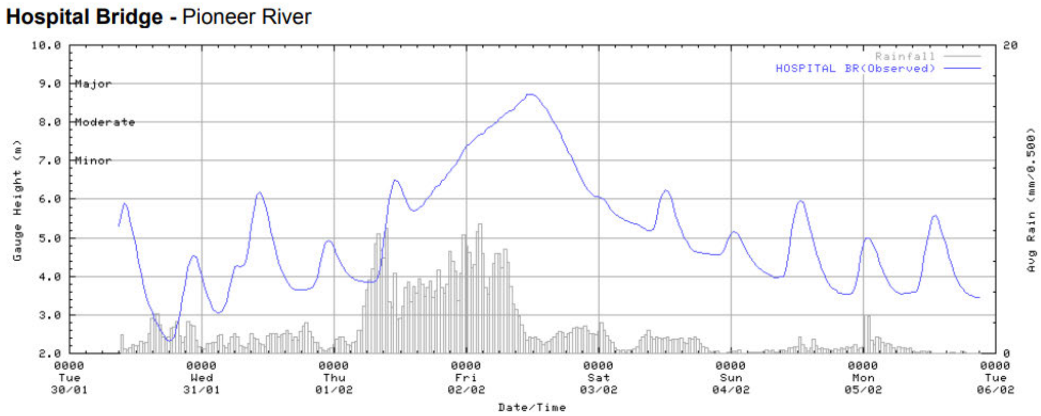

Figure 3-8: Pioneer River Height and Rainfall at Hospital Bridge - 2007 Event (BoM, 2007)

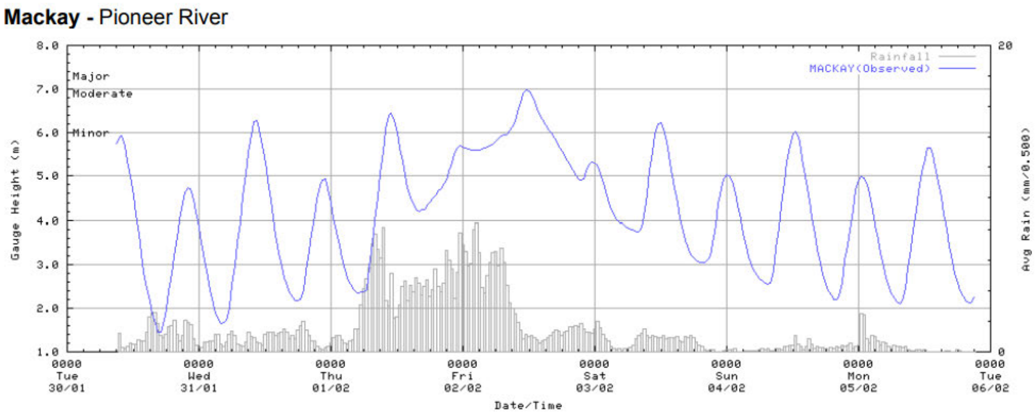

Figure 3-9: Pioneer River Height and Rainfall at Mackay Alert - 2007 Event (BoM, 2007)

3.3. At Site Flood Frequency Analysis

As described in the literature review, at site Flood Frequency Analysis (FFA) is the process where recorded flood data is analysed to estimate the probability model of flood peaks, which is then used to determine flow quantum's for design and risk assessments. This will determine the 'true' design discharges for the sites at which the FFA is undertaken. For this investigation, DNRM stream gauge sites (refer Section 3.1.2) were used due to the availability of continuous data.

3.3.1. Annual Series FFA

FLIKE (BMT WBM 2015), a software platform that calculates flood frequency distributions for at site locations has been used for the study (refer Section 2.3.5). This software has been recommended as ‘best practice’ in the 2016 Revision of the ARR Guidelines.

The software has the capability to calculate flood frequency distributions using the following probabilistic models:

- Log-Normal
- Log Pearson III (LP3)
- Gumbel
- Generalised Extreme Value (GEV)
- Generalised Pareto

Data from each stream gauge was processed to determine the water year annual maximum (1^{st} October through 30^{th} September) in a format to suit the required input for the FLIKE software. Data input to the software is to censor data points that may skew or distort the flood frequency distribution. From this the distribution curve is generated by fitting the gauged data to the various probability models above using the Bayesian method (refer Section 2.3.6), the distribution method that is built into the FLIKE software package. Confidence limits are also exported from FLIKE for comparison.

3.3.2. Partial Series FFA

As mentioned in the literature review (refer Section 2.3.4), Annual Series FFA is only sufficient for determining the magnitude of rainfall events smaller than the 10% AEP. Therefore, a Partial Series FFA was also undertaken for each site to determine the flood frequency distribution for low order events.

This method was undertaken by selecting a peak flood threshold that resulted in the number of flood peaks in the analysis (K) is no more than 3N where N is the number of years of gauge records (Kuczera & Franks 2006). From this a NExp distribution was fit to the data to produce a flood frequency curve.

3.4. Determination of Hydrologic Inputs

Hydrologic inputs for the modelling were determined using the ‘best practice’ methods outlined in the Australian Rainfall and Runoff (ARR) Guidelines, the Simple Event and Ensemble Event from the 1987 and 2016 revisions respectively. These methods utilise a database of design rainfall Intensity-Frequency-Duration (IFD) values accessible through the BoM website. The IFD databases vary for the two methodologies, this is due to the length of rainfall data on record with the 2016 data having almost 30 years extra historical rainfall.

As the greater Pioneer River catchment covers such a large area, it is expected that these IFD values will vary based on the location in the catchment. To ensure that the most accurate hydrologic data was applied to the models, subcatchments were grouped by their geographical location to make a subset of smaller catchments for which unique parameters were applied. The IFD calculation areas are shown geographically in Figure 3-10.

The two methodologies also vary in the application of design rainfall temporal patterns, loss values and areal reduction factors applying to each storm. The specific requirements of each revision are noted below.

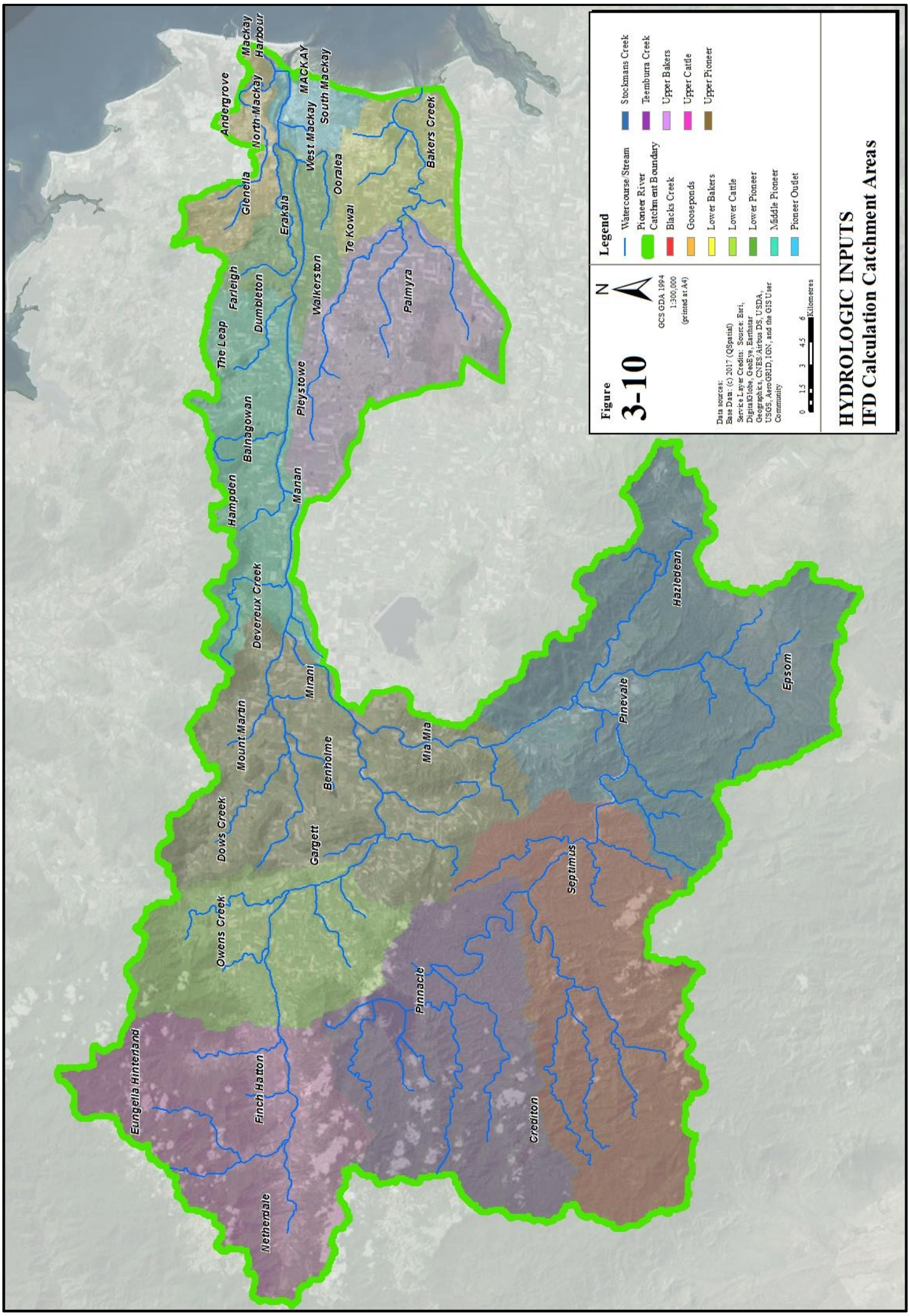

Figure 3-10: IFD Calculation Areas for Hydrological Inputs

3.4.1. ARR1987 Simple Event Hydrology

The following procedure was used for the determination of hydrologic inputs for modelling using the Simple Event ARR1987 methodology:

i. Determine the rainfall catchment boundary and centroid location.
ii. Extract ARR87 IFD data from BoM online for the catchment.
iii. Determine IFD information for all design storms using procedures in Section 1.3 Book II of ARR87 (Canterford et al. 1987).
iv. Apply Areal Reduction Factors (ARF) to design rainfall for long duration storms based on the Queensland Extreme Rainfall Estimation Project (EREP) (Hargraves 2004). For this investigation the ARF's from the Pioneer River Flood Study (WRM Water & Environment 2011) were used for consistency.
v. Fit design rainfall to the ARR1987 temporal patterns for the relevant zone (Zone 3) as per Section 2 Book II of ARR87 (Pilgrim et al. 1987) to determine design rainfall to be applied to model for a variety of storm durations.
vi. Determine Initial Loss (IL) and Continuing Loss (CL) values based on Section 3 Book II of ARR87 (Cordery 1987), in this case 15mm IL and 2.5mm/hr CL for eastern Queensland.

3.4.2. ARR2016 Ensemble Event Hydrology

The following procedure was used for the determination of hydrologic inputs for modelling using the Ensemble Event ARR2016 methodology:

i. Determine the rainfall catchment boundary and centroid location.
ii. Extract ARR16 IFD design rainfall depth from BoM online for the catchment.
iii. Use the ARR16 Data Hub site to extract the design losses, areal reduction factors, preburst depths and ensemble temporal patterns for the catchment.
iv. Apply areal reduction factors to the relevant design rainfall depths.
v. Plot the rainfall per period for all 10 ensemble temporal patterns for the required events and storm durations.
vi. Calculate the amount of preburst/initial loss required by analysis of the two values and add required values to the design rainfall or model parameters.

3.5. Runoff Routing Model Development

A Watershed Bounded Network Model (WBNM) was developed as part of the investigation for the greater Pioneer River Catchment. As mentioned in the literature review (refer Section 2.6) the model calculates the flood hydrograph from design or recorded rainfall using a runoff routing approach where a stream network is divided into smaller subcatchments. A routing method is then assigned to each subcatchment as well as loss values which are applied to the calculation of flow on to the next downstream subcatchment. The processes in setting up the WNBM runoff routing model for the greater Pioneer River catchment is outlined in the subsequent sections below.

3.5.1. Subcatchment Delineation

Subcatchments for the WBNM model were selected from the Australian Hydrological Geospatial Fabric (Geofabric) (BoM, 2017). This methodology is similar to that currently undertaken within the industry and also aligns with previous investigations undertaken in the area as part of the Pioneer River Flood Study (WRM Water & Environment 2011).

Each subcatchment was then assigned a unique identification and linked to a downstream catchment for routing. The WBNM model catchments are shown geographically in Figure 3-11.

3.5.2. Routing Method

The non-linear routing method was selected for the WBNM model developed as part of the investigation. This routing method is the default method built into the software for routing through natural catchments. Although there is urbanisation throughout the Pioneer River catchment this method still proven to calibrate best to the historical rainfall events modelled with minimal modification required.

The default Lag Parameter of 1.6 was applied to the model with some variation in this value (ranging from 1.2 to 1.8) applied to match the modelled discharge to the recorded discharge of historic events.

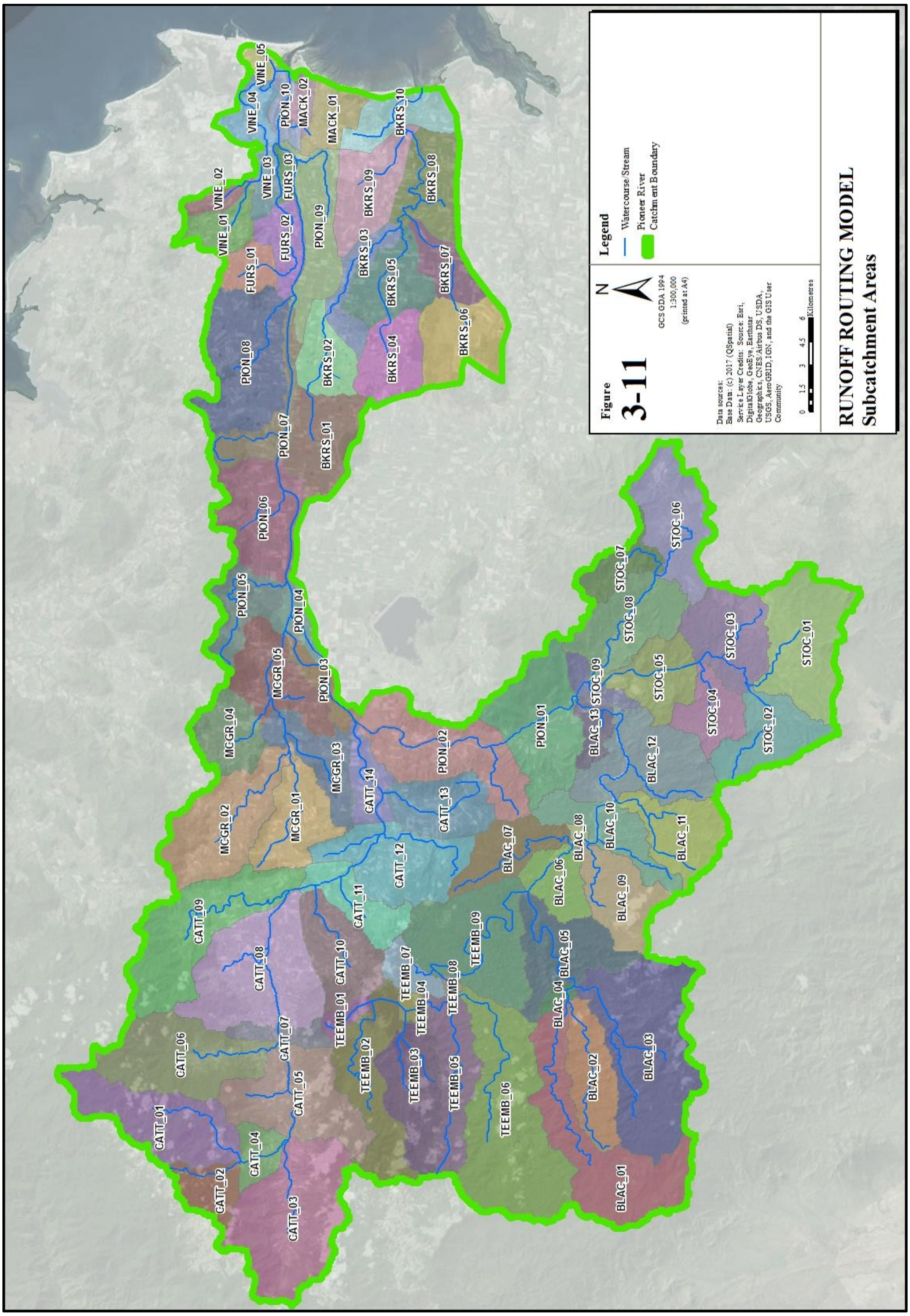

Figure 3-11: WBNM Runoff Routing Model Subcatchment Areas

3.5.3. Design Hydrograph Extraction

From the WBNM runoff routing model developed as part of the investigation, design discharge hydrographs were extracted from the results using excel coding developed for the WBNM software platform. These hydrographs were then used for comparison and the extraction of the design peak discharge for each event modelled.

3.6. Hydrodynamic Model Development

A TUFLOW hydrodynamic model was developed as part of the investigation for the lower, more urbanised reaches of the Pioneer River Catchment. As described in the literature review (refer Section 2.7) a hydrodynamic model aims to provide a realistic representation of flow behaviour in a particular environment.

As mentioned in the literature review "The physical system being modelled may be schematised in many different ways depending on the selection of model elements within the modelling tool and the choices made by the modeller. The accuracy, reliability and usefulness of the model are significantly influenced by the skill of the modeller in completing this process." (Babister & Barton 2012).

Each of the elements selected to prepare the hydrodynamic model are described and justified in the subsequent sections.

3.6.1. Model Parameters

The latest version of the TUFLOW hydrodynamic modelling software available at the time of the investigation was used to run the model, that being TUFLOW 2016-03-AD. All setup parameters such as viscosities and cell wetting/drying depths were set as per the advice given in the TUFLOW manual.

A 20m grid spacing with a 5 second timestep was chosen for the model simulation, this is deemed to be acceptable with the terrain used (refer Section 3.6.2).

3.6.2. Model Terrain

The terrain used within the TUFLOW model was sourced from the Australian Shuttle Radar Topography Mission (SRTM). This dataset is publicly available online and was compiled as part of a NASA program in the early 2000's which digitised the topography of the globe on a 30m grid. The digital terrain produced as part of the SRTM program is reported to have a vertical accuracy of ±10m however it is regarded as an appropriate topography source for large scale catchment modelling. To improve flow through watercourses in the model, the Geoscience Australia SRTM Derived Hydrological Digital Elevation Model (DEM-H) was used. This dataset is a hydrology conditioned and drainage enforced subset of the SRTM project that enforces catchment delineation and watercourse connectivity. This dataset was also used for the development of the TUFLOW model boundary.

Figure 3-12 shows a map of the boundary and terrain applied to the TUFLOW hydrodynamic model.

3.6.3. Model Boundary Conditions

The Pioneer River and Bakers Creek are heavily tidally influenced estuaries, with some of the Highest Astronomical Tides (HAT) in Australia being experienced along the Mackay coastline (EPA 2005). The tide planes for the Mackay Outer Harbour are shown in Table 3-3 as sourced from the Semidiurnal Tidal Planes tables (Maritime Safety Queensland, 2017)

The appropriate downstream water level (representative of a predicted tide) for the probability of exceedance (refer Table 3-3) of the modelled event was selected and applied as a HT boundary applying a constant water surface level (H) over the time of the simulation (T).

Table 3-3: Predicted Tide Planes for Mackay Outer Harbour (Maritime Safety Queensland, 2017)

Defined Tide	Predicted Tide Level (mAHD)	Probability of Annual Exceedance (%)
Highest Astronomical Tide (HAT)	3.64	0
Mean High Water Springs (MHWS)	2.35	5
Mean High Water Neaps (MHWN)	1.12	26
Mean Low Water Neaps (MLWN)	-0.98	74
Mean Low Water Springs (MLWS)	-2.20	97
Lowest Astronomical Tide (LAT)	-2.94	100

3.6.4. Model Roughness Layer

TUFLOW hydrodynamic modelling supports the application of a material roughness layer where set Manning's roughness value or a depth varying Manning's roughness can be applied spatially to the terrain. To develop this layer for the Pioneer River catchment, MRC's land use dataset and aerial photography were used to categorise areas into very high level materials groups. The materials layer groups and relevant attributes are shown in Table 3-4, these values and groups were chosen to match those modelled as part of the Pioneer River Flood Study (WRM Water & Environment 2011) The model roughness layer being shown geographically in Figure 3-13.

Table 3-4: Model Roughness Layer Attributes

Material	Mannings 'n'
Pasture, Cane Fields & Open Space	0.070
Dense Vegetation & Riparian	0.090
Urbanised Areas	0.100
Roads	0.020
Pioneer River	0.040
Vegetated Creeks & Channels	0.100

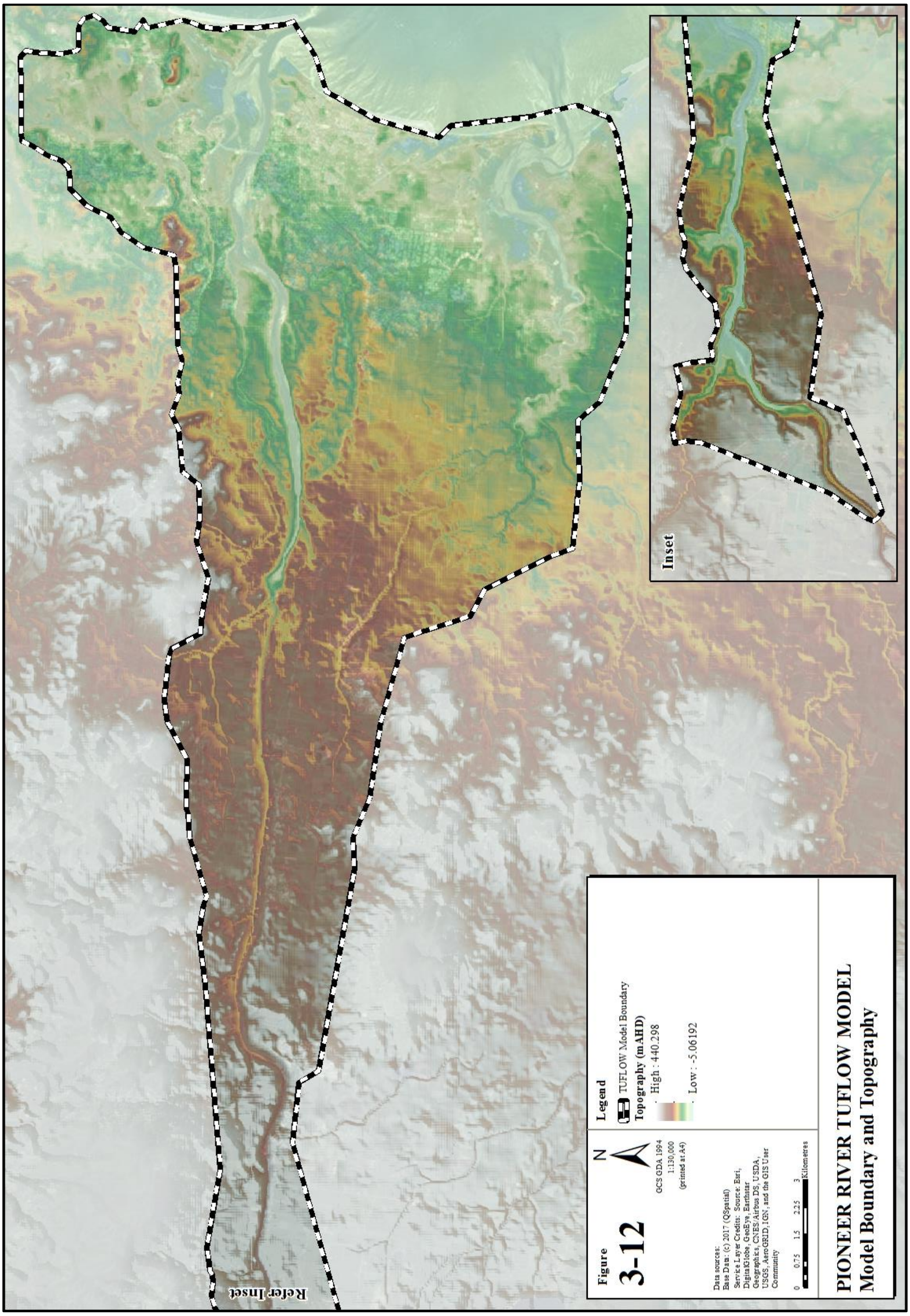

Figure 3-12: TUFLOW Hydrodynamic Model Boundary and Terrain

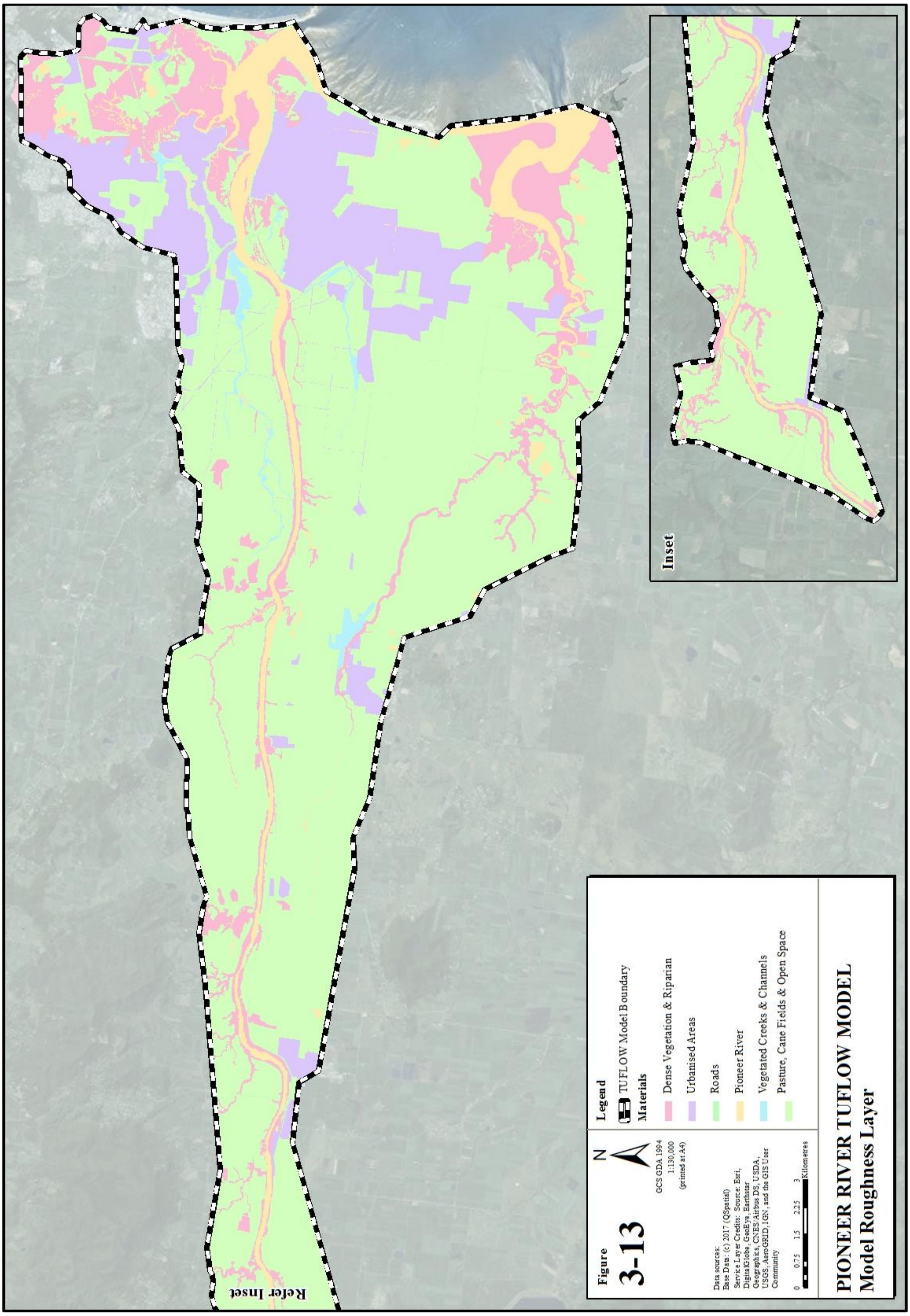

Figure 3-13: TUFLOW Hydrodynamic Model Roughness Layer

3.6.1. Design Hydrograph Extraction

From the TUFLOW hydrodynamic model developed as part of the investigation, design discharge hydrographs were extracted from the results using Plot Output (PO) coding within the TUFLOW model, creating time series data outputs that were then used to graph the design discharge hydrographs. These hydrographs were then used for comparison and the extraction of the design peak discharge for each event modelled.

3.7. Model Calibration

To ensure that models created as part of the investigation are performing adequately and representing the actual hydraulic conditions of the catchment it is important that the models are calibrated to historical recorded flood events. As mentioned in Section 3.2, the February 2008 and January/February 2007 events were selected for calibration due to the availability of data at the time of the investigation.

Sourced pluviograph rainfall data was applied to the WBNM runoff routing model and the peak discharges at locations with historical records were compared to that of the discharges output from the model. The model parameters including loss values and routing parameters were then modified to match the shape and peak of the model hydrograph to that of the recorded hydrograph at the same location. This procedure is used in hydrologic and hydraulic investigations throughout the industry and is critical in ensuring model outputs can be trusted.

It is not expected that the model will perfectly represent the recorded hydrograph as there are many external factors that may determine the recorded flood hydrograph that cannot be represented in the conceptual modelling process. However, a calibration that delivers a hydrograph with the same shape (discharge over time) and a similar peak discharge is considered acceptable.

It is important that the model calibration is undertaken for more than one historic event to ensure that the model is not being calibrated to false or inaccurate data sourced from a single event. For this investigation calibration to two historic flood events was deemed to be sufficient, however it is quite common in the industry for calibration to be undertaken on many events where data is sufficient.

3.8. Regional Flood Frequency Estimation (RFFE)

The online Regional Flood Frequency Estimation (RFFE) Model was used to estimate the peak discharges at key locations. The flood frequency distribution was output from the tool using the same catchment areas as derived for the WBNM runoff routing model.

Although this model does not accurately produce discharge estimates for all catchment sizes and configurations, its output and limitations at each location was recorded for the comparative purposes of this investigation.

3.9. Rational Method Estimation

The Rational Method was used to estimate the design discharge at catchment outlets for a variety of storm events. Although (as mentioned in the literature review) this method is now only used as a 'sanity check' in the industry and has been excluded from the 2016 revision of the Australian Rainfall and Runoff Guidelines, it is still documented as an acceptable assessment method in the Queensland Urban Drainage Manual (Department of Energy and Water Supply 2013).

The calculation utilises catchment boundaries and areas which, for consistency, were taken as a sum of the reporting subcatchments derived for the WBNM runoff routing model. Whilst SRTM topography as utilised for the TUFLOW hydrodynamic model was used to estimate the slope of the catchments.

To determine the time of concentration for each calculation, the Bransby-Williams' equation was selected to be a consistent approach across the study (refer Equation 3-1). Although this method is generally only recommended for rural and creek catchments (Department of Energy and Water Supply 2013), it was selected the most appropriate for the greater catchment as majority of the upstream reaches satisfy the calculations requirements. The Bransby-Williams' formula is as follows:

$$t_c = \frac{58L}{\left(A^{0.1} \cdot S_e^{0.2}\right)}$$

Equation 3-1

where t_c is the time of concentration (min), L is the length of the flowpath from the outlet upstream to the catchment divide (km), A is the catchment area (ha) and Se is the equal area slope of the flowpath (%).

Although this calculation may not be suitable for all catchment sizes and configurations, its output and limitations at each location was recorded for the comparative purposes of this investigation.

3.10. Results Presentation

The results for the investigation have been sourced throughout a variety of data analysis and modelling computations as mentioned in the above sections. These results will be presented in tabular and graphical format and summarised in a dissertation format within Chapter 4 of this document. A more detailed presentation of results from the above processes is provided in the appendices of this document.

3.11. Discussion/Comparison of Results

The findings of the investigation will be delivered in a detailed discussion comparing the results of each design discharge estimation method, reported in Chapter 5 of this document. The results will be compared over a range of exceedance probabilities to determine whether one approach is more suitable for specific events or a range of events. The results will also be compared by inputs and catchment parameters to determine if particular approaches perform better in certain geographies or catchment sizes/configurations. All findings will then be summarised, discussed and justified in the conclusions of this dissertation (refer Chapter 6).

CHAPTER 4

4. INVESTIGATION AND RESULTS

This chapter contains the results of investigation processes outlined in the study methodology (refer Chapter 3). The results shown have been summarised into a dissertation format with more detailed outputs and calculations available in the appendices. A list of the relevant context contained in the appendices is as follows:

- Appendix B: ARR1987 Design Rainfall Information
- Appendix C: ARR2016 Design Rainfall Information
- Appendix D: WBNM Model Subcatchment Configuration
- Appendix E: Critical Duration Assessment Results
- Appendix F: TUFLOW Model Development and Results Mapping
- Appendix G: RFFE Model Results
- Appendix H: Rational Method Calculations
- Appendix J: DNRM Stream Gauge Station Information

The peak flow determination methods (refer Chapter 3) were applied throughout the catchment with results extracted from key locations for comparison. These key locations are described in Sections 3.1.2 and 3.1.3 and have been summarised in Table 4-1. This table also provides a summary of what assessments have been applied at each location. Some methods of estimation could only be applied in areas where data permitted or were restricted to the boundary of the model developed.

The assessment locations have also been shown geographically in Figure 4-1. This figure also shows the locations relative to the boundaries of the runoff routing (WBNM) and hydrodynamic (TUFLOW) model that were developed for the investigation.

Table 4-1: Assessment Methods and Locations for the Investigation

Assessment Location	At Site FFA	WBNM Model	TUFLOW Model	RFFE Model	Rational Method
Cattle Creek at Highams Bridge	✓	✓		✓	✓
Cattle Creek at Gargett	✓	✓		✓	✓
Blacks Creek at Whitefords	✓	✓		✓	✓
Pioneer River at Sarichs	✓	✓		✓	✓
Pioneer River at Mirani Weir Tailwater	✓	✓		✓	✓
Pioneer River at Dumbleton Weir Tailwater		✓		✓	✓
Hospital Bridge Alert		✓	✓	✓	✓
Mackay Alert		✓	✓	✓	✓
Gooseponds Alert		✓	✓	✓	✓
Bakers Creek Alert		✓	✓	✓	✓

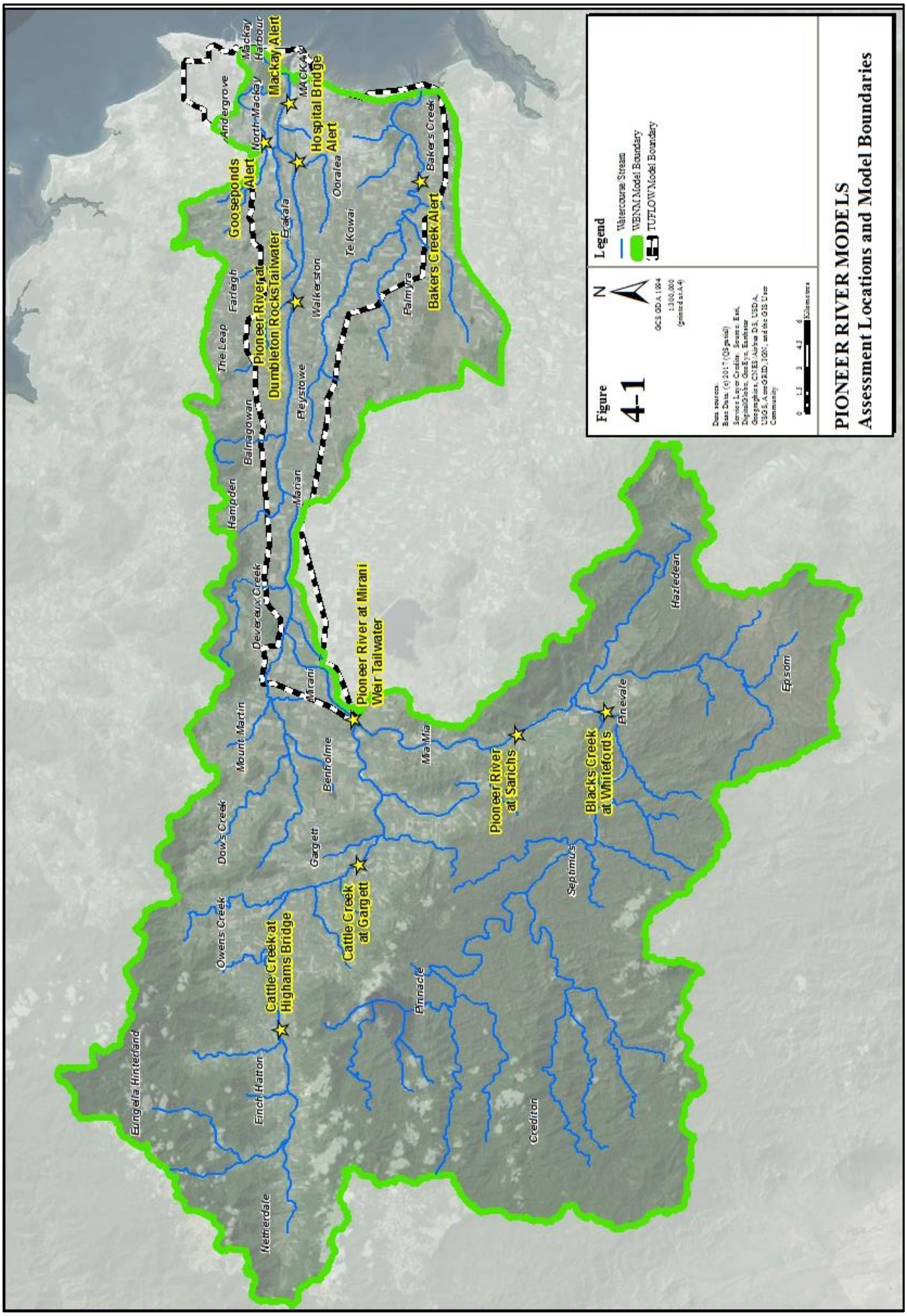

Figure 4-1: Assessment Locations within the Pioneer River Catchment

4.1. At Site Flood Frequency Analysis

As per the methodology outlined in Section 3.3.1 the FLIKE software package was used to undertake an at site annual maximum flood frequency analysis. Similarly as per section 3.3.2 a peak over threshold partial series flood frequency analysis was undertaken by fitting flood peaks to a NExp distribution. The following sections present the results of the FFA calculations undertaken on the DRNM sites with adequate data quality and history for assessment.

4.1.1. Cattle Creek at Highams Bridge (125009A)

The results of the FLIKE annual flood frequency analysis for the stream gauge on Cattle Creek at Highams Bridge (125009A) is shown in Figure 4-2. It should be noted that there are only 12 water years on record for this stream gauge, with the largest magnitude being estimated at as a 1 in 27 year event (3.7% AEP), therefore large magnitude events may not be accurately estimated at this site.

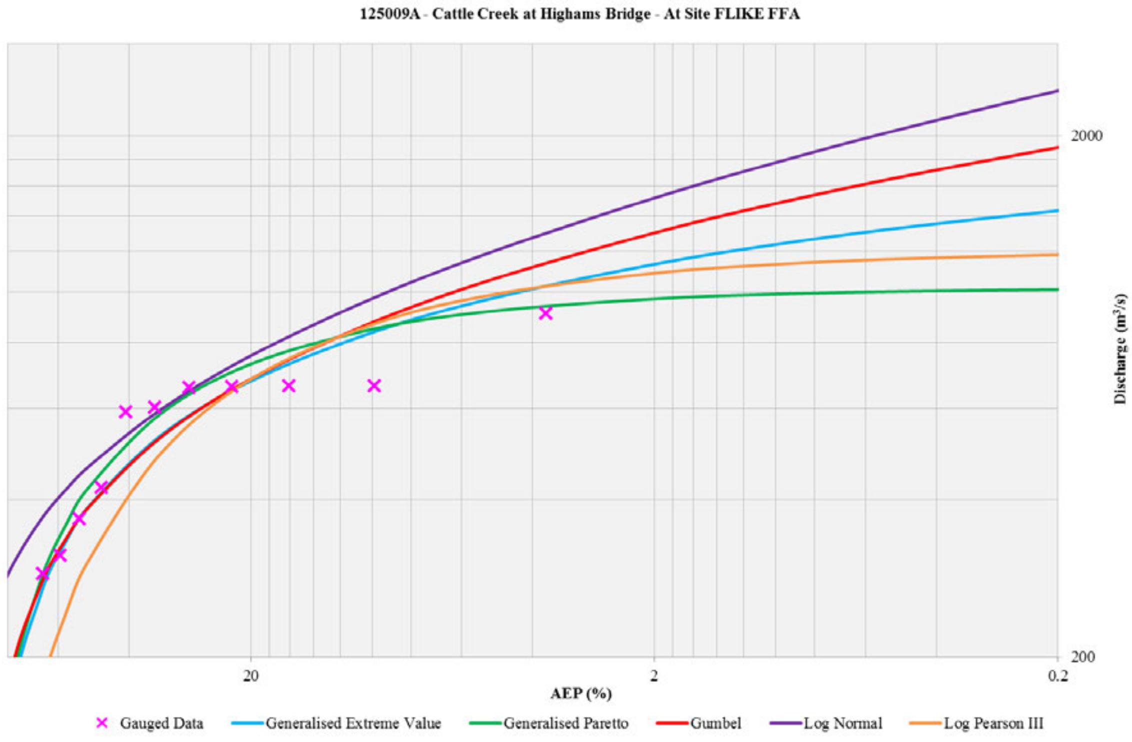

Figure 4-2: FLIKE Annual FFA Results – Cattle Creek at Highams Bridge

The probability model that best represents the 'true' flood frequency distribution at the stream gauge site was chosen to be the Generalised Pareto (GP) model. The GP model was deemed as a best fit to the recorded levels through computation to the R^2 value. The R^2 values for each probability model at the site can be found in Table 4-2.

Table 4-2: FLIKE Annual FFA Results – Cattle Creek at Highams Bridge: Probability Model R^2 Value

Probability Model	R^2 Value
Generalised Pareto	0.967
Generalised Extreme Value	0.965
Log Pearson III	0.952
Gumbel	0.948
Log Normal	0.889

The 90% confidence limits (exported from FLIKE) have been plotted against the Generalised Pareto flood frequency distribution in Figure 4-3. This shows that there is a good correlation between the gauged data at the site and the accepted distribution curve.

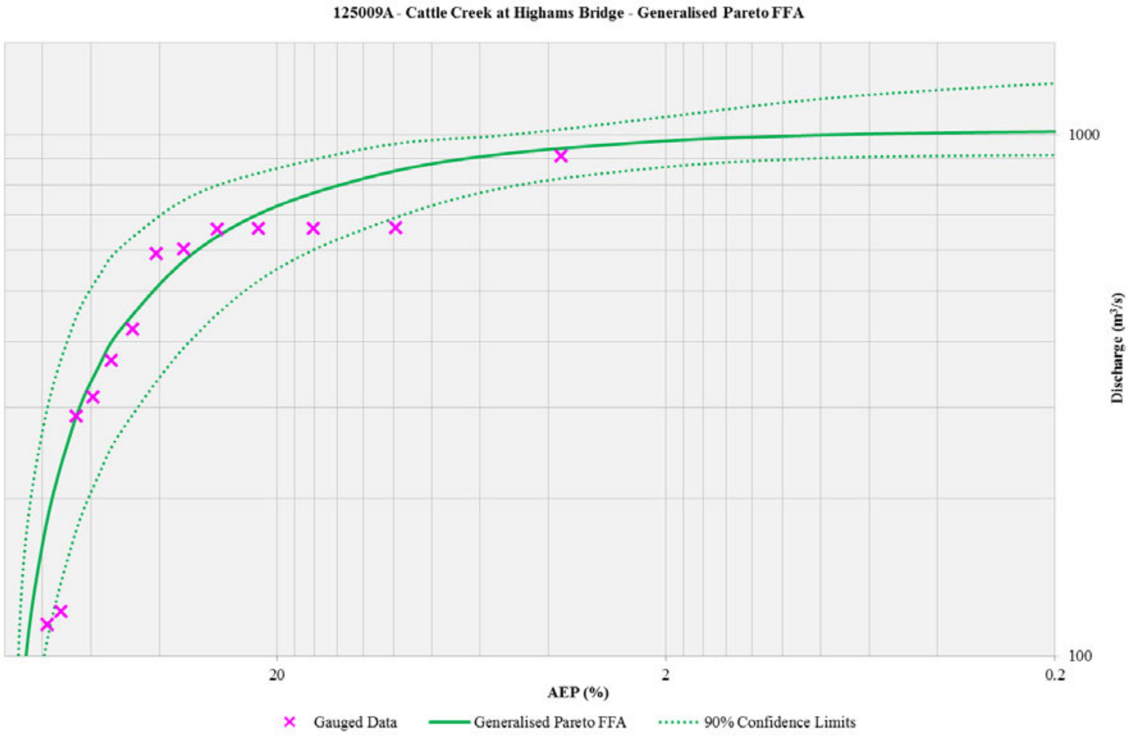

Figure 4-3: Annual Flood Frequency Distribution – Cattle Creek at Highams Bridge (Generalised Pareto)

Discharges from the FLIKE FFA as well as the relevant confidence limits have been listed in Table 4-3.

Table 4-3: Annual Series FFA Estimated Quantities – Cattle Creek at Highams Bridge

AEP (%)	Estimated Peak Discharge (m^3/s)						
	99	50	20	10	5	2	1
Upper 90% Confidence	9	610	861	959	999	1081	1151
Generalised Pareto	15	426	729	850	921	972	992
Lower 90% Confidence	6	271	552	691	795	867	895

For the partial series flood frequency analysis, a cut-off threshold of $100m^3/s$ was selected to return 26 peak flood events over the 16 years of gauge data on record. The distribution of the selected flood peaks over time is shown in Figure 4-4.

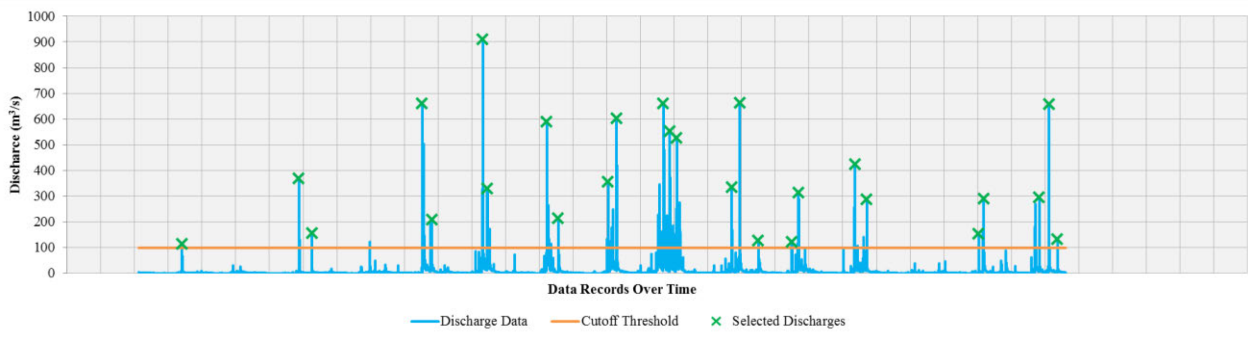

Figure 4-4: Partial Flood Frequency Analysis – Cattle Creek at Highams Bridge

The NExp flood frequency distribution for the selected flood peaks over threshold at the site has been plotted in Figure 4-5. The graph shows that there is a good correlation between the recorded peaks and the distribution function in the lower order events, as expected for a partial series analysis (refer Section 2.3.4).

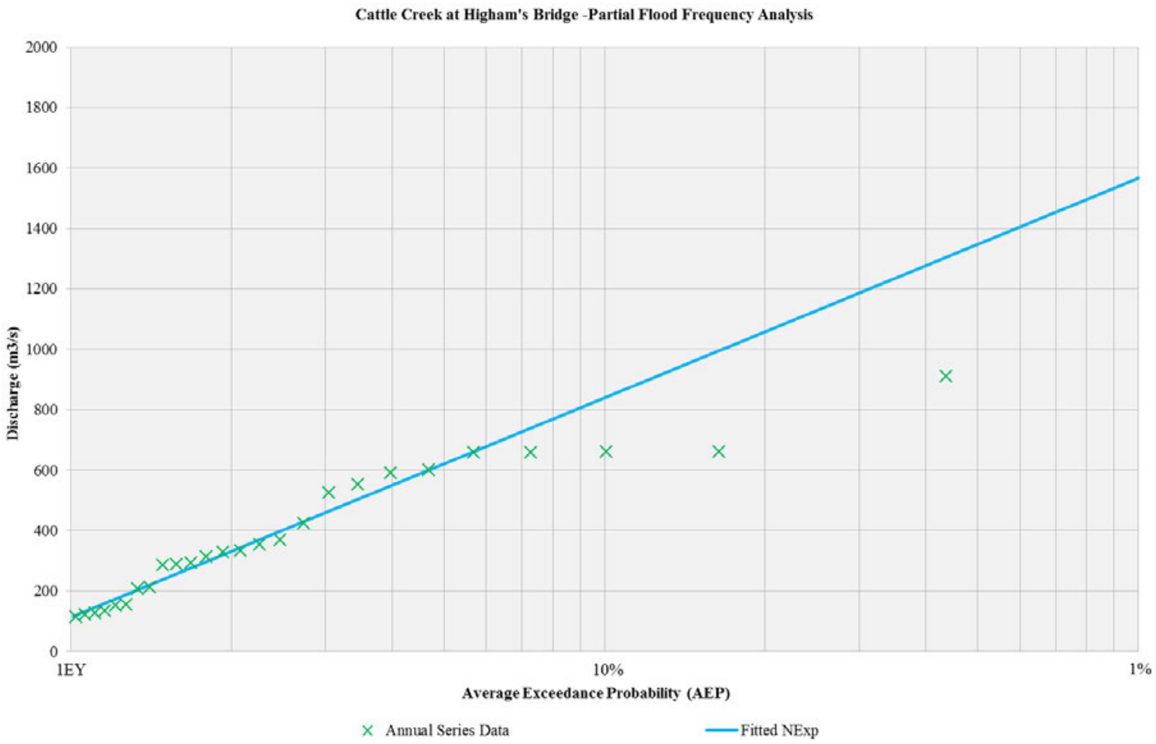

Figure 4-5: Partial Flood Frequency Distribution – Cattle Creek at Highams Bridge

Discharges from the partial series FFA have been listed in Table 4-4. However, it is to be noted that the results from the partial series FFA are not appropriate for events larger than 10% AEP.

Table 4-4: Partial Series FFA Estimated Quantities – Cattle Creek at Highams Bridge

AEP (%)	Estimated Peak Discharge (m^3/s)						
	99	50	20	10	5	2	1
Partial Series NExp	116	332	621	840	1058	1347	1566

4.1.2. Cattle Creek at Gargett (125004B)

The results of the FLIKE annual flood frequency analysis for the stream gauge on Cattle Creek at Gargett (125004B) is shown in Figure 4-6.

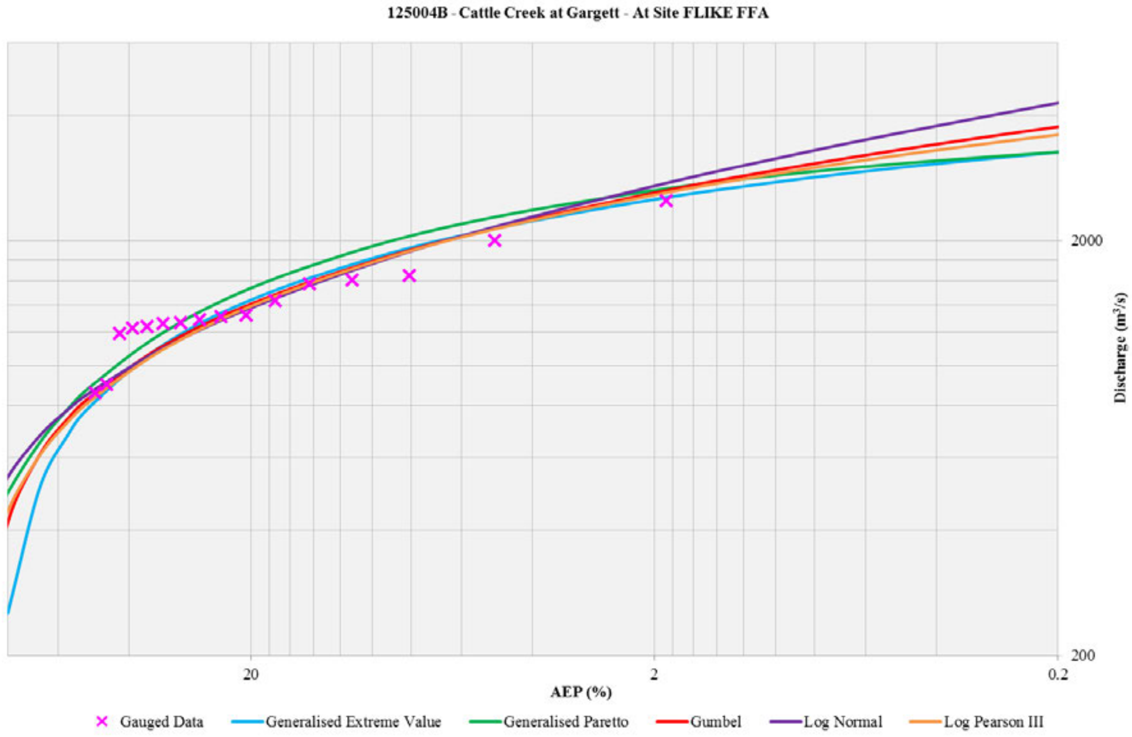

Figure 4-6: FLIKE Annual FFA Results – Cattle Creek at Gargett

The probability model that best represents the 'true' flood frequency distribution at the stream gauge site was chosen to be the Generalised Extreme Value (GEV) model. The GEV model was deemed as a best fit to the recorded levels through computation to the R^2 value. The R^2 values for each probability model at the site can be found in Table 4-5.

Table 4-5: FLIKE Annual FFA Results – Cattle Creek at Gargett: Probability Model R^2 Value

Probability Model	R^2 Value
Generalised Extreme Value	0.807
Generalised Pareto	0.796
Log Pearson III	0.793
Gumbel	0.792
Log Normal	0.774

The 90% confidence limits (exported from FLIKE) have been plotted against the GEV flood frequency distribution in Figure 4-7. This shows that there is a good correlation between the gauged data at the site and the accepted distribution curve.

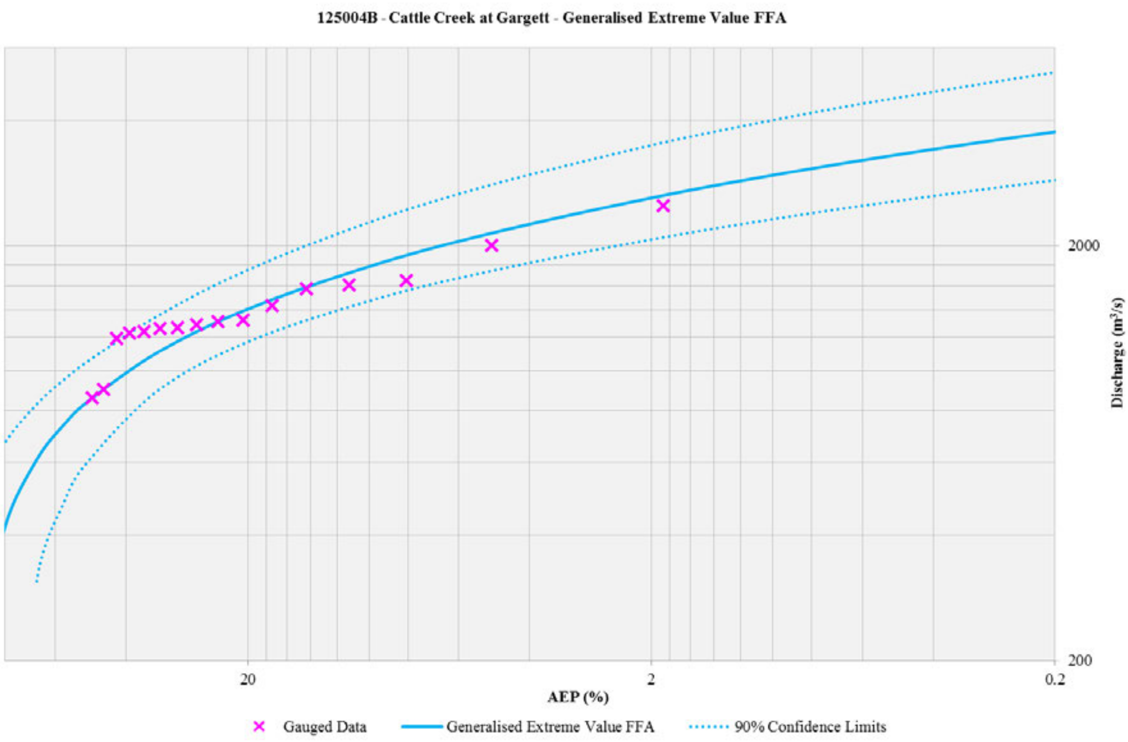

Figure 4-7: Annual Flood Frequency Distribution – Cattle Creek at Gargett (GEV)

Discharges from the FLIKE FFA as well as the relevant confidence limits have been listed in Table 4-6.

Table 4-6: Annual FFA Estimated Quantities – Cattle Creek at Gargett

AEP (%)	Estimated Peak Discharge (m^3/s)						
	80	**50**	**20**	**10**	**5**	**2**	**1**
Upper 90% Confidence	631	1061	1810	2254	3041	4265	5505
Generalised Extreme Value	253	797	1438	1812	2137	2514	2767
Lower 90% Confidence	0	504	1216	1517	1778	2055	2281

For the partial series flood frequency analysis, a cut-off threshold of $500m^3/s$ was selected to return 50 peak flood events over the 31 years of gauge data on record. The distribution of the selected flood peaks over time is shown in Figure 4-8.

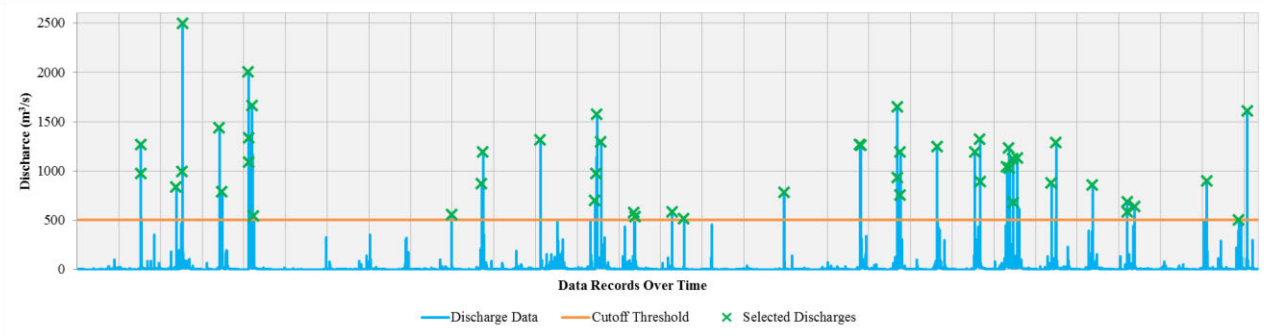

Figure 4-8: Partial Flood Frequency Analysis – Cattle Creek at Gargett

The NExp flood frequency distribution for the selected flood peaks over threshold at the site has been plotted in Figure 4-9. The graph shows that there is a good correlation between the recorded peaks and the distribution function in the lower order events, as expected for a partial series analysis (refer Section 2.3.4).

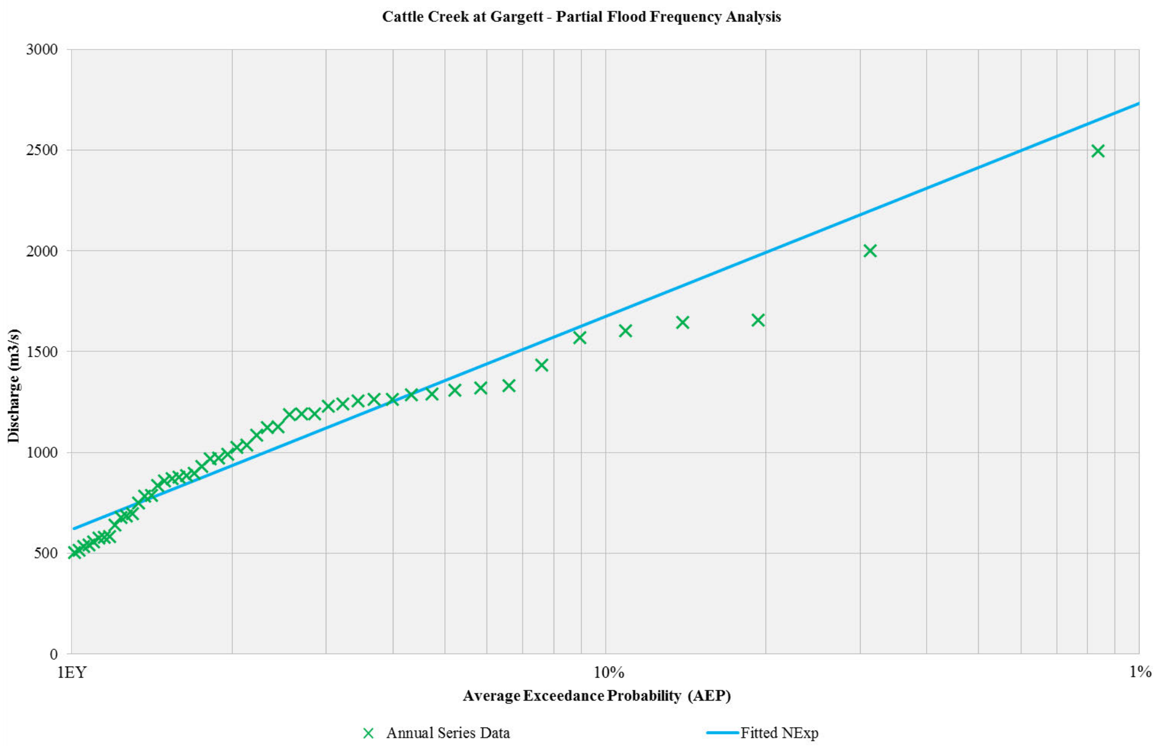

Figure 4-9: Partial Flood Frequency Distribution – Cattle Creek at Gargett

Discharges from the partial series FFA have been listed in Table 4-7. However, it is to be noted that the results from the partial series FFA are not appropriate for events larger than 10% AEP.

Table 4-7: Partial Series FFA Estimated Quantities – Cattle Creek at Gargett

AEP (%)	Estimated Peak Discharge (m^3/s)						
	99	**50**	**20**	**10**	**5**	**2**	**1**
Partial Series NExp	622	936	1357	1675	1993	2414	2732

4.1.3. Blacks Creek at Whitefords (125005A)

The results of the FLIKE annual flood frequency analysis for the stream gauge on Blacks Creek at Whitefords (125005A) is shown in Figure 4-10.

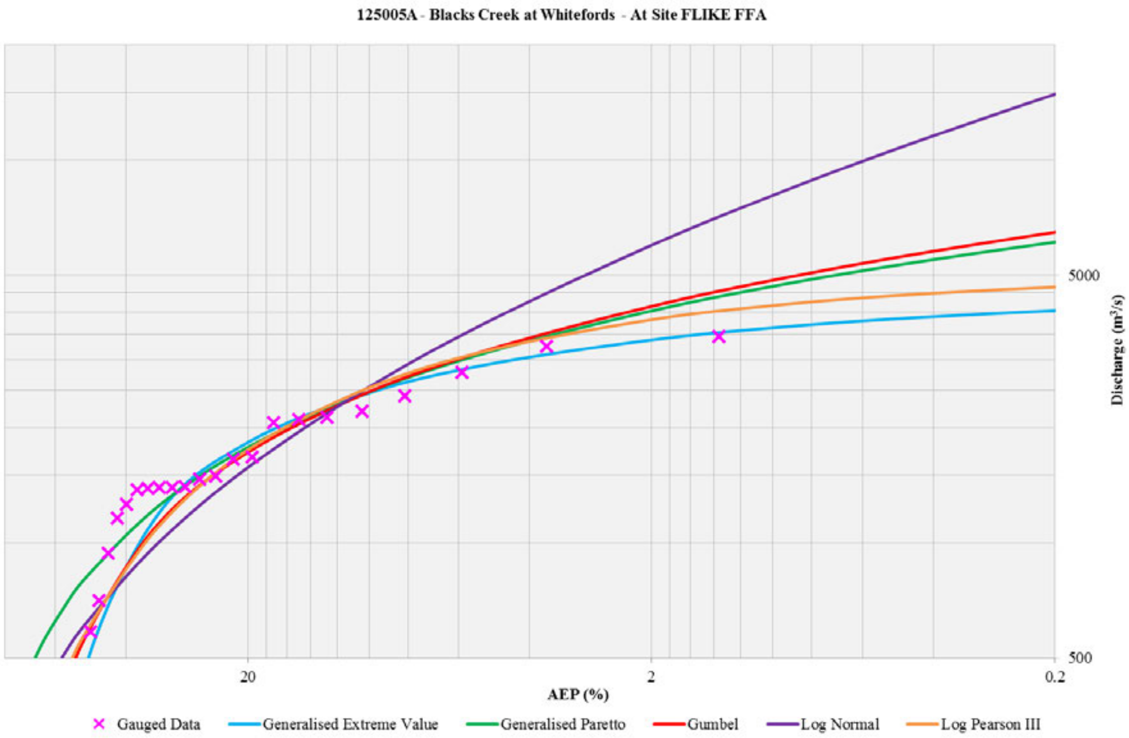

Figure 4-10: FLIKE Annual FFA Results – Blacks Creek at Whitefords

The probability model that best represents the ‘true’ flood frequency distribution at the stream gauge site was chosen to be the Generalised Extreme Value (GEV) model. The GEV model was deemed as a best fit to the recorded levels through computation to the R^2 value. The R^2 values for each probability model at the site can be found in Table 4-8.

Table 4-8: FLIKE Annual FFA Results – Blacks Creek at Whitefords: Probability Model R^2 Value

Probability Model	R^2 Value
Generalised Extreme Value	0.912
Log Pearson III	0.883
Generalised Pareto	0.862
Gumbel	0.860
Log Normal	0.695

The 90% confidence limits (exported from FLIKE) have been plotted against the GEV flood frequency distribution in Figure 4-11. This shows that there is a good correlation between the gauged data at the site and the accepted distribution curve.

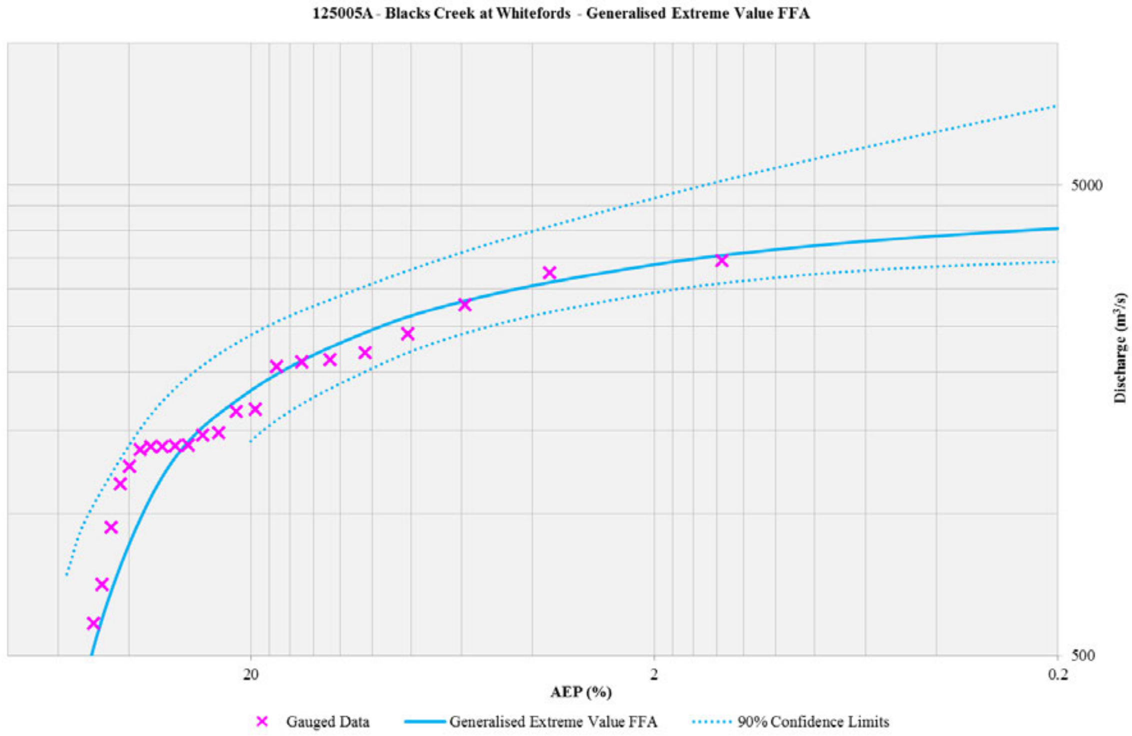

Figure 4-11: Annual Flood Frequency Distribution – Blacks Creek at Whitefords (GEV)

Discharges from the FLIKE FFA as well as the relevant confidence limits have been listed in Table 4-9.

Table 4-9: Annual FFA Estimated Quantities – Blacks Creek at Whitefords

AEP (%)	Estimated Peak Discharge (m³/s)					
	50	20	10	5	2	1
Upper 90% Confidence	1014	2398	3083	3769	4687	5431
Generalised Extreme Value	487	1826	2461	2930	3386	3642
Lower 90% Confidence	0	1426	2038	2515	2949	3175

For the partial series flood frequency analysis, a cut-off threshold of 300m^3/s was selected to return 63 peak flood events over the 44 years of gauge data on record. The distribution of the selected flood peaks over time is shown in Figure 4-12.

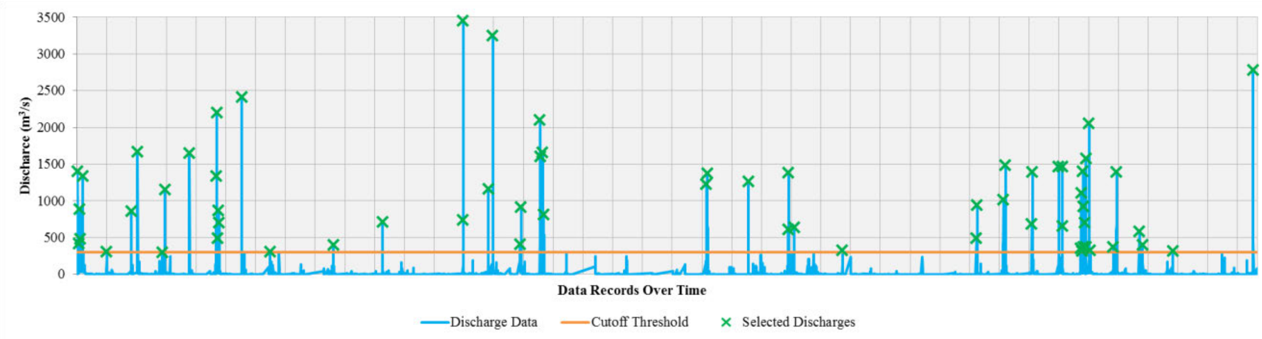

Figure 4-12: Partial Flood Frequency Analysis – Blacks Creek at Whitefords

The NExp flood frequency distribution for the selected flood peaks over threshold at the site has been plotted in Figure 4-13. The graph shows that there is a good correlation between the recorded peaks and the distribution function in the lower order events, as expected for a partial series analysis (refer Section 2.3.4).

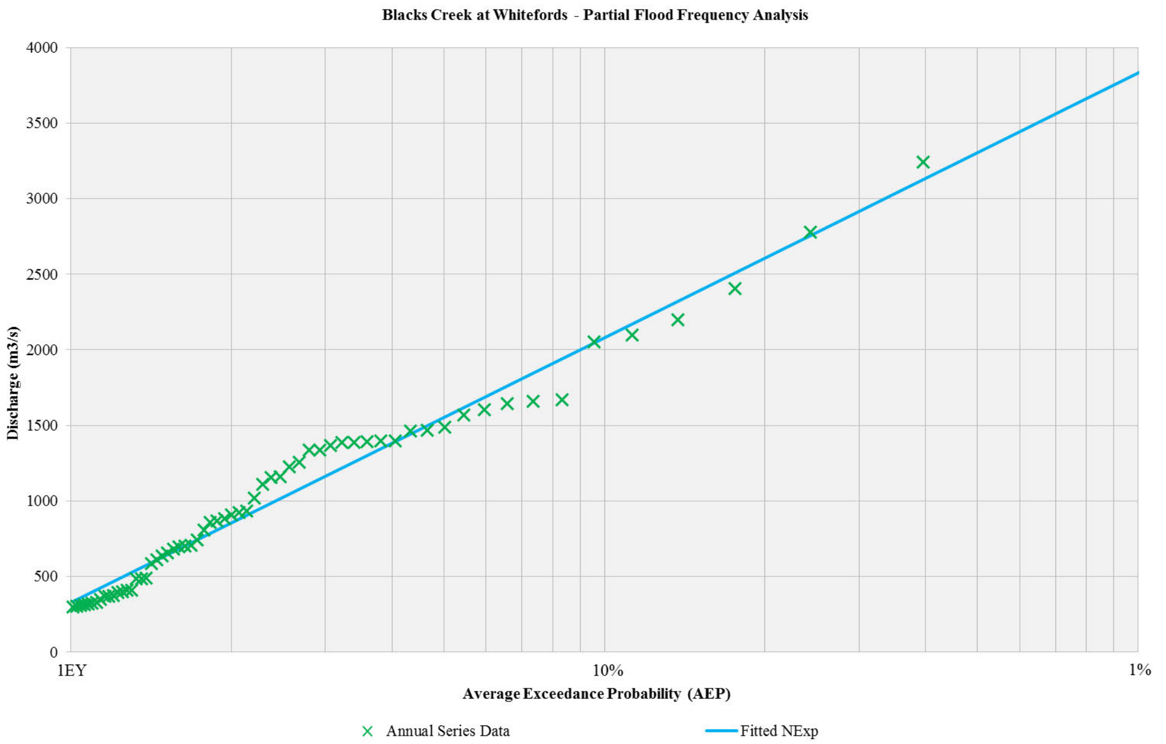

Figure 4-13: Partial Flood Frequency Distribution – Blacks Creek at Whitefords

Discharges from the partial series FFA have been listed in Table 4-10. However, it is to be noted that the results from the partial series FFA are not appropriate for events larger than 10% AEP.

Table 4-10: Partial Series FFA Estimated Quantities – Blacks Creek at Whitefords

AEP (%)	Estimated Peak Discharge (m^3/s)						
	99	**50**	**20**	**10**	**5**	**2**	**1**
Partial Series NExp	335	855	1552	2080	2607	3304	3832

4.1.4. Pioneer River at Sarichs (125002C)

The results of the FLIKE annual flood frequency analysis for the stream gauge on the Pioneer River at Sarichs (125002C) is shown in Figure 4-14.

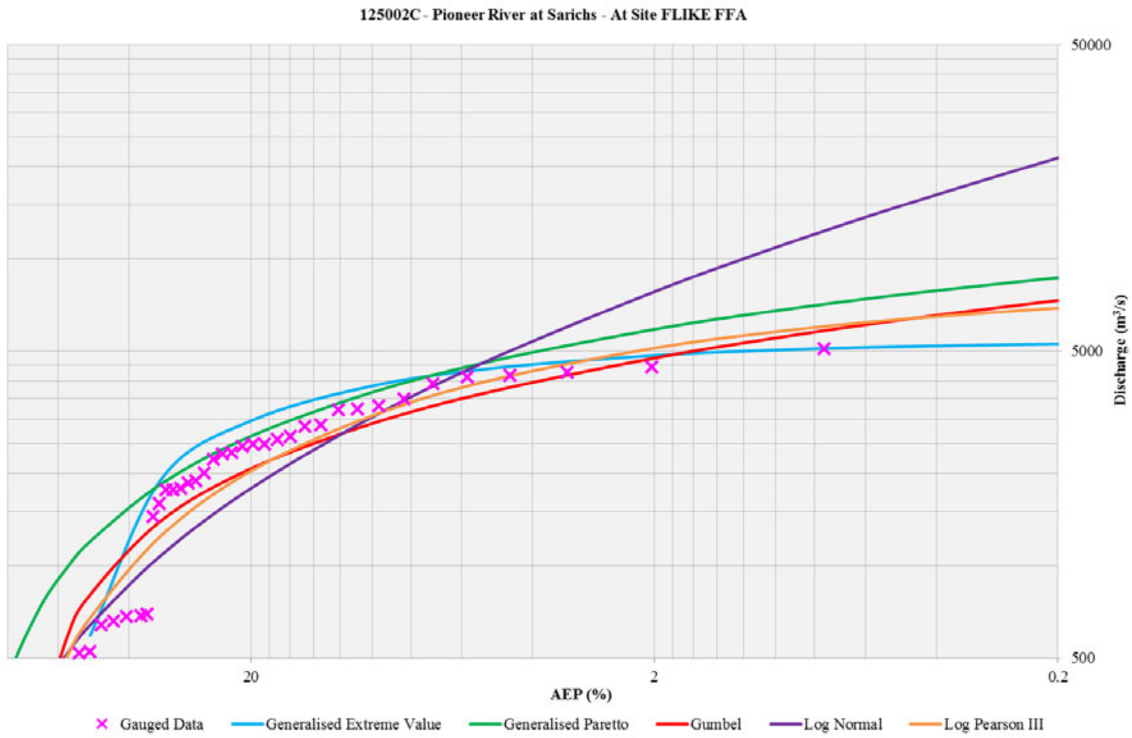

Figure 4-14: FLIKE Annual FFA Results – Pioneer River at Sarichs

The probability model that best represents the 'true' flood frequency distribution at the stream gauge site was chosen to be the Generalised Extreme Value (GEV) model. The GEV model was deemed as a best fit to the recorded levels through computation to the R^2 value. The R^2 value for each probability model at the site can be found in Table 4-11.

Table 4-11: FLIKE Annual FFA Results – Pioneer River at Sarichs: Probability Model R^2 Value

Probability Model	R^2 Value
Generalised Extreme Value	0.858
Gumbel	0.846
Log Pearson III	0.839
Generalised Pareto	0.798
Log Normal	0.626

The 90% confidence limits (exported from FLIKE) have been plotted against the Generalised Extreme Value (GEV) flood frequency distribution in Figure 4-15. This shows that there is a good correlation between the gauged data at the site and the accepted distribution curve

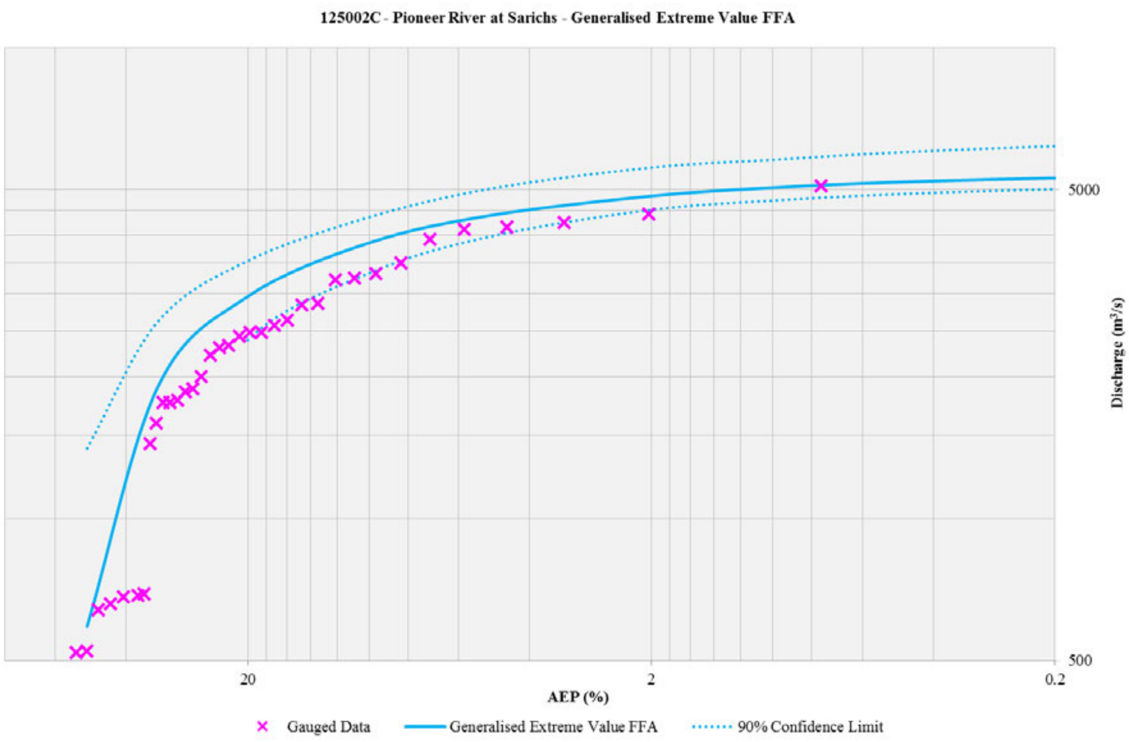

Figure 4-15: Annual Flood Frequency Distribution – Pioneer River at Sarichs (GEV)

Discharges from the FLIKE FFA as well as the relevant confidence limits have been listed in Table 4-12.

Table 4-12: Annual FFA Estimated Quantities – Pioneer River at Sarichs

AEP (%)	Estimated Peak Discharge (m^3/s)					
	50	20	10	5	2	1
Upper 90% Confidence	1410	3527	4367	4020	5567	5783
Generalised Extreme Value	591	2963	3852	4403	4843	5045
Lower 90% Confidence	0	2395	3321	3975	4525	4737

For the partial series flood frequency analysis, a cut-off threshold of $1{,}000m^3/s$ was selected to return 65 peak flood events over the 55 years of gauge data on record. The distribution of the selected flood peaks over time is shown in Figure 4-16.

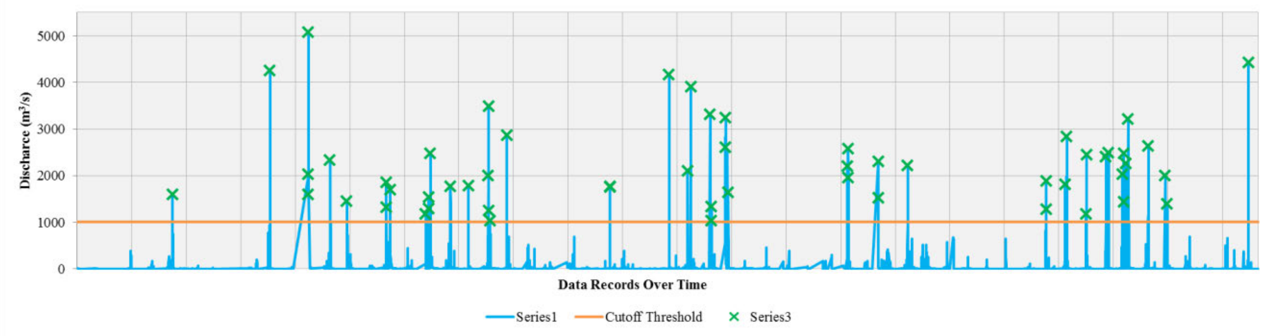

Figure 4-16: Partial Flood Frequency Analysis – Pioneer River at Sarichs

The NExp flood frequency distribution for the selected flood peaks over threshold at the site has been plotted in Figure 4-17. The graph shows that there is a good correlation between the recorded peaks and the distribution function in the lower order events, as expected for a partial series analysis (refer Section 2.3.4).

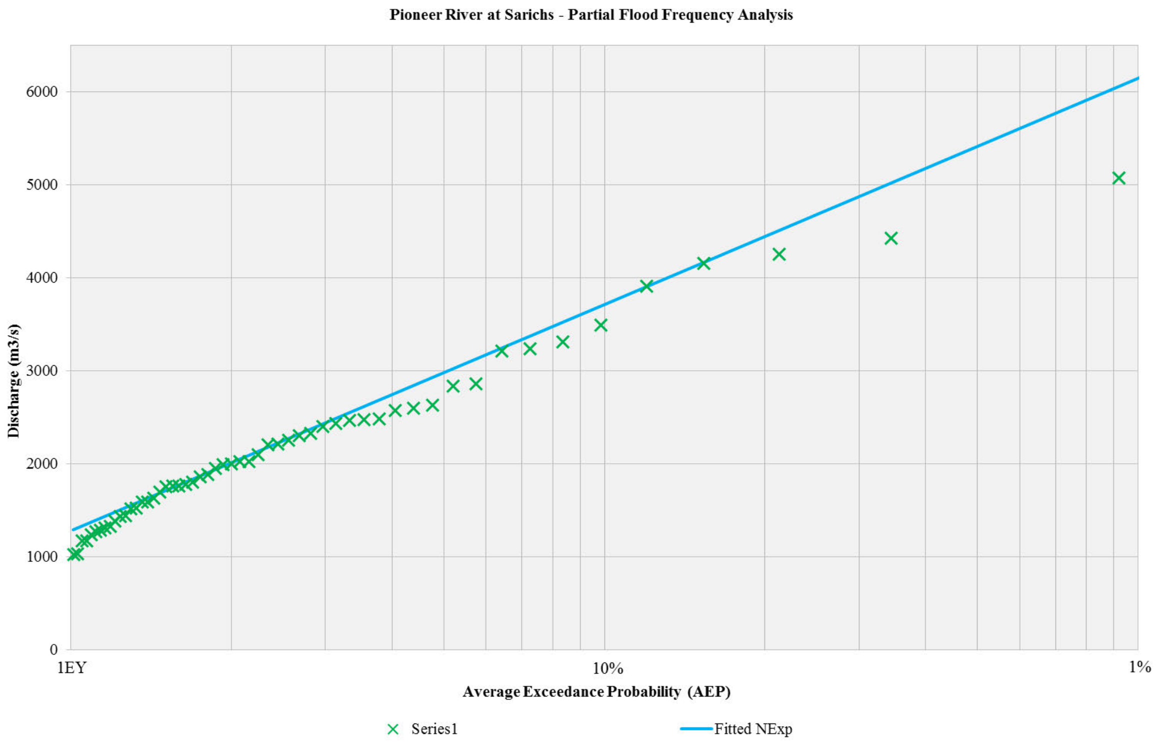

Figure 4-17: Partial Flood Frequency Distribution – Pioneer River at Sarichs

Discharges from the partial series FFA have been listed in Table 4-13. However, it is to be noted that the results from the partial series FFA are not appropriate for events larger than 10% AEP.

Table 4-13: Partial Series FFA Estimated Quantities – Pioneer River at Sarichs

AEP (%)	Estimated Peak Discharge (m^3/s)						
	99	50	20	10	5	2	1
Partial Series NExp	1292	2013	2981	3712	4444	5411	6143

4.1.5. Pioneer River at Mirani Weir Tailwater (125007A)

The results of the FLIKE annual flood frequency analysis for the stream gauge the Pioneer River at the Mirani Weir Tailwater (125007A) is shown in Figure 4-18.

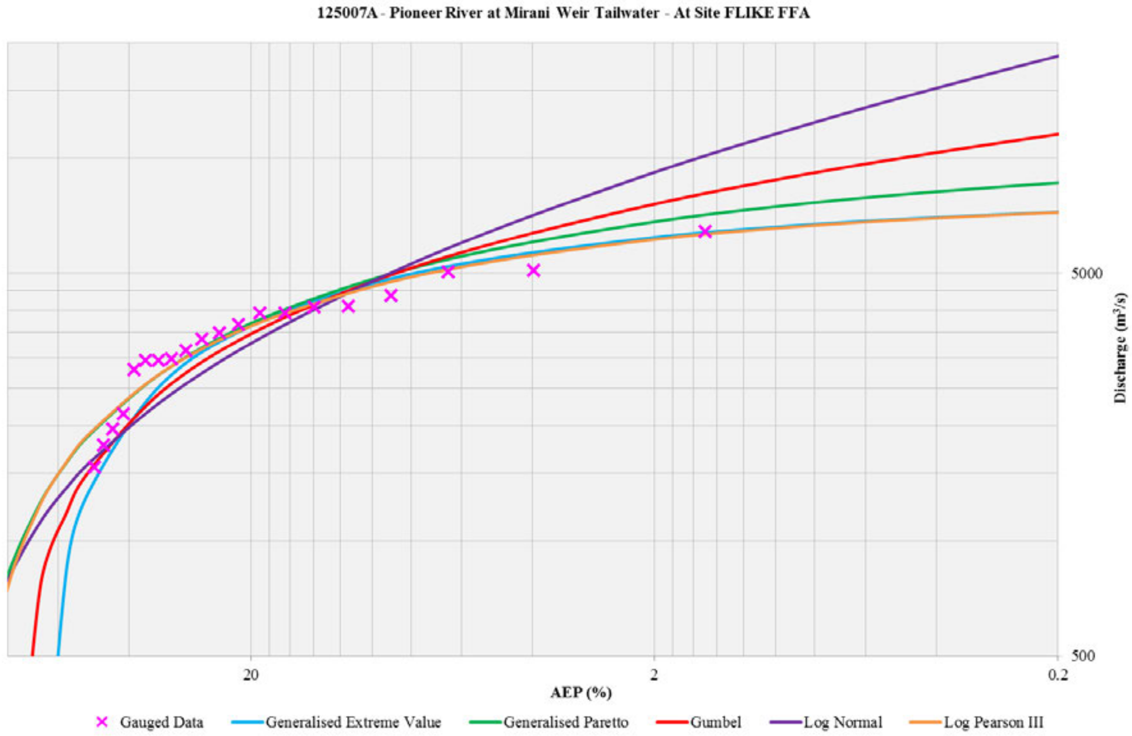

Figure 4-18: FLIKE Annual FFA Results – Pioneer River at Mirani Weir Tailwater

The probability model that best represents the 'true' flood frequency distribution at the stream gauge site was chosen to be the Generalised Extreme Value (GEV) model. The GEV model was deemed as a best fit to the recorded levels through computation to the R^2 value. The R^2 values for each probability model at the site can be found in Table 4-14.

Table 4-14: FLIKE Annual FFA Results – Pioneer River at Mirani Weir Tailwater: Probability Model R^2 Value

Probability Model	R^2 Value
Generalised Extreme Value	0.936
Log Pearson III	0.931
Generalised Pareto	0.907
Gumbel	0.871
Log Normal	0.774

The 90% confidence limits (exported from FLIKE) have been plotted against the GEV flood frequency distribution in Figure 4-19. This shows that there is a good correlation between the gauged data at the site and the accepted distribution curve

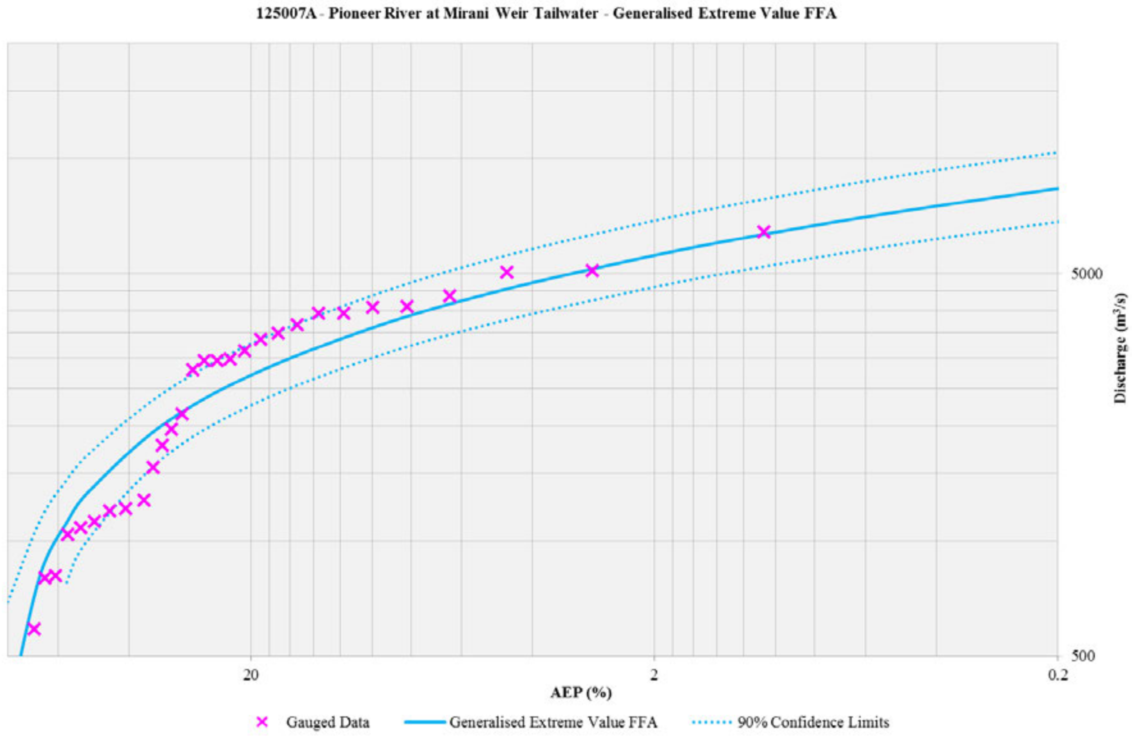

Figure 4-19: Annual Flood Frequency Distribution – Pioneer River at Mirani Weir Tailwater (GEV)

Discharges from the FLIKE FFA as well as the relevant confidence limits have been listed in Table 4-15.

Table 4-15: Annual FFA Estimated Quantities – Pioneer River at Mirani Weir Tailwater

AEP (%)	**Estimated Peak Discharge (m^3/s)**					
	50	**20**	**10**	**5**	**2**	**1**
Upper 90% Confidence	2289	4600	5782	6834	8085	9225
Generalised Extreme Value	1362	3637	4692	5462	6195	6600
Lower 90% Confidence	359	2940	4007	4771	5496	5865

For the partial series flood frequency analysis, a cut-off threshold of 1,000m^3/s was selected to return 63 peak flood events over the 39 years of gauge data on record. The distribution of the selected flood peaks over time is shown in Figure 4-20.

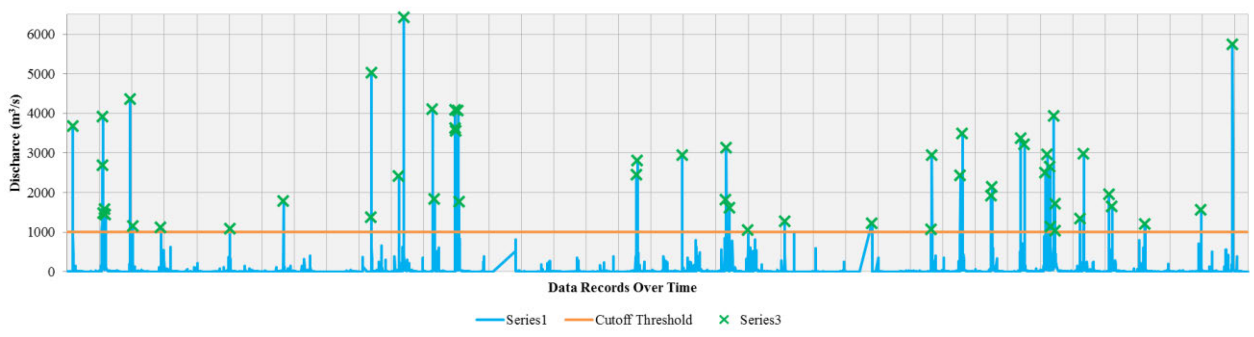

Figure 4-20: Partial Flood Frequency Analysis – Pioneer River at Mirani Weir Tailwater

The NExp flood frequency distribution for the selected flood peaks over threshold at the site has been plotted in Figure 4-21. The graph shows that there is a good correlation between the recorded peaks and the distribution function in the lower order events, as expected for a partial series analysis (refer Section 2.3.4).

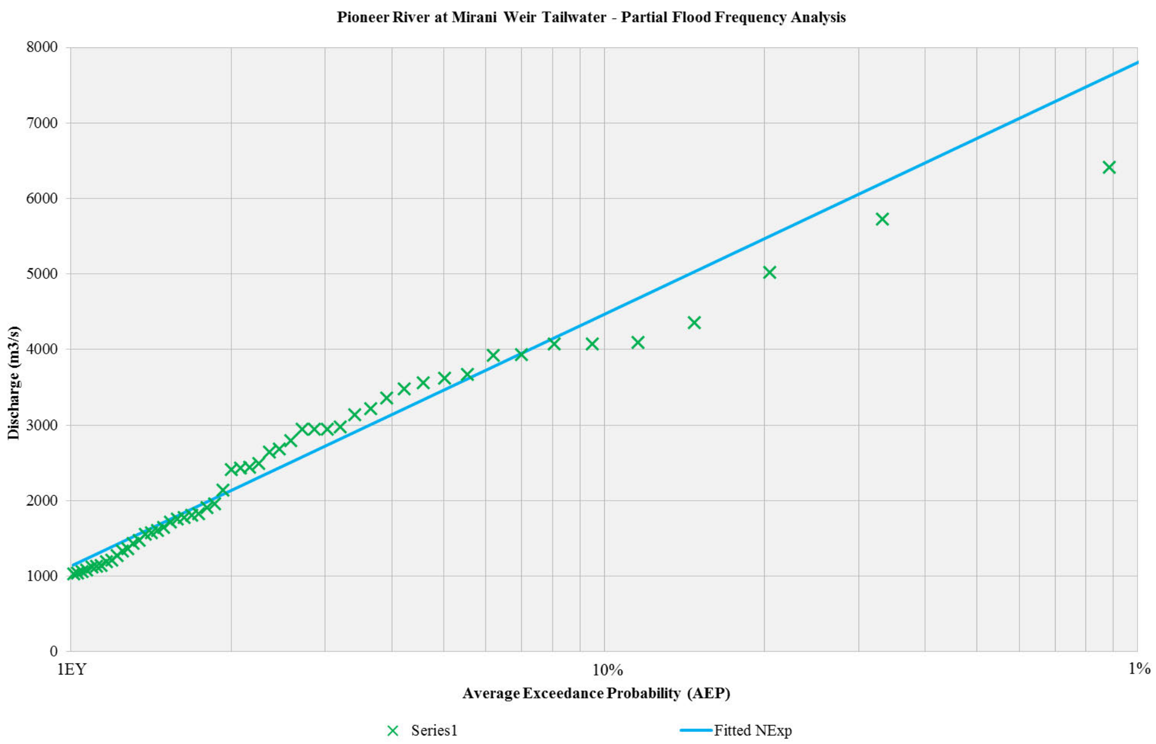

Figure 4-21: Partial Flood Frequency Distribution – Pioneer River at Mirani Weir Tailwater

Discharges from the partial series FFA have been listed in Table 4-16. However, it is to be noted that the results from the partial series FFA are not appropriate for events larger than 10% AEP.

Table 4-16: Partial Series FFA Estimated Quantities – Pioneer River at Mirani Weir Tailwater

AEP (%)	Estimated Peak Discharge (m^3/s)						
	99	**50**	**20**	**10**	**5**	**2**	**1**
Partial Series NExp	1147	2136	3463	4467	5470	6796	7801

4.2. Stage-Storage-Discharge Relationships for Structures

There are four large storages situated along the Pioneer River that have a pivotal impact on the discharge at downstream locations. It is important that these storages are modelled correctly to accurately represent the conditions that would be experienced on site. For this, stage-storage-discharge relationships were developed from sourced data and are described in more detail in the subsequent sections.

4.2.1. Teemburra Dam

The stage-storage-discharge relationship for Teemburra Dam is shown in Figure 4-22. Teemburra Dam is located in the upper Pioneer River catchment situated on Teemburra Creek. The dam has a storage capacity of approximately 147,500 ML at its spillway level of 290 mAHD. The data in shown in Figure 4-22 was sourced from the BoM and was presented as part of the Pioneer River Flood Study (WRM Water & Environment 2011).

Figure 4-22: Stage-Storage-Discharge Relationship for Teemburra Dam (WRM, 2011)

4.2.2. Mirani Weir

The stage-storage-discharge relationship for the Mirani Weir is shown in Figure 4-23. The Mirani Wier is located on the Pioneer River in the upper reaches to the south west of the Mirani Township. The weir has a storage capacity of approximately 4,600 ML at its spillway level of 47 mAHD. The data shown in Figure 4-23 was obtained from DERM and was presented as part of the Pioneer River Flood Study (WRM Water & Environment 2011).

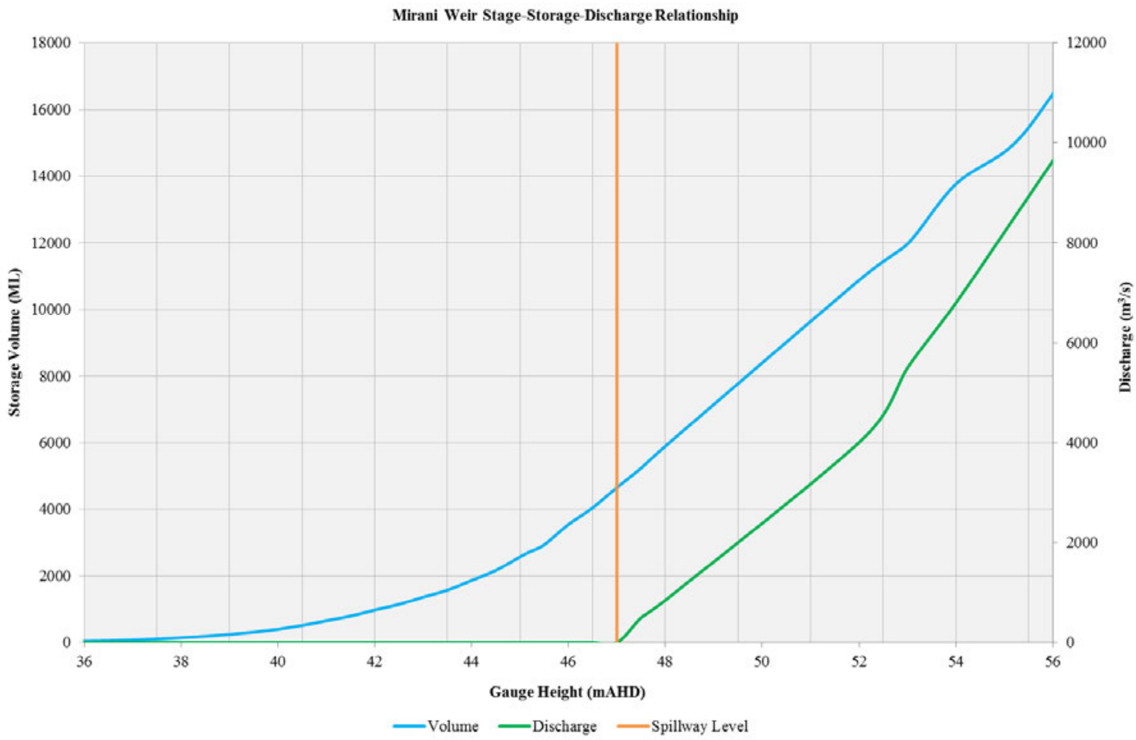

Figure 4-23: Stage-Storage-Discharge Relationship for Mirani Weir (WRM, 2011)

4.2.3. Marian Weir

The stage-storage-discharge relationship for the Marian Weir is shown in Figure 4-24. The Marian Wier is located on the Pioneer River in the middle reaches to the west of the Marian Township. The weir has a storage capacity of approximately 3,830 ML at its spillway level of 31.9 mAHD. The data shown in Figure 4-24 was obtained from DERM and was presented as part of the Pioneer River Flood Study (WRM Water & Environment 2011).

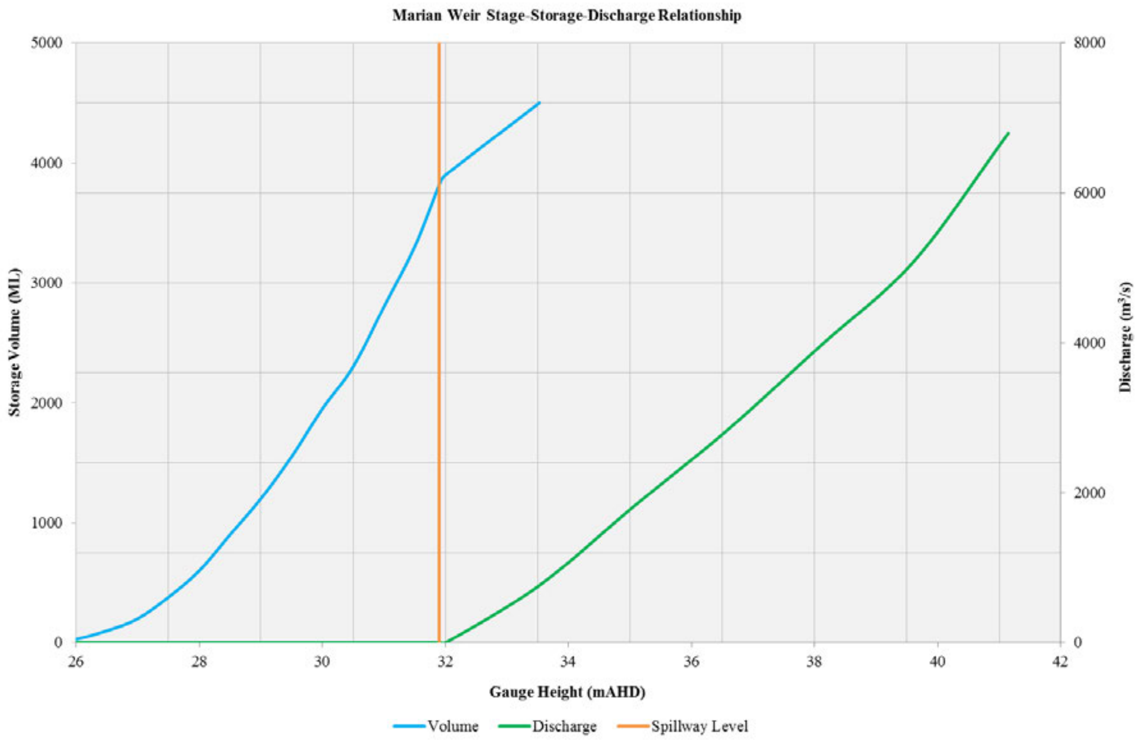

Figure 4-24: Stage-Storage-Discharge Relationship for Marian Weir (WRM, 2011)

4.2.4. Dumbleton Rocks Weir

The stage-storage-discharge relationship for the Dumbleton Rocks Weir is shown in Figure 4-25. The Dumbleton Rocks Weir is located on the Pioneer River in the middle to lower reaches to the south of the Dumbleton Township. The weir has a storage capacity of approximately 6,540 ML at its spillway level of 14.4 mAHD. The data shown in Figure 4-25 was obtained from DERM and was presented as part of the Pioneer River Flood Study (WRM Water & Environment 2011).

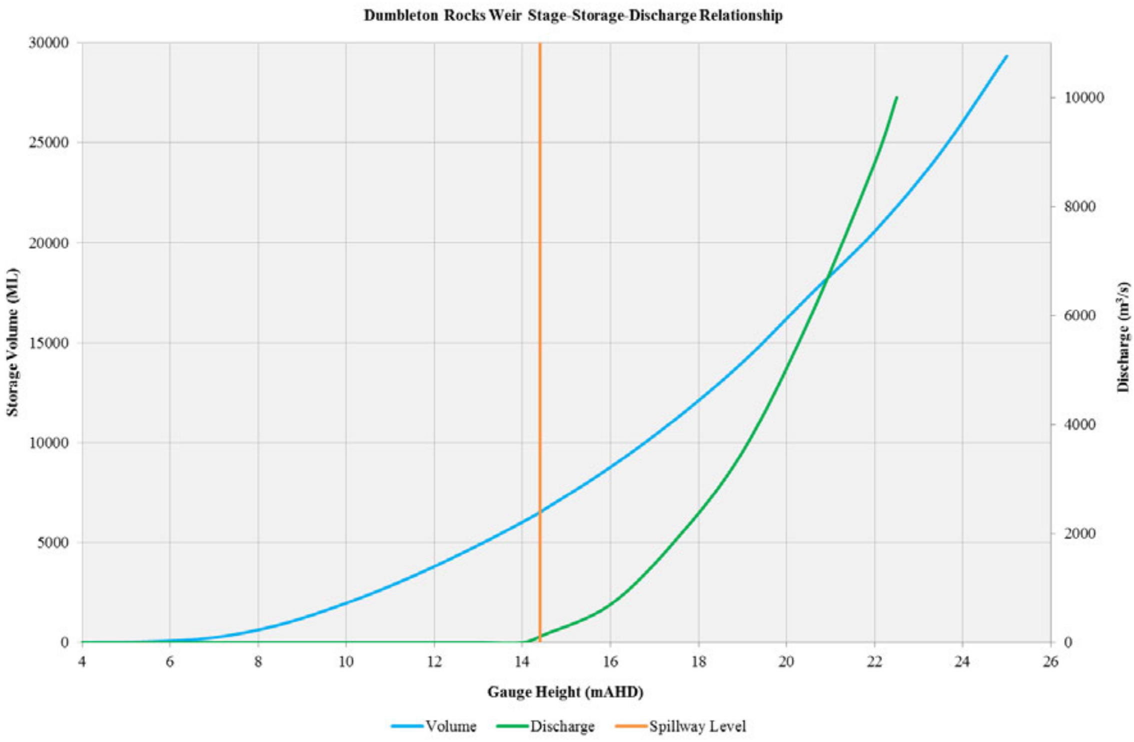

Figure 4-25: Stage-Storage-Discharge Relationship for Dumbleton Rocks Weir (WRM, 2011)

4.3. Runoff Routing Model Development

As per the methodology outlined in Section 3.5 a runoff routing model was developed using the Watershed Bounded Network Model (WBNM) (refer Section 2.6.6) to estimate the discharge hydrograph at subcatchments within the wider Pioneer River catchment.

The subsequent sections display the results of the model calibration to historical events as well as the design flood hydrographs extracted at key locations (refer Figure 4-1) for both ARR 1987 and 2016 hydrological inputs.

4.3.1. Model Calibration

To ensure that the runoff routing model was accurately representing the catchment conditions the model was calibrated against two historical rain events (refer Section 3.2). The following figures show the results of the calibration, comparing the modelled hydrograph (solid line) to the recorded hydrograph (dotted line) at the same location.

Overall, the model calibrated well to the historical events providing confidence in the findings of the investigation. A significant variance between the modelled results and recorded flows can be seen for the 2008 event at Bakers Creek (Figure 4-33). As found from the Pioneer River Flood Study (WRM Water & Environment 2011) the gauge was reported to be faulty for this event and underestimated recorded levels. A peak of 4.15mAHD was measured at the gauge in the February 2008 event, whereas surveyed flood marks in the area reported that the creek may have peaked at over 4.6mAHD.

The gauges located in the downstream reaches of the river at the Hospital Bridge and Mackay Alert (refer Figure 4-30 and Figure 4-31) have recorded tidal peaks of the river for both events prior to the storm peak. These were not evident in the model as the tidal conditions for the event were not incorporated into the calibration model.

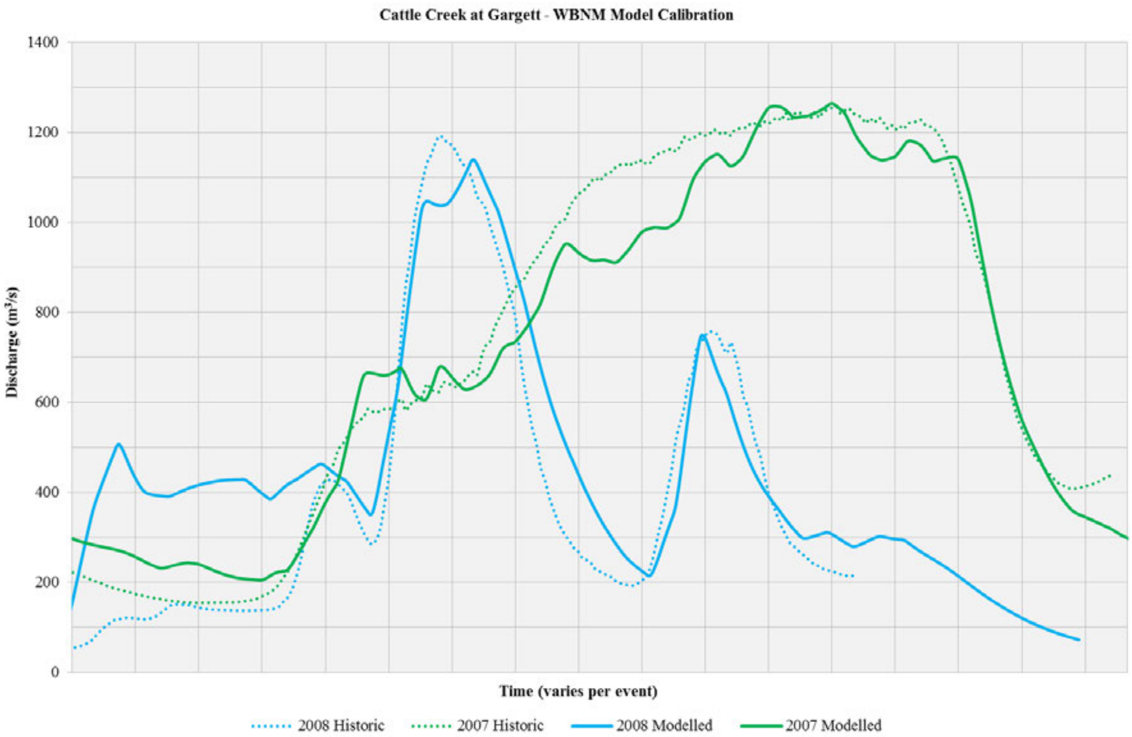

Figure 4-26: WBNM Model Calibration - Cattle Creek at Gargett

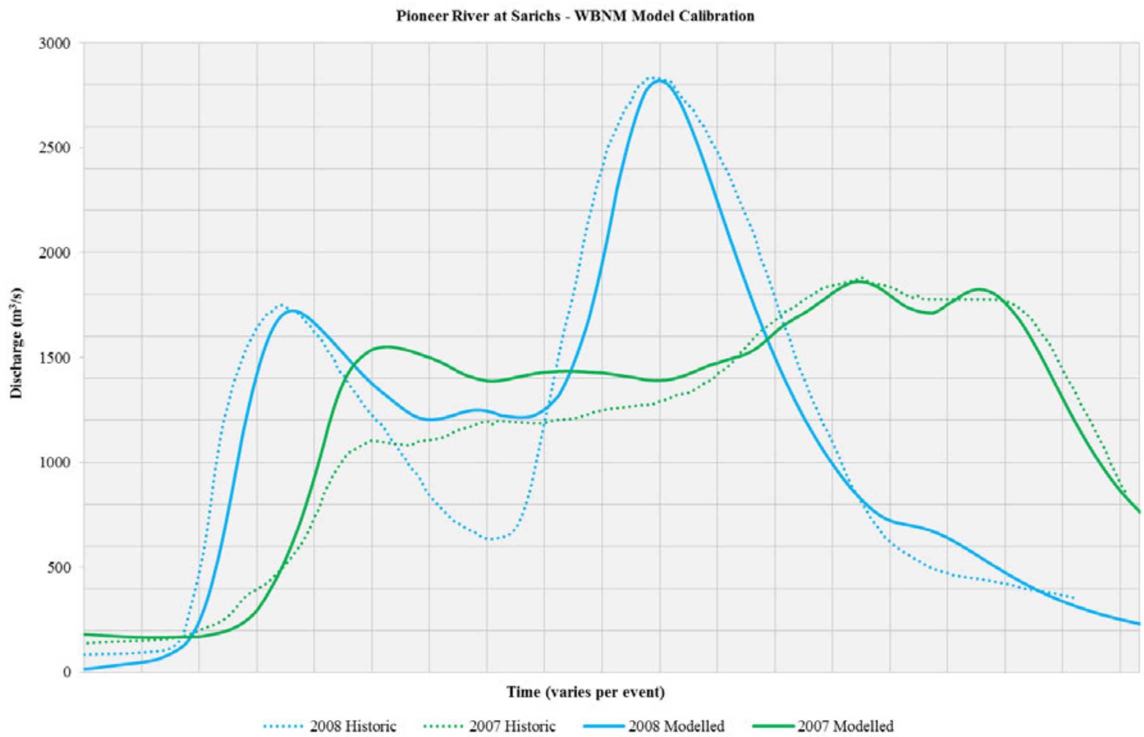

Figure 4-27: WBNM Model Calibration - Pioneer River at Sarichs

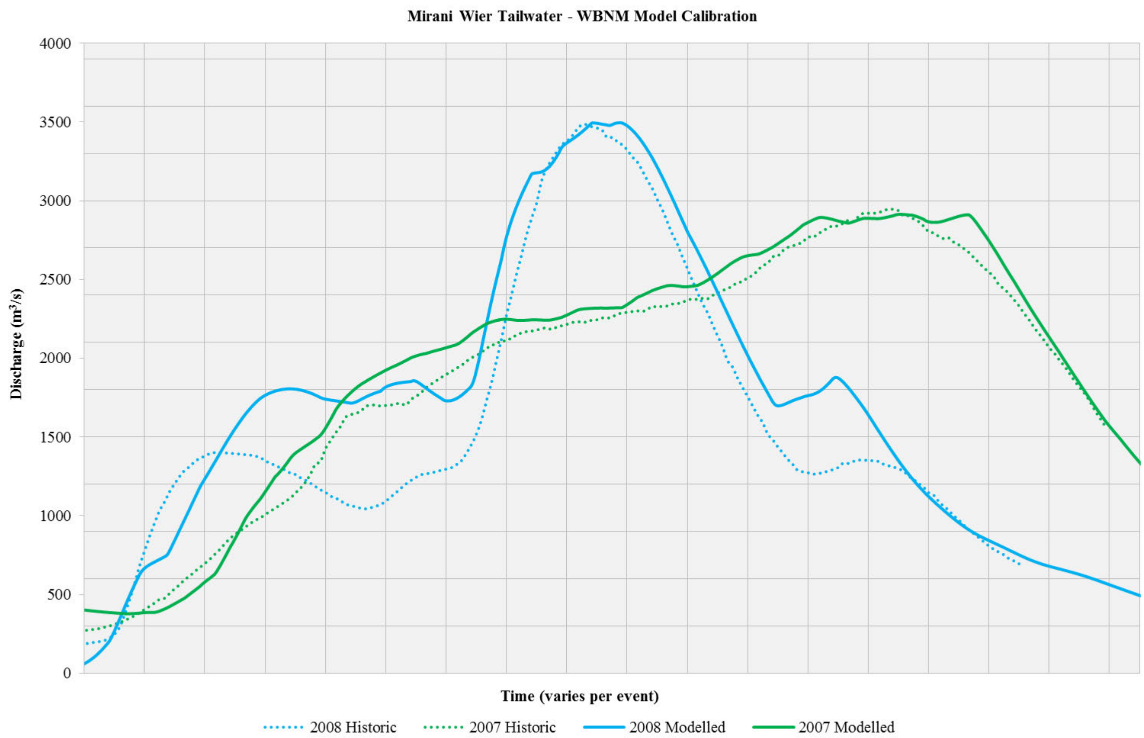

Figure 4-28: WBNM Model Calibration - Mirani Weir Tailwater

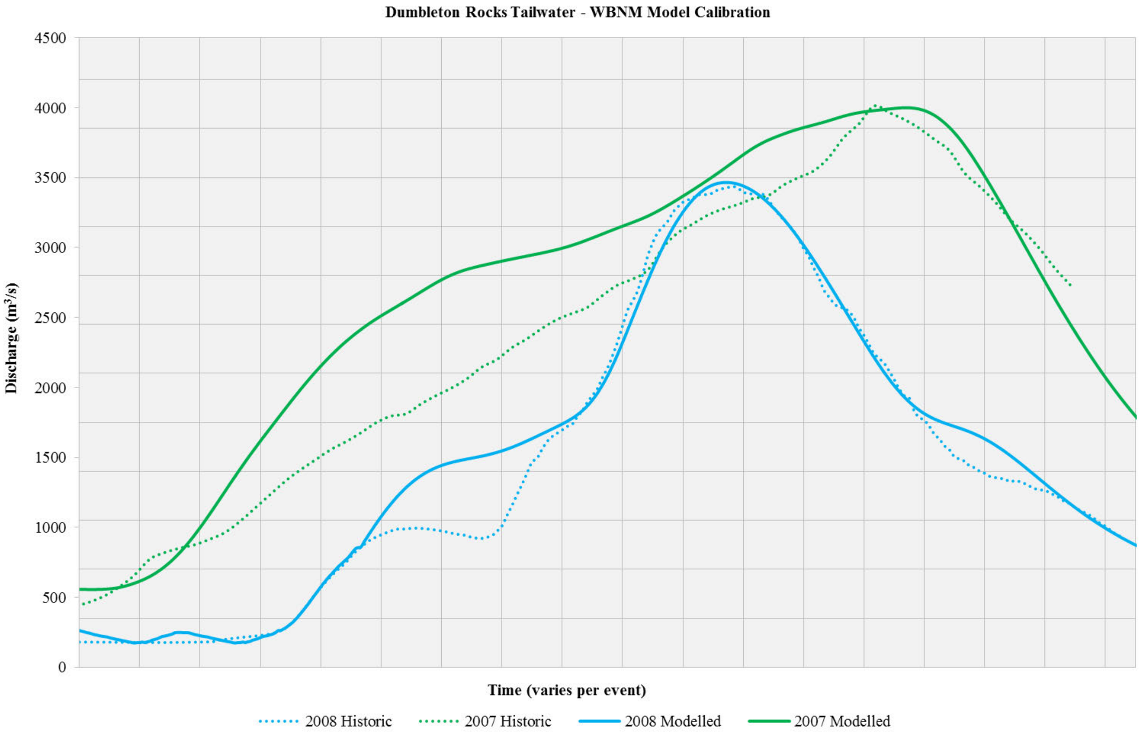

Figure 4-29: WBNM Model Calibration - Dumbleton Rocks Tailwater

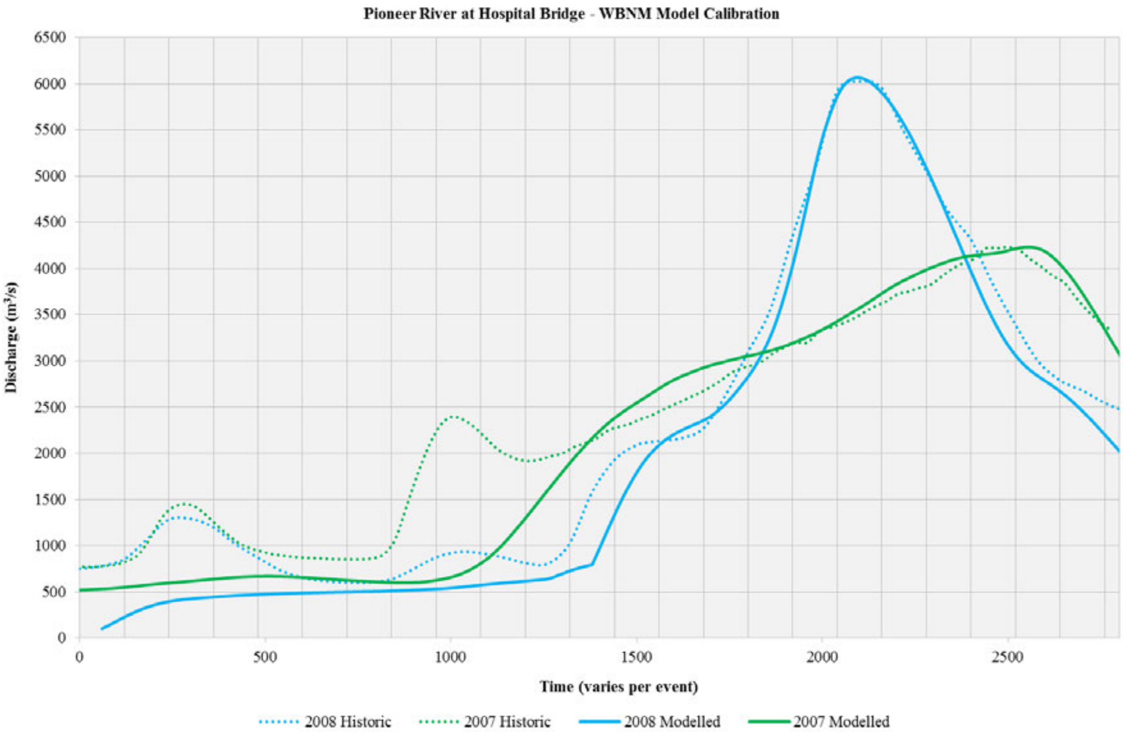

Figure 4-30: WBNM Model Calibration - Pioneer River at Hospital Bridge

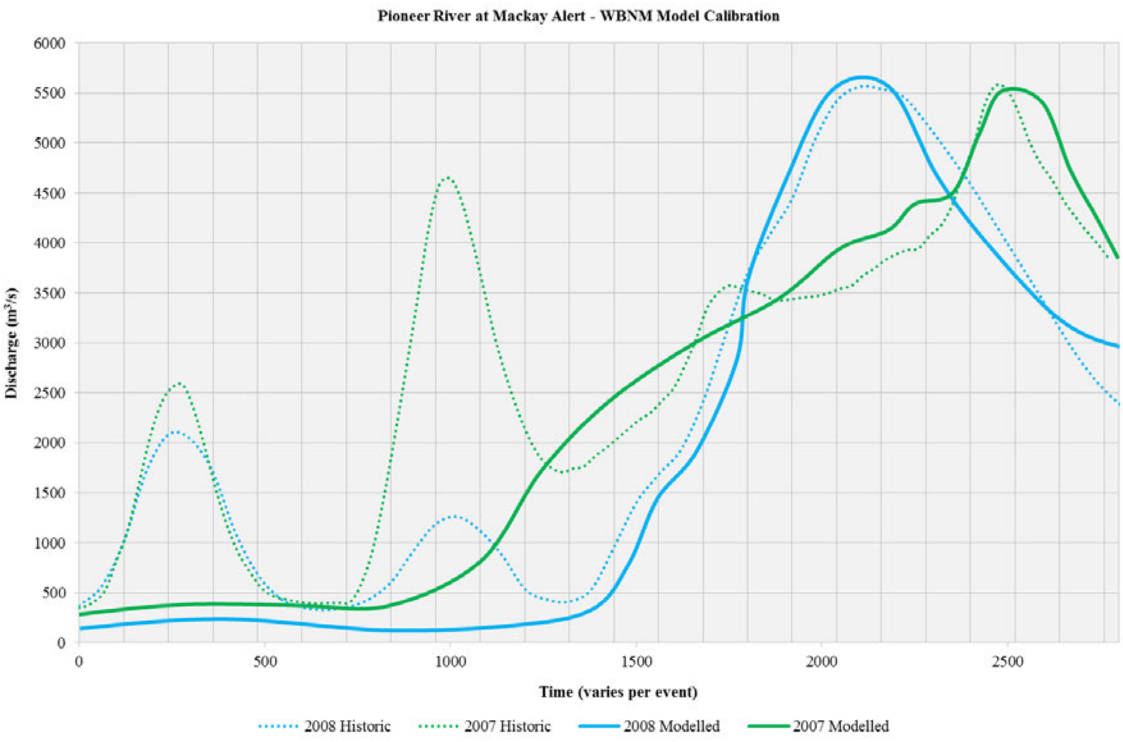

Figure 4-31: WBNM Model Calibration - Pioneer River at Mackay Alert

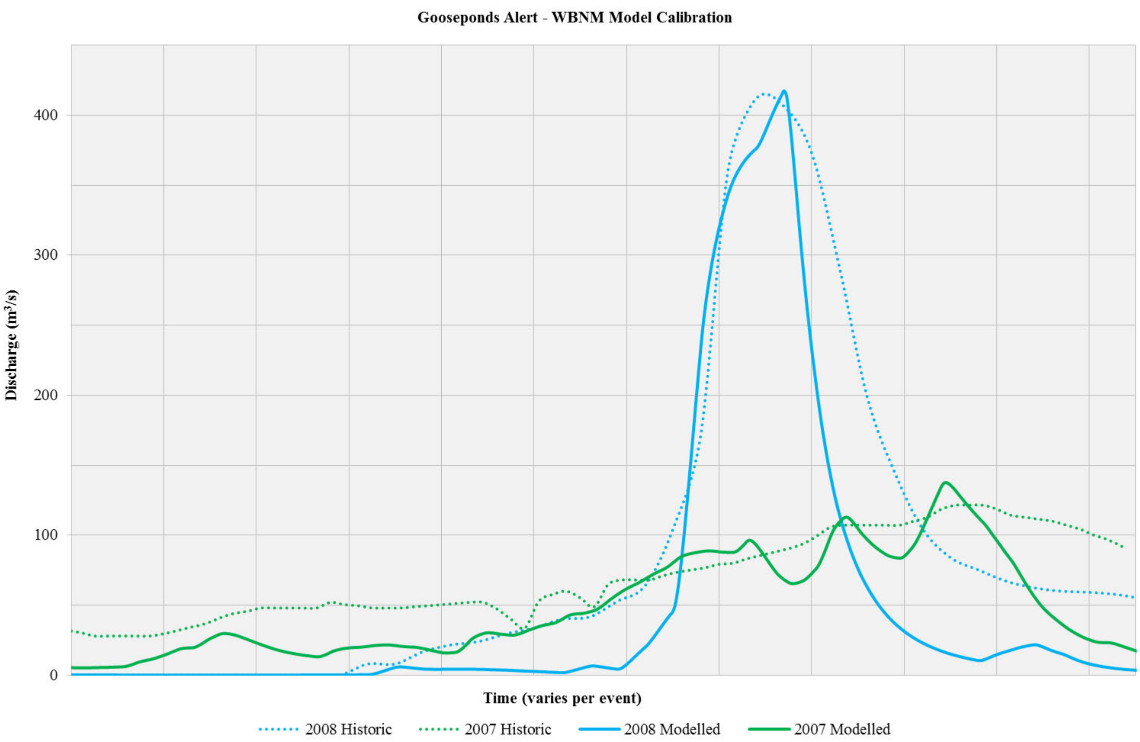

Figure 4-32: WBNM Model Calibration – Gooseponds Creek at Gooseponds Alert

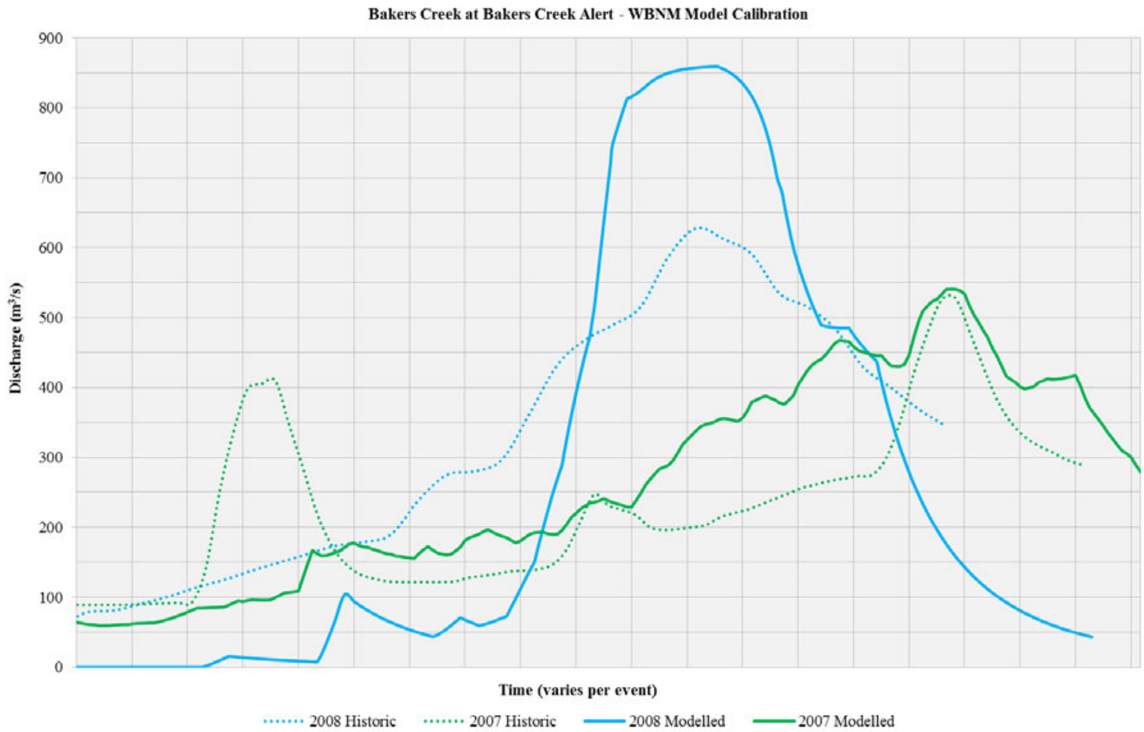

Figure 4-33: WBNM Model Calibration - Bakers Creek at Bakers Creek Alert

4.3.2. ARR1987 Critical Duration Assessment

To ease the amount of modelling required, a critical duration assessment was undertaken on the 1% AEP (100 year ARI event). This methodology is in line with current industry procedures and ensures that the storm duration that delivers the 'worst case' flood peak is identified.

For this, a number of design storm durations were modelled and assessed using the WBNM runoff routing model. When using the ARR 1897 hydrologic inputs the durations modelled were the 12, 24, 36 and 48 hour storm events.

The 24 hour (1440 minute) storm was found to produce the highest peak discharge at all assessment locations and therefore was taken forward as the critical duration storm for the Pioneer River catchment (when applying ARR 1987 inputs).

The results of the critical duration assessment for each location can be found in Appendix E.

4.3.3. ARR2016 Critical Duration Assessment

As per Section 4.3.2, a number of design storm durations were modelled and assessed using the WBNM runoff routing model to determine the critical duration storm. When using the ARR 2016 hydrologic inputs the durations modelled were the 3, 6, 12, 18 and 24 hour storm events.

The 12 hour (720 minute) storm was found to produce the highest peak discharge at the majority of assessment locations and therefore was taken forward as the critical duration storm for the Pioneer River catchment (when applying ARR 2016 inputs).

The results of the critical duration assessment for each location can be found in Appendix E.

4.3.4. ARR1987 Results

The following figures show the resultant flood hydrographs from the WBNM catchment modelling using the hydrologic parameters from the ARR 1987 guidelines (refer Section 3.4.1). Results have been generated for each assessment location as shown in Figure 4-1. The peak discharge values for each location have been summarised in the comparison of results table for each location (refer Chapter 5).

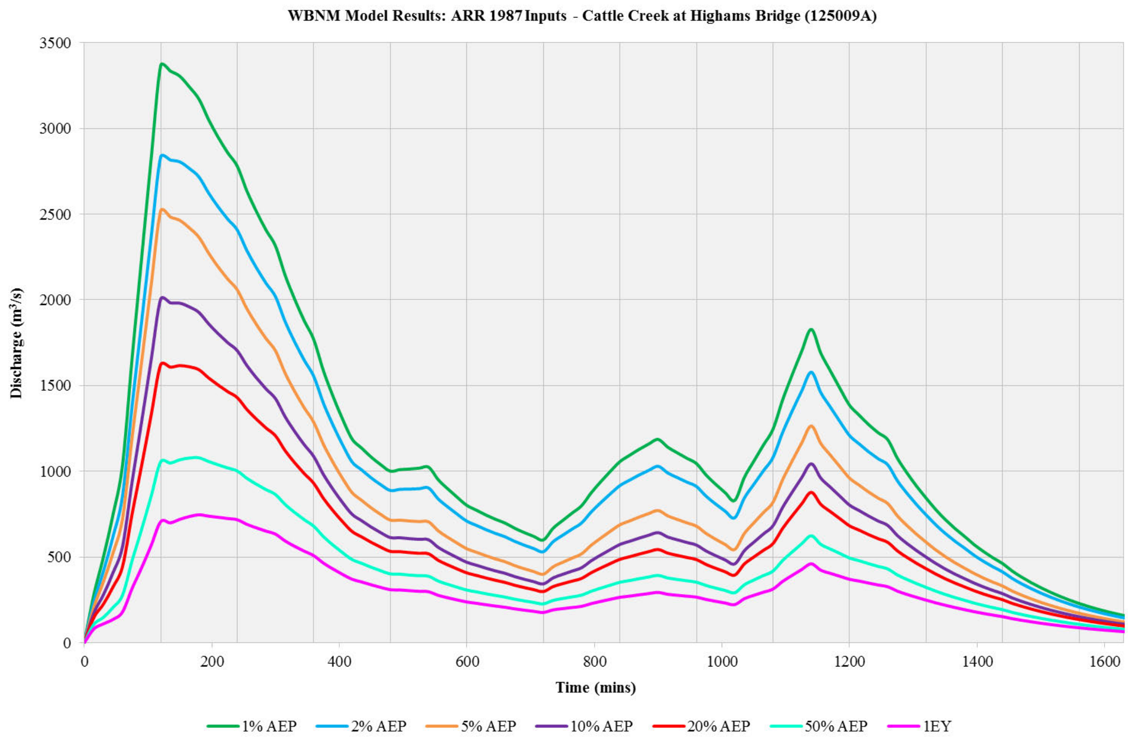

Figure 4-34: Runoff Routing Model Flood Hydrographs (ARR 1987) – Cattle Creek at Highams Bridge

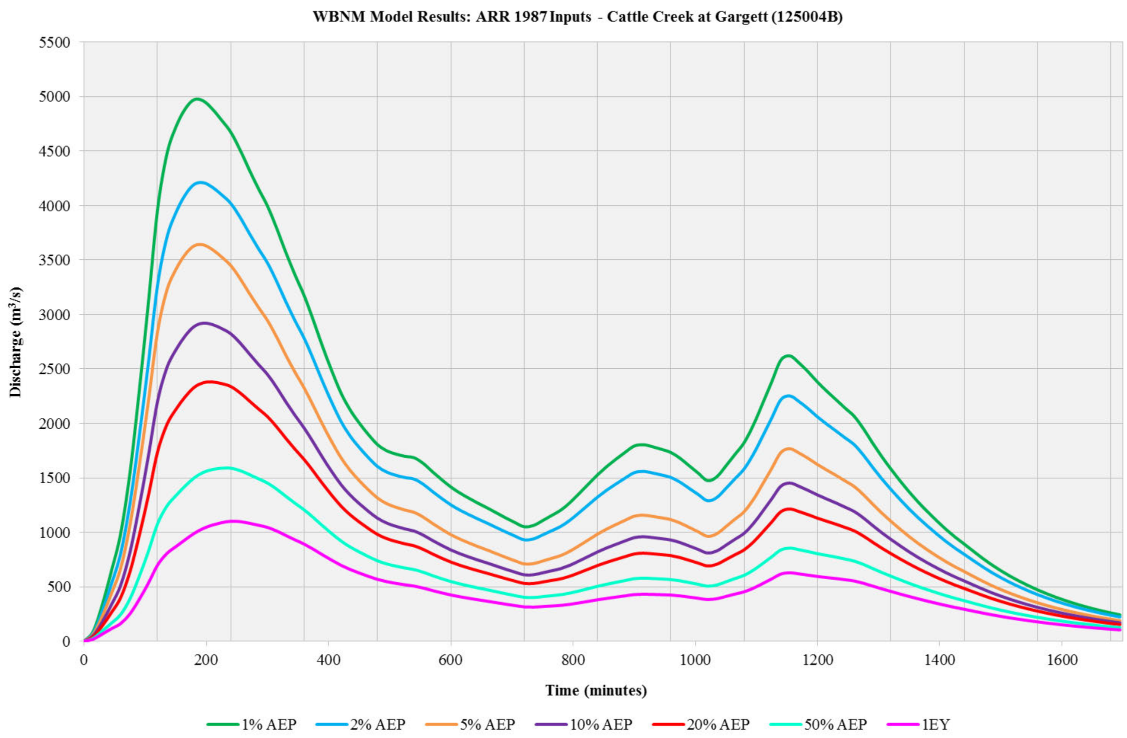

Figure 4-35: Runoff Routing Model Flood Hydrographs (ARR 1987) – Cattle Creek at Gargett

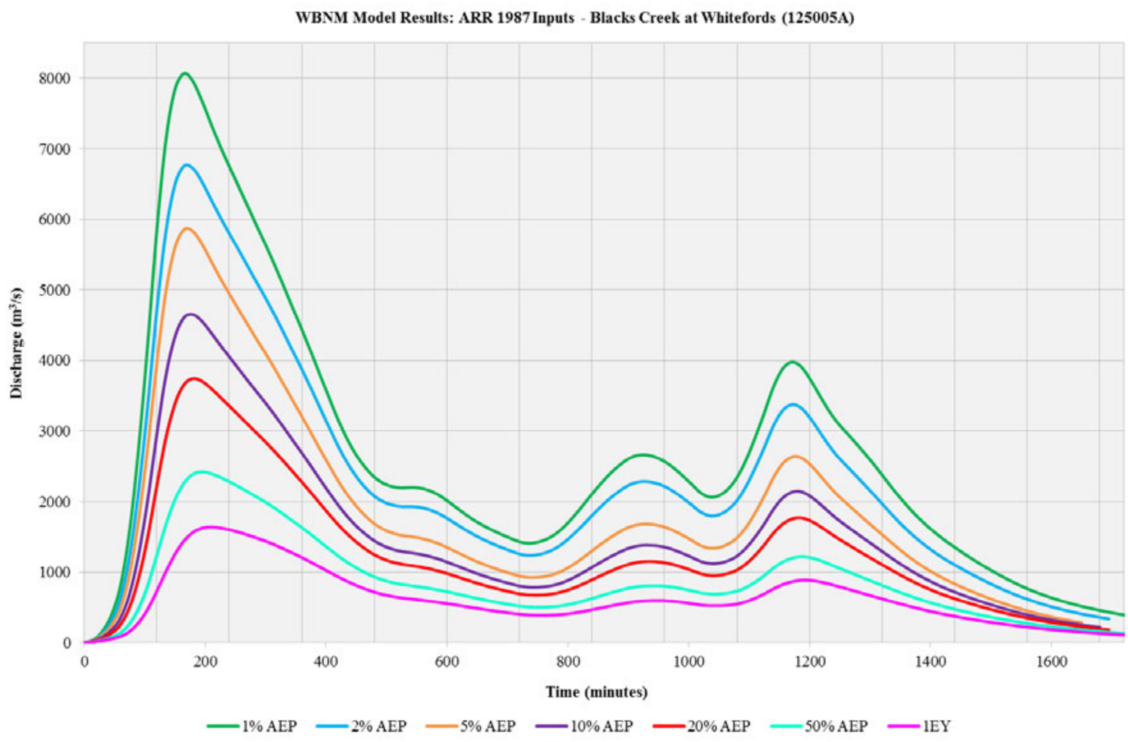

Figure 4-36: Runoff Routing Model Flood Hydrographs (ARR 1987) – Blacks Creek at Whitefords

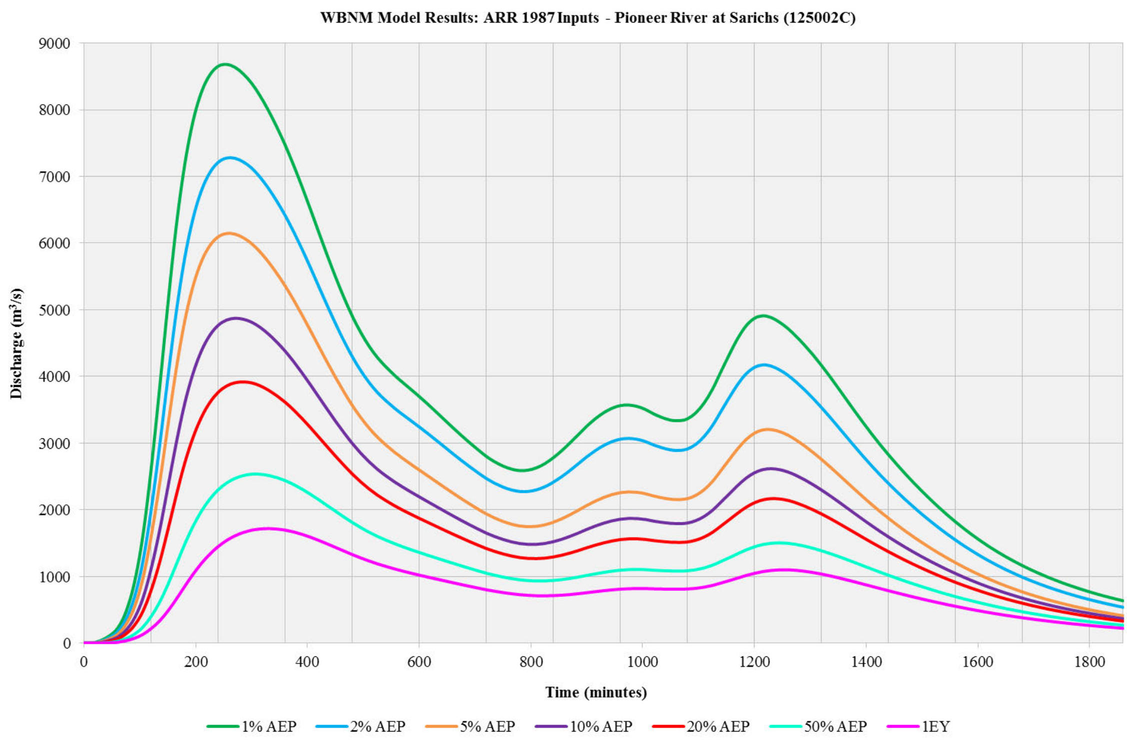

Figure 4-37: Runoff Routing Model Flood Hydrographs (ARR 1987) – Pioneer River at Sarichs

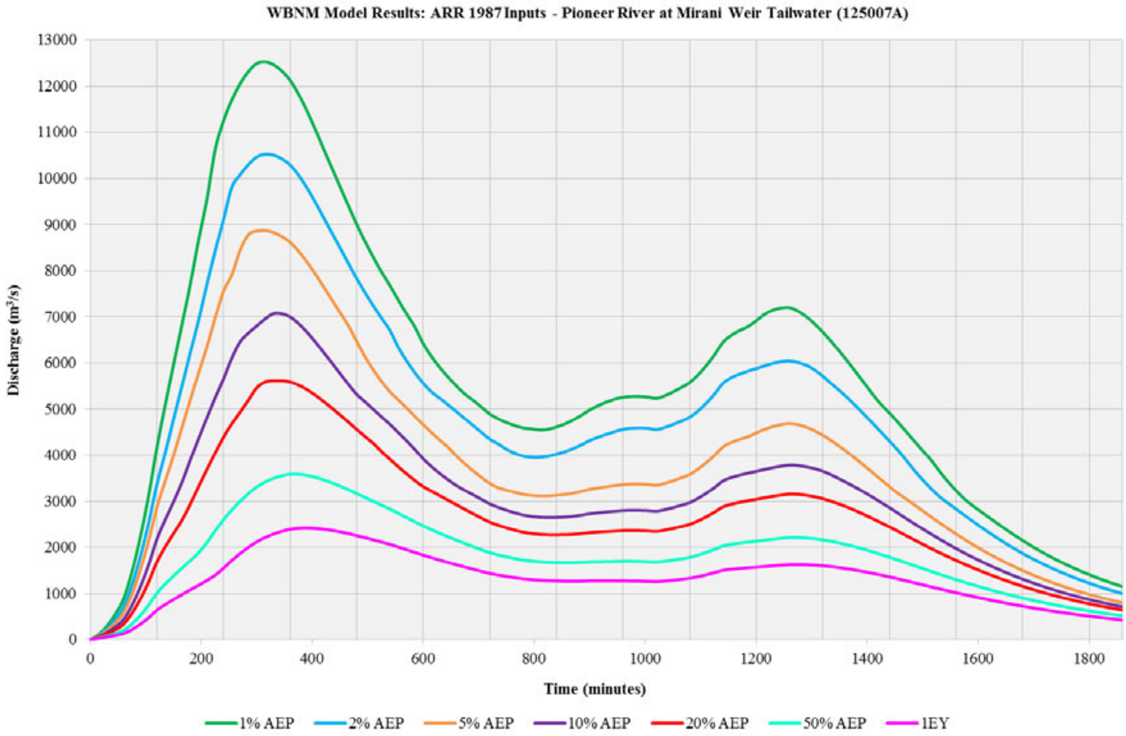

Figure 4-38: Runoff Routing Model Flood Hydrographs (ARR 1987) – Pioneer River at Mirani Weir Tailwater

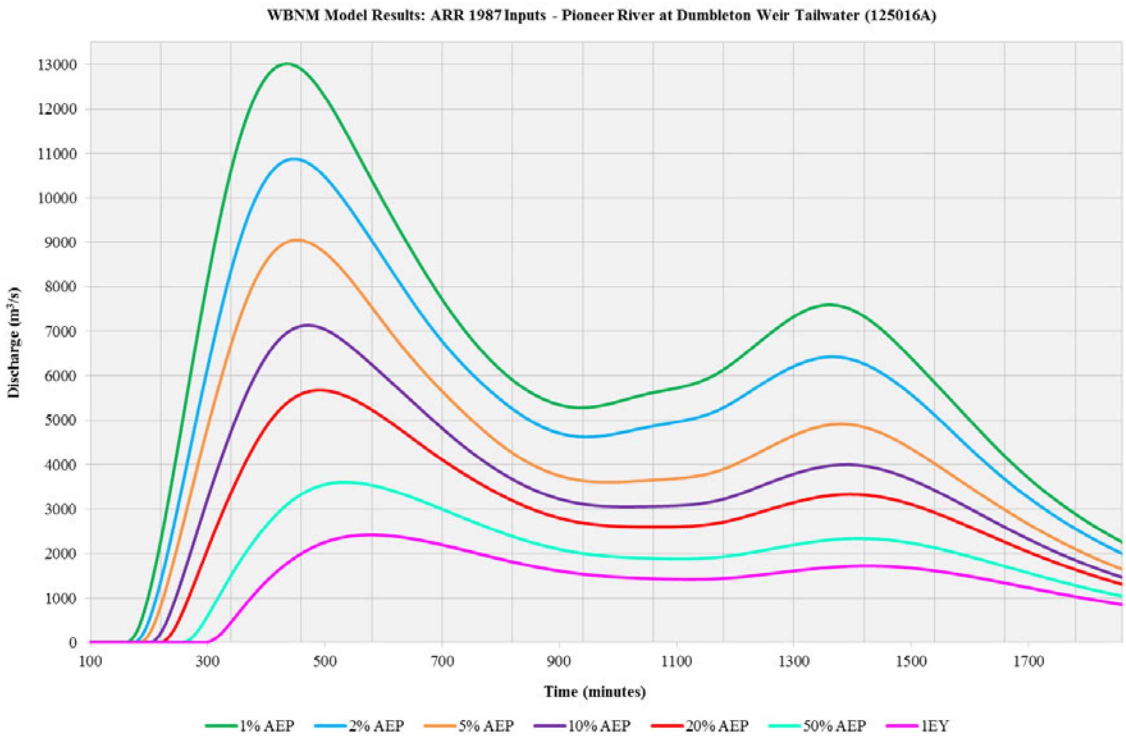

Figure 4-39: Runoff Routing Model Flood Hydrographs (ARR 1987) – Pioneer River at Dumbleton Weir Tailwater

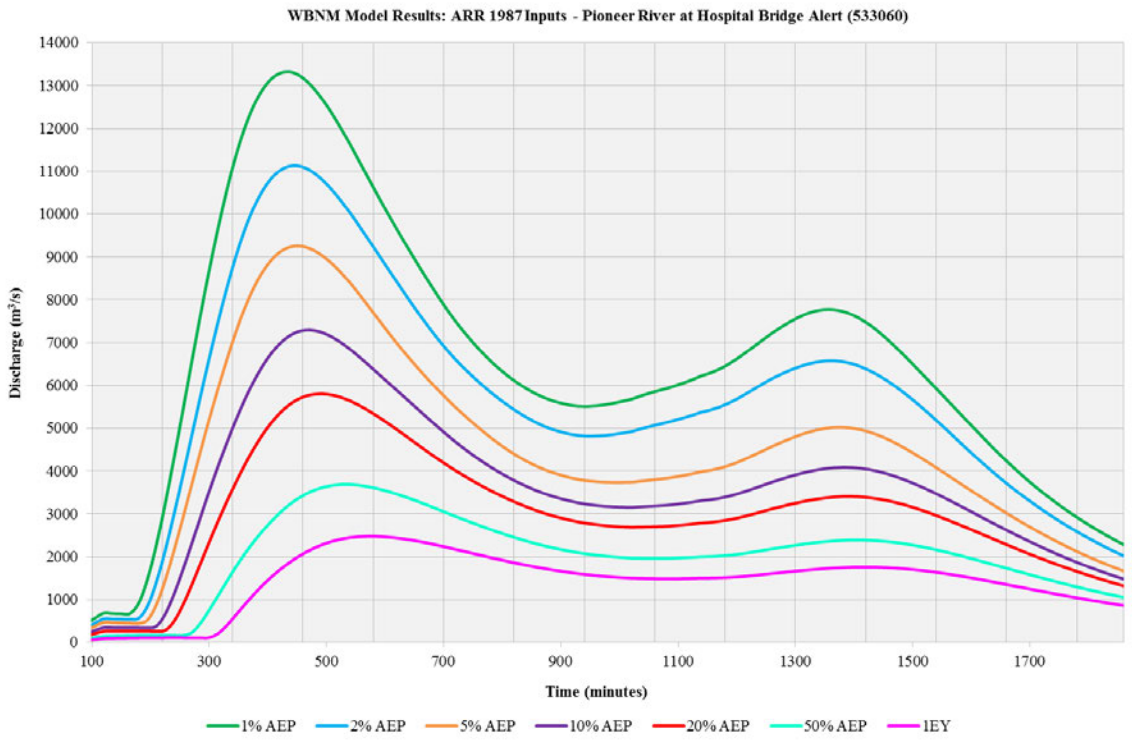

Figure 4-40: Runoff Routing Model Flood Hydrographs (ARR 1987) – Pioneer River at Hospital Bridge

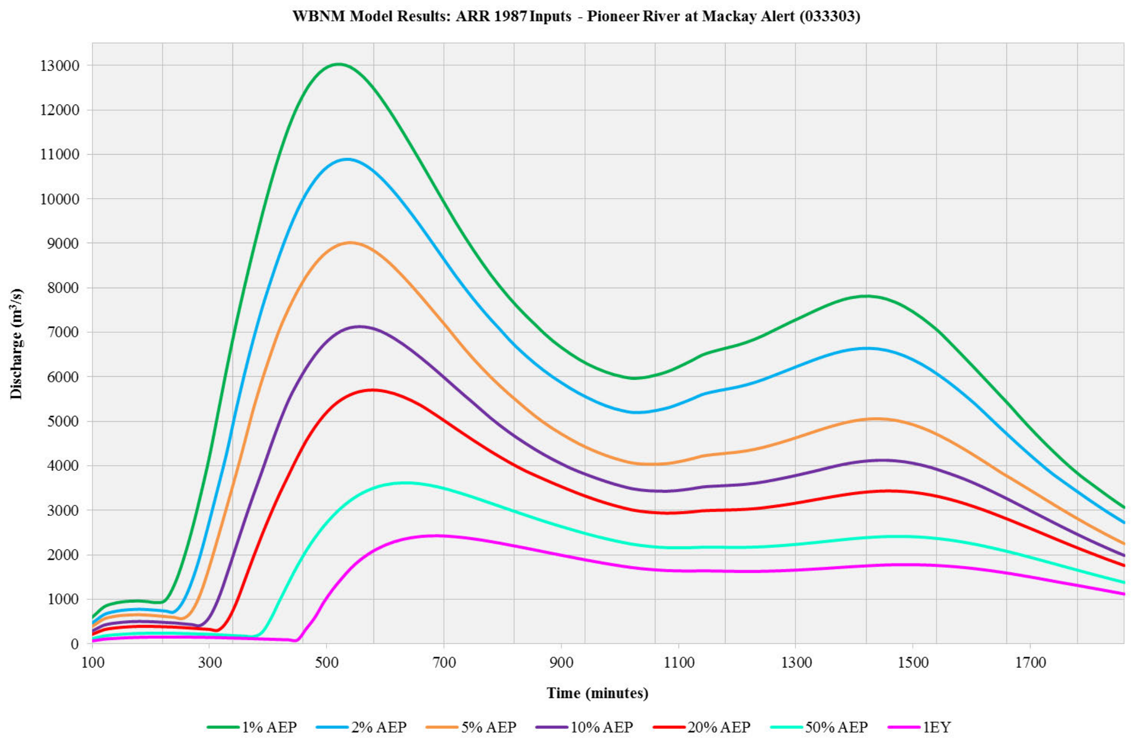

Figure 4-41: Runoff Routing Model Flood Hydrographs (ARR 1987) – Pioneer River at Mackay Alert

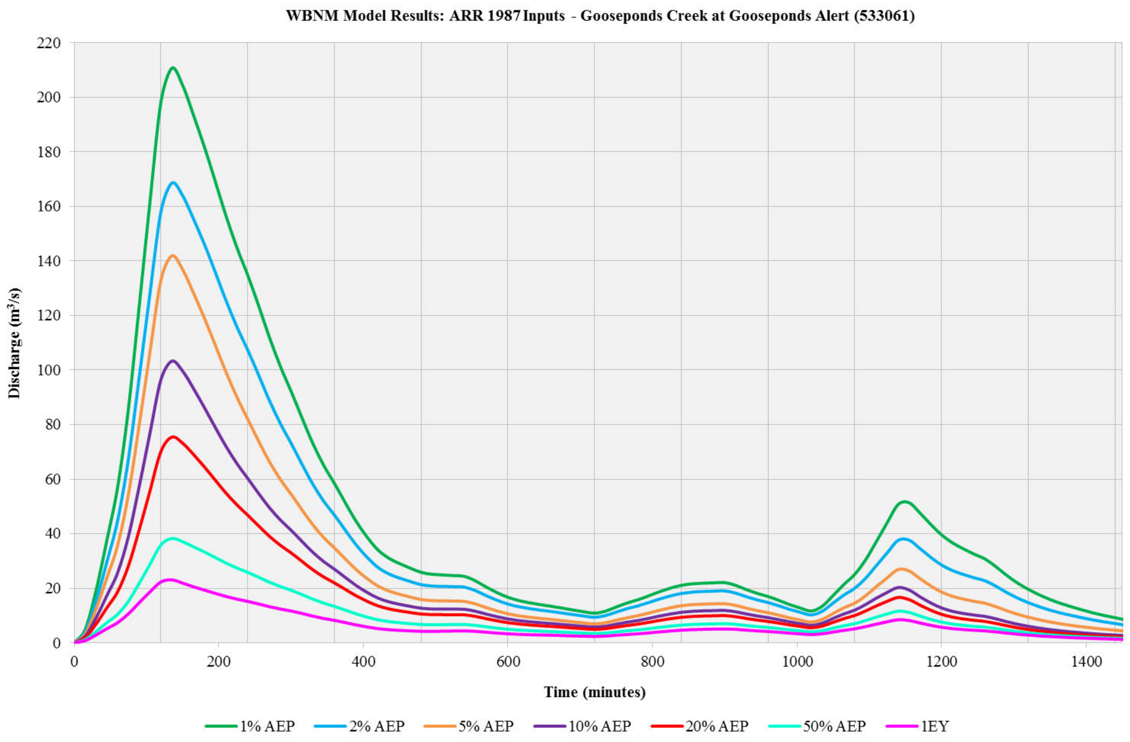

Figure 4-42: Runoff Routing Model Flood Hydrographs (ARR 1987) – Gooseponds Creek at Gooseponds Alert

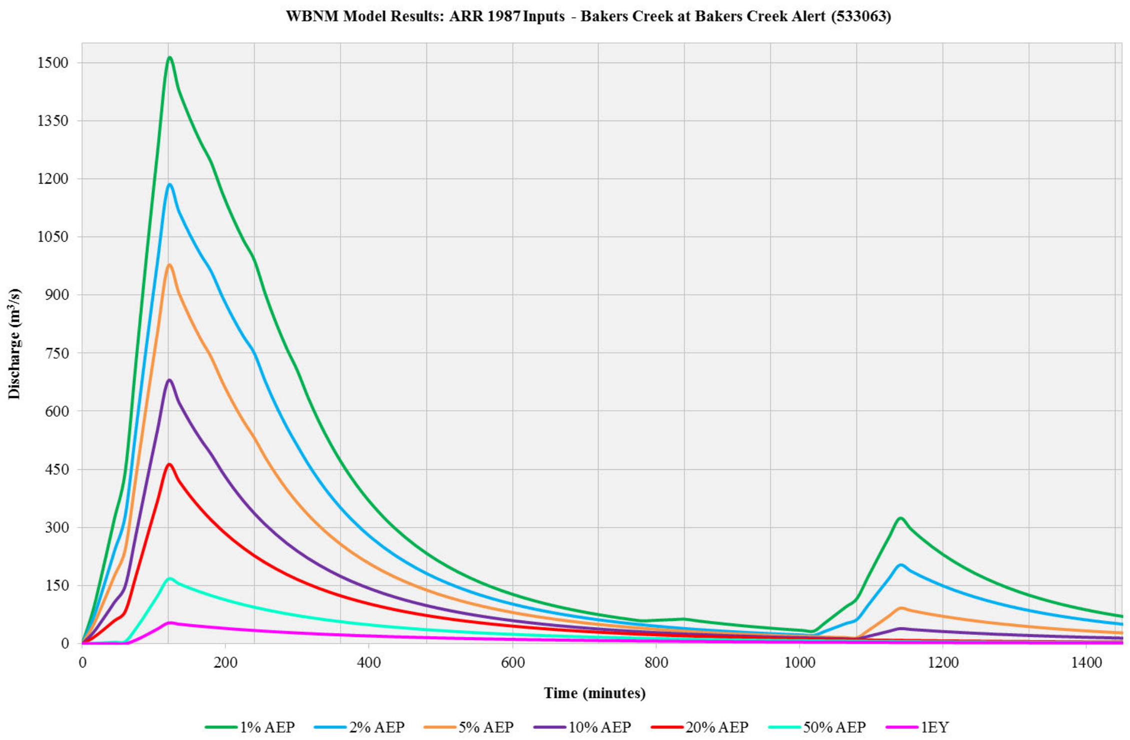

Figure 4-43: Runoff Routing Model Flood Hydrographs (ARR 1987) – Bakers Creek at Bakers Creek Alert

4.3.5. ARR2016 Results

The following figures show the resultant flood hydrographs from the WBNM catchment modelling using the hydrologic parameters from the ARR 2016 guidelines (refer Section 3.4.2) and the Ensemble Event modelling approach. Results have been generated for each assessment location as shown in Figure 4-1. The peak discharge values for each location have been summarised in the comparison of results table for each location (refer Chapter 5).

Unlike the ARR 1987 hydrology results, the 2016 hydrographs vary in shape dependent on the magnitude of the event. This is due to the different Ensemble patterns prescribed for rare (1% & 2% AEP events), intermediate (5%, 10% & 20% AEP events) and frequent (50% AEP & 1EY events) rainfall. The difference in the rainfall patterns (% of total rainfall per 30min period) for Ensemble pattern 4 (found to be the median for the 1% AEP event, refer Appendix C) can be seen in Figure 4-44.

As seen in Figure 4-44, the rare rainfall (green) is a twin peak storm which is being reflected in the results compared to the intermediate (blue) and frequent (purple) rainfall events reporting a single peak. The increase in rainfall intensities over the 150 to 240 minute periods for the intermediate rainfall is also causing some 5% AEP peak discharges to report higher than the 2% AEP peak discharge. In theory this is not possible and in these instances the larger peak of the two events has been taken forward as the peak for both events for comparison purposes. Further investigation into the selection of different ensemble patterns would be required to solve this issue and this has been identified in the further work proceeding this dissertation (refer Section 5.12).

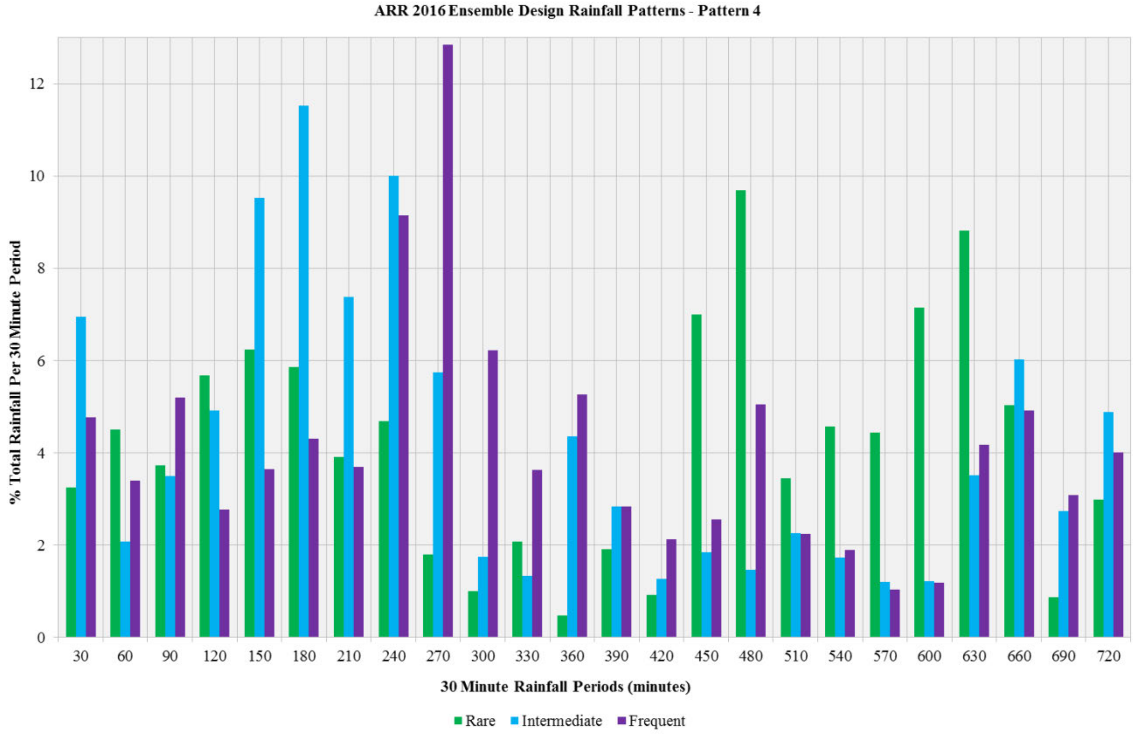

Figure 4-44: ARR 2016 Ensemble Pattern 4 Design Rainfall Patterns

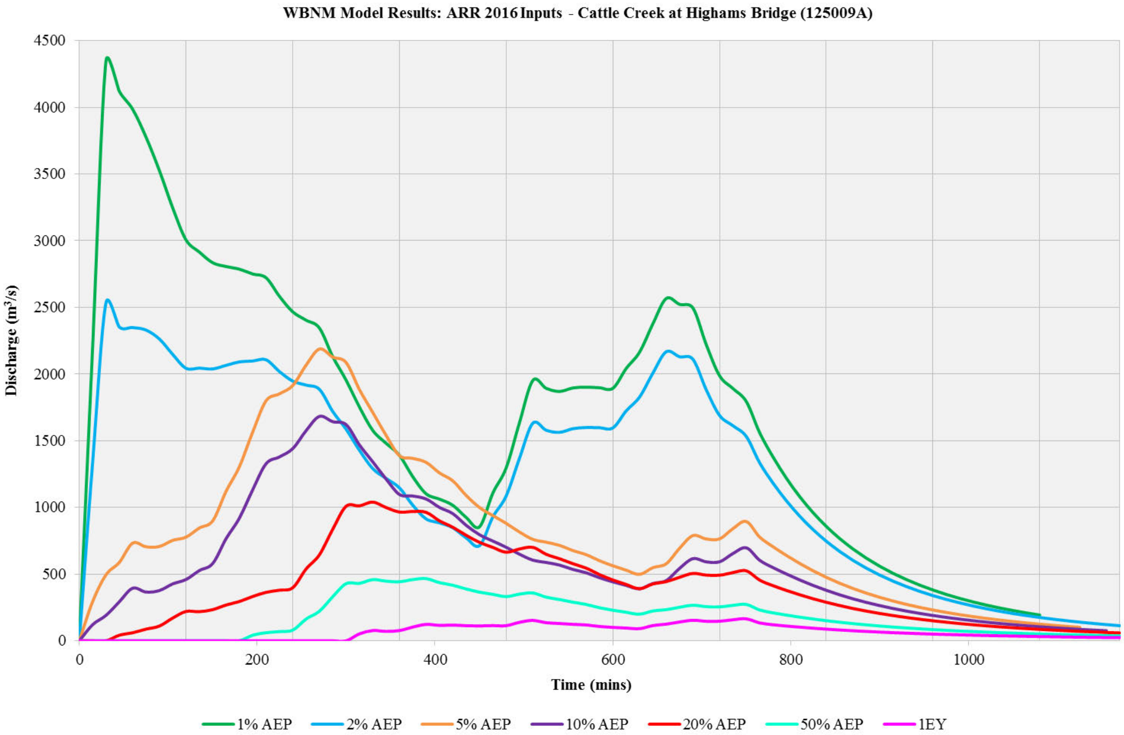

Figure 4-45: Runoff Routing Model Flood Hydrographs (ARR 2016) – Cattle Creek at Highams Bridge

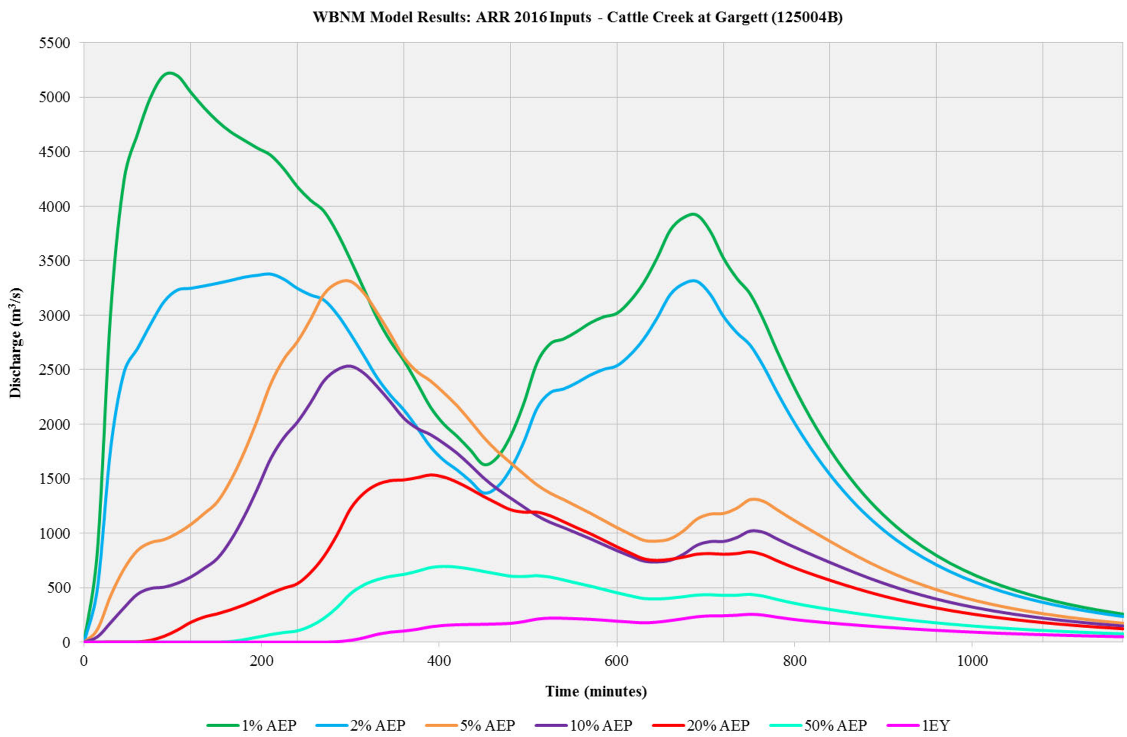

Figure 4-46: Runoff Routing Model Flood Hydrographs (ARR 2016) – Cattle Creek at Gargett

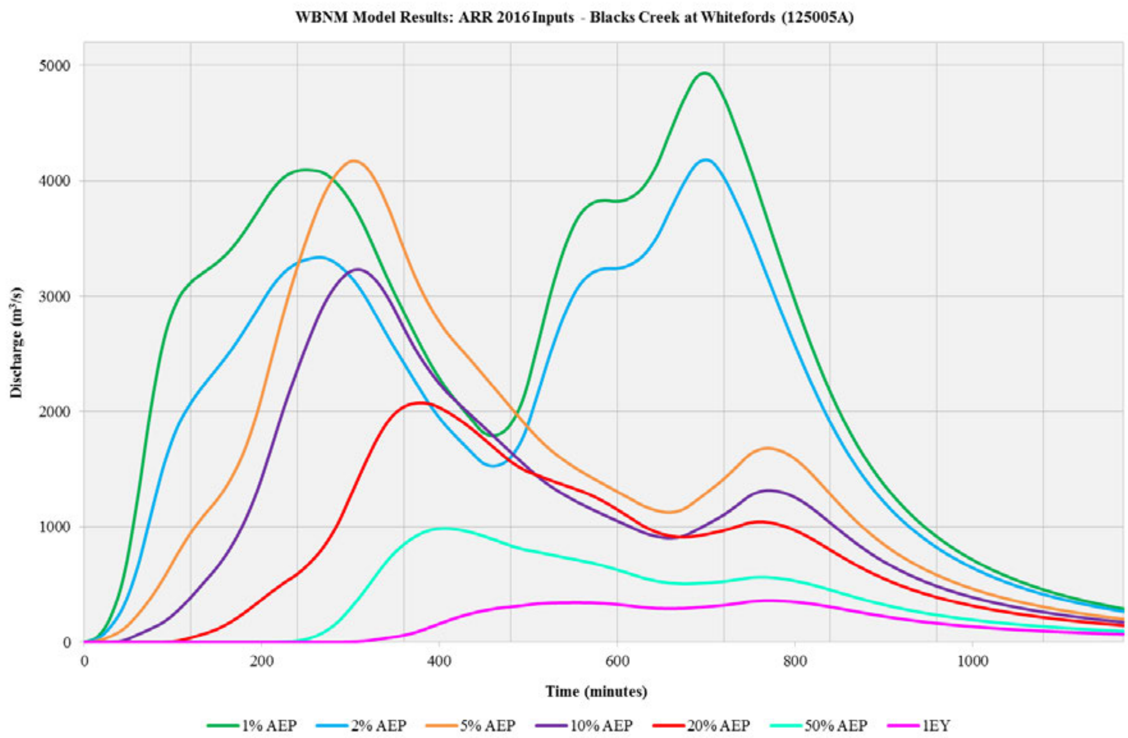

Figure 4-47: Runoff Routing Model Flood Hydrographs (ARR 2016) – Blacks Creek at Whitefords

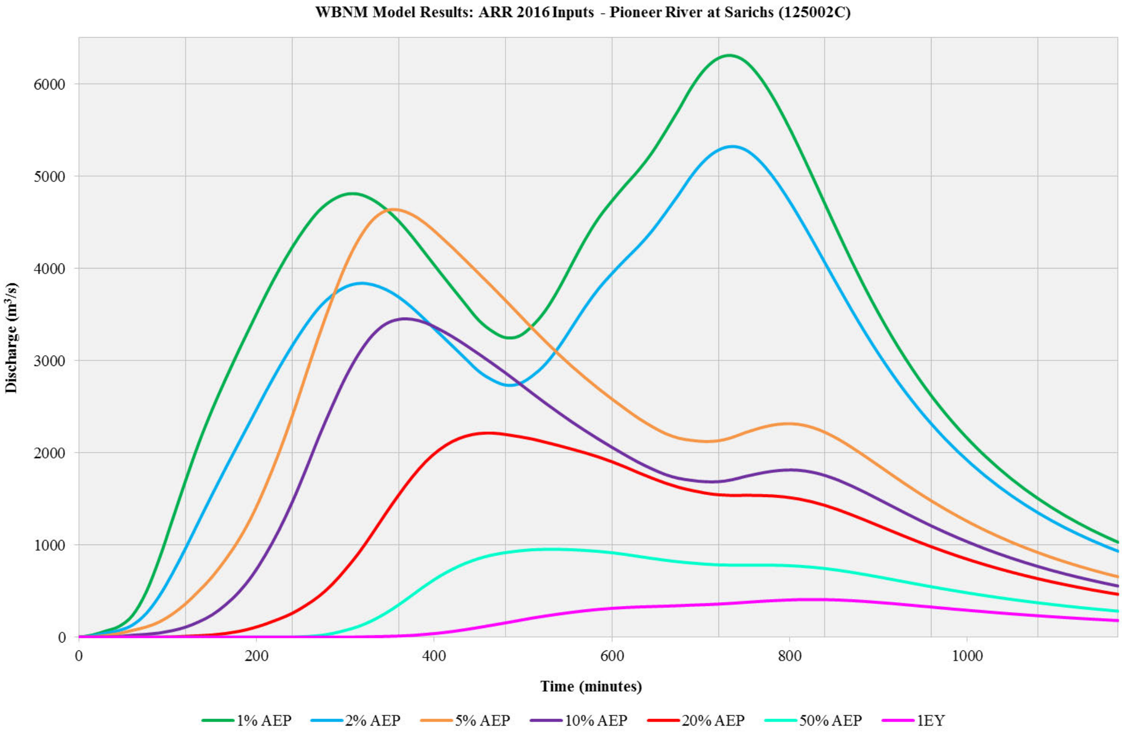

Figure 4-48: Runoff Routing Model Flood Hydrographs (ARR 2016) – Pioneer River at Sarichs

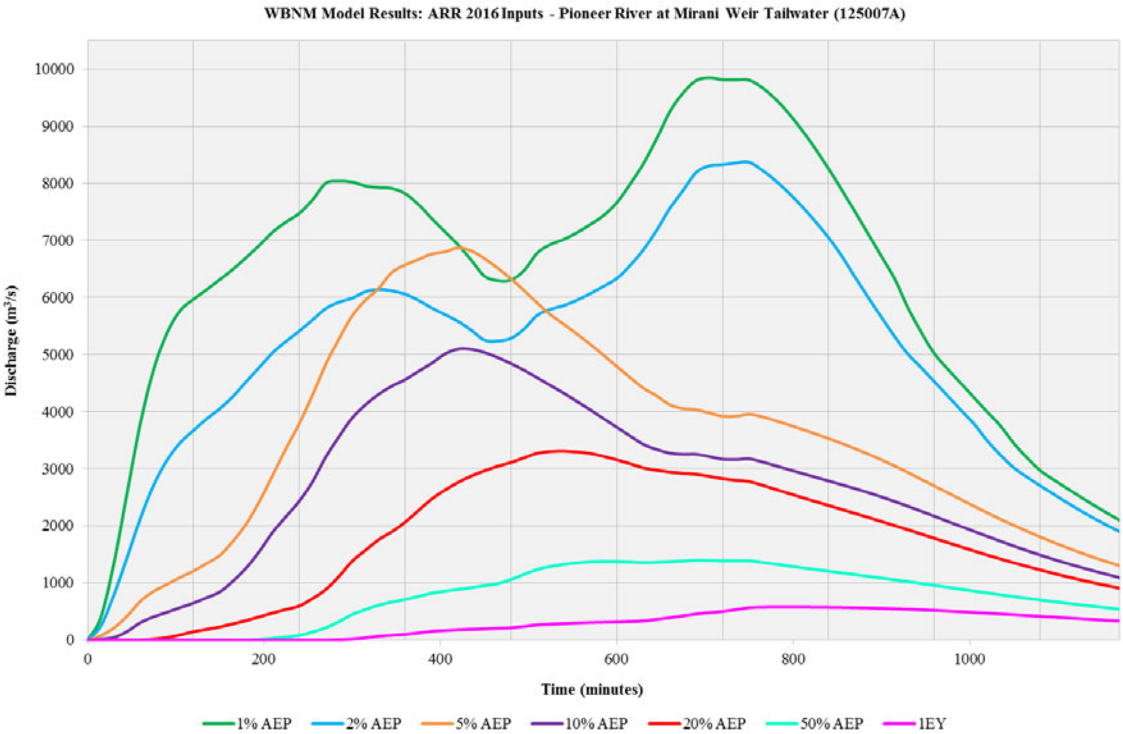

Figure 4-49: Runoff Routing Model Flood Hydrographs (ARR 2016) – Pioneer River at Mirani Weir Tailwater

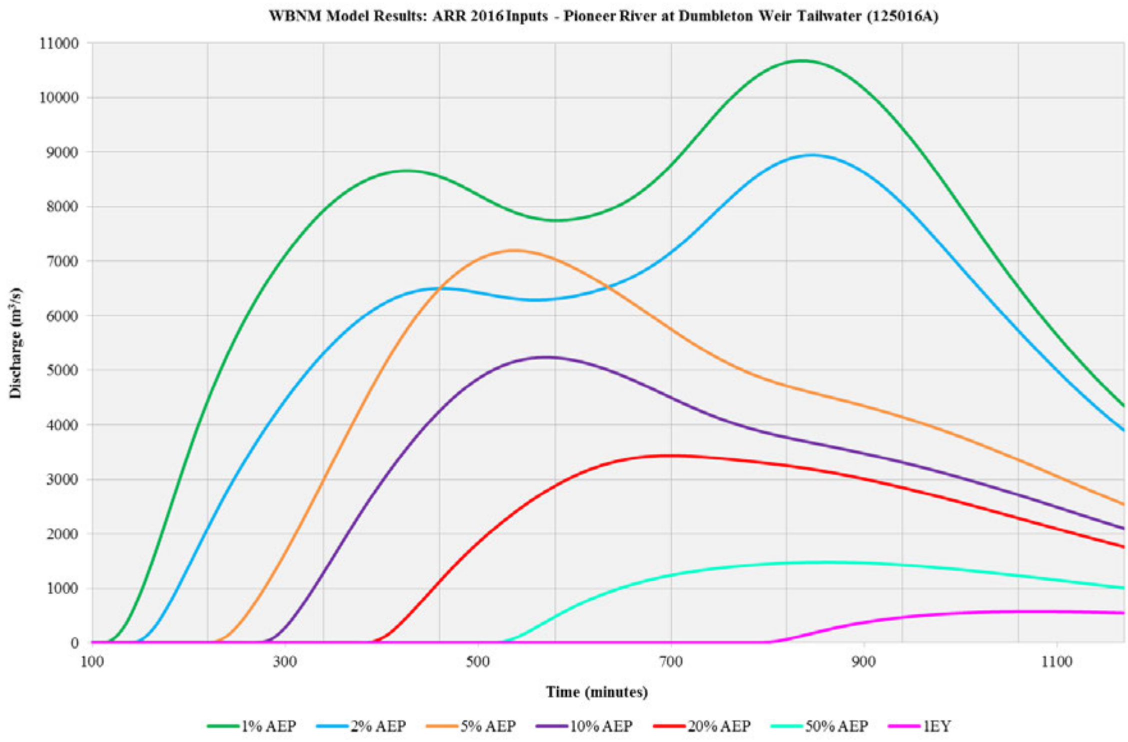

Figure 4-50: Runoff Routing Model Flood Hydrographs (ARR 2016) – Pioneer River at Dumbleton Weir Tailwater

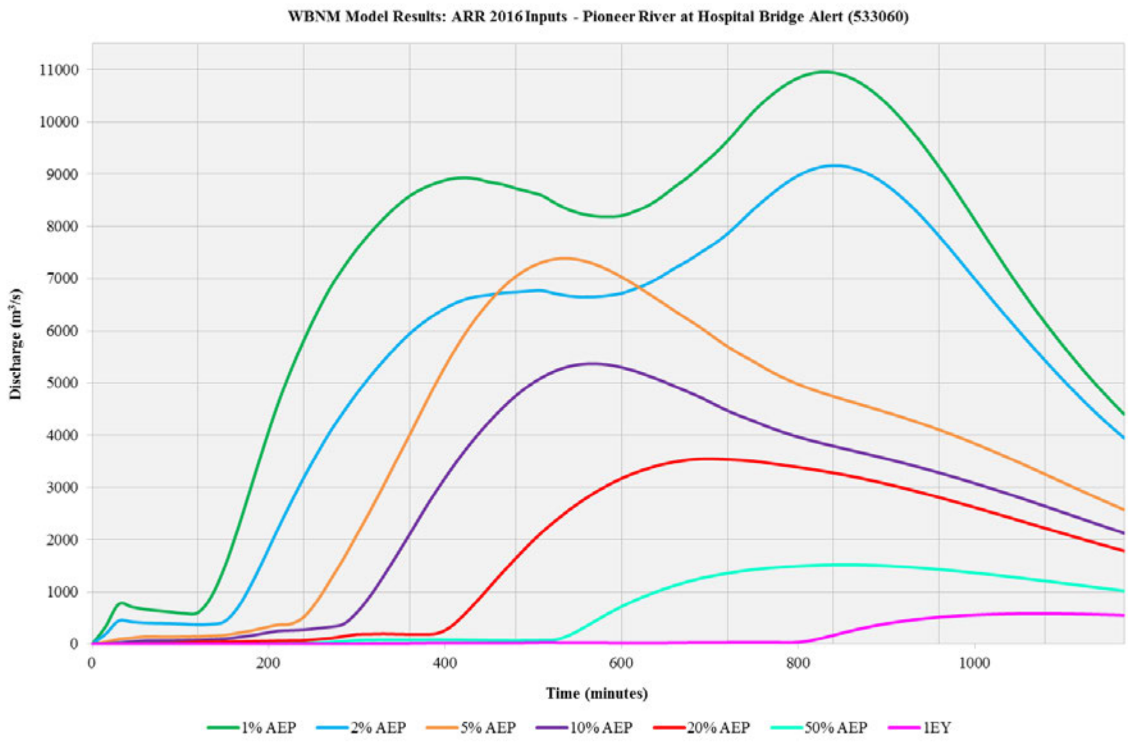

Figure 4-51: Runoff Routing Model Flood Hydrographs (ARR 2016) – Pioneer River at Hospital Bridge

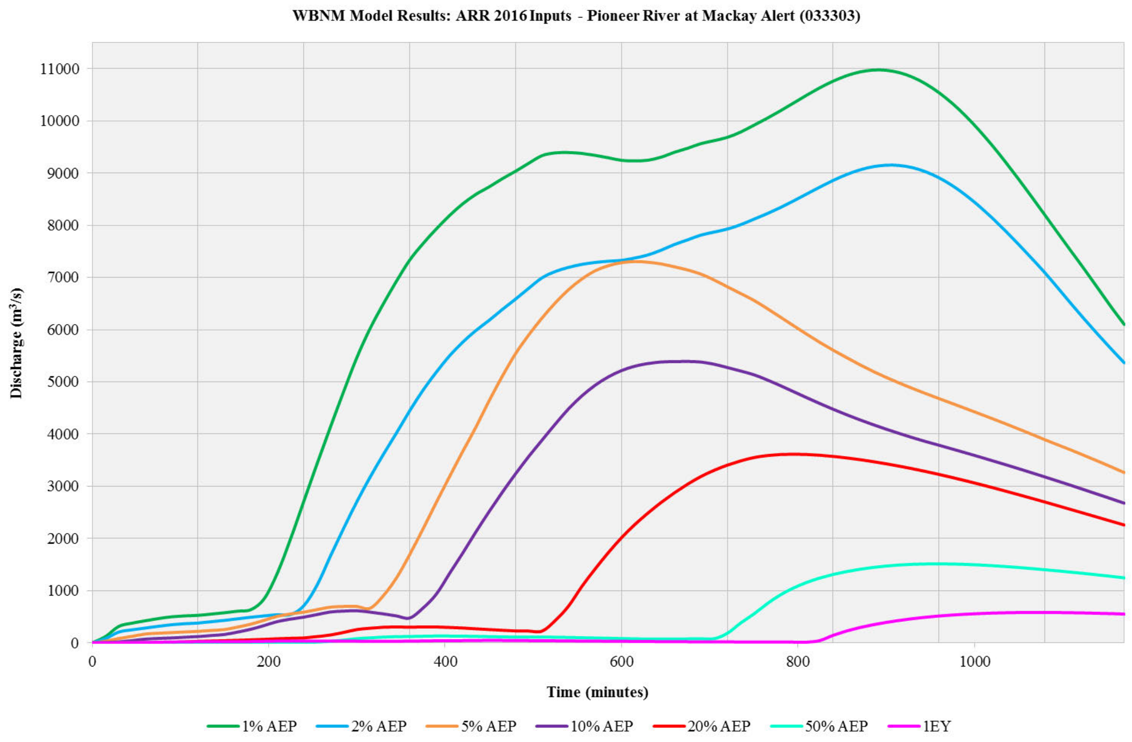

Figure 4-52: Runoff Routing Model Flood Hydrographs (ARR 2016) – Pioneer River at Mackay Alert

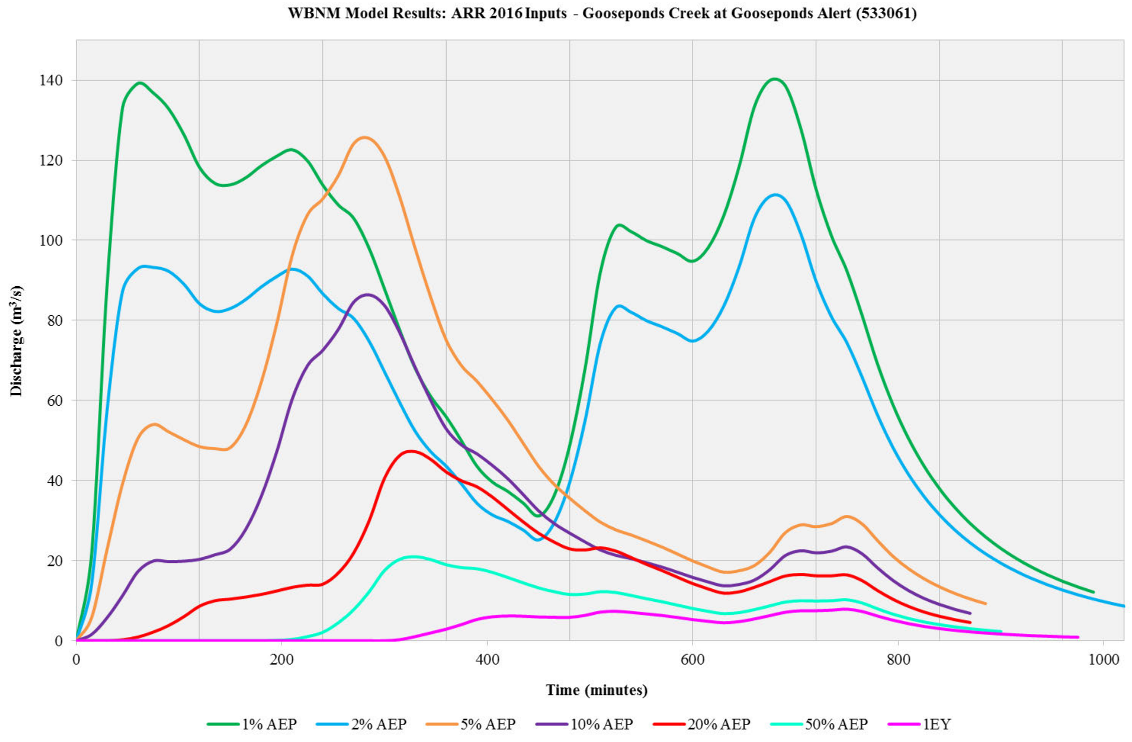

Figure 4-53: Runoff Routing Model Flood Hydrographs (ARR 2016) – Gooseponds Creek at Gooseponds Alert

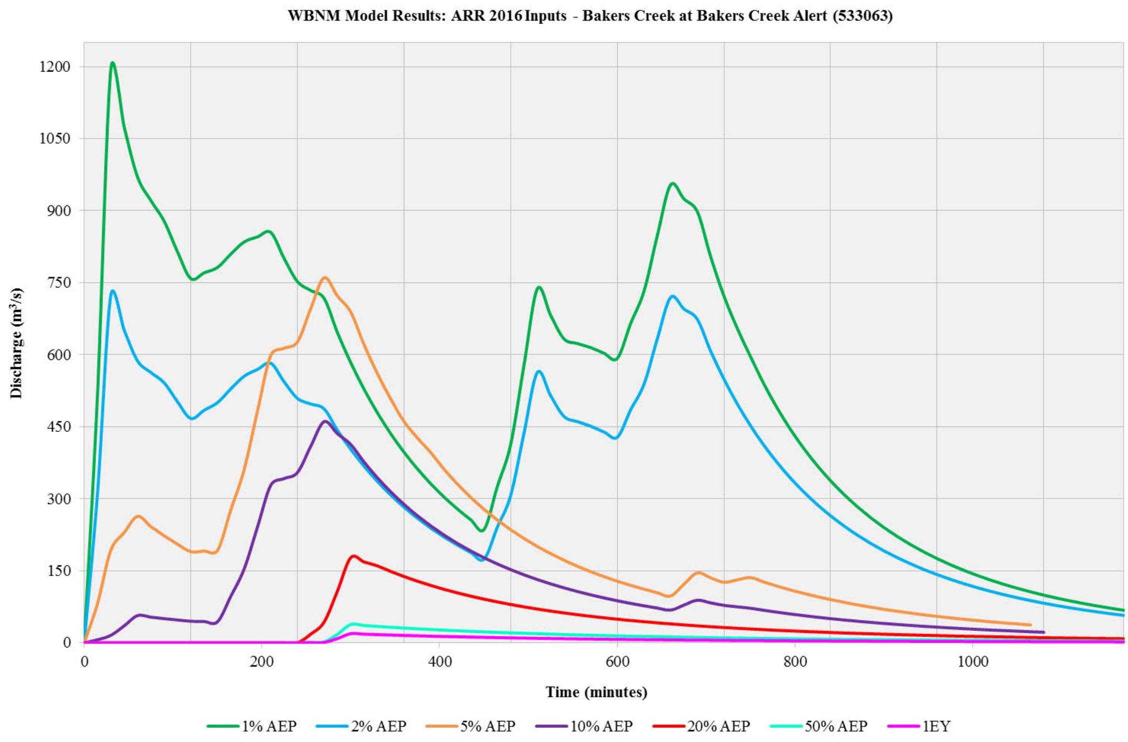

Figure 4-54: Runoff Routing Model Flood Hydrographs (ARR 2016) – Bakers Creek at Bakers Creek Alert

4.4. Hydrodynamic Model Development

As per the methodology outlined in Section 3.5.3 a hydrodynamic model was developed using the TUFLOW software (refer Section 2.7.3) to estimate the discharge hydrograph and flooding extents for subcatchments within the lower, more urbanised reaches of the Pioneer River catchment.

The subsequent sections display the results of the model calibration to historical events as well as the design flood hydrographs extracted at key locations (refer Figure 4-1) for both ARR 1987 and 2016 hydrological inputs.

4.4.1. Model Calibration

To ensure that the TUFLOW hydrodynamic model was accurately representing the catchment conditions the model was calibrated against two historical rain events (refer Section 3.2). The following figures show the results of the calibration, comparing the modelled hydrograph (solid line) to the recorded hydrograph (dotted line) at the same location.

Overall, the model calibrated well to the historical events providing confidence in the findings of the investigation. A significant variance between the modelled results and recorded flows can be seen for the 2008 event at Bakers Creek (refer Figure 4-58). As found from the Pioneer River Flood Study (WRM Water & Environment 2011) the gauge was reported to be faulty for this event and underestimated recorded levels. A peak of 4.15mAHD was measured at the gauge in the February 2008 event, whereas surveyed flood marks in the area reported that the creek may have peaked at over 4.6mAHD.

The gauges located in the downstream reaches of the river at the Hospital Bridge and Mackay Alert (refer Figure 4-55 and Figure 4-56) have recorded tidal peaks of the river for both events prior to the storm peak. Although the tidal levels for the event were incorporated into the TUFLOW model, the interactions between the tide and localised runoff did not calibrate as recorded. However, the storm peak was accurately represented for each event and this was deemed as sufficient calibration.

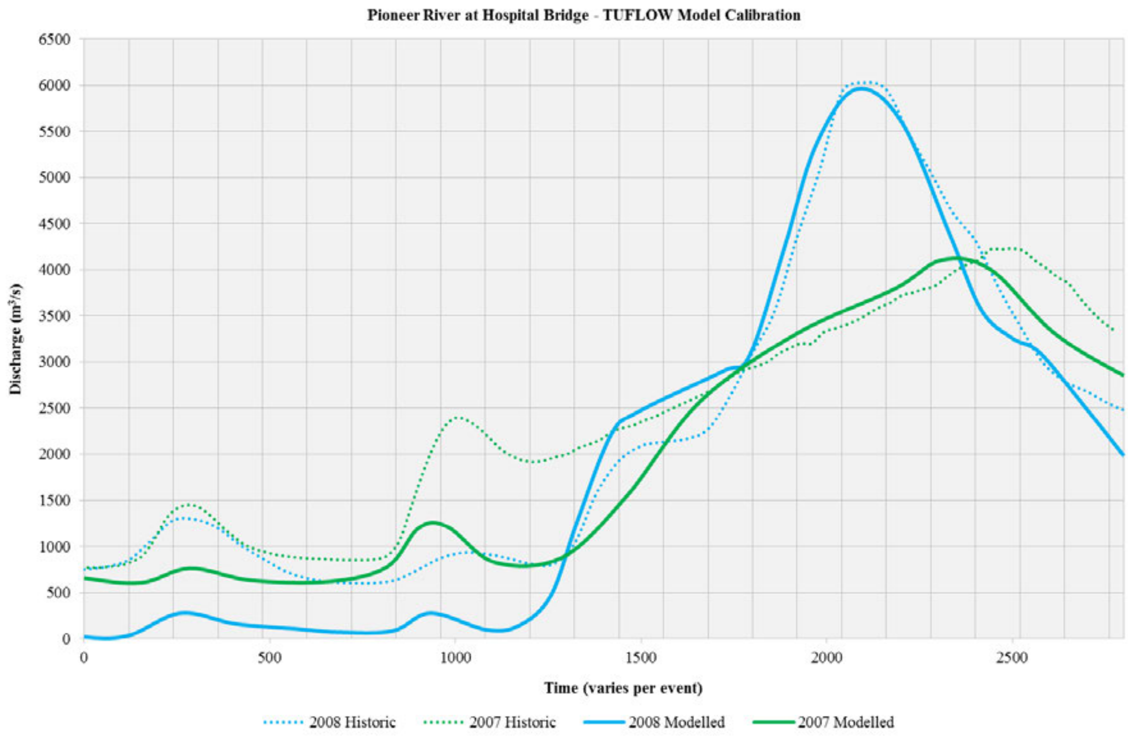

Figure 4-55: TUFLOW Model Calibration – Pioneer River at Hospital Bridge

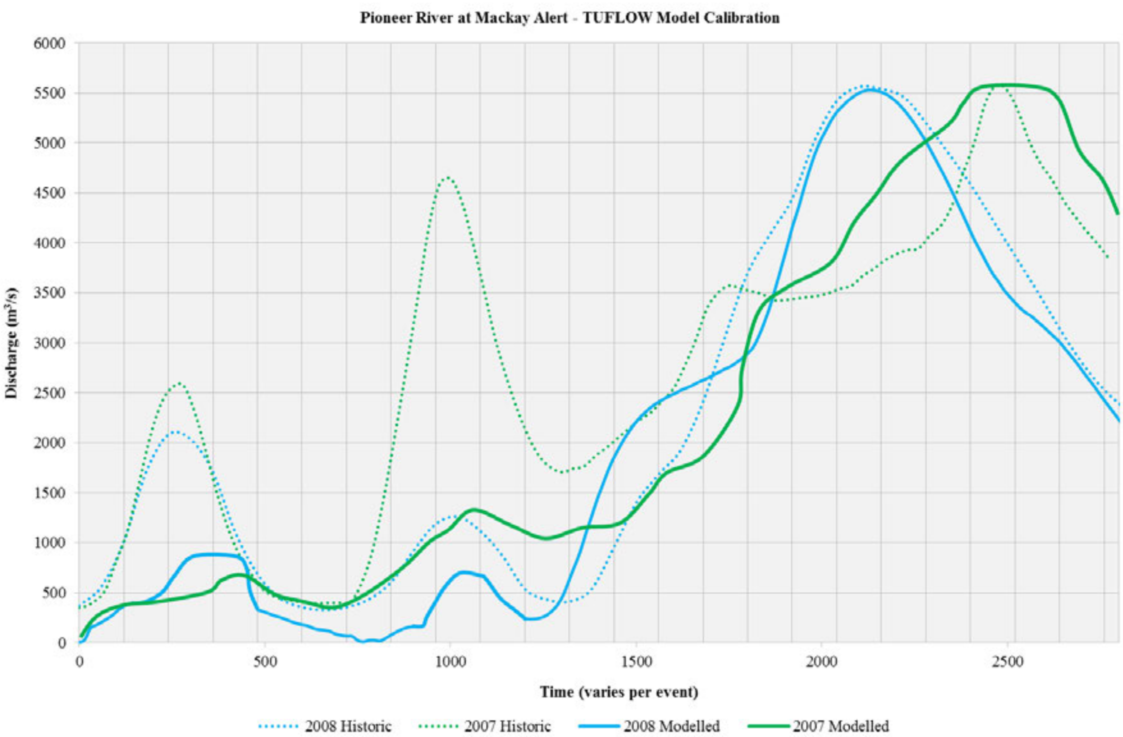

Figure 4-56: TUFLOW Model Calibration – Pioneer River at Mackay Alert

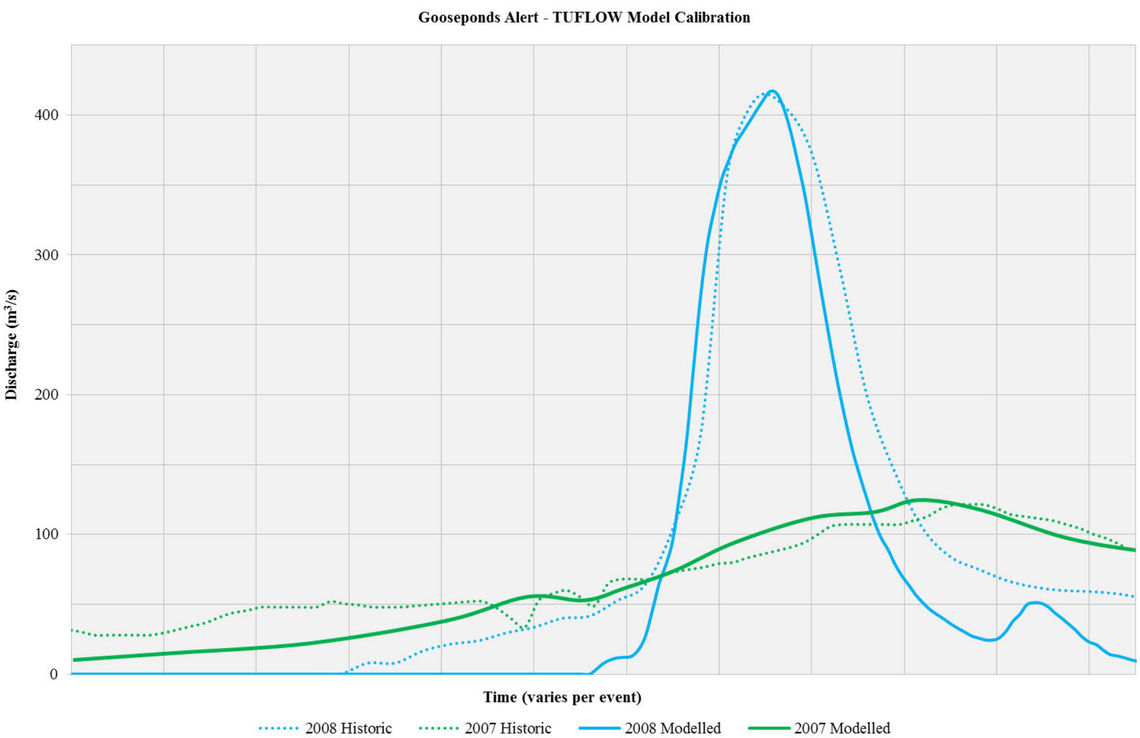

Figure 4-57: TUFLOW Model Calibration – Gooseponds Creek at Gooseponds Alert

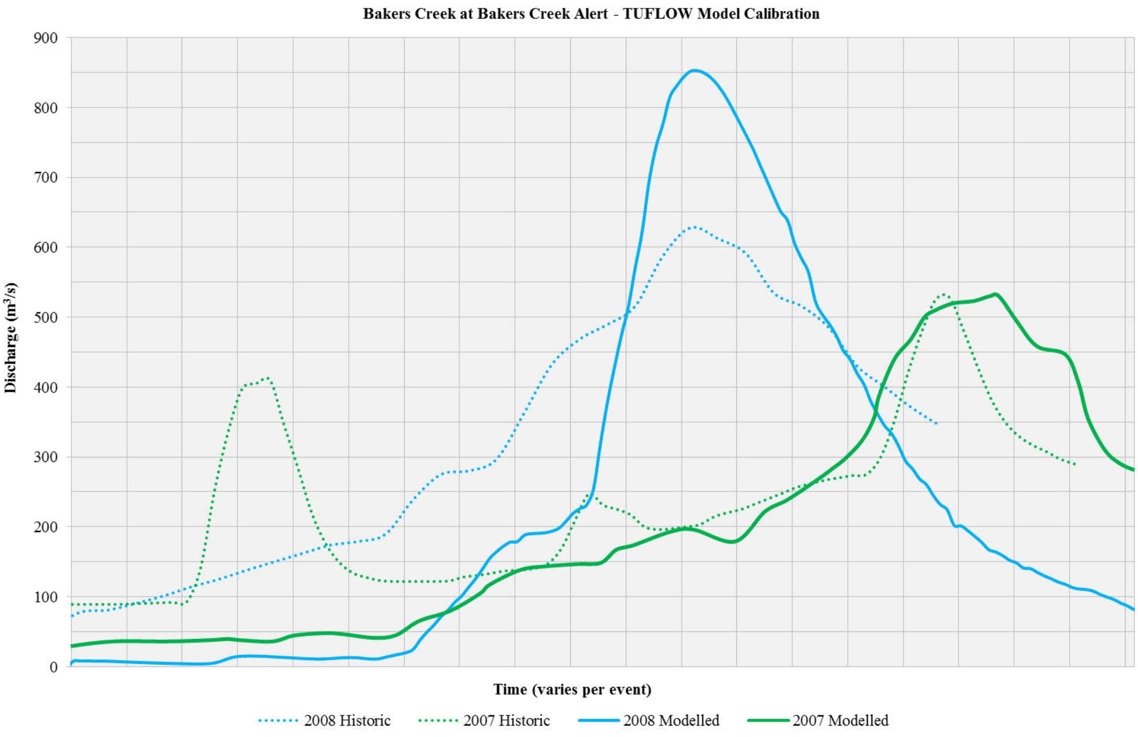

Figure 4-58: TUFLOW Model Calibration – Bakers Creek at Bakers Creek Alert

4.4.1. ARR1987 Results

The following figures show the resultant flood hydrographs from the TUFLOW hydrodynamic modelling using the hydrologic parameters from the ARR 1987 guidelines (refer Section 3.4.1). Results have been generated for each assessment location as shown in Figure 4-1. The peak discharge values for each location have been summarised in the comparison of results table for each location (refer Chapter 5).

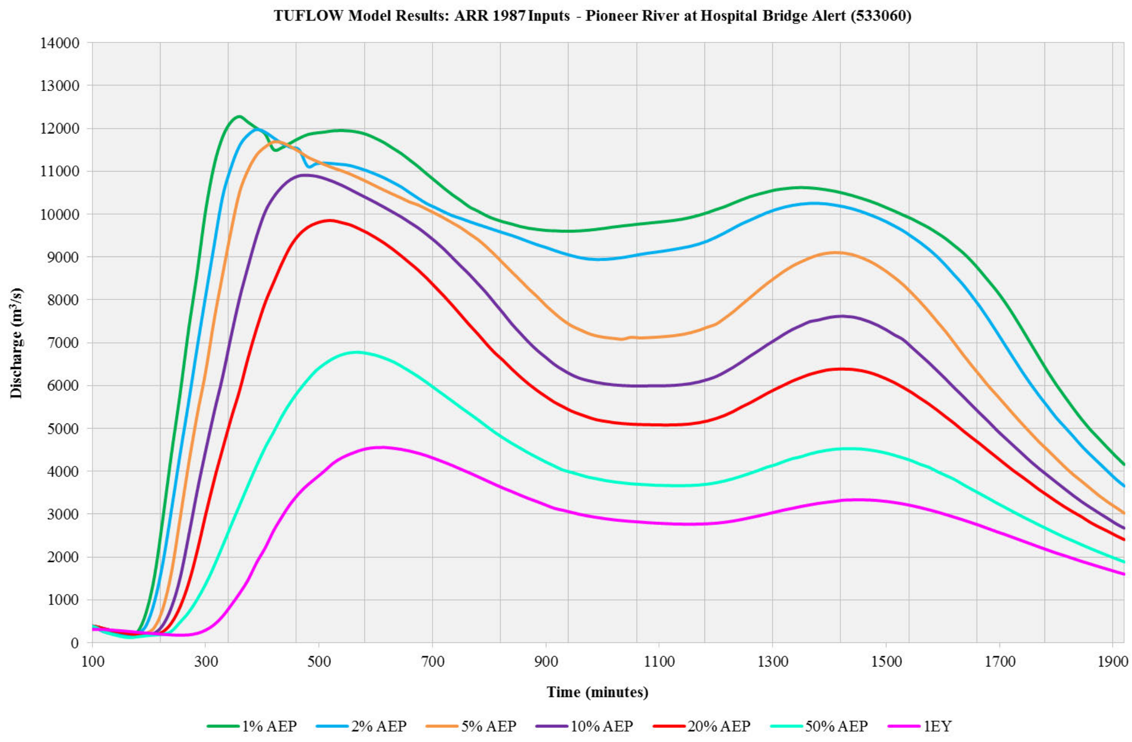

Figure 4-59: Hydrodynamic Model Hydrographs (ARR 1987) – Pioneer River at Hospital Bridge

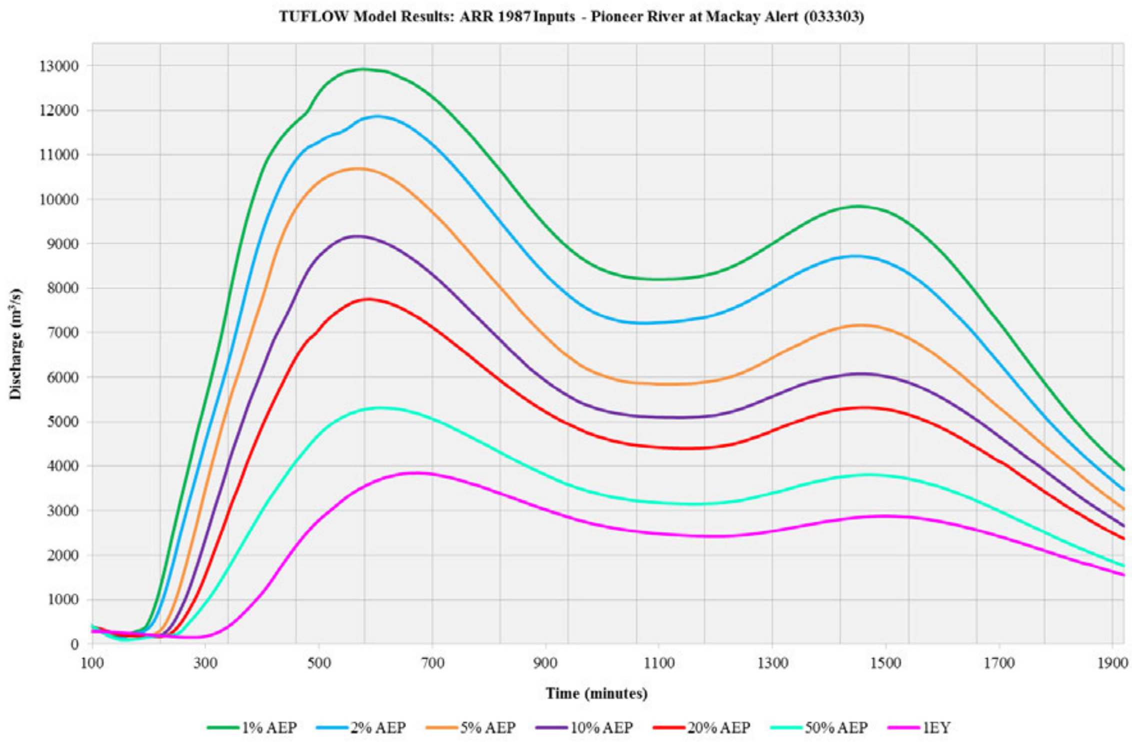

Figure 4-60: Hydrodynamic Model Hydrographs (ARR 1987) – Pioneer River at Mackay Alert

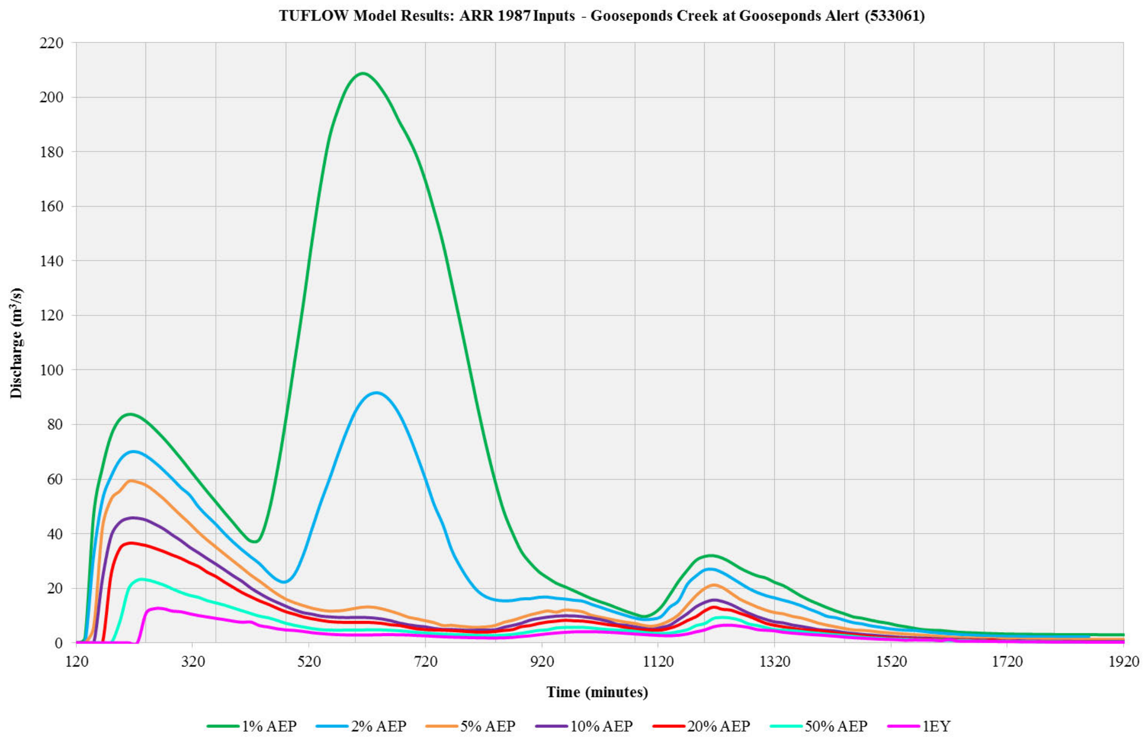

Figure 4-61: Hydrodynamic Model Hydrographs (ARR 1987) – Gooseponds Creek at Gooseponds Alert

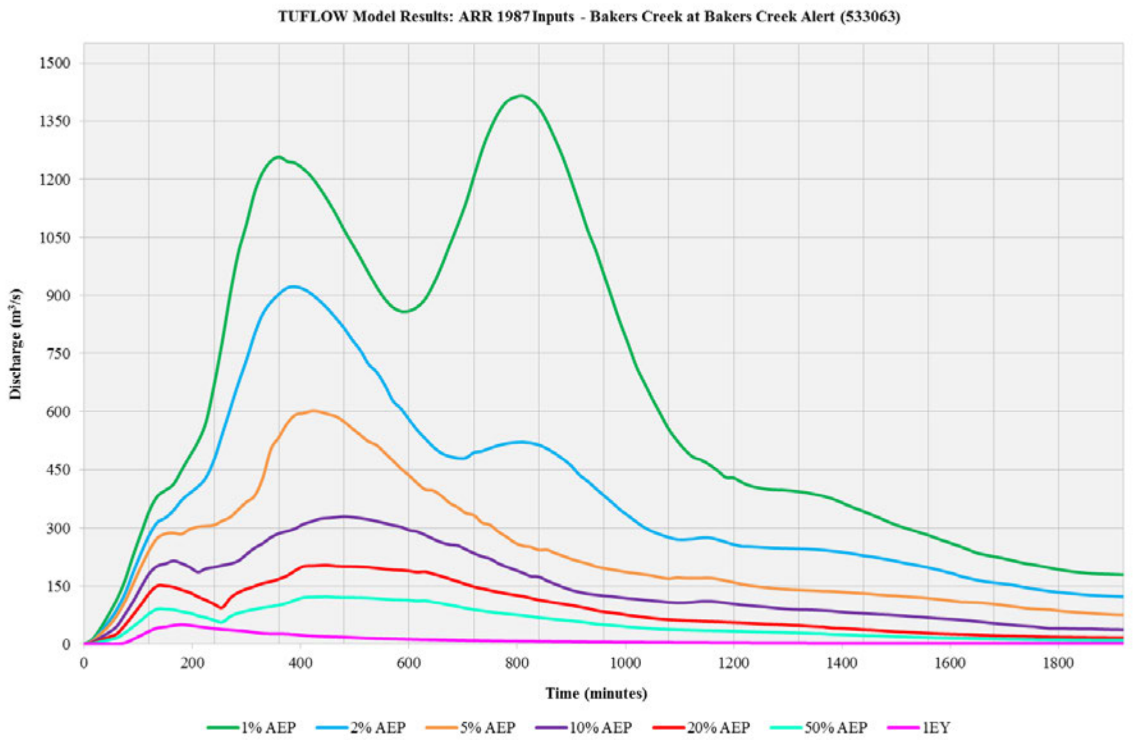

Figure 4-62: Hydrodynamic Model Hydrographs (ARR 1987) – Bakers Creek at Bakers Creek Alert

4.4.2. ARR2016 Results

The following figures show the resultant flood hydrographs from the TUFLOW hydrodynamic modelling using the hydrologic parameters from the ARR 2016 guidelines (refer Section 3.4.2) and the Ensemble Event modelling approach. Results have been generated for each assessment location as shown in Figure 4-1. The peak discharge values for each location have been summarised in the comparison of results table for each location (refer Chapter 5).

Different Ensemble method rainfall patterns for different events have been used as per the ARR 2016 guidelines. The different patterns are evident through the varying shape of the flood hydrographs. This has been explained in more detail around the runoff routing model results (refer Section 4.3.5).

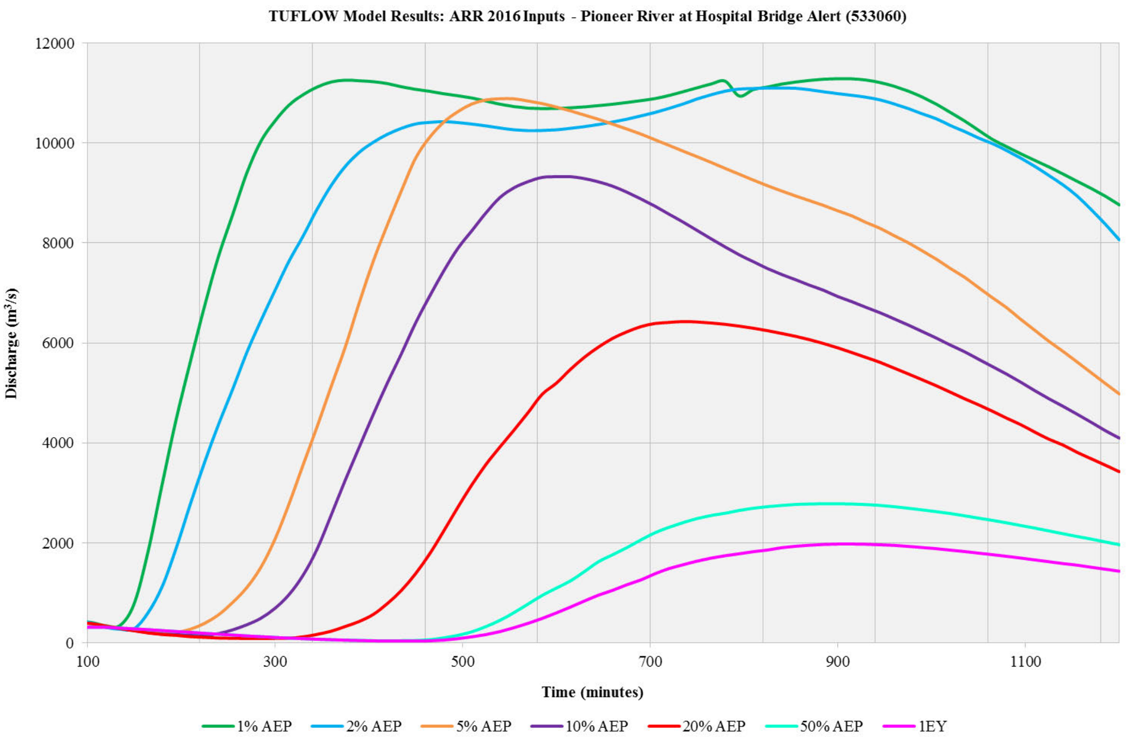

Figure 4-63: Hydrodynamic Model Hydrographs (ARR 2016) – Pioneer River at Hospital Bridge

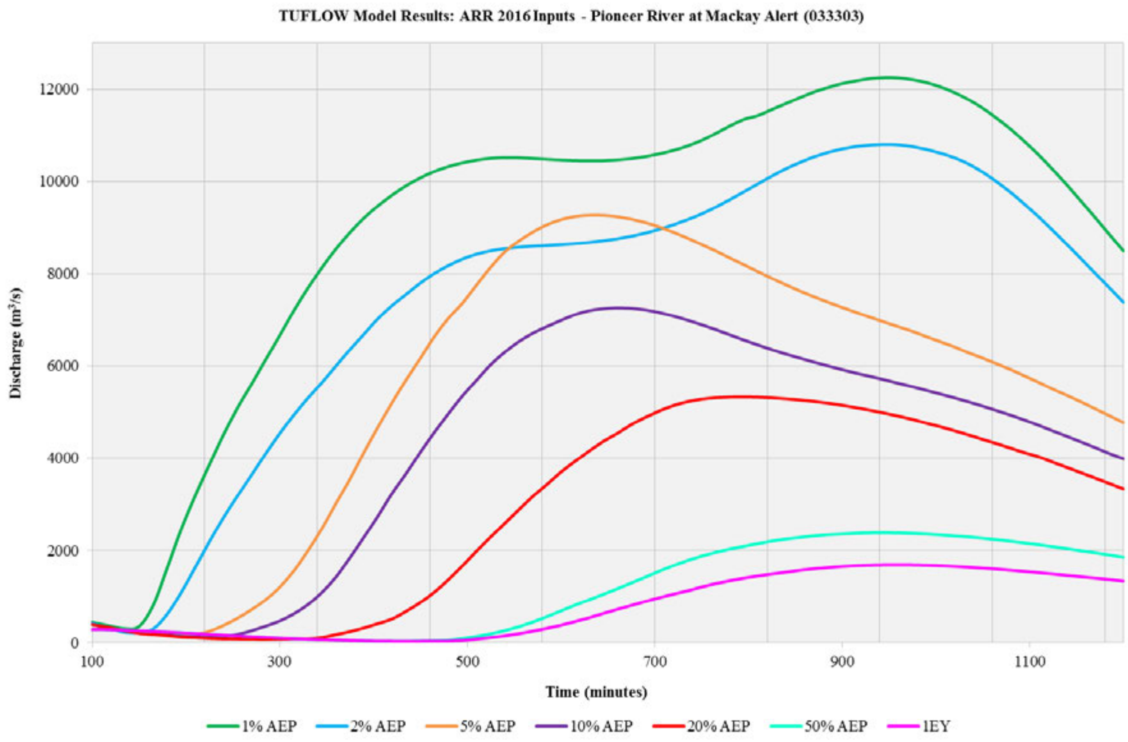

Figure 4-64: Hydrodynamic Model Hydrographs (ARR 2016) – Pioneer River at Mackay Alert

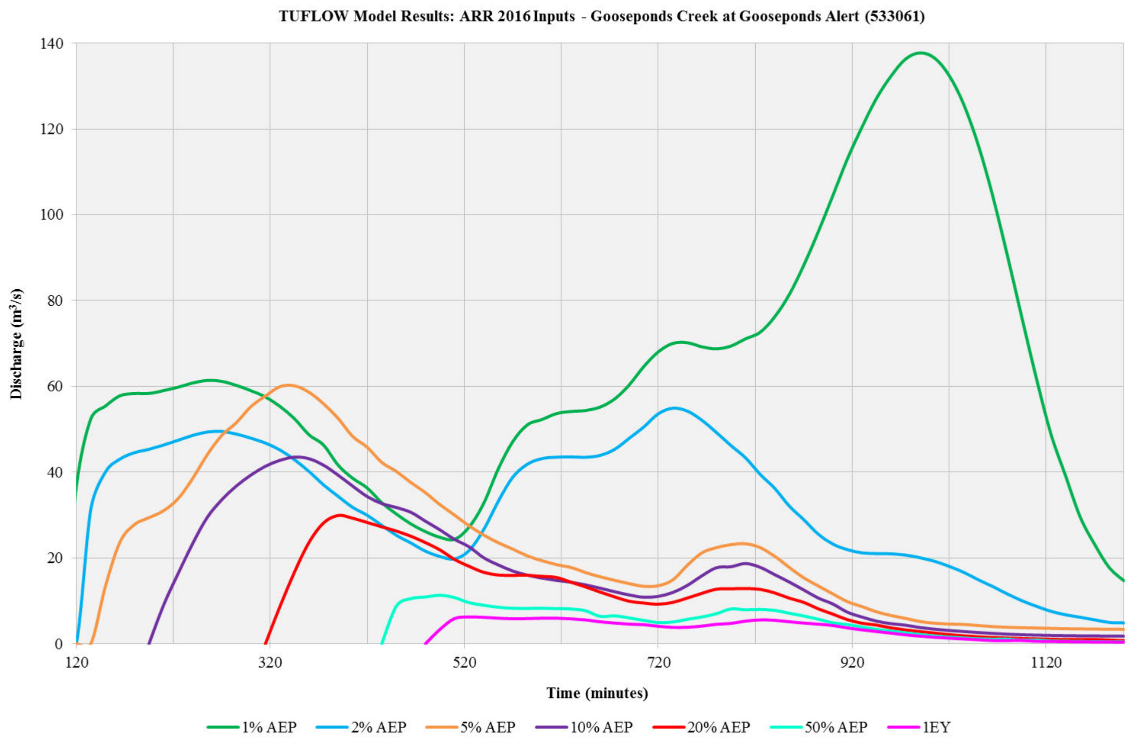

Figure 4-65: Hydrodynamic Model Hydrographs (ARR 2016) – Gooseponds Creek at Gooseponds Alert

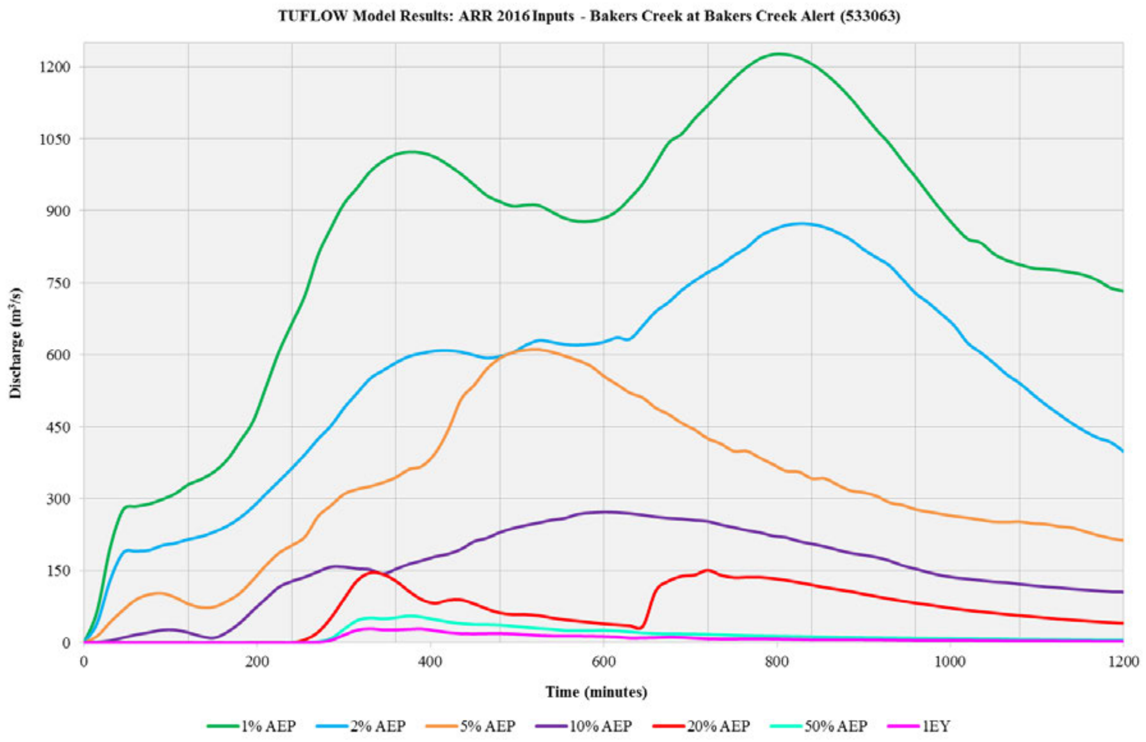

Figure 4-66: Hydrodynamic Model Hydrographs (ARR 2016) – Bakers Creek at Bakers Creek Alert

4.5. Regional Flood Frequency Estimation (RFFE)

The following figures show the resultant flood hydrographs from the Regional Flood Frequency Estimation (RFFE) model using the catchment parameters input from the WBNM subcatchment delineation (refer Figure 3-11). Results have been generated for each assessment location as shown in Figure 4-1. The estimated discharge values for each location have been summarised in the comparison of results table for each location (refer Chapter 5).

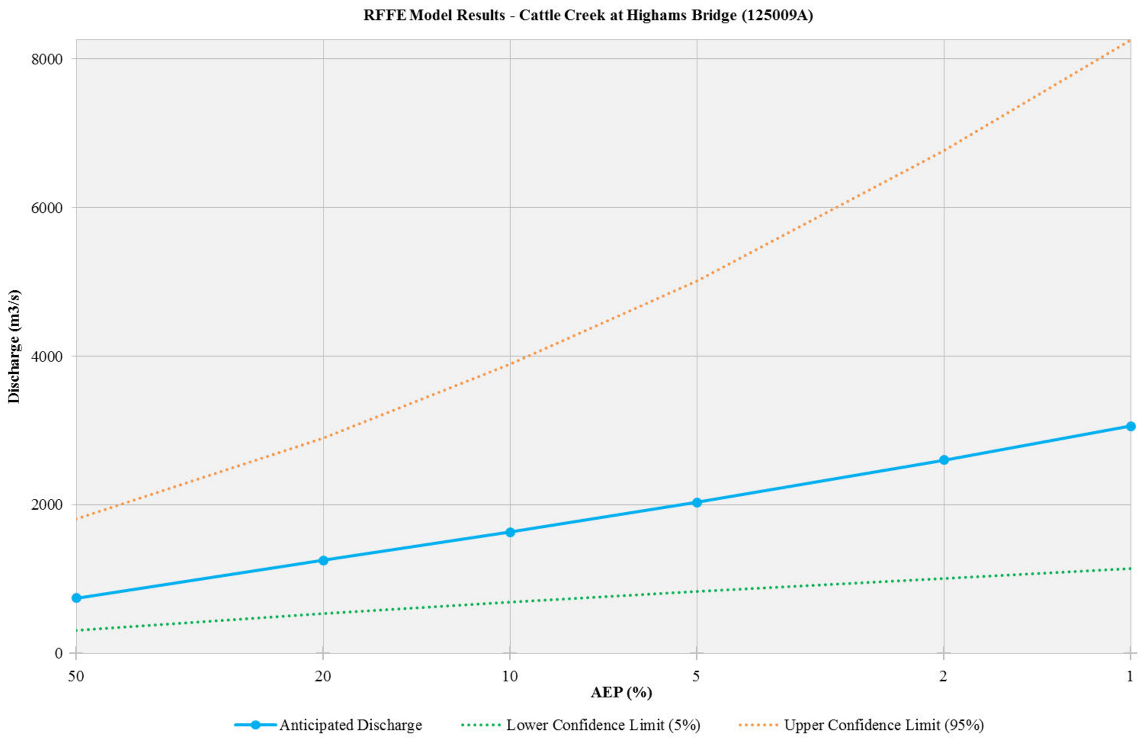

Figure 4-67: RFFE Model Results – Cattle Creek at Highams Bridge

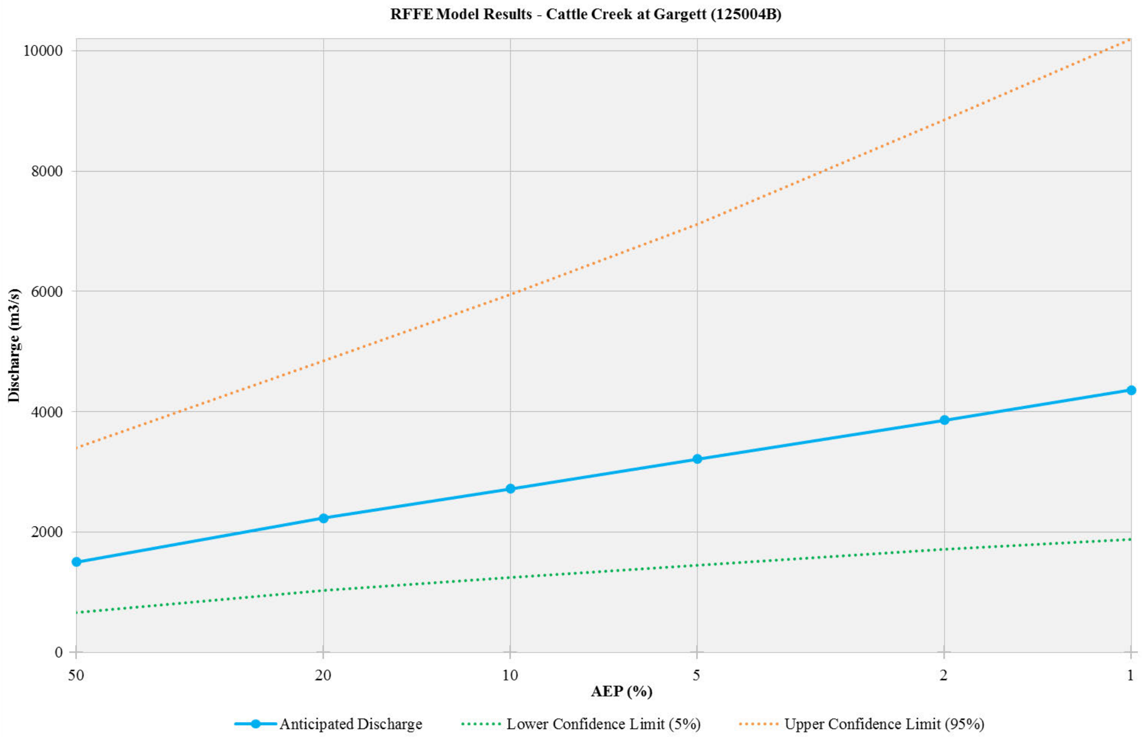

Figure 4-68: RFFE Model Results – Cattle Creek at Gargett

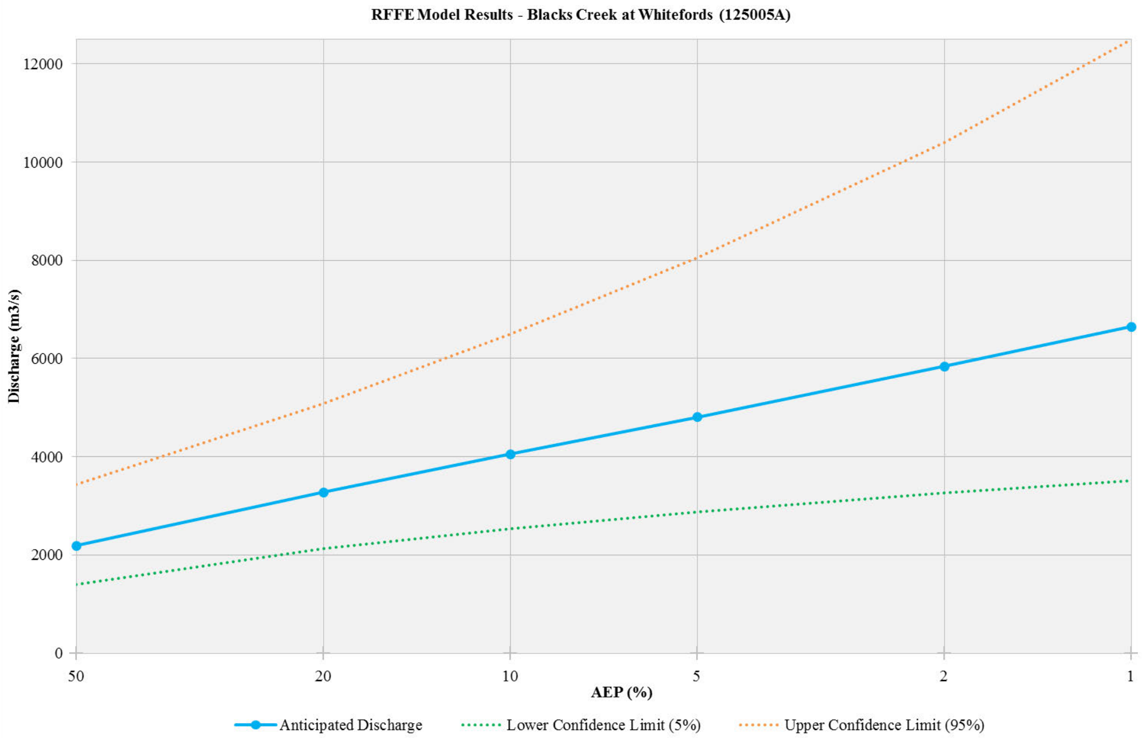

Figure 4-69: RFFE Model Results – Blacks Creek at Whitefords

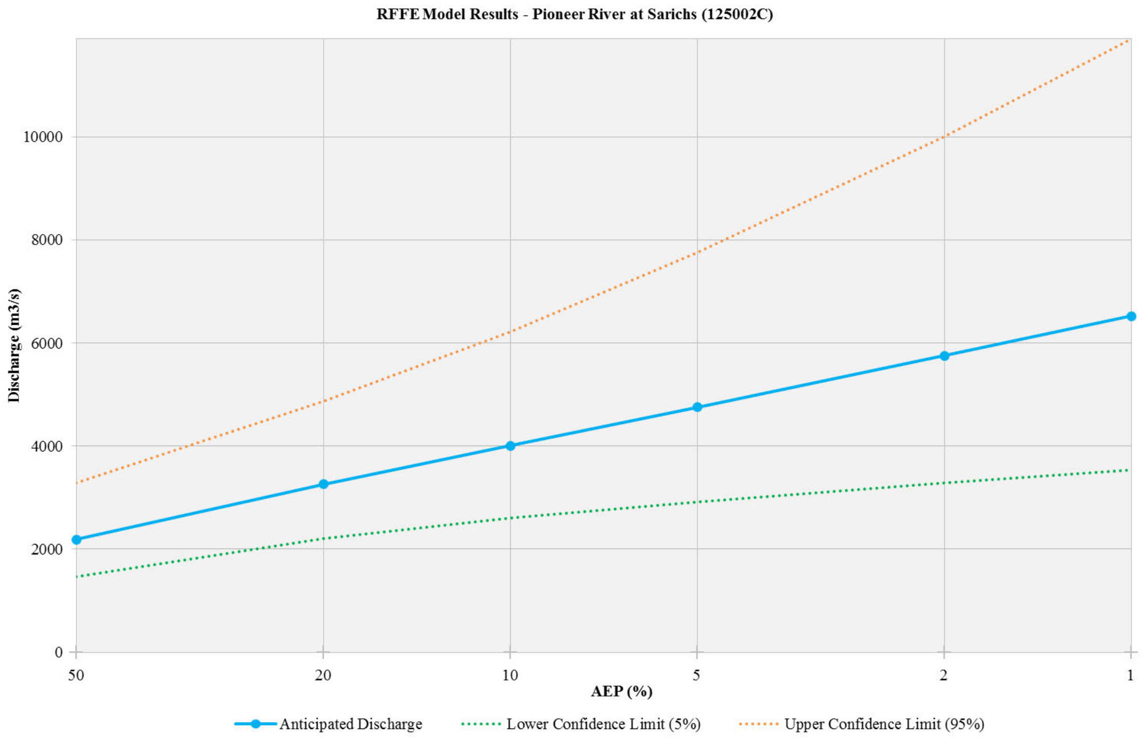

Figure 4-70: RFFE Model Results – Pioneer River at Sarichs

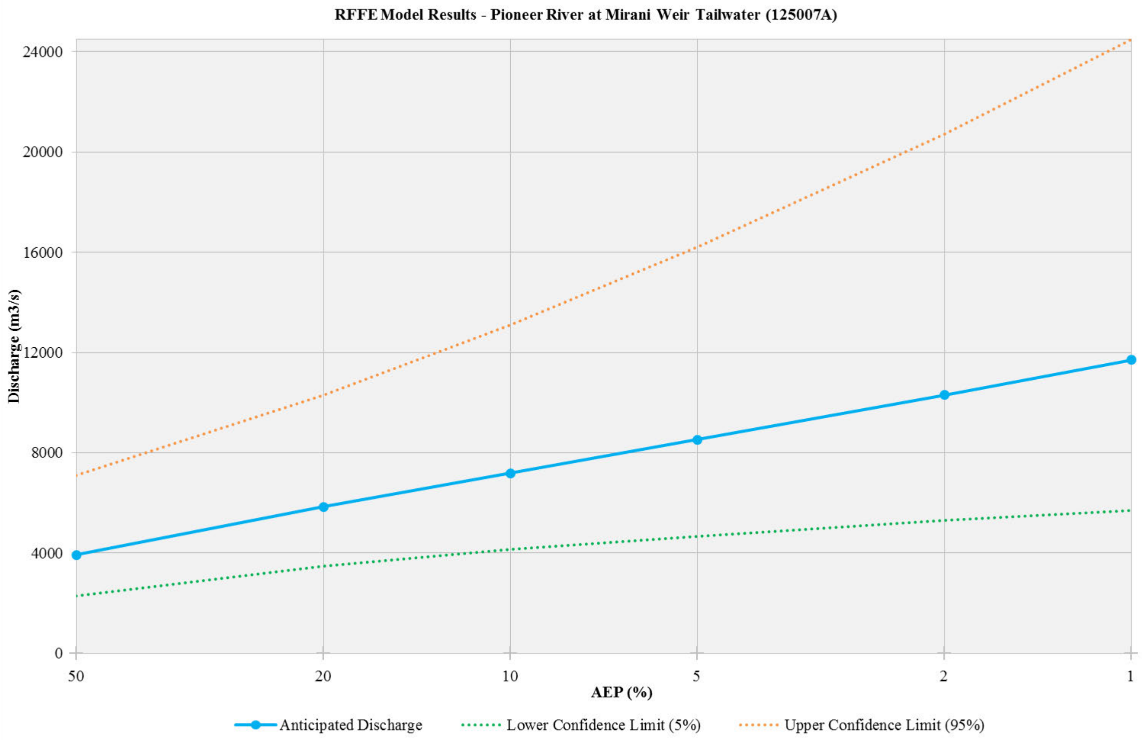

Figure 4-71: RFFE Model Results – Pioneer River at Mirani Weir Tailwater

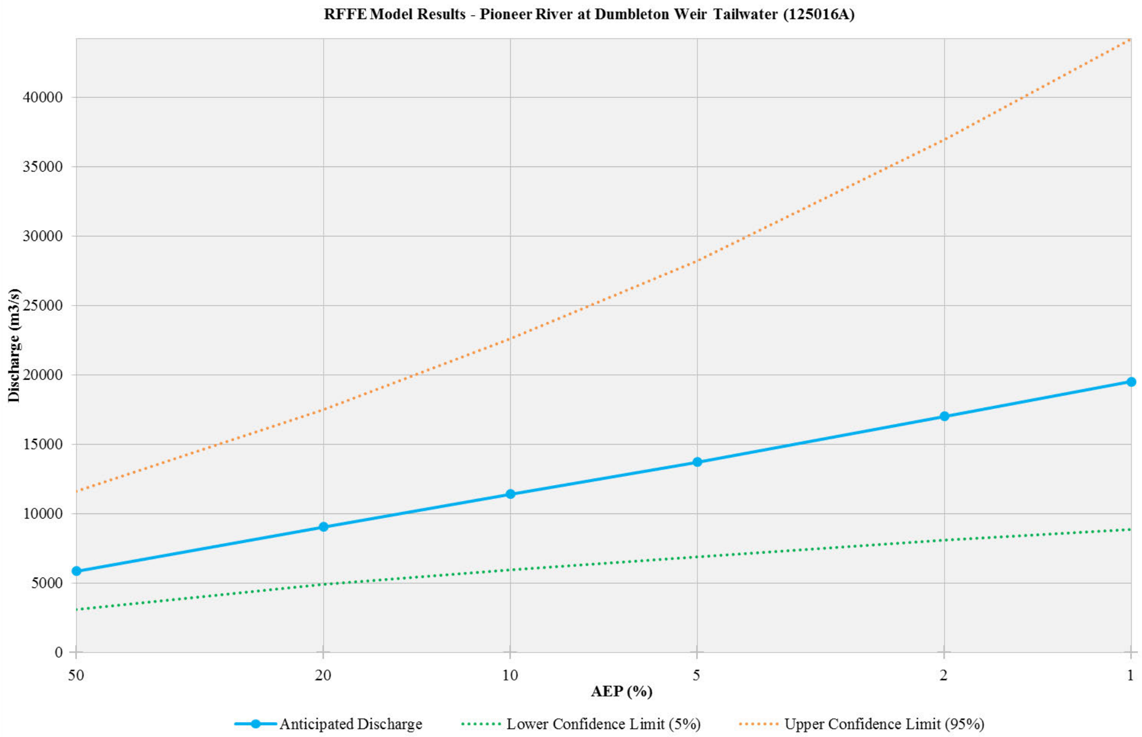

Figure 4-72: RFFE Model Results – Pioneer River at Dumbleton Weir Tailwater

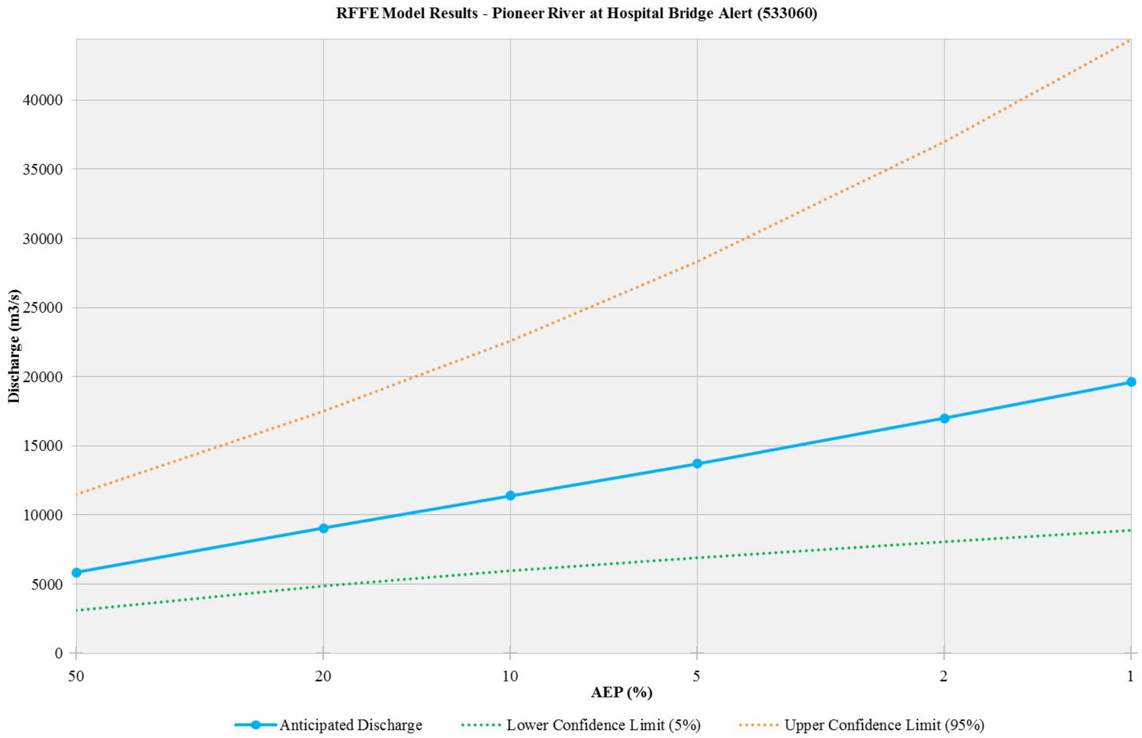

Figure 4-73: RFFE Model Results – Pioneer River at Hospital Bridge Alert

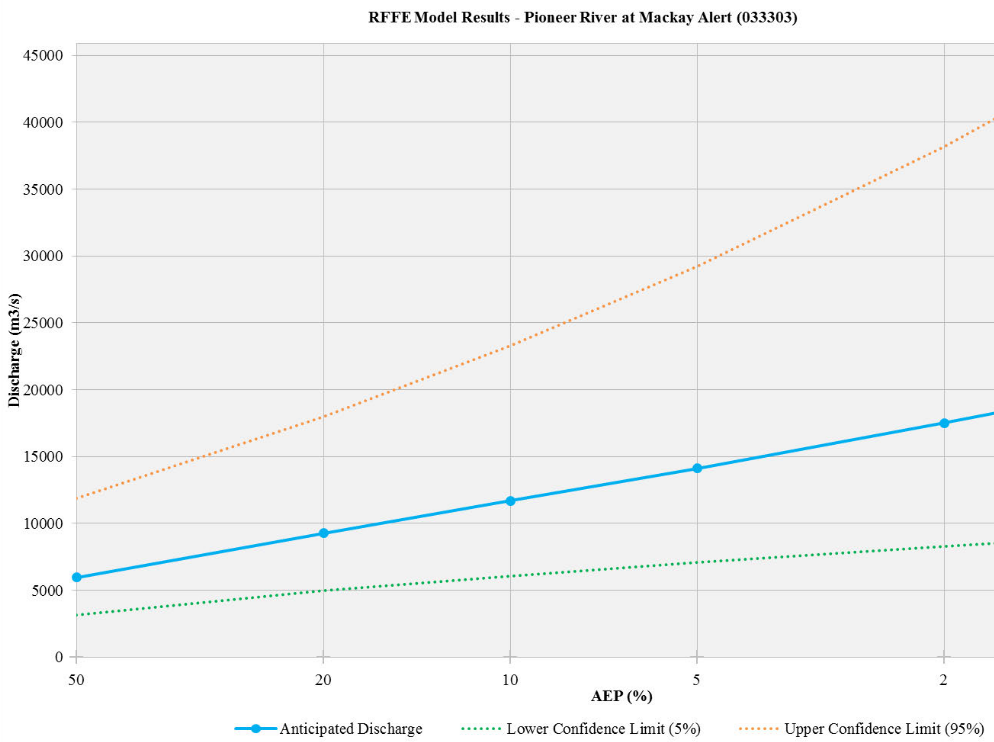

Figure 4-74: RFFE Model Results – Pioneer River at Mackay

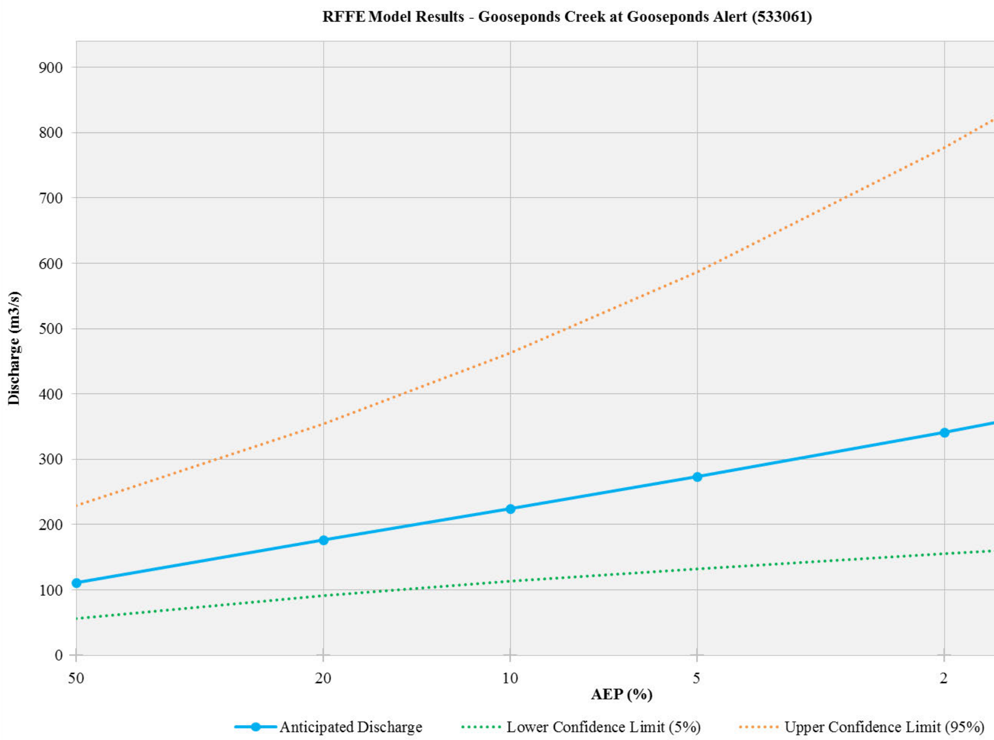

Figure 4-75: RFFE Model Results – Gooseponds Creek at Gooseponds Alert

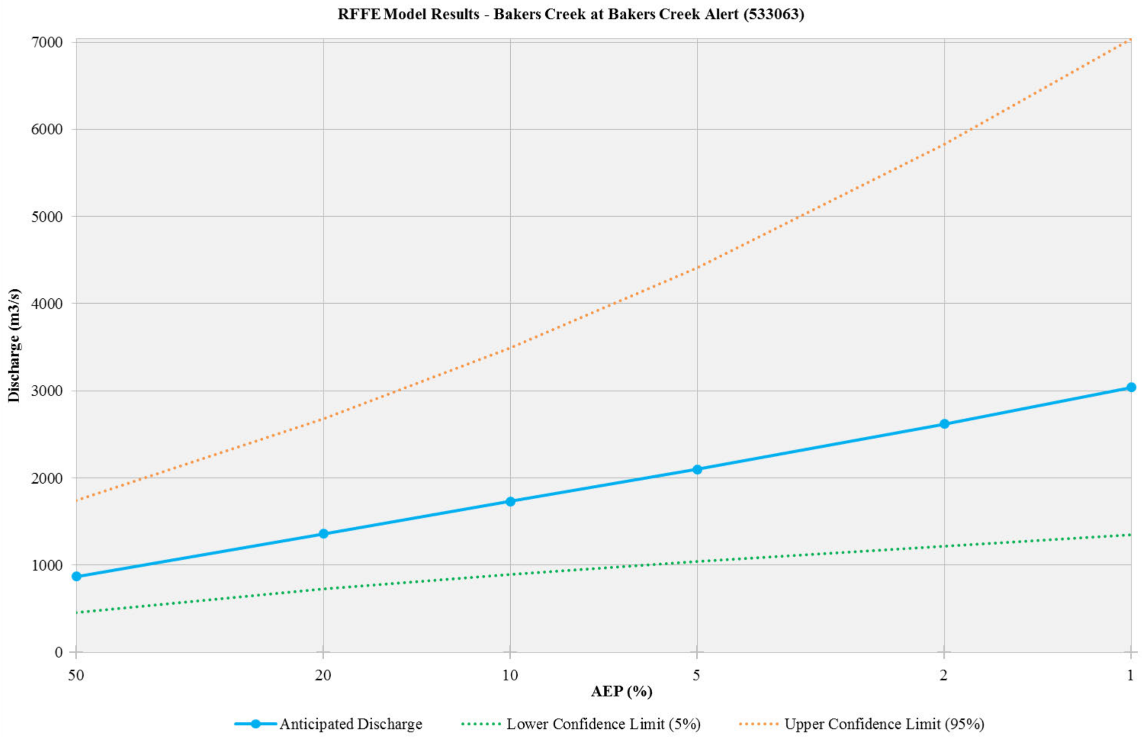

Figure 4-76: RFFE Model Results – Bakers Creek at Bakers Creek Alert

4.6. Rational Method Estimation

Table 4-17 shows the resultant peak design discharge estimates from the Rational Method estimation (refer methodology Section 3.9), using the catchment parameters input from the WBNM subcatchment delineation (refer Figure 3-11). Results have been generated for each assessment location as shown in Figure 4-1. The estimated discharge values for each location have been summarised in the comparison of results table for each location (refer Chapter 5).

Table 4-17: Rational Method Estimation – Comparison of Results

Location	Annual Exceedance Probability (AEP) (%)						
	1EY	50%	20%	10%	5%	2%	1%
Cattle Creek at Highams	839	1098	1449	1670	2205	2950	3509
Cattle Creek at Gargett	1140	1495	1987	2303	3050	4118	4885
Blacks Creek at Whitefords	1713	2243	2986	3461	4586	6176	7354
Pioneer River at Sarichs	1907	2515	3382	3942	5252	7078	8514
Pioneer River at Mirani Weir	2669	3507	4744	5540	7400	9997	11991
Pioneer River at Dumbleton	2611	3438	4699	5493	7365	10002	12012
Pioneer River at Hospital Bridge	2774	3653	4994	5838	7828	10631	12767
Pioneer River at Mackay Alert	2799	3684	5039	5893	7902	10732	12888
Gooseponds Alert	56	73	99	115	153	207	249
Bakers Creek Alert	308	407	552	643	861	1167	1401

CHAPTER 5

5. DISCUSSION

This chapter provides a direct comparison of the results from investigation processes (refer Chapter 4) at each key assessment location (refer Figure 4-1).

The results have been separated into a tabular and graphical format for each location and are described in more detail in the subsequent sections. The overall correlation of the results from each assessment method (refer Section 5.11) and areas for further work (refer Section 5.12) have also been identified.

The at site Flood Frequency Analysis (FFA) results are made up of the partial series estimation (refer Section 2.3.3) for the events ranging from 1EY to 10% AEP, with the annual series FLIKE assessment results (refer Section 2.3.2) for the 5% to 1% AEP events as per recommendations from the ARR 2016 Guidelines (refer Section 2.3.4).

5.1. Cattle Creek at Highams Bridge (125009A)

The results from the analysis at the DNRM Cattle Creek at Highams Bridge site can be seen in Figure 5-1. Overall, there is a good degree of correlation between the analysis methods undertaken at this site, with the at site FFA results falling below the other methods. As this site is in the upper, more rural reaches of the Pioneer River the TUFLOW hydrodynamic model extents did not cover the site. The RFFE model produced an estimation similar to that of the runoff routing model and Rational Method estimation. This is most likely due to the small, circular size of the catchment which seems to deliver the best results. The correlation of the results is discussed further in Section 5.11.

Table 5-1: Comparison of Results – Cattle Creek at Highams Bridge

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
FFA	116	332	621	840	921	972	992
WBNM 2016	166	466	1040	1683	2188	2533	4345
WBNM 1987	746	1079	1622	2004	2519	2834	3367
RFFE	-	745	1250	1630	2030	2600	3060
Rational	839	1097	1449	1670	2204	2950	3509

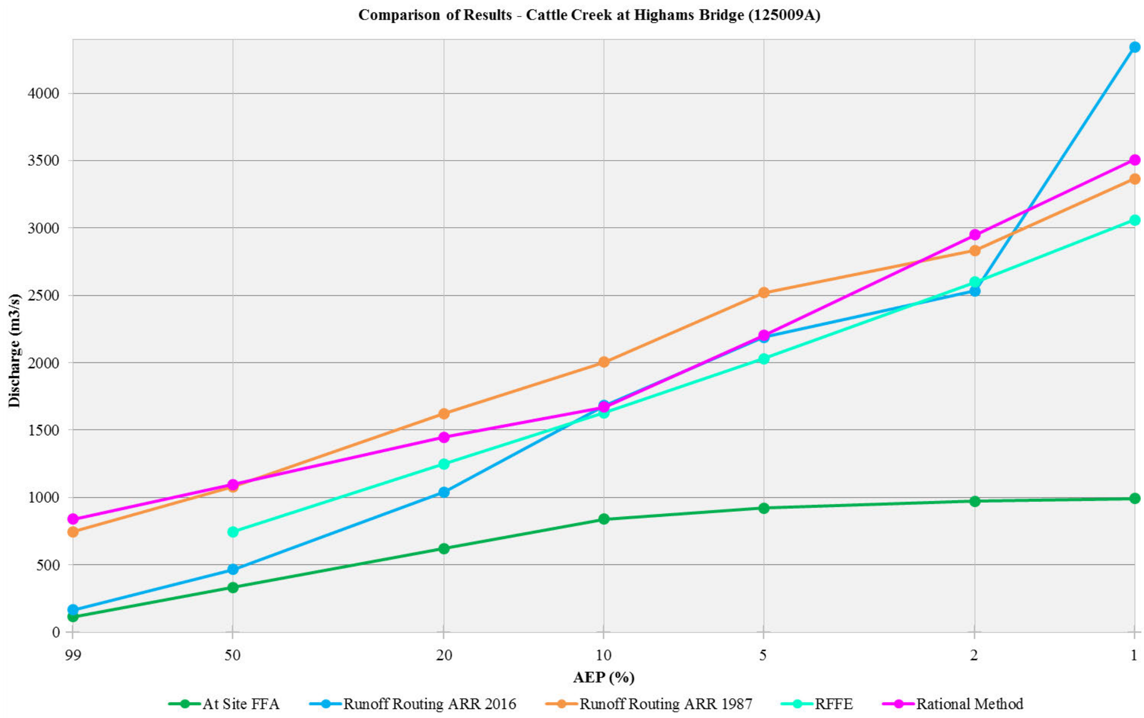

Figure 5-1: Comparison of Results – Cattle Creek at Highams Bridge

5.2. Cattle Creek at Gargett (125004B)

The results from the analysis at the DNRM Cattle Creek at Gargett site can be seen in Figure 5-2. Similar to other sites nearby, there is a good degree of correlation between the analysis methods undertaken at the site, with the at site FFA results falling only slightly below the other methods. As this site is in the upper, more rural reaches of the Pioneer River the TUFLOW hydrodynamic model extents did not cover the site. The RFFE model produced an estimation similar to that of the runoff routing model and Rational Method estimation. This is most likely due to the small, more circular size of the catchment which seems to deliver the best results. The correlation of the results is discussed further in Section 5.11.

Table 5-2: Comparison of Results – Cattle Creek at Gargett

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
FFA	622	936	1357	1675	2137	2514	2767
WBNM 2016	256	695	1534	2532	3308	3377	5202
WBNM 1987	1101	1588	2379	2919	3638	4209	4971
RFFE	-	1500	2230	2720	3210	3860	4360
Rational	1140	1494	1986	2303	3050	4117	4884

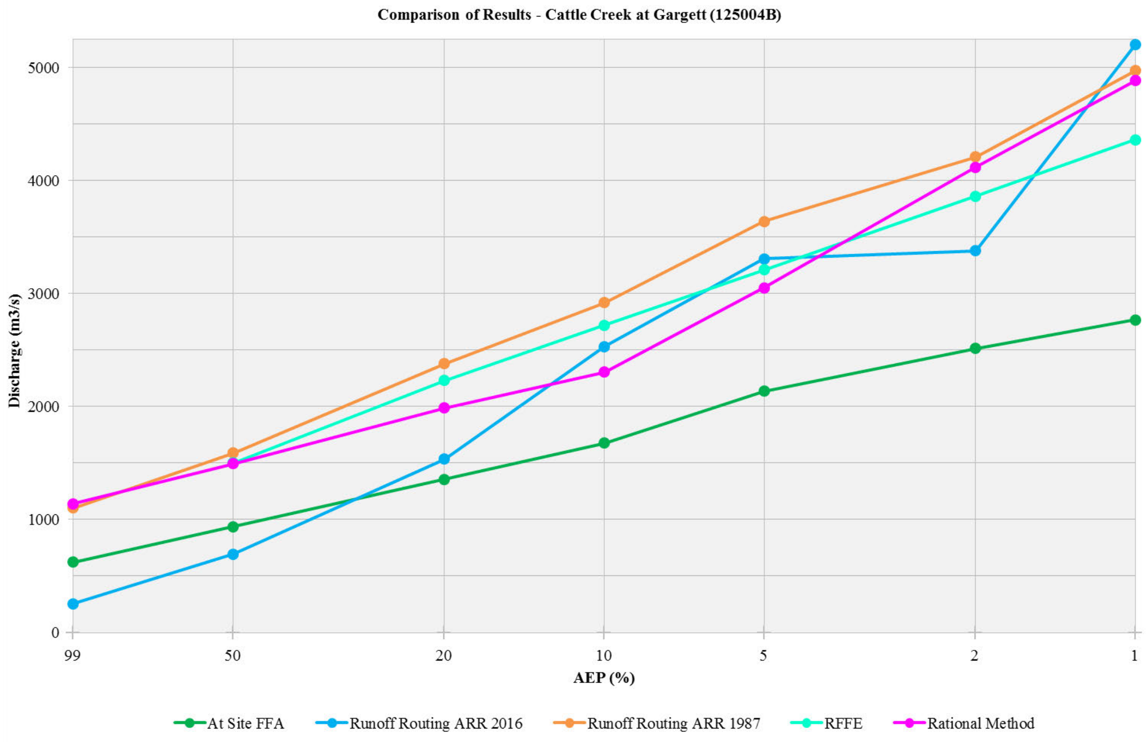

Figure 5-2: Comparison of Results – Cattle Creek at Gargett

5.3. Blacks Creek at Whitefords (125005A)

The results from the analysis at the DNRM Blacks Creek at Whitefords site can be seen in Figure 5-3. At this site the correlation between the results of the different estimation methods begins to spread, bounded by the ARR 1987 runoff routing model and the at site FFA as the upper and lower bounds respectively. As this site is in the upper, more rural reaches of the Pioneer River the TUFLOW hydrodynamic model extents did not cover the site. The RFFE model resulted in a peak discharge similar to that of the Rational Method estimation. This is most likely due to the small, more circular size of the catchment which seems to deliver the best results. The correlation of the results is discussed further in Section 5.11.

Table 5-3: Comparison of Results – Blacks Creek at Whitefords

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
FFA	335	855	1552	2080	2930	3386	3642
WBNM 2016	360	987	2074	3221	4169	4173	4915
WBNM 1987	1639	2421	3745	4651	4854	6755	8067
RFFE	-	2190	3280	4050	4810	5840	6650
Rational	1712	2243	2985	3461	4586	6175	7354

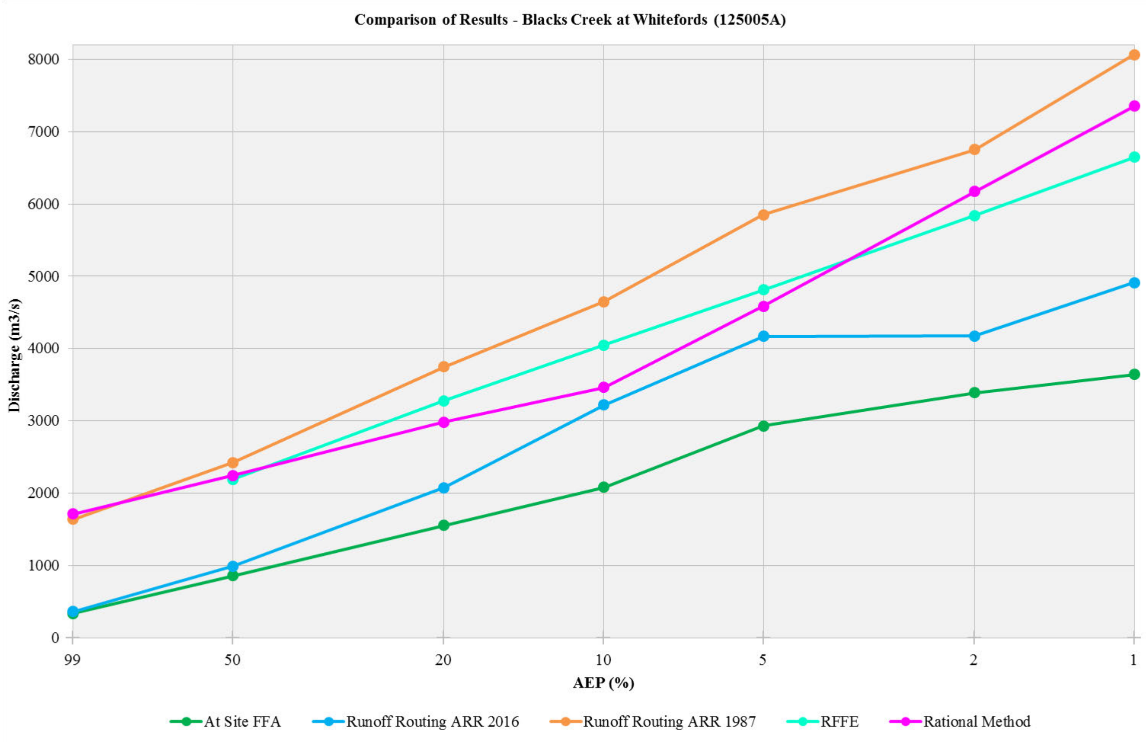

Figure 5-3: Comparison of Results – Blacks Creek at Whitefords

5.4. Pioneer River at Sarichs (125002C)

The results from the analysis at the DNRM Pioneer River at Sarichs site can be seen in Figure 5-4. Similar to other sites nearby, the correlation between the results of the different estimation methods begins to spread, bounded by the ARR 1987 runoff routing model and the at site FFA as the upper and lower bounds respectively for the higher order events. As this site is in the upper, more rural reaches of the Pioneer River the TUFLOW hydrodynamic model extents did not cover the site. The RFFE model resulted in a peak discharge similar to that of the Rational Method estimation and ARR 2016 runoff routing modelling. This is most likely due to the smaller, more circular size of the catchment which seems to deliver the best results. The correlation of the results is discussed further in Section 5.11.

Table 5-4: Comparison of Results – Pioneer River at Sarichs

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
FFA	1292	2013	2981	3712	4403	4843	5045
WBNM 2016	408	952	2212	3446	4632	5320	6306
WBNM 1987	1718	2534	3918	4872	6144	7275	8681
RFFE	-	2190	3260	4010	4750	5750	6520
Rational	1906	2515	3381	3942	5251	7077	8513

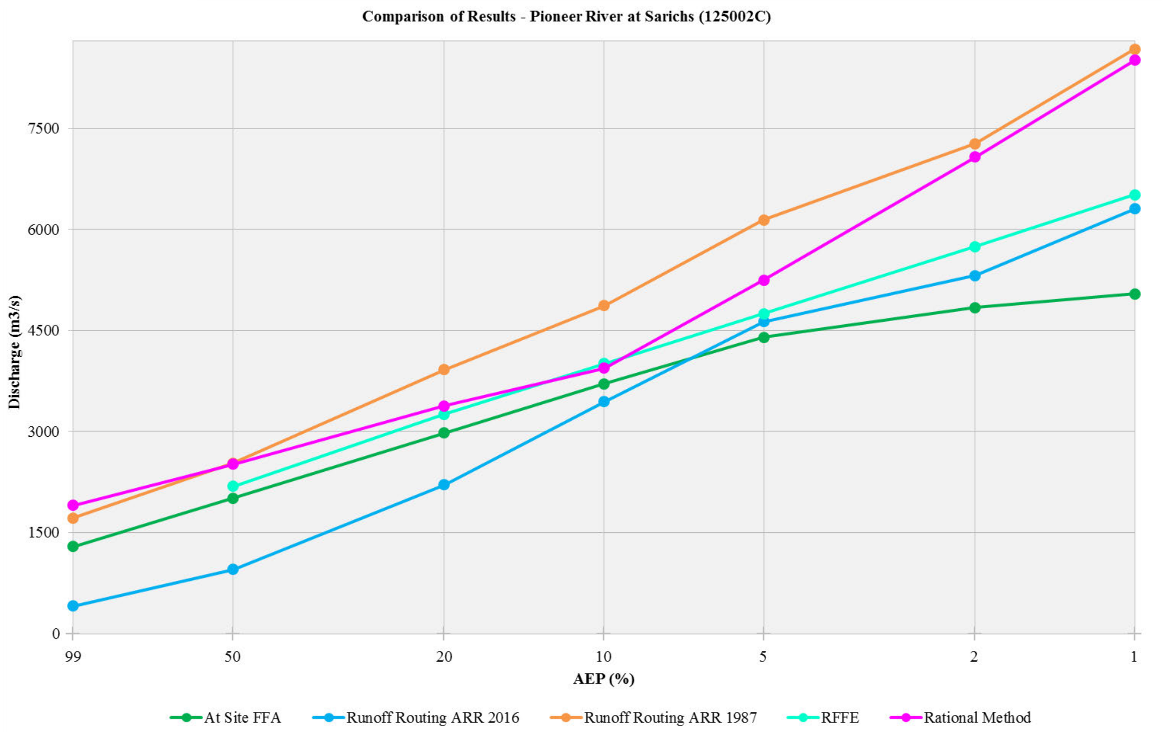

Figure 5-4: Comparison of Results – Pioneer River at Sarichs

5.5. Pioneer River at Mirani Weir Tailwater (125007A)

The results from the analysis at the DNRM Pioneer River at Mirani Weir Tailwater site can be seen in Figure 5-5. Similar to other sites nearby, there is a spread between the results of the different estimation methods, increasing in the higher order events. The results are bounded by the ARR 1987 runoff routing model and the at site FFA as the upper and lower bounds respectively for the majority of events. As this site is in the upper, more rural reaches of the Pioneer River the TUFLOW hydrodynamic model extents did not cover the site. The RFFE model begins to overestimate the peak discharge at this site when compared to the other methods. This is most likely due to the increasing size and unusual shape of the catchment causing uncertainties in the model. The correlation of the results is discussed further in Section 5.11.

Table 5-5: Comparison of Results – Pioneer River at Mirani Weir Tailwater

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
FFA	1147	2136	3463	4467	5462	6195	6600
WBNM 2016	582	1400	3312	5097	6868	8370	9848
WBNM 1987	2422	3589	5613	7067	8870	10524	12525
RFFE	-	3920	5850	7190	8530	10300	11700
Rational	2668	3507	4744	5540	7400	9997	11991

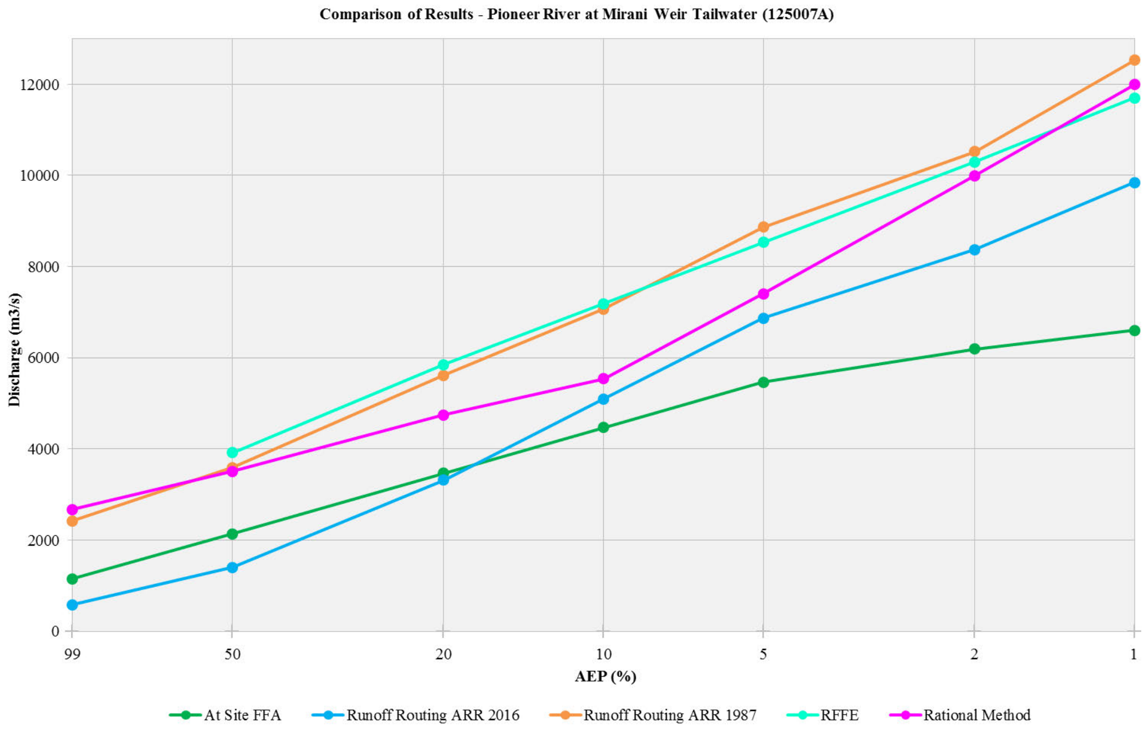

Figure 5-5: Comparison of Results – Pioneer River at Mirani Weir Tailwater

5.6. Pioneer River at Dumbleton Weir Tailwater (125016A)

The results from the analysis at the DNRM Pioneer River at Dumbleton Weir Tailwater site can be seen in Figure 5-6. Similar to other sites nearby, there is a spread between the results of the different estimation methods, increasing in the higher order events. The data available at this site did not allow for an at site FFA assessment to be undertaken. The results are bounded by the RFFE model and ARR 2016 runoff routing model as the upper and lower bounds respectively for all events. As this site is in the middle reaches of the Pioneer River the TUFLOW hydrodynamic model was not used as an assessment location for the comparison of methods. The RFFE model begins to significantly overestimate the peak discharge at this site when compared to the other methods. This is due to the catchment size being over the models threshold and unusual shape of the catchment causing uncertainties in the model. The correlation of the results is discussed further in Section 5.11.

Table 5-6: Comparison of Results – Pioneer River at Dumbleton Weir Tailwater

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
WBNM 2016	571	1477	3430	5233	7189	8939	10676
WBNM 1987	2419	3604	5672	7127	9050	10877	13015
RFFE	-	5850	9040	11400	13700	1700	19500
Rational	2610	3438	4698	5493	7364	10002	12012

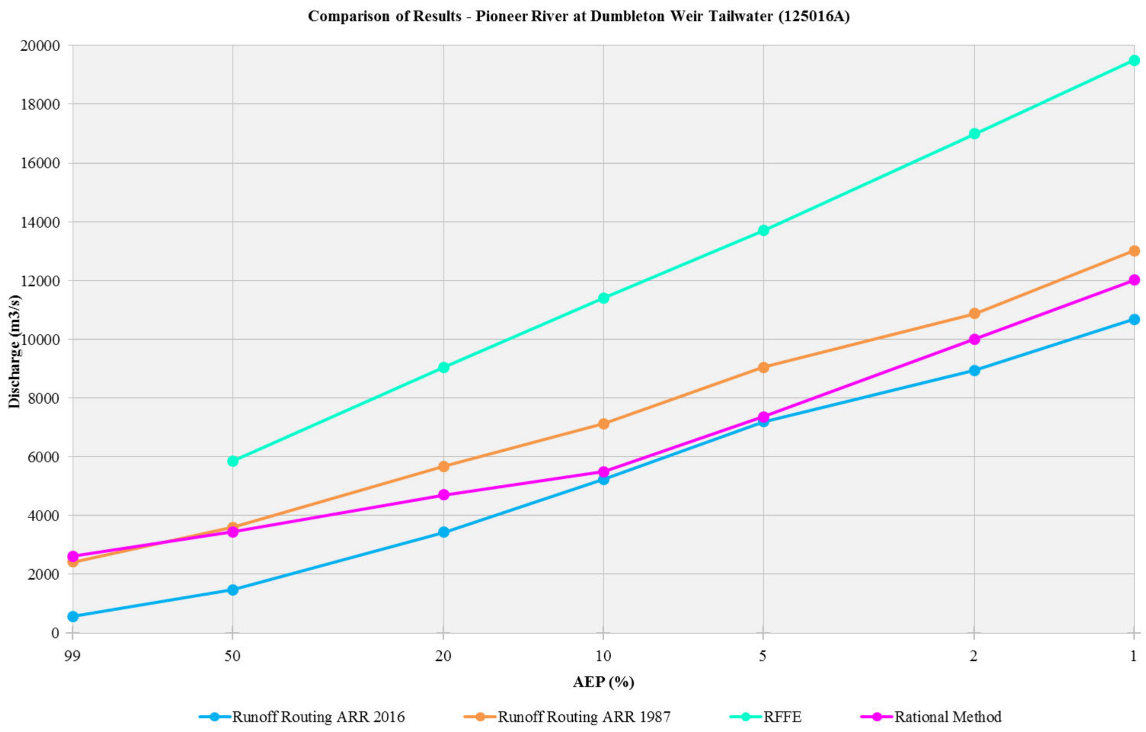

Figure 5-6: Comparison of Results – Pioneer River at Dumbleton Weir Tailwater

5.7. Pioneer River at Hospital Bridge Alert (533060)

The results from the analysis at the BoM Pioneer River at Hospital Bridge alert station can be seen in Figure 5-7. Similar to other sites nearby, there is a spread between the results of the different estimation methods, increasing in the higher order events. As there was no streamflow data at this site an at site FFA assessment could not be undertaken. The results are bounded by the RFFE model and ARR 2016 runoff routing model as the upper and lower bounds respectively for all events, similar to the upstream and downstream assessment locations. This site is located within TUFLOW hydrodynamic model extents, which from the results shows that overbank breakout flows occur in higher order events with the peak discharge being less than that of the runoff routing model. The RFFE model significantly overestimates the peak discharge at this site compared to the other methods. This is due to the catchment size being over the models threshold and unusual shape of the catchment causing uncertainties in the model. The correlation of the results is discussed further in Section 5.11.

Table 5-7: Comparison of Results – Pioneer River at Hospital Bridge Alert

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
WBNM 2016	582	1518	3546	5368	7383	9165	10951
WBNM 1987	2477	3692	5802	7292	9257	11131	13321
TUFLOW 2016	1977	2784	6428	9328	10887	11100	11288
TUFLOW 1987	4558	6777	9847	10905	11683	11973	12276
RFFE	-	5830	9030	11400	13700	17000	19600
Rational	2774	3652	4993	5838	7827	10630	12767

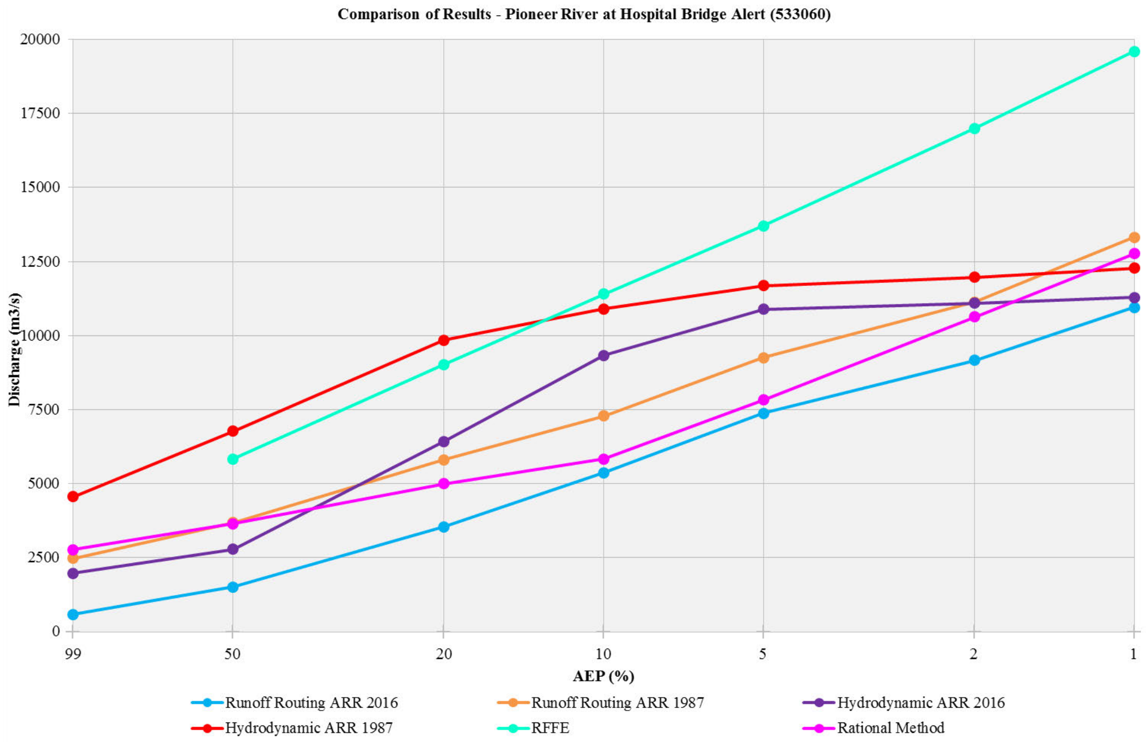

Figure 5-7: Comparison of Results – Pioneer River at Hospital Bridge

5.8. Pioneer River at Mackay Alert (033303)

The results from the analysis at the BoM Pioneer River at Mackay Alert station can be seen in Figure 5-8. Similar to other sites nearby, there is a spread between the results of the different estimation methods, increasing in the higher order events. As there was no streamflow data at this site an at site FFA assessment could not be undertaken. The results are bounded by the RFFE model and ARR 2016 runoff routing model as the upper and lower bounds respectively for all events, similar to the upstream assessment locations. This site is located within TUFLOW hydrodynamic model extents, which from the results shows that overbank breakout flows occur in higher order events with the peak discharge being less than that of the runoff routing model. The RFFE model significantly overestimates the peak discharge at this site compared to the other methods. This is due to the catchment size being over the models threshold and unusual shape of the catchment causing uncertainties in the model. The correlation of the results is discussed further in Section 5.11.

Table 5-8: Comparison of Results – Pioneer River at Mackay Alert

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
WBNM 2016	582	1513	3614	5391	7305	9152	10974
WBNM 1987	2426	3617	5700	7128	9017	10888	13024
TUFLOW 2016	1690	2383	5330	7258	9265	10801	12246
TUFLOW 1987	3845	5312	7749	9171	10692	11860	12917
RFFE	-	5960	9260	11700	14100	17500	20200
Rational	2798	3683	5039	5893	7902	10732	12888

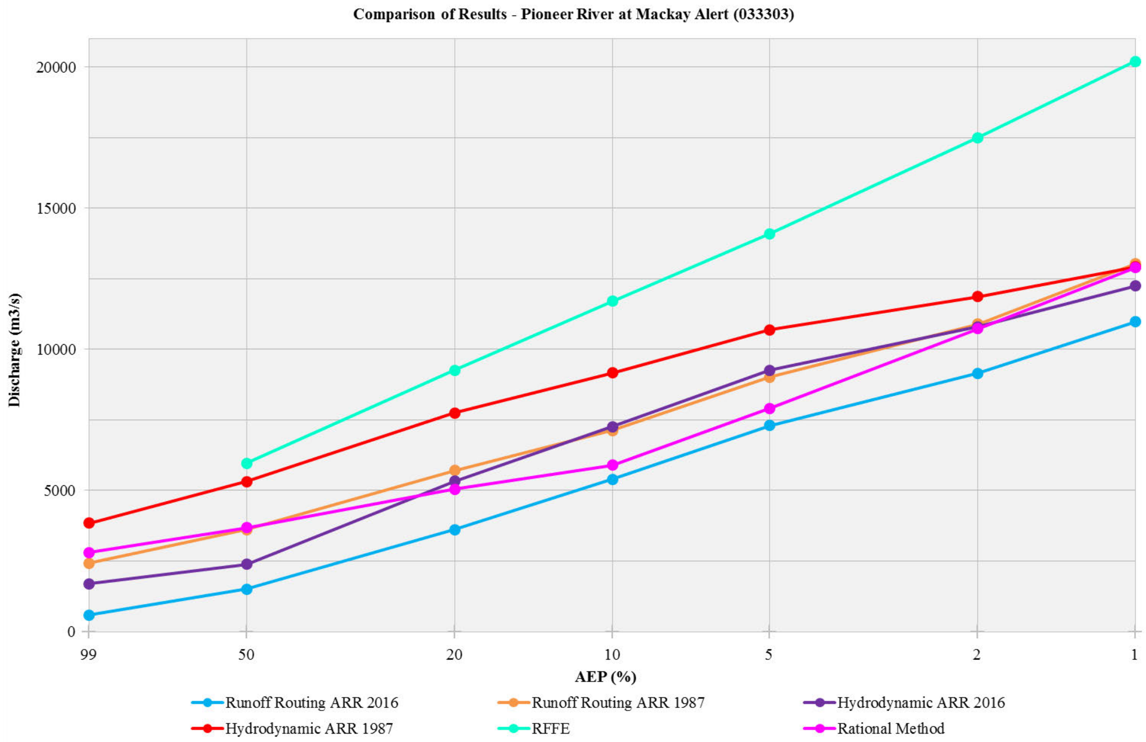

Figure 5-8: Comparison of Results – Pioneer River at Mackay Alert

5.9. Gooseponds Creek at Gooseponds Alert (533061)

The results from the analysis at the BoM Gooseponds Creek at Gooseponds Alert station can be seen in Figure 5-9. There is a spread between the results of the different estimation methods, increasing in the higher order events. As there was no streamflow data at this site an at site FFA assessment could not be undertaken. The results are bounded by the RFFE model and ARR 2016 hydrodynamic model as the upper and lower bounds respectively for all events. This site is located within TUFLOW hydrodynamic model extents, which from the results shows peak discharges significantly lower than that of the runoff routing model. This is most likely due to complex flowpaths within the catchment and cross catchment flow that is not represented within the runoff routing model. The RFFE model significantly overestimates the peak discharge at this site compared to the other methods. This is due to the unusual shape of the catchment causing uncertainties in the model and the lack of streamflow gauges influencing the estimation. The correlation of the results is discussed further in Section 5.11.

Table 5-9: Comparison of Results – Goosepondss Creek at Gooseponds Alert

Method	Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)						
	1EY	50%	20%	10%	5%	2%	1%
WBNM 2016	8	21	47	86	125	125	140
WBNM 1987	23	38	75	103	141	168	210
TUFLOW 2016	6	11	30	44	60	60	137
TUFLOW 1987	13	23	36	46	92	92	200
RFFE	-	111	176	224	273	341	396
Rational	55	73	98	115	153	206	248

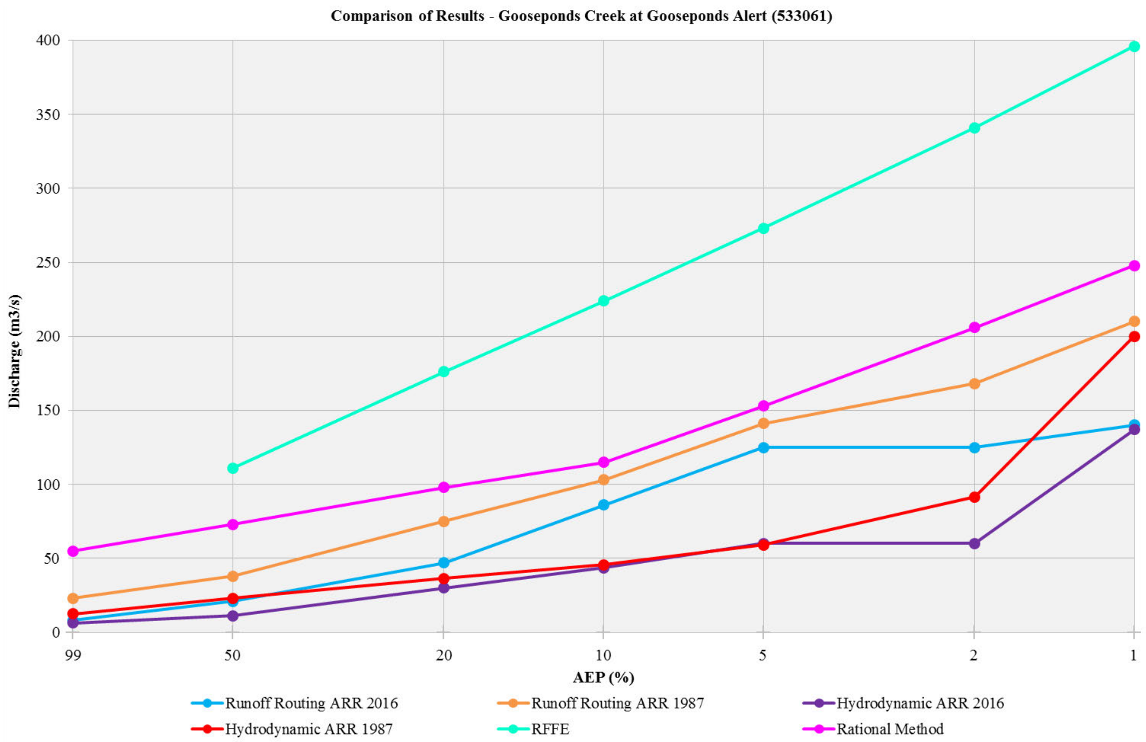

Figure 5-9: Comparison of Results – Gooseponds Creek at Gooseponds Alert

5.10. Bakers Creek at Bakers Creek Alert (533063)

The results from the analysis at the BoM Bakers Creek at Bakers Creek Alert station can be seen in Figure 5-10. There is a high level of correlation between the results of the different estimation methods, with the RFFE model results being an outlier of the other methods. As there was no streamflow data at this site an at site FFA assessment could not be undertaken. The results are bounded by the RFFE model and ARR 2016 hydrodynamic model as the upper and lower bounds respectively for the majority of events. This site is located within TUFLOW hydrodynamic model extents, which from the results shows peak discharges are on average lower than that of the runoff routing model. This is most likely due to complex flowpaths within the catchment and cross catchment flow that is not represented within the runoff routing model. The RFFE model significantly overestimates the peak discharge at this site compared to the other methods. This is due to the unusual shape of the catchment causing uncertainties in the model and the lack of streamflow gauges influencing the estimation. The correlation of the results is discussed further in Section 5.11.

Table 5-10: Comparison of Results – Bakers Creek at Bakers Creek Alert

Method	**Annual Exceedance Probability (AEP) (%) Peak Discharge (m^3/s)**						
	1EY	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
WBNM 2016	19	38	177	460	760	760	1194
WBNM 1987	53	167	461	678	975	1181	1508
TUFLOW 2016	29	56	151	272	611	873	1226
TUFLOW 1987	50	112	203	329	601	922	1415
RFFE	-	868	1360	1730	2100	2620	3040
Rational	308	406	551	643	860	1167	1401

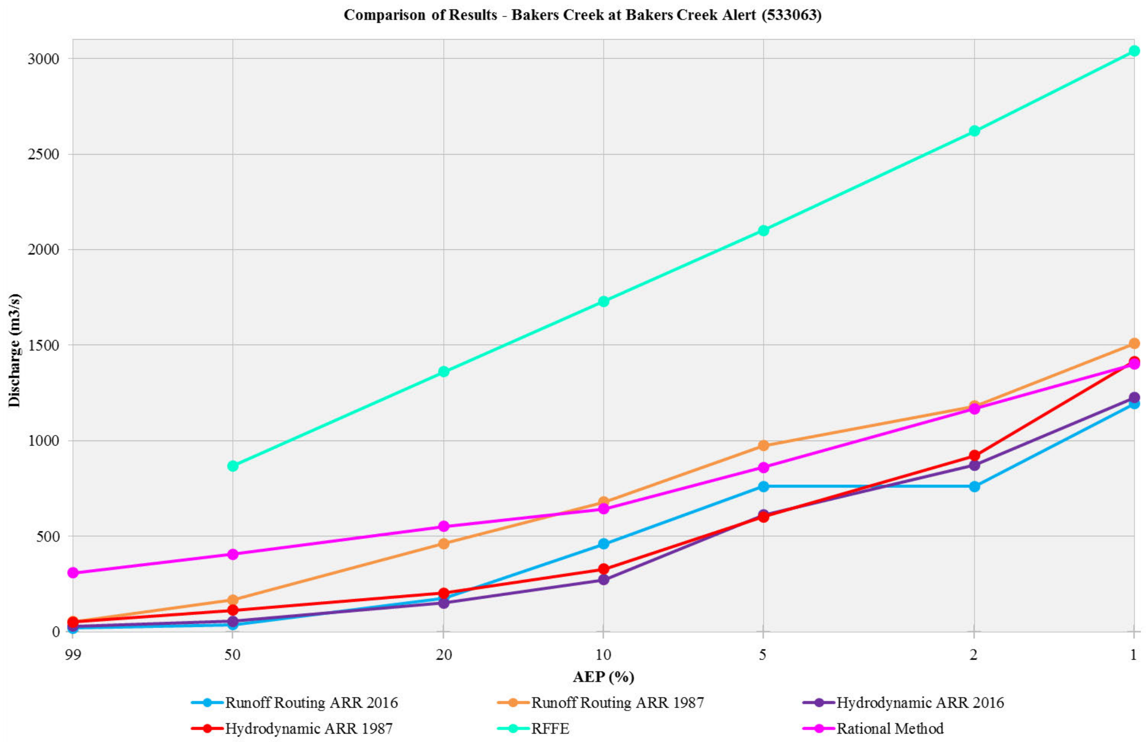

Figure 5-10: Comparison of Results – Bakers Creek at Bakers Creek Alert

5.11. Correlation of Results

From the results in the previous sections, the following observations have been made on the correlation of results between peak flow estimation methods.

The at site FFA assessments seem to return design peak discharges that are lower than that of other estimation methods at most locations. Upon review of the stream gauge data, it was found that when gauges in the region capture a flood peak the classification of the quality of the captured data changes. In most cases, flood peaks were captured as a 'derived height' or 'estimate' reading by the stream gauge and in some cases classified as a 'poor' reading. The combination of this possible inaccuracy of results along with the limited length of stream gauge data on record is suspected to be the reasoning behind the lower peak discharge estimates. The average length of data on record for the stream gauges is around 20 years, making it difficult to extrapolate and predict a 100 year (1% AEP) peak discharge.

The WBNM runoff routing model was found to estimate discharges on the higher end of the results spectrum. This is somewhat expected and is common with runoff routing modelling methods as cross catchment connectivity and breakout or overbank flows are not represented in the modelling process. The discharge estimates produced from the runoff routing model using the ARR 1987 hydrology are larger than that of the ARR 2016 hydrology counterparts (with the exception of some boundary catchments). This is attributed to the increase in the continuing loss rate and also the decrease in total rainfall depths and lowering of intensities (refer Appendix C).

On the contrary to the runoff routing model, the TUFLOW hydrodynamic model results show peak discharges slightly lower than that of the WBNM runoff routing model. This is attributed to the hydrodynamic model's ability to model breakout and overbank flows (particularly in higher order events) and create a cross connectivity between subcatchments allowing for 'free' routing of flow. Similar to the runoff routing model, discharge estimates produced using the ARR 1987 hydrology are larger than that of the ARR 2016 hydrology estimates. Likewise, this is attributed to the increase in the continuing loss rate and also the decrease in total rainfall depths and lowering of intensities (refer Appendix C).

The Regional Flood Frequency Estimation (RFFE) Model appears to deliver a variety of results dependent on the catchment configuration and input parameters into the online model. In the upper reaches of the Pioneer River catchment, where catchment sizes are smaller than the models threshold (>1,000km^2) and the shape of the catchments is more circular in nature (i.e. centroid and outlet location relative to the catchment area), peak discharge estimations seem to be comparative to more complex modelling methods. However, in catchments further downstream, where the total catchment area exceeds that of the threshold and the long, skinny shape comes into effect, results begin to become exaggerated and exceed that of other methods (up to 55% increase at the Mackay Alert).

The Rational Method estimation was found to deliver results acceptable for its current prescription in the industry. In most cases the Rational Method estimation was similar to that of the ARR 1987 runoff routing model results. This is because ARR 1987 rainfall was used for the Rational Method calculation (refer Section 2.5). As this method has not been recommended for use with ARR 2016 hydrology it has not been assessed. As the results don't seem to follow a trend of being consistently higher or lower the relationship between the Rational Method and other estimation techniques is hard to derive, however its application as a ‘sanity check’ of modelled results proves as acceptable as per the investigation findings.

5.12. Further Work

In order to gain a full understanding of how sensitive each estimation method is in regards to catchment input parameters, an investigation into each parameter separately would be required. As this investigation was focussed more about a large scale catchment comparison of calculation methods/software, each parameter was not investigated for its possible influences on the result. Further work may involve an assessment into the variability of results for a smaller catchment based on the input parameters. These parameters have been identified as:

- Initial/continuing loss values,
- Selection method of the ‘most representative’ Ensemble temporal pattern,
- Preburst rainfall values (compared to the median preburst value), and
- Co-incident events between catchments including tidal events.

Each of these parameters have been identified and described as part of the literature review (refer Chapter 2) with the assumptions made as part of this investigation being detailed in the study methodology (refer Chapter 3).

Another possible area for further work would be the application of direct rainfall modelling (refer Section 2.7.6). This method would remove the reliance on the WBNM runoff routing model flows in the TUFLOW hydrodynamic modelling investigation. This approach was excluded due to the long duration model run time that would be required for such a large catchment. By estimating peak flow using the direct rainfall modelling method, the design rainfall would be applied straight on to the hydrodynamic terrain with flowpaths and losses calculated within the model computations. This would be expected to deliver different and possibly substantially dissimilar results.

CHAPTER 6

6. CONCLUSION

From the investigations, it was determined that the Rational Method appears to still be acceptable as a high level design peak discharge estimation or as a ‘sanity check’ for outputs from more complex modelling techniques. In most cases the Rational Method estimation was similar to that of the ARR 1987 runoff routing model, this is attributed to the input hydrology the Rational Method requires. The results from the investigation deliver discharge estimates acceptable for its current prescription in the industry. As the Rational Method is still published in the latest version of the Queensland Urban Drainage Manual (QUDM) (IPWEAQ 2017) it can therefore still be regarded as a prescribed estimation method within the state of Queensland.

Unlike the Rational Method, the Regional Flood Frequency Estimation (RFFE) Model was found to only deliver acceptable results in some cases, with skewed estimates and limitations in other catchments. The RFFE online tool has in input catchment size limitation of 1,000km^2, where the catchment size was larger than that of the threshold a catchment size error was returned and although discharge estimates were still computed, they were found to be much larger than that of other estimation methods. The RFFE Model also has a catchment shape factor limitation, determined from the size of the catchment in relation to the catchment’s outlet and centroid. Where the shape factor was below that of the cut off threshold (obscure shape catchment rather than circular), resultant discharge estimates were also found to be much higher than that of other methods. Therefore, in conclusion, the RFFE Model should be applied with caution as a high level design peak discharge estimation or as a ‘sanity check’ for outputs from more complex modelling techniques, similar to that of the Rational Method. If the catchment input parameters are not ideal the model will struggle to interpolate between the surrounding gauged catchments and more than likely overestimate the design discharge.

The runoff routing modelling (WBNM) was found to deliver results that in the majority of cases acted as the upper bound of the design discharge distribution. This was somewhat expected and is a common limitation of runoff routing modelling methods as cross catchment connectivity and breakout or overbank flows within the terrain are not represented in the modelling process. From this, it is recommended that this modelling method be used with caution in the Mackay Region with their limitations recognised. This method may be acceptable for high level conceptual studies or estimates as the modelling procedure is quick to undertake and will most likely deliver a conservative estimation. However, it would be highly recommended that the estimation is revised using a more rigorous technique for detailed design or hydraulically sensitive design work.

The final method assessed, the hydrodynamic modelling (TUFLOW) technique resulted in peak discharge estimates that were slightly lower than that output from the runoff routing model. This was determined to mainly be attributed to the ability that a hydrodynamic model has to model breakout and overbank flows (particularly in higher order events) over the terrain, creating cross connectivity between subcatchments allowing for 'free' routing of flow. This in turn, was found to deliver the most accurate peak design discharge estimation at the sites modelled and in turn serves as the ultimate recommendation for the estimation of peak design discharges in the Mackay Region.

The application of the Australian Rainfall and Runoff (ARR) 2016 Guideline's hydrology parameters and design rainfall depths reported lower peak discharges for the majority of events in both the runoff routing and hydrodynamic modelling applications. This was mainly due to the reduction in total design rainfall depth when compared to the ARR 1987 parameters, as well as the change in temporal patterns applying design rainfall with a lower intensity of that of the 1987 methods. These changes can be attributed to the increase in historical data that the 2016 parameters have been built upon, as well as the incorporation of climate change over time. The increase of the initial and continual rainfall loss values as per the ARR 2016 guidelines also attribute to the reduction is peak discharge estimates, however the implementation of preburst rainfall is aimed to reduce this difference.

It is recommended that the input parameters for each estimation method studied be taken forward for further investigation with a sensitivity analysis or similar to be undertaken on a smaller scale individual catchment analysis to complement this larger, more regional, high level investigation.

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APPENDIX A: PROJECT SPECIFICATION

University of Southern Queensland
FACULTY OF ENGINEERING AND SURVEYING

ENG4111 & ENG4112 Research Project

PROJECT SPECIFICATION

FOR:	Hayden BRIGG (0061075507)
TITLE:	Estimation of Peak Flows within the Mackay Region
MAJOR:	Civil Engineering (Hydrology)
SUPERVISORS:	USQ – Ian Brodie (Environmental Engineering Discipline Leader) AECOM – Melanie Collett (Technical Practice Lead – Water Resources)
SPONSORSHIP:	AECOM Australia Pty Ltd Mackay Regional Council (MRC)
CONFIDENTIALITY:	Existing flood study projects currently in the public domain will be used for the study to ensure confidentiality requirements will not be breached. Mackay Regional Council will provide any other data with consent for use in the investigation.
ENROLMENT:	ENG4111 – Semester 1, 2017 ENG4112 – Semester 2, 2017
PROJECT AIM	This project seeks to deliver a comparison between methods of estimation peak discharges within rivers and streams and their accuracy in application across the Mackay Region. The study also aims to compare changes in best practice techniques presented in the 2016 Revision of the Australian Rainfall and Runoff Guide.
PROGRAMME:	Revision A – March 15, 2017

1) Undertake a literature review to research theoretical practices for estimation of peak flows and catchment simulation such as:
   a. Rational Method,
   b. Regional Flood Frequency Estimation (RFFE),
   c. At site partial and annual Flood Frequency Analysis (FFA),
   d. Ensemble Event
   e. Hydrologic modelling software (such as XP-RAFTS, URBS, WBNM).
   f. Hydrodynamic rain on grid modelling software (TUFLOW)
2) Select a variety of (3 to 4) catchments within the Mackay Region of varying parameters and review previous hydrological investigations conducted within the area.
3) Collect and assemble data required for multiple approach hydrological assessment as researched in the literature review. Assemble hydrologic and hydrodynamic models based on available data.
4) Undertake peak flow calculations using different methodologies and approaches, tabulate/graphing results.
5) Draw conclusions on the results and why variances exist, including mathematical phenomena behind the calculation/modelling processes.
6) Ultimately recommend a method or rank methods in regards to their accuracy within the Mackay Region.
7) Write a dissertation on the findings of the project in the required format.

AGREED

(H. Brigg)	Submission Acknowledged (I. Brodie)	(M. Collett)
Date: 15 / 03 / 2017	Date: 11 / 04 / 2017	Date: 15 / 03 / 2017

APPENDIX B: ARR 1987 DESIGN RAINFALL INFORMATION

IFD Parameters

The following sections contain ARR 1987 Intensity Frequency Duration (IFD) design rainfall parameters for each rainfall catchment.

Upper Cattle Creek Catchment

Table B1 – Upper Cattle Creek ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.100°
Catchment Centroid Longitude	149.575°
${}^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	54.94 mm/hr
${}^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	14.13 mm/hr
${}^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	5.75 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	100.75 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	30.71 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	13.41 mm/hr
Skewness (G)	0.140

Table B2 – Upper Cattle Creek ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	122.0	157.0	199.0	224.0	258.0	303.0	338.0
6Mins	115.0	148.0	188.0	212.0	244.0	287.0	321.0
10Mins	96.2	124.0	157.0	177.0	203.0	239.0	267.0
20Mins	72.2	92.7	117.0	131.0	151.0	177.0	197.0
30Mins	60.0	77.0	97.0	109.0	125.0	146.0	163.0
1Hr	42.5	54.5	69.0	77.6	89.2	105.0	117.0
2Hrs	29.1	37.6	48.2	54.6	63.2	74.7	83.6
3Hrs	23.1	30.0	38.9	44.4	51.6	61.3	69.0
6Hrs	15.6	20.4	27.0	31.2	36.6	44.0	49.9
12Hrs	10.7	14.1	19.0	22.2	26.3	32.0	36.4
24Hrs	7.6	10.0	13.7	16.1	19.2	23.4	26.8

Table B3 – Upper Cattle Creek ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.75E+00	-5.24E-01	-3.39E-02	4.32E-03	1.97E-03	1.88E-04	-1.15E-04
2	4.00E+00	-5.19E-01	-2.99E-02	4.29E-03	1.60E-03	2.00E-04	-1.06E-04
5	4.23E+00	-5.06E-01	-1.94E-02	3.89E-03	7.16E-04	2.56E-04	-9.08E-05

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
10	4.35E+00	-4.99E-01	-1.38E-02	3.88E-03	2.06E-04	2.70E-04	-7.89E-05
20	4.49E+00	-4.93E-01	-9.22E-03	3.59E-03	-1.57E-04	3.06E-04	-7.48E-05
50	4.65E+00	-4.87E-01	-3.61E-03	3.49E-03	-6.57E-04	3.25E-04	-6.39E-05
100	4.76E+00	-4.82E-01	-7.54E-05	3.40E-03	-9.61E-04	3.40E-04	-5.77E-05

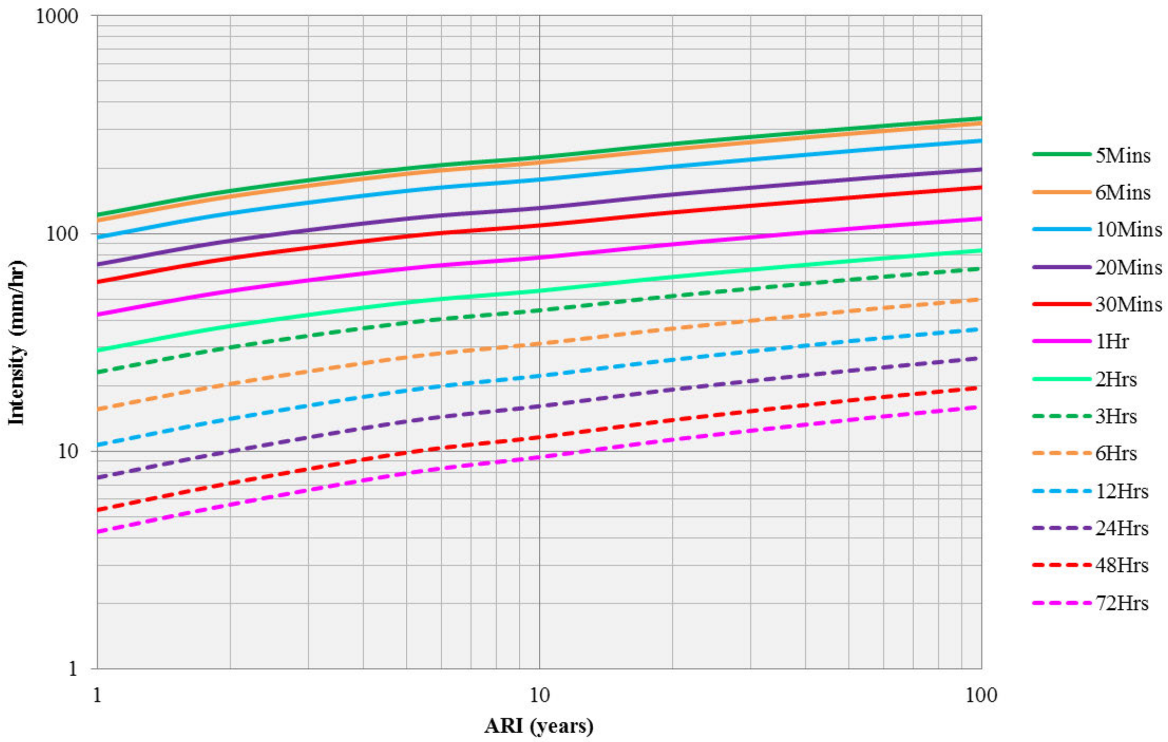


Figure B1 – Upper Cattle Creek Intensity Frequency Duration (IFD) Chart

Lower Cattle Creek Catchment

Table B4 – Lower Cattle Creek ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.125°
Catchment Centroid Longitude	148.700°
${}^{1}I_2$ (1 hour, 2 year ARI rainfall intensity)	55.97 mm/hr
${}^{12}I_2$ (12 hour, 2 year ARI rainfall intensity)	12.93 mm/hr
${}^{72}I_2$ (72 hour, 2 year ARI rainfall intensity)	4.60 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	108.17 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	29.60 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	10.84 mm/hr
Skewness (G)	0.150

Table B5 – Lower Cattle Creek ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	124.0	160.0	205.0	232.0	269.0	318.0	356.0
6Mins	117.0	150.0	193.0	219.0	254.0	300.0	336.0
10Mins	97.1	126.0	161.0	183.0	212.0	251.0	281.0
20Mins	73.3	94.7	122.0	138.0	160.0	189.0	212.0
30Mins	60.8	78.6	101.0	114.0	133.0	157.0	176.0
1Hr	42.6	55.1	71.2	81.0	94.1	112.0	126.0
2Hrs	28.6	37.2	48.8	55.9	65.4	78.3	88.4
3Hrs	22.3	29.2	38.7	44.7	52.6	63.3	71.8
6Hrs	14.6	19.2	26.1	30.5	36.2	44.1	50.4
12Hrs	9.7	12.8	17.7	20.9	25.0	30.7	35.3
24Hrs	6.6	8.7	12.1	14.4	17.3	21.3	24.6

Table B6 – Lower Cattle Creek ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.75E+00	-5.48E-01	-4.50E-02	6.29E-03	2.33E-03	-9.60E-05	-8.05E-05
2	4.01E+00	-5.43E-01	-4.11E-02	6.41E-03	1.92E-03	-1.05E-04	-6.75E-05
5	4.27E+00	-5.28E-01	-3.20E-02	6.52E-03	1.04E-03	-1.32E-04	-3.94E-05
10	4.39E+00	-5.19E-01	-2.70E-02	6.37E-03	5.66E-04	-1.19E-04	-2.86E-05
20	4.54E+00	-5.13E-01	-2.27E-02	6.40E-03	1.23E-04	-1.26E-04	-1.49E-05
50	4.72E+00	-5.05E-01	-1.77E-02	6.37E-03	-3.57E-04	-1.27E-04	-1.44E-06
100	4.83E+00	-4.99E-01	-1.43E-02	6.46E-03	-7.12E-04	-1.39E-04	1.03E-05

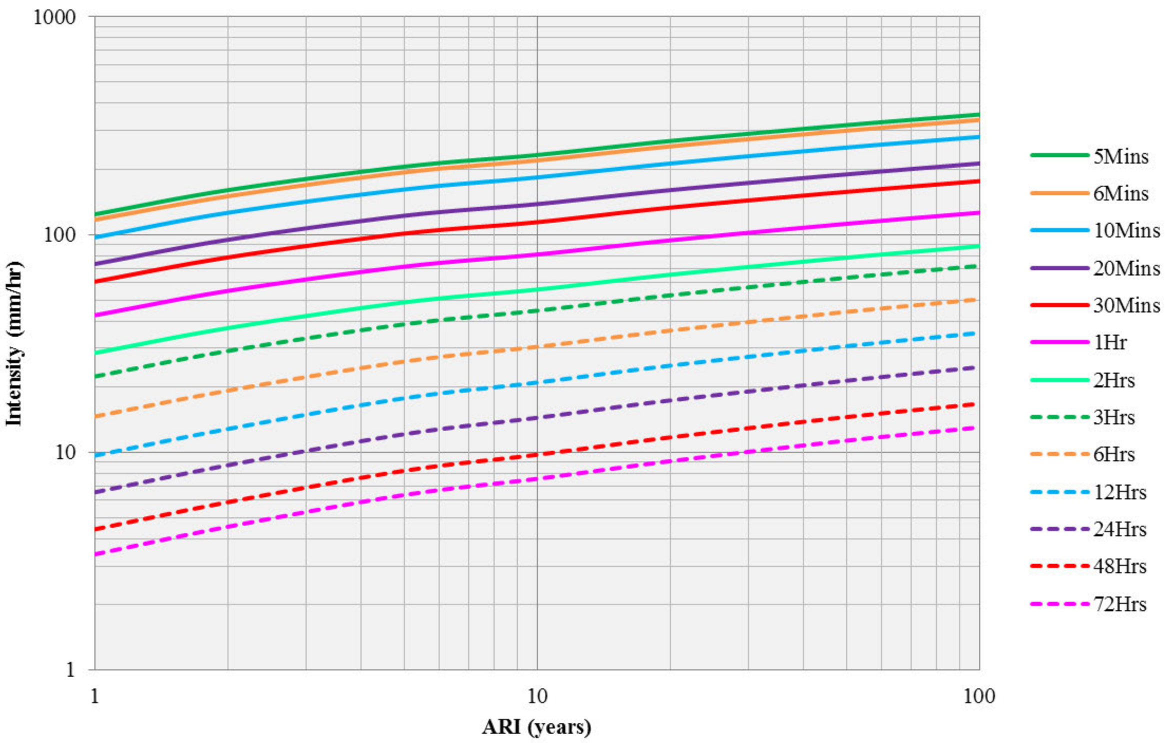


Figure B2 – Lower Cattle Creek Intensity Frequency Duration (IFD) Chart

Teemburra Creek Catchment

Table B7 – Teemburra Creek ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.225°
Catchment Centroid Longitude	148.625°
${}^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	56.13 mm/hr
${}^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	13.67 mm/hr
${}^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	4.76 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	103.16 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	29.50 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	11.63 mm/hr
Skewness (G)	0.150

Table B8 – Teemburra Creek ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	125.0	160.0	202.0	228.0	262.0	308.0	343.0
6Mins	117.0	151.0	191.0	215.0	248.0	291.0	325.0
10Mins	98.0	126.0	159.0	179.0	206.0	243.0	271.0
20Mins	74.0	94.9	120.0	134.0	154.0	181.0	201.0
30Mins	61.5	78.7	99.2	111.0	128.0	150.0	167.0
1Hr	43.2	55.4	70.2	79.0	90.9	107.0	119.0

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
2Hrs	29.3	37.8	48.5	55.0	63.7	75.3	84.4
3Hrs	23.1	30.0	38.9	44.3	51.5	61.3	69.0
6Hrs	15.4	20.1	26.6	30.6	36.0	43.2	49.0
12Hrs	10.3	13.5	18.3	21.3	25.2	30.6	35.0
24Hrs	6.9	9.2	12.6	14.9	17.7	21.8	25.0

Table B9 – Teemburra Creek ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.77E+00	-5.38E-01	-3.67E-02	7.43E-03	1.20E-03	-2.59E-04	-2.49E-05
2	4.01E+00	-5.33E-01	-3.30E-02	6.83E-03	9.76E-04	-1.78E-04	-3.13E-05
5	4.25E+00	-5.19E-01	-2.45E-02	6.16E-03	3.90E-04	-8.07E-05	-2.97E-05
10	4.37E+00	-5.12E-01	-1.99E-02	5.89E-03	7.15E-05	-3.74E-05	-2.75E-05
20	4.51E+00	-5.06E-01	-1.60E-02	5.49E-03	-1.78E-04	1.93E-05	-2.94E-05
50	4.67E+00	-4.99E-01	-1.12E-02	5.14E-03	-5.22E-04	7.40E-05	-2.82E-05
100	4.78E+00	-4.94E-01	-8.39E-03	4.87E-03	-6.89E-04	1.14E-04	-3.04E-05

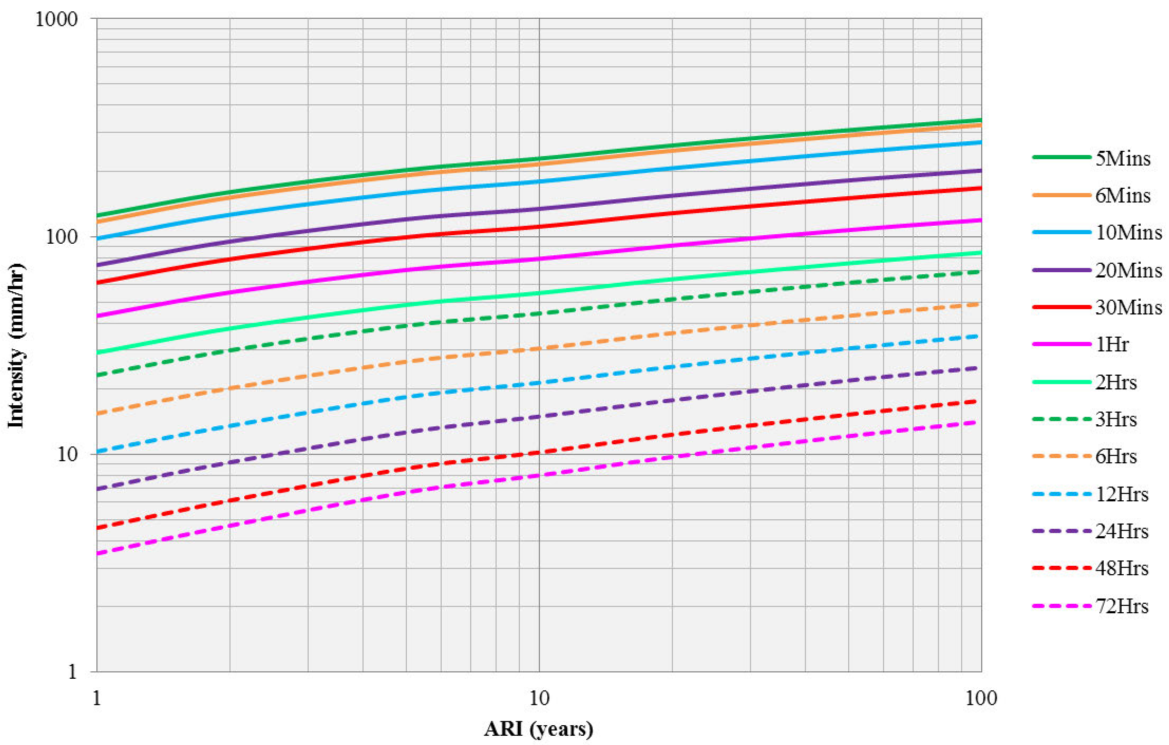


Figure B3 – Teemburra Creek Intensity Frequency Duration (IFD) Chart

Blacks Creek Catchment

Table B10 – Blacks Creek ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.325°
Catchment Centroid Longitude	148.675°
${}^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	55.59 mm/hr
${}^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	13.97 mm/hr
${}^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	4.96 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	98.27 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	32.46 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	12.70 mm/hr
Skewness (G)	0.150

Table B11 – Blacks Creek ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	124.0	159.0	199.0	223.0	256.0	299.0	332.0
6Mins	117.0	150.0	188.0	211.0	242.0	283.0	315.0
10Mins	97.7	125.0	157.0	176.0	201.0	235.0	261.0
20Mins	73.7	94.0	117.0	130.0	149.0	173.0	192.0
30Mins	61.2	78.0	96.9	108.0	123.0	143.0	158.0
1Hr	43.1	55.0	68.8	76.8	87.9	103.0	114.0
2Hrs	29.3	37.7	48.2	54.5	63.0	74.4	83.2
3Hrs	23.2	30.1	39.1	44.7	52.1	62.1	69.9
6Hrs	15.5	20.3	27.4	31.9	37.8	45.9	52.3
12Hrs	10.4	13.8	19.2	22.7	27.3	33.7	38.8
24Hrs	7.1	9.4	13.4	16.0	19.3	24.1	27.9

Table B12 – Blacks Creek ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.76E+00	-5.35E-01	-3.59E-02	6.77E-03	1.30E-03	-1.77E-04	-4.03E-05
2	4.01E+00	-5.27E-01	-2.87E-02	6.54E-03	6.33E-04	-1.37E-04	-2.75E-05
5	4.23E+00	-5.06E-01	-1.21E-02	6.31E-03	-8.42E-04	-9.27E-05	5.81E-06
10	4.34E+00	-4.94E-01	-2.76E-03	5.79E-03	-1.64E-03	-2.61E-05	1.77E-05
20	4.48E+00	-4.85E-01	4.86E-03	5.70E-03	-2.33E-03	-1.07E-05	3.41E-05
50	4.63E+00	-4.74E-01	1.36E-02	5.57E-03	-3.10E-03	1.36E-05	5.14E-05
100	4.73E+00	-4.66E-01	1.94E-02	5.30E-03	-3.60E-03	4.95E-05	5.96E-05

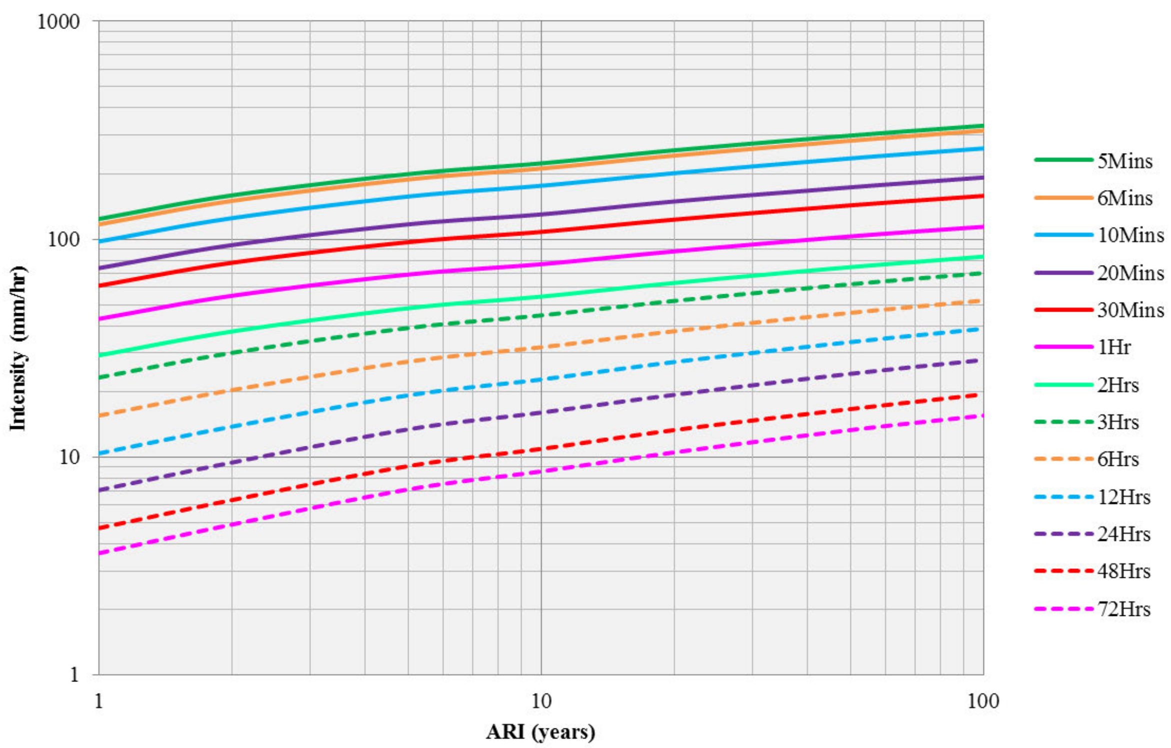


Figure B4 – Blacks Creek Intensity Frequency Duration (IFD) Chart

Stockmans Creek Catchment

Table B13 – Stockmans Creek ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.350°
Catchment Centroid Longitude	148.850°
${}^{1}I_2$ (1 hour, 2 year ARI rainfall intensity)	51.76 mm/hr
${}^{12}I_2$ (12 hour, 2 year ARI rainfall intensity)	11.70 mm/hr
${}^{72}I_2$ (72 hour, 2 year ARI rainfall intensity)	3.93 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	97.21 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	27.13 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	9.75 mm/hr
Skewness (G)	0.160

Table B14 – Stockmans Creek ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	116.0	149.0	192.0	218.0	253.0	299.0	336.0
6Mins	109.0	141.0	181.0	205.0	238.0	283.0	317.0
10Mins	90.5	117.0	150.0	171.0	198.0	235.0	263.0
20Mins	68.3	88.1	112.0	127.0	147.0	174.0	194.0
30Mins	56.6	73.0	93.0	105.0	121.0	143.0	160.0
1Hr	39.4	50.9	65.2	73.8	85.4	101.0	113.0

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
2Hrs	26.3	34.2	44.5	50.8	59.3	70.7	79.7
3Hrs	20.5	26.7	35.3	40.7	47.8	57.4	65.1
6Hrs	13.3	17.5	23.8	27.8	33.0	40.3	46.1
12Hrs	8.7	11.6	16.1	19.0	22.9	28.2	32.5
24Hrs	5.8	7.7	10.9	13.0	15.7	19.5	22.6

Table B15 – Stockmans Creek ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.67E+00	-5.57E-01	-4.66E-02	7.39E-03	2.12E-03	-2.55E-04	-5.06E-05
2	3.93E+00	-5.51E-01	-4.16E-02	7.29E-03	1.68E-03	-2.33E-04	-4.19E-05
5	4.18E+00	-5.35E-01	-2.81E-02	7.02E-03	4.71E-04	-1.92E-04	-1.51E-05
10	4.30E+00	-5.27E-01	-2.09E-02	6.95E-03	-1.91E-04	-1.72E-04	-2.16E-07
20	4.45E+00	-5.19E-01	-1.45E-02	6.54E-03	-7.34E-04	-1.19E-04	6.43E-06
50	4.61E+00	-5.11E-01	-7.67E-03	6.38E-03	-1.34E-03	-9.66E-05	1.96E-05
100	4.73E+00	-5.06E-01	-2.78E-03	6.36E-03	-1.80E-03	-8.85E-05	3.11E-05

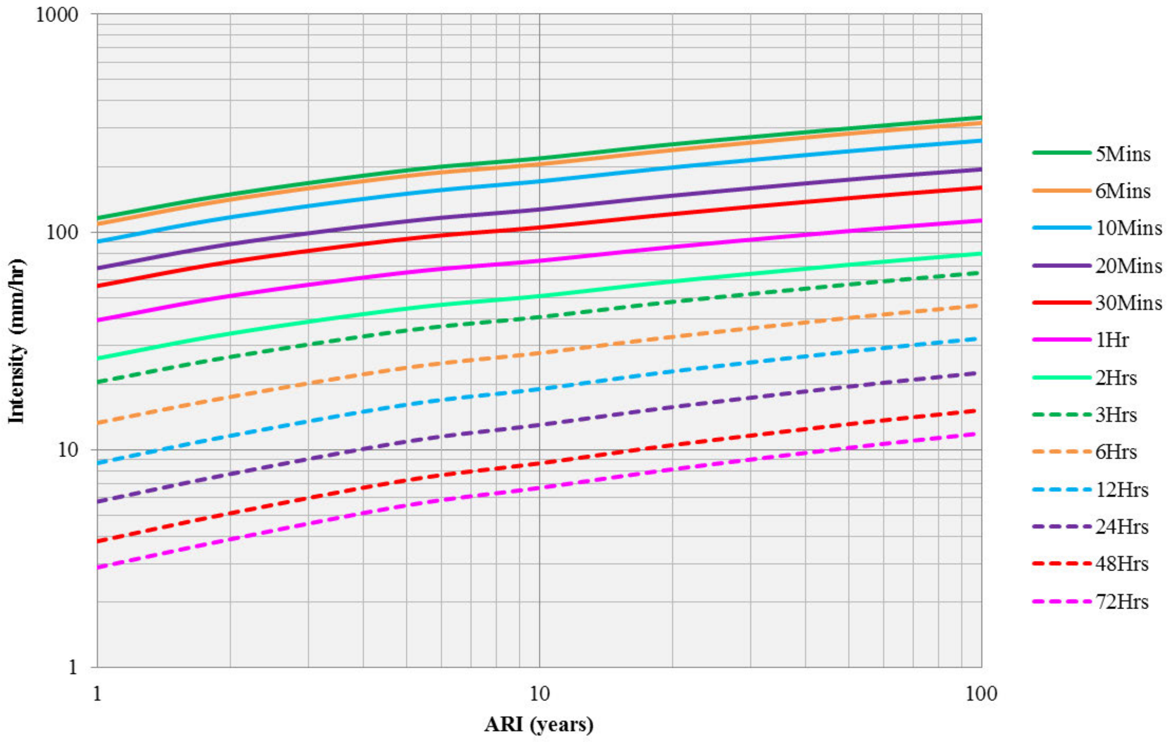


Figure B5 – Stockmans Creek Intensity Frequency Duration (IFD) Chart

Upper Pioneer River Catchment

Table B16 – Upper Pioneer River ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.175°
Catchment Centroid Longitude	148.800°
${}^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	55.10 mm/hr
${}^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	11.95 mm/hr
${}^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	4.02 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	108.06 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	27.87 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	10.11 mm/hr
Skewness (G)	0.150

Table B17 – Upper Pioneer River ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	116.0	149.0	192.0	218.0	253.0	299.0	336.0
6Mins	109.0	141.0	181.0	205.0	238.0	283.0	317.0
10Mins	90.5	117.0	150.0	171.0	198.0	235.0	263.0
20Mins	68.3	88.1	112.0	127.0	147.0	174.0	194.0
30Mins	56.6	73.0	93.0	105.0	121.0	143.0	160.0
1Hr	39.4	50.9	65.2	73.8	85.4	101.0	113.0
2Hrs	26.3	34.2	44.5	50.8	59.3	70.7	79.7
3Hrs	20.5	26.7	35.3	40.7	47.8	57.4	65.1
6Hrs	13.3	17.5	23.8	27.8	33.0	40.3	46.1
12Hrs	8.7	11.6	16.1	19.0	22.9	28.2	32.5
24Hrs	5.8	7.7	10.9	13.0	15.7	19.5	22.6

Table B18 – Upper Pioneer River ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.67E+00	-5.57E-01	-4.66E-02	7.39E-03	2.12E-03	-2.55E-04	-5.06E-05
2	3.93E+00	-5.51E-01	-4.16E-02	7.29E-03	1.68E-03	-2.33E-04	-4.19E-05
5	4.18E+00	-5.35E-01	-2.81E-02	7.02E-03	4.71E-04	-1.92E-04	-1.51E-05
10	4.30E+00	-5.27E-01	-2.09E-02	6.95E-03	-1.91E-04	-1.72E-04	-2.16E-07
20	4.45E+00	-5.19E-01	-1.45E-02	6.54E-03	-7.34E-04	-1.19E-04	6.43E-06
50	4.61E+00	-5.11E-01	-7.67E-03	6.38E-03	-1.34E-03	-9.66E-05	1.96E-05
100	4.73E+00	-5.06E-01	-2.78E-03	6.36E-03	-1.80E-03	-8.85E-05	3.11E-05

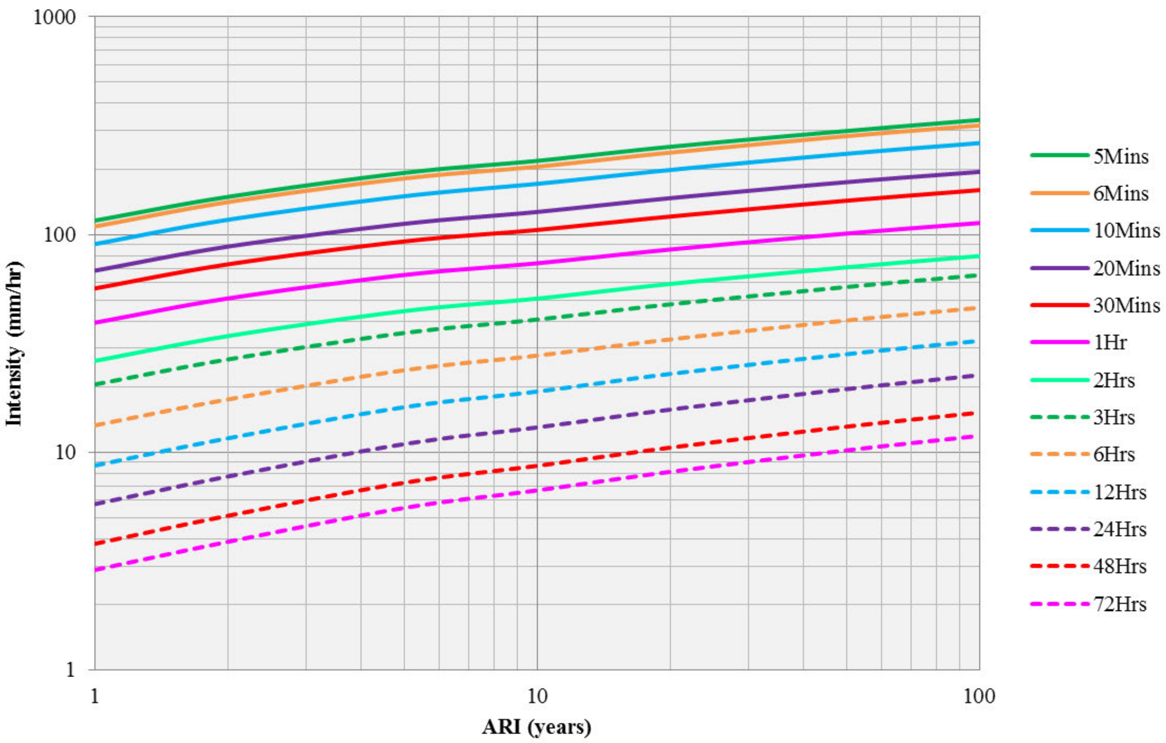


Figure B6 – Upper Pioneer River Intensity Frequency Duration (IFD) Chart

Middle Pioneer River Catchment

Table B19 – Middle Pioneer River ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.125°
Catchment Centroid Longitude	148.975°
$^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	57.12 mm/hr
$^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	13.52 mm/hr
$^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	4.71 mm/hr
$^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	108.48 mm/hr
$^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	30.48 mm/hr
$^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	11.90 mm/hr
Skewness (G)	0.160

Table B20 – Middle Pioneer River ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	127.0	163.0	208.0	235.0	272.0	321.0	359.0
6Mins	119.0	154.0	197.0	222.0	257.0	304.0	340.0
10Mins	99.5	128.0	164.0	186.0	215.0	254.0	284.0
20Mins	75.0	96.7	123.0	139.0	161.0	190.0	212.0
30Mins	62.2	80.2	102.0	116.0	134.0	158.0	176.0
1Hr	43.5	56.3	72.2	82.0	95.0	113.0	126.0

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
2Hrs	29.4	38.2	49.7	56.9	66.3	79.2	89.2
3Hrs	23.1	30.1	39.7	45.6	53.5	64.3	72.7
6Hrs	15.2	20.0	26.9	31.3	37.1	45.1	51.4
12Hrs	10.1	13.4	18.4	21.7	25.9	31.8	36.5
24Hrs	6.8	9.1	12.7	15.1	18.2	22.5	26.0

Table B21 – Middle Pioneer River ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.77E+00	-5.44E-01	-3.99E-02	6.71E-03	1.61E-03	-1.62E-04	-5.15E-05
2	4.03E+00	-5.39E-01	-3.69E-02	6.98E-03	1.33E-03	-1.90E-04	-3.79E-05
5	4.28E+00	-5.23E-01	-2.80E-02	6.14E-03	7.22E-04	-7.88E-05	-3.88E-05
10	4.41E+00	-5.15E-01	-2.35E-02	5.91E-03	3.99E-04	-4.25E-05	-3.56E-05
20	4.55E+00	-5.08E-01	-1.95E-02	5.62E-03	1.09E-04	-3.63E-06	-3.36E-05
50	4.72E+00	-5.00E-01	-1.51E-02	5.30E-03	-2.08E-04	4.02E-05	-3.16E-05
100	4.84E+00	-4.94E-01	-1.19E-02	5.01E-03	-4.24E-04	7.82E-05	-3.15E-05

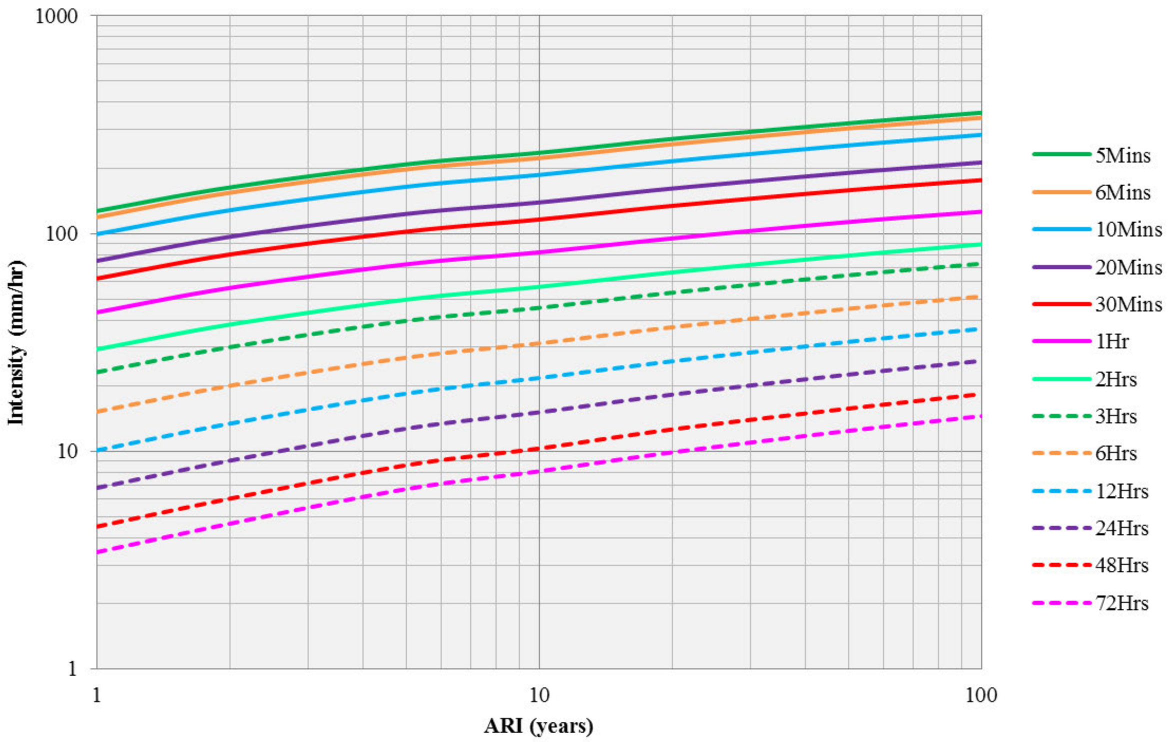


Figure B7 – Middle Pioneer River Intensity Frequency Duration (IFD) Chart

Lower Pioneer River Catchment

Table B22 – Lower Pioneer River ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.150°
Catchment Centroid Longitude	149.100°
${}^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	56.33 mm/hr
${}^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	12.49 mm/hr
${}^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	4.14 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	106.24 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	27.50 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	10.68 mm/hr
Skewness (G)	0.160

Table B23 – Lower Pioneer River ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	125.0	162.0	206.0	233.0	269.0	318.0	356.0
6Mins	118.0	152.0	194.0	220.0	254.0	301.0	337.0
10Mins	98.2	127.0	162.0	183.0	212.0	250.0	280.0
20Mins	74.2	95.6	122.0	138.0	159.0	187.0	209.0
30Mins	61.6	79.3	101.0	114.0	132.0	155.0	174.0
1Hr	42.8	55.3	70.8	80.2	92.9	110.0	123.0
2Hrs	28.5	37.0	48.0	54.7	63.7	75.9	85.4
3Hrs	22.2	28.9	37.8	43.4	50.7	60.7	68.6
6Hrs	14.3	18.8	25.1	29.1	34.3	41.5	47.2
12Hrs	9.4	12.4	16.8	19.7	23.5	28.7	32.9
24Hrs	6.2	8.2	11.5	13.6	16.3	20.2	23.3

Table B24 – Lower Pioneer River ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.76E+00	-5.58E-01	-4.73E-02	7.67E-03	2.02E-03	-3.03E-04	-3.97E-05
2	4.01E+00	-5.53E-01	-4.48E-02	7.44E-03	1.92E-03	-2.63E-04	-4.44E-05
5	4.26E+00	-5.40E-01	-3.68E-02	6.70E-03	1.45E-03	-1.44E-04	-4.90E-05
10	4.39E+00	-5.32E-01	-3.29E-02	6.06E-03	1.30E-03	-5.40E-05	-5.94E-05
20	4.53E+00	-5.26E-01	-2.94E-02	5.53E-03	1.12E-03	2.41E-05	-6.65E-05
50	4.70E+00	-5.19E-01	-2.53E-02	5.04E-03	8.89E-04	9.36E-05	-7.06E-05
100	4.81E+00	-5.14E-01	-2.29E-02	4.68E-03	7.97E-04	1.49E-04	-7.70E-05

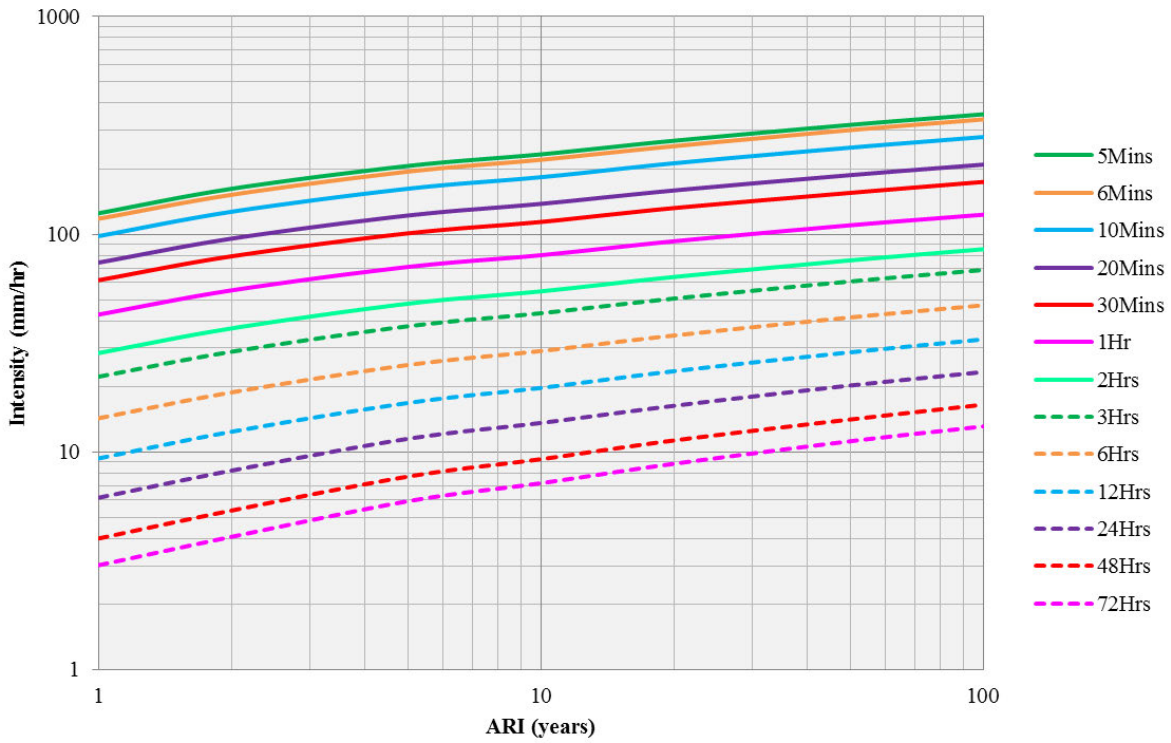


Figure B8 – Lower Pioneer River Intensity Frequency Duration (IFD) Chart

Pioneer River Outlet Catchment

Table B25 – Pioneer River Outlet ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.150°
Catchment Centroid Longitude	149.175°
$^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	55.55 mm/hr
$^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	11.62 mm/hr
$^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	3.83 mm/hr
$^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	106.26 mm/hr
$^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	26.02 mm/hr
$^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	9.29 mm/hr
Skewness (G)	0.160

Table B26 – Pioneer River Outlet ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	124.0	160.0	205.0	233.0	269.0	319.0	358.0
6Mins	116.0	150.0	193.0	219.0	254.0	301.0	338.0
10Mins	96.6	125.0	161.0	182.0	211.0	251.0	281.0
20Mins	73.1	94.5	121.0	137.0	159.0	188.0	211.0
30Mins	60.7	78.3	100.0	114.0	132.0	156.0	174.0
1Hr	42.0	54.4	70.0	79.6	92.3	109.0	123.0

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
2Hrs	27.7	35.9	46.9	53.7	62.6	74.8	84.4
3Hrs	21.3	27.8	36.6	42.2	49.5	59.4	67.3
6Hrs	13.5	17.8	23.9	27.8	32.9	40.0	45.6
12Hrs	8.7	11.5	15.8	18.5	22.1	27.1	31.1
24Hrs	5.7	7.6	10.6	12.5	15.0	18.5	21.4

Table B27 – Pioneer River Outlet ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.74E+00	-5.70E-01	-5.61E-02	7.36E-03	3.02E-03	-2.58E-04	-7.45E-05
2	4.00E+00	-5.66E-01	-5.24E-02	7.42E-03	2.66E-03	-2.49E-04	-6.63E-05
5	4.25E+00	-5.52E-01	-4.39E-02	7.06E-03	1.99E-03	-2.02E-04	-5.46E-05
10	4.38E+00	-5.45E-01	-3.91E-02	6.79E-03	1.61E-03	-1.64E-04	-5.06E-05
20	4.53E+00	-5.39E-01	-3.52E-02	6.60E-03	1.29E-03	-1.38E-04	-4.55E-05
50	4.70E+00	-5.32E-01	-3.05E-02	6.38E-03	9.10E-04	-1.03E-04	-4.07E-05
100	4.81E+00	-5.27E-01	-2.74E-02	6.24E-03	6.49E-04	-8.34E-05	-3.66E-05

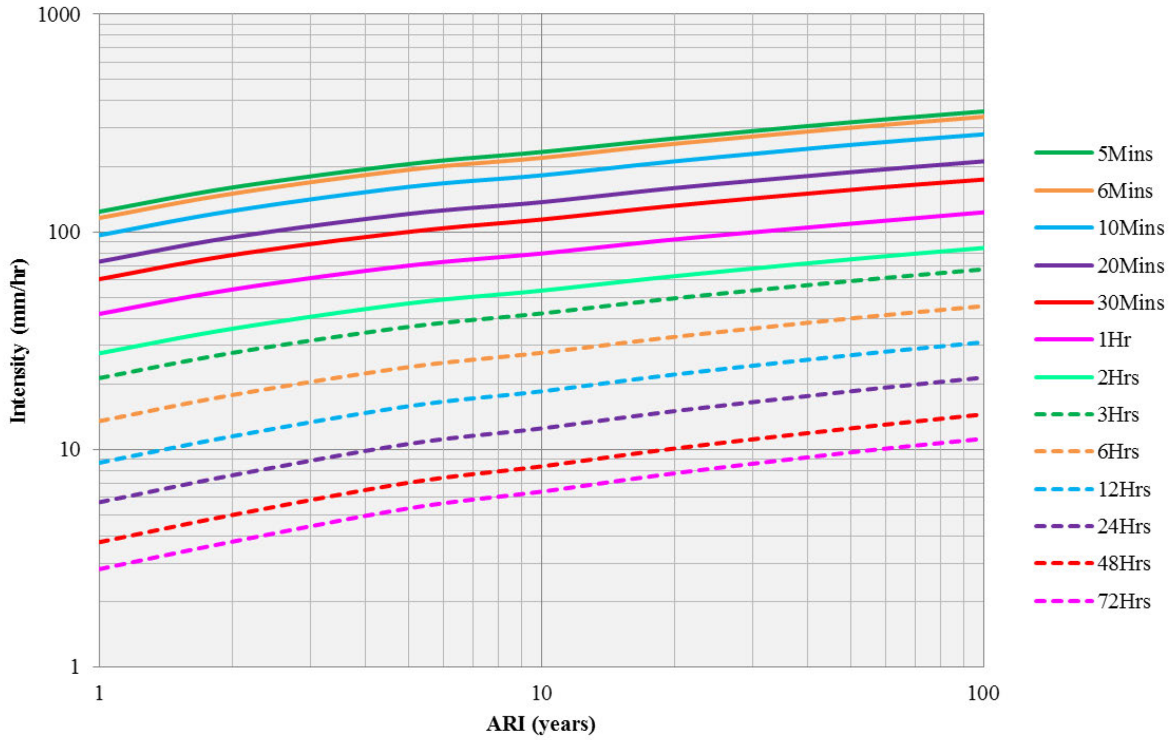


Figure B9 – Pioneer River Outlet Intensity Frequency Duration (IFD) Chart

Gooseponds Catchment

Table B28 – Gooseponds ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.100°
Catchment Centroid Longitude	149.150°
${}^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	56.12 mm/hr
${}^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	11.79 mm/hr
${}^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	3.89 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	108.12 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	26.63 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	9.54 mm/hr
Skewness (G)	0.160

Table B29 – Gooseponds ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	125.0	161.0	207.0	235.0	272.0	322.0	362.0
6Mins	117.0	151.0	195.0	221.0	256.0	304.0	341.0
10Mins	97.6	126.0	162.0	184.0	214.0	254.0	285.0
20Mins	73.8	95.4	122.0	139.0	161.0	191.0	214.0
30Mins	61.2	79.1	102.0	115.0	133.0	158.0	177.0
1Hr	42.4	55.0	71.0	80.7	93.8	111.0	125.0
2Hrs	27.9	36.3	47.6	54.5	63.8	76.3	86.1
3Hrs	21.5	28.1	37.2	42.9	50.4	60.6	68.7
6Hrs	13.7	18.0	24.3	28.3	33.6	40.9	46.6
12Hrs	8.8	11.7	16.1	18.9	22.6	27.7	31.9
24Hrs	5.8	7.7	10.8	12.8	15.4	19.0	22.0

Table B30 – Gooseponds ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.75E+00	-5.70E-01	-5.55E-02	7.61E-03	2.93E-03	-2.74E-04	-7.00E-05
2	4.01E+00	-5.65E-01	-5.23E-02	7.58E-03	2.66E-03	-2.70E-04	-6.32E-05
5	4.26E+00	-5.50E-01	-4.38E-02	7.04E-03	1.98E-03	-1.98E-04	-5.59E-05
10	4.39E+00	-5.43E-01	-3.92E-02	6.70E-03	1.63E-03	-1.54E-04	-5.29E-05
20	4.54E+00	-5.36E-01	-3.53E-02	6.53E-03	1.31E-03	-1.33E-04	-4.72E-05
50	4.71E+00	-5.29E-01	-3.09E-02	6.33E-03	9.51E-04	-1.06E-04	-4.11E-05
100	4.83E+00	-5.24E-01	-2.77E-02	6.20E-03	6.80E-04	-8.53E-05	-3.69E-05

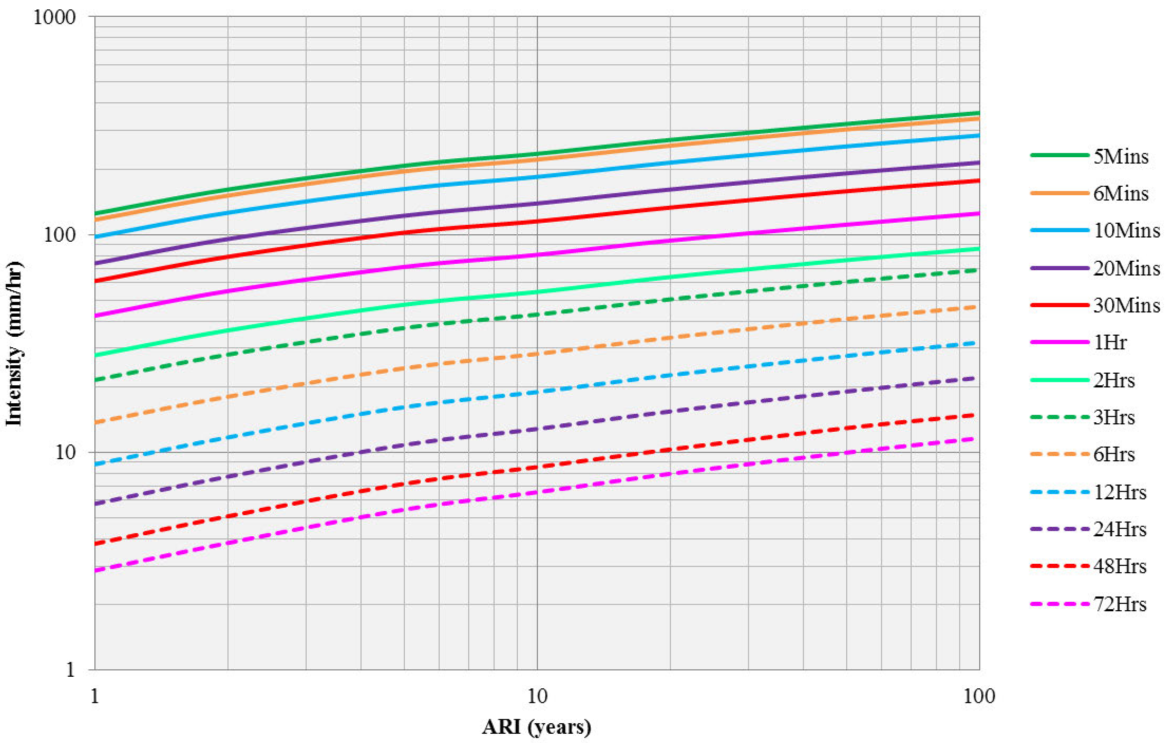


Figure B10 – Gooseponds Intensity Frequency Duration (IFD) Chart

Upper Bakers Creek Catchment

Table B31 – Upper Bakers Creek ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.200°
Catchment Centroid Longitude	149.050°
${}^{1}I_2$ (1 hour, 2 year ARI rainfall intensity)	56.03 mm/hr
${}^{12}I_2$ (12 hour, 2 year ARI rainfall intensity)	13.02 mm/hr
${}^{72}I_2$ (72 hour, 2 year ARI rainfall intensity)	4.30 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	103.91 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	28.87 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	12.00 mm/hr
Skewness (G)	0.160

Table B32 – Upper Bakers Creek ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	125.0	161.0	204.0	230.0	266.0	313.0	350.0
6Mins	117.0	151.0	193.0	218.0	251.0	296.0	332.0
10Mins	97.9	126.0	160.0	181.0	209.0	247.0	276.0
20Mins	74.0	95.0	120.0	135.0	156.0	183.0	205.0
30Mins	61.3	78.8	99.7	112.0	129.0	152.0	169.0
1Hr	42.8	55.1	70.2	79.3	91.5	108.0	121.0

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
2Hrs	28.8	37.2	48.1	54.7	63.6	75.5	84.9
3Hrs	22.5	29.3	38.3	43.8	51.2	61.1	69.0
6Hrs	14.8	19.4	25.9	29.9	35.3	42.7	48.5
12Hrs	9.7	12.9	17.6	20.7	24.7	30.2	34.7
24Hrs	6.4	8.6	12.1	14.4	17.5	21.8	25.2

Table B33 – Upper Bakers Creek ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.76E+00	-5.50E-01	-4.09E-02	8.21E-03	1.24E-03	-3.89E-04	-5.09E-06
2	4.01E+00	-5.44E-01	-3.75E-02	7.71E-03	1.12E-03	-3.03E-04	-1.54E-05
5	4.25E+00	-5.28E-01	-2.91E-02	6.18E-03	8.43E-04	-7.88E-05	-4.24E-05
10	4.37E+00	-5.20E-01	-2.44E-02	5.44E-03	6.63E-04	4.18E-05	-5.67E-05
20	4.52E+00	-5.13E-01	-2.05E-02	4.73E-03	5.29E-04	1.42E-04	-6.80E-05
50	4.68E+00	-5.05E-01	-1.61E-02	4.03E-03	3.82E-04	2.51E-04	-8.13E-05
100	4.79E+00	-4.99E-01	-1.31E-02	3.44E-03	2.96E-04	3.36E-04	-9.21E-05

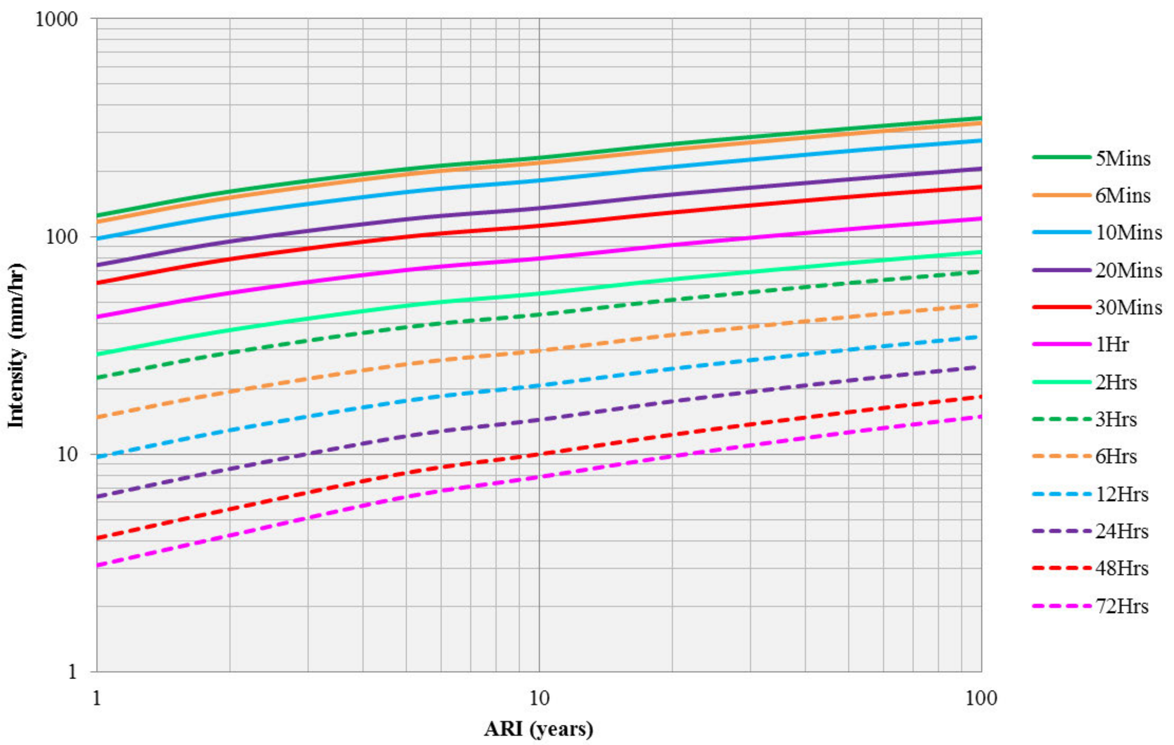


Figure B11 – Upper Bakers Creek Intensity Frequency Duration (IFD) Chart

Lower Bakers Creek Catchment

Table B34 – Lower Bakers Creek ARR87 Intensity Frequency Duration (IFD) Raw Data

Parameter	Value
Catchment Centroid Latitude	-21.200°
Catchment Centroid Longitude	149.150°
${}^{1}I_{2}$ (1 hour, 2 year ARI rainfall intensity)	55.52 mm/hr
${}^{12}I_{2}$ (12 hour, 2 year ARI rainfall intensity)	12.01 mm/hr
${}^{72}I_{2}$ (72 hour, 2 year ARI rainfall intensity)	3.98 mm/hr
${}^{1}I_{50}$ (1 hour, 50 year ARI rainfall intensity)	104.16 mm/hr
${}^{12}I_{50}$ (12 hour, 50 year ARI rainfall intensity)	26.39 mm/hr
${}^{72}I_{50}$ (72 hour, 50 year ARI rainfall intensity)	9.98 mm/hr
Skewness (G)	0.160

Table B35 – Lower Bakers Creek ARR87 Intensity Frequency Duration (IFD) Table (BoM)

Duration	Rainfall Intensity (mm/hr) per ARI						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
5Mins	124.0	160.0	204.0	231.0	267.0	315.0	352.0
6Mins	116.0	150.0	192.0	217.0	251.0	297.0	333.0
10Mins	96.9	125.0	160.0	181.0	209.0	247.0	277.0
20Mins	73.3	94.3	120.0	136.0	156.0	184.0	206.0
30Mins	60.8	78.2	99.5	112.0	130.0	153.0	171.0
1Hr	42.2	54.4	69.6	78.7	91.0	108.0	120.0
2Hrs	28.0	36.2	46.9	53.4	62.2	73.9	83.2
3Hrs	21.7	28.2	36.8	42.2	49.3	59.0	66.6
6Hrs	13.9	18.2	24.3	28.1	33.2	40.1	45.6
12Hrs	9.0	11.9	16.2	18.9	22.5	27.5	31.5
24Hrs	5.9	7.9	11.0	13.0	15.6	19.2	22.1

Table B36 – Lower Bakers Creek ARR87 Intensity Frequency Duration (IFD) Coefficients (BoM)

ARI in years	Coefficient Value						
	A	B	C	D	E	F	G
1	3.74E+00	-5.64E-01	-5.13E-02	7.61E-03	2.50E-03	-2.87E-04	-5.70E-05
2	4.00E+00	-5.59E-01	-4.79E-02	7.58E-03	2.23E-03	-2.76E-04	-5.01E-05
5	4.24E+00	-5.46E-01	-3.95E-02	6.71E-03	1.71E-03	-1.41E-04	-5.76E-05
10	4.37E+00	-5.38E-01	-3.48E-02	6.19E-03	1.42E-03	-7.00E-05	-6.02E-05
20	4.51E+00	-5.32E-01	-3.08E-02	5.76E-03	1.17E-03	-6.84E-06	-6.32E-05
50	4.68E+00	-5.25E-01	-2.63E-02	5.30E-03	8.71E-04	6.22E-05	-6.57E-05
100	4.79E+00	-5.21E-01	-2.34E-02	5.05E-03	6.94E-04	9.79E-05	-6.62E-05

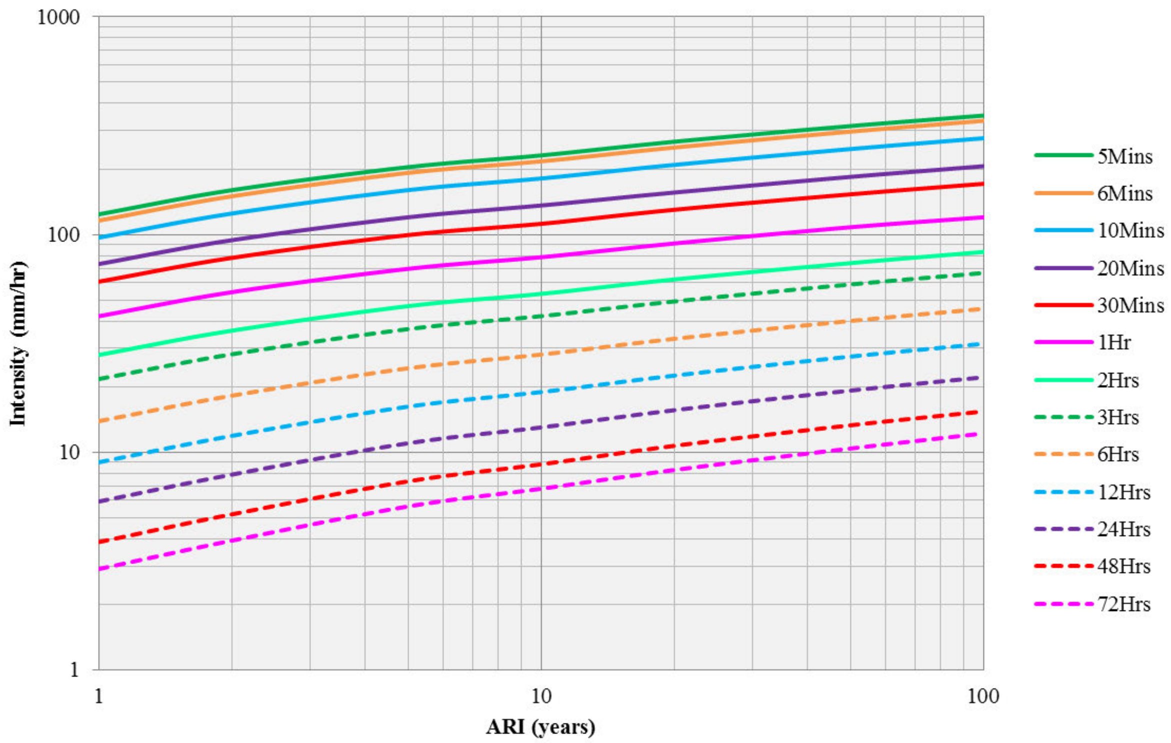


Figure B12 – Lower Bakers Creek Intensity Frequency Duration (IFD) Chart

Temporal Patterns

The temporal patterns associated with the ARR 1987 design rainfall calculations separate Australia into 8 zones. All of the catchments within the Mackay region are located in the North-East Coast Division (Zone 3).

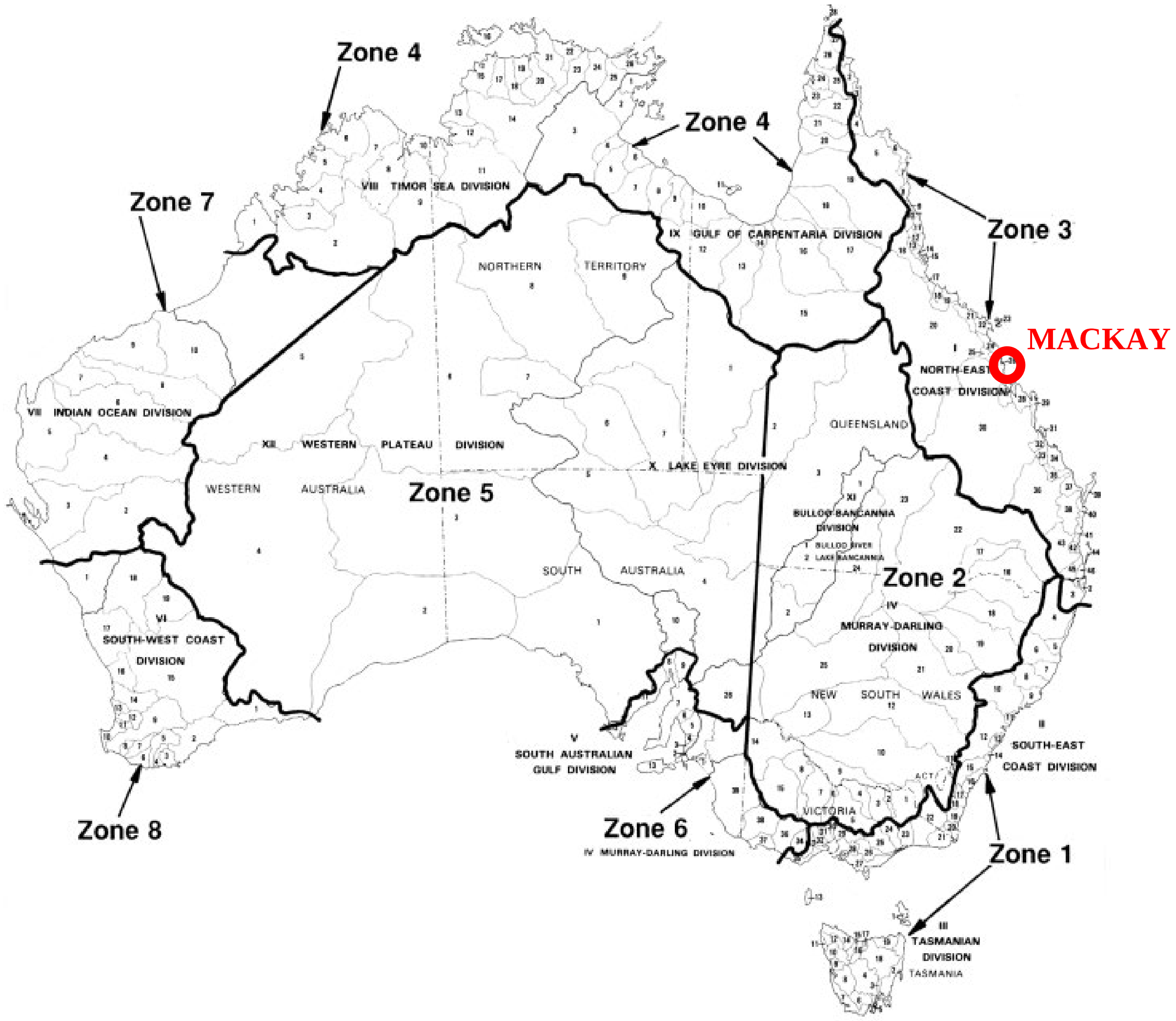


Figure 13 - ARR87 Temporal Pattern Zones

Temporal Patterns: Percentages of Rainfall Per Period for Zone 3 (ARR, 1987)

The following rainfall percentages per period have been derived for design storm events within Zone 3.

Table 37 - Zone 3 Temporal Pattern for 10 Minute Storm Duration in 2 Periods of 5 Minutes (ARR, 1987)

Period	**1**	**2**
ARI ≤ 30yrs	57	43
ARI > 30yrs	54	46

Table 38 - Zone 3 Temporal Pattern for 15 Minute Storm Duration in 3 Periods of 5 Minutes (ARR, 1987)

Period	**1**	**2**	**3**
ARI ≤ 30yrs	32	50	18
ARI > 30yrs	33	47	20

Table 39 - Zone 3 Temporal Pattern for 20 Minute Storm Duration in 4 Periods of 5 Minutes (ARR, 1987)

Period	1	2	3	4
ARI ≤ 30yrs	19	43	30	8
ARI > 30yrs	20	40	30	10

Table 40 - Zone 3 Temporal Pattern for 25 Minute Storm Duration in 5 Periods of 5 Minutes (ARR, 1987)

Period	1	2	3	4	5
ARI ≤ 30yrs	17	28	39	9	7
ARI > 30yrs	18	26	35	11	10

Table 41 - Zone 3 Temporal Pattern for 30 Minute Storm Duration in 6 Periods of 5 Minutes (ARR, 1987)

Period	1	2	3	4	5	6
ARI ≤ 30yrs	16	25	33	9	11	6
ARI > 30yrs	16	24	30	10	12	8

Table 42 - Zone 3 Temporal Pattern for 45 Minute Storm Duration in 9 Periods of 5 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9
ARI ≤ 30yrs	4.8	14.2	24.7	18.3	9.5	11.6	7.5	6.1	3.3
ARI > 30yrs	5.3	13.9	23.3	17.7	9.8	11.7	7.9	6.5	3.9

Table 43 - Zone 3 Temporal Pattern for 1 Hour Storm Duration in 12 Periods of 5 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12
ARI ≤ 30yrs	3.9	7.0	16.8	12.0	23.2	10.1	8.9	5.7	4.8	3.1	2.6	1.9
ARI > 30yrs	4.3	7.3	16.1	11.6	21.7	10.0	9.0	6.0	5.2	3.5	3.0	2.3

Table 44 - Zone 3 Temporal Pattern for 1.5 Hour Storm Duration in 18 Periods of 5 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
ARI ≤ 30yrs	3.8	6.9	8.8	4.6	12.8	5.7	16.7	10.4	5.1	4.8	4.3	2.3	3.4	3.0	2.7	2.0	1.6	1.1
ARI > 30yrs	4.1	6.8	8.6	5.0	11.7	5.8	14.7	9.9	5.3	5.1	4.7	2.7	3.8	3.4	3.1	2.3	1.8	1.2

Table 45 - Zone 3 Temporal Pattern for 2 Hour Storm Duration in 24 Periods of 5 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
ARI ≤ 30yrs	2.3	3.8	6.2	4.2	11.3	4.3	14.5	9.0	7.3	4.4	4.2	3.8	3.4	3.1	2.8	2.4	2.6	2.3	1.8	1.4	1.7	1.3	1.1	0.8
ARI > 30yrs	2.7	4.0	6.0	4.2	10.2	4.2	12.6	8.4	7.0	4.3	4.4	4.0	3.7	3.4	3.1	2.7	3.0	2.8	2.1	1.6	2.0	1.5	1.3	0.8

Table 46 - Zone 3 Temporal Pattern for 3 Hour Storm Duration in 12 Periods of 15 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12
ARI ≤ 30yrs	3.6	16.8	11.4	24.1	9.0	8.1	6.9	4.8	5.8	4.1	3.1	2.3
ARI > 30yrs	4.2	15.6	11.1	21.4	9.0	8.4	7.3	5.4	6.3	4.7	3.7	2.9

Table 47 - Zone 3 Temporal Pattern for 4.5 Hour Storm Duration in 18 Periods of 15 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
ARI ≤ 30yrs	2.1	10.1	13.8	18.7	7.1	6.8	5.8	3.5	3.1	4.4	5.0	5.7	3.9	2.8	2.4	1.9	1.6	1.3
ARI > 30yrs	2.5	9.6	12.6	16.4	6.9	6.7	5.9	3.9	3.5	4.7	5.3	5.9	4.3	3.2	2.8	2.3	1.9	1.6

Table 48 - Zone 3 Temporal Pattern for 6 Hour Storm Duration in 12 Periods of 30 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12
ARI ≤ 30yrs	4.3	16.5	25.6	4.8	12.6	8.9	7.7	4.9	5.8	3.6	3.0	2.3
ARI > 30yrs	4.9	15.4	22.8	5.4	12.3	9.0	8.0	5.3	6.2	4.2	3.6	2.9

Table 49 - Zone 3 Temporal Pattern for 9 Hour Storm Duration in 18 Periods of 30 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
ARI ≤ 30yrs	8.8	13.0	21.8	3.7	3.2	4.4	6.0	1.9	5.1	2.7	7.2	11.4	2.8	1.9	1.6	1.3	2.1	1.1
ARI > 30yrs	8.7	12.0	19.3	4.0	3.6	4.7	6.2	2.2	5.4	3.1	7.2	11.0	3.2	2.9	1.9	1.5	2.4	1.3

Table 50 - Zone 3 Temporal Pattern for 12 Hour Storm Duration in 24 Periods of 30 Minutes (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
ARI ≤ 30yrs	3.8	9.1	20.3	3.7	6.6	13.7	1.8	1.7	2.2	4.3	3.0	6.6	4.9	2.7	2.5	1.5	1.5	3.4	2.0	1.2	1.0	1.1	0.8	0.6
ARI > 30yrs	4.0	8.6	17.9	3.9	6.4	12.5	2.1	2.0	2.6	4.4	3.3	6.5	4.9	3.0	2.9	1.8	1.8	3.7	2.3	1.4	1.2	1.3	0.9	0.6

Table 51 - Zone 3 Temporal Pattern for 18 Hour Storm Duration in 18 Periods of 1 Hour (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
ARI ≤ 30yrs	3.0	24.2	3.5	2.2	4.1	11.4	1.5	1.0	8.8	7.0	1.8	1.2	4.9	15.9	5.8	2.6	0.7	0.4
ARI > 30yrs	3.4	21.5	3.9	2.6	4.5	11.1	1.9	1.2	8.8	7.1	2.2	1.5	5.2	14.8	6.1	3.0	0.8	0.4

Table 52 - Zone 3 Temporal Pattern for 24 Hour Storm Duration in 24 Periods of 1 Hour (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
ARI ≤ 30yrs	12.9	22.0	8.1	6.8	4.9	2.6	0.7	1.8	2.9	0.9	1.3	1.0	3.3	4.3	3.8	2.0	1.1	5.8	8.5	1.5	2.3	0.6	0.5	0.4
ARI > 30yrs	11.9	19.5	8.0	6.8	5.0	2.9	0.8	2.1	3.2	1.1	1.5	1.2	3.6	4.5	4.1	2.3	1.3	6.0	8.1	1.8	2.6	0.7	0.6	0.4

Table 53 - Zone 3 Temporal Pattern for 30 Hour Storm Duration in 15 Periods of 2 Hours (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
ARI ≤ 30yrs	1.5	28.3	3.7	4.7	0.8	3.1	1.1	1.9	2.4	9.2	7.3	17.2	12.3	6.0	0.5
ARI > 30yrs	1.9	25.3	4.2	5.1	11.1	3.6	1.5	2.3	2.9	9.3	7.5	16.1	12.0	6.4	0.8

Table 54 - Zone 3 Temporal Pattern for 36 Hour Storm Duration in 18 Periods of 2 Hours (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
ARI ≤ 30yrs	26.2	5.8	7.1	1.8	3.3	2.7	1.1	0.6	0.9	2.2	0.5	1.4	4.7	16.5	12.0	9.0	3.9	0.3
ARI > 30yrs	23.3	6.1	7.2	2.2	3.7	3.1	1.4	0.9	1.3	2.6	0.8	1.7	5.0	15.3	11.6	9.0	4.3	0.5

Table 55 - Zone 3 Temporal Pattern for 48 Hour Storm Duration in 24 Periods of 2 Hours (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
ARI ≤ 30yrs	22.4	11.4	5.9	0.7	1.1	0.8	0.6	0.9	1.4	4.2	7.0	2.2	2.6	1.9	8.7	11.4	5.0	1.6	3.6	1.3	1.1	3.1	0.6	0.5
ARI > 30yrs	19.8	10.5	6.1	0.8	1.3	0.9	0.7	1.1	1.6	4.5	7.0	2.5	2.9	2.2	8.6	11.0	5.2	1.8	3.9	1.5	1.3	3.5	0.7	0.6

Table 56 - Zone 3 Temporal Pattern for 72 Hour Storm Duration in 18 Periods of 4 Hours (ARR, 1987)

Period	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
ARI ≤ 30yrs	28.9	4.3	1.3	0.5	1.0	2.2	12.3	18.2	15.6	2.7	0.7	1.7	7.3	9.3	3.4	0.3	0.2	0.1
ARI > 30yrs	25.8	4.7	1.7	0.7	1.3	2.6	12.0	17.0	6.0	3.1	1.0	2.1	7.5	9.4	3.8	0.5	0.5	0.3

APPENDIX C: ARR 2016 DESIGN RAINFALL INFORMATION

IFD Parameters

The following sections contain ARR 2016 Intensity Frequency Duration (IFD) design rainfall parameters for each rainfall catchment.

Upper Cattle Creek Catchment

Table C1 – Upper Cattle Creek ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	114.96	128.40	169.20	194.40	218.40	249.60	271.20
10 min	94.20	105.00	137.40	158.40	178.20	202.20	220.20
15 min	80.40	90.00	117.60	135.20	151.60	172.40	187.20
20 min	71.10	79.20	103.50	119.10	133.50	151.50	164.70
25 min	64.08	71.28	93.12	107.28	120.24	136.56	148.32
30 min	58.40	65.20	85.20	98.20	110.00	125.00	136.00
45 min	47.20	52.80	69.47	80.13	90.13	102.67	111.87
1 hour	40.20	45.10	59.90	69.40	78.20	89.50	97.70
1.5 hour	31.73	35.93	48.53	56.67	64.33	74.00	81.33
2 hour	26.80	30.55	41.85	49.25	56.50	65.50	72.00
3 hour	21.03	24.23	34.00	40.67	47.00	55.33	61.33
4.5 hour	16.49	19.24	28.00	33.78	39.56	47.11	53.11
6 hour	13.88	16.37	24.17	29.67	35.17	42.33	47.83
9 hour	10.90	13.00	19.89	24.67	29.56	36.22	41.44
12 hour	9.17	11.08	17.17	21.58	26.08	32.17	37.17
18 hour	7.17	8.72	13.89	17.67	21.56	26.89	31.33
24 hour	6.04	7.38	11.83	15.13	18.58	23.33	27.29
30 hour	5.27	6.43	10.37	13.33	16.40	20.70	24.27
36 hour	4.69	5.72	9.28	11.94	14.72	18.64	21.89
48 hour	3.90	4.77	7.73	9.96	12.31	15.58	18.33
72 hour	2.96	3.63	5.88	7.57	9.36	11.83	13.89
96 hour	2.43	2.96	4.79	6.18	7.65	9.65	11.35
120 hour	2.06	2.52	4.08	5.28	6.55	8.24	9.67
144 hour	1.80	2.20	3.58	4.65	5.80	7.29	8.47
168 hour	1.60	1.96	3.22	4.20	5.27	6.61	7.68

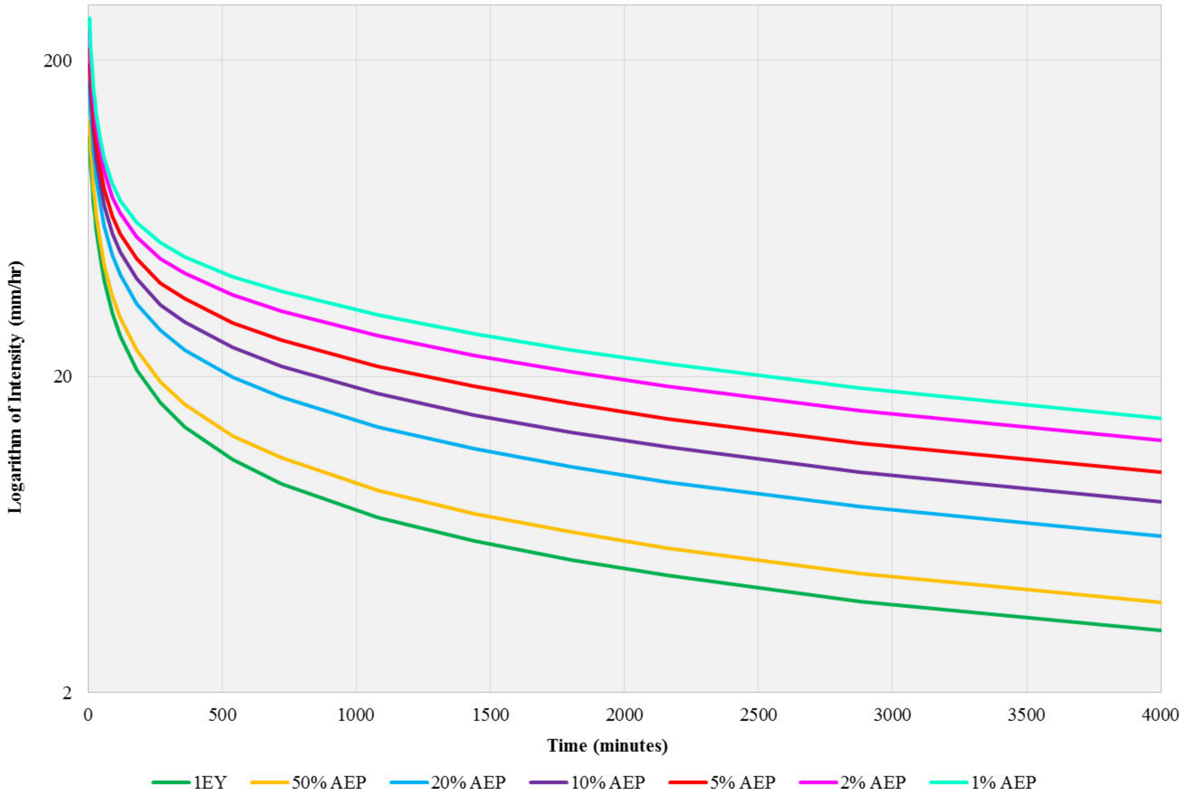


Figure C1 – Upper Cattle Creek Intensity Frequency Duration (IFD) Chart

Lower Cattle Creek Catchment

Table C2 - Lower Cattle Creek ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	118.68	133.20	175.20	201.60	226.80	258.00	280.80
10 min	98.40	109.80	144.00	165.60	185.40	209.40	227.40
15 min	84.80	94.80	123.60	141.60	158.40	179.20	194.40
20 min	75.00	83.40	108.90	125.10	139.80	158.40	171.60
25 min	67.20	75.12	98.16	112.56	126.24	143.04	155.04
30 min	61.20	68.40	89.60	103.00	115.60	131.20	142.40
45 min	48.93	54.80	72.53	83.87	94.27	107.60	117.20
1 hour	41.30	46.40	62.00	72.00	81.30	93.20	102.00
1.5 hour	32.07	36.40	49.47	57.93	66.00	76.00	84.00
2 hour	26.70	30.50	42.10	49.70	57.00	66.50	73.50
3 hour	20.57	23.77	33.67	40.33	46.67	54.67	61.00
4.5 hour	15.89	18.58	26.89	32.67	38.22	45.56	51.11
6 hour	13.27	15.63	23.17	28.33	33.33	40.17	45.50
9 hour	10.36	12.33	18.67	23.22	27.67	33.78	38.56
12 hour	8.75	10.50	16.17	20.17	24.25	29.83	34.33
18 hour	6.89	8.33	13.06	16.50	20.06	24.94	29.00
24 hour	5.83	7.08	11.21	14.25	17.42	21.88	25.54
30 hour	5.13	6.23	9.93	12.67	15.53	19.60	23.03

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
36 hour	4.61	5.61	8.97	11.47	14.08	17.86	21.06
48 hour	3.88	4.71	7.56	9.71	11.98	15.27	18.06
72 hour	2.99	3.64	5.85	7.53	9.32	11.94	14.17
96 hour	2.44	2.97	4.79	6.18	7.68	9.84	11.67
120 hour	2.06	2.51	4.06	5.25	6.54	8.42	9.92
144 hour	1.77	2.16	3.51	4.56	5.72	7.29	8.61
168 hour	1.55	1.89	3.10	4.04	5.07	6.43	7.62

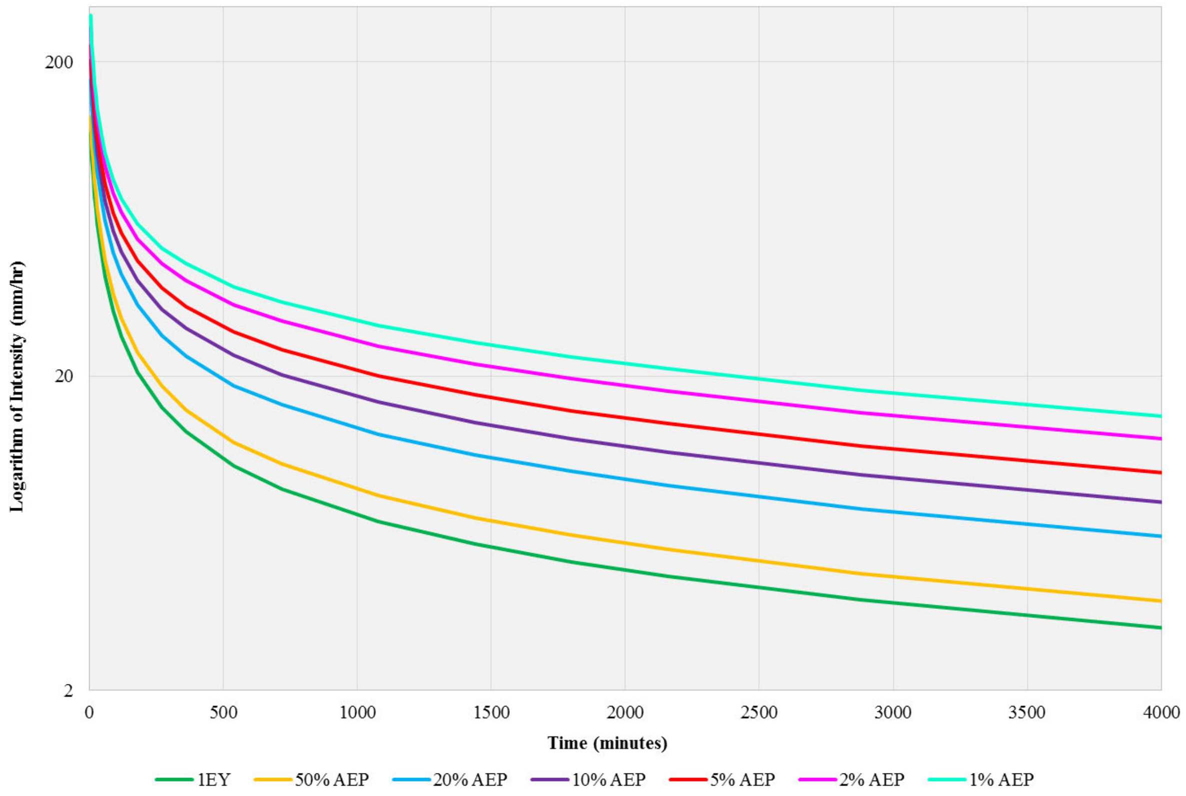


Figure C2 - Lower Cattle Creek Intensity Frequency Duration (IFD) Chart

Teemburra Creek Catchment

Table C3 - Teemburra Creek ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	108.48	121.20	159.60	184.80	206.40	235.20	255.60
10 min	90.00	100.80	132.00	151.80	170.40	193.80	210.00
15 min	77.20	86.40	112.80	129.60	145.60	164.80	179.20
20 min	68.10	75.90	99.30	114.00	127.80	144.90	157.50
25 min	61.20	68.16	89.28	102.48	114.96	130.32	141.60
30 min	55.60	62.20	81.40	93.60	105.00	119.00	129.40

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
45 min	44.40	49.73	65.60	75.73	85.07	96.93	105.47
1 hour	37.50	42.20	56.00	64.90	73.20	83.70	91.40
1.5 hour	29.27	33.13	44.80	52.33	59.40	68.67	75.33
2 hour	24.40	27.85	38.25	45.05	51.50	60.00	66.00
3 hour	18.93	21.83	30.77	36.67	42.33	50.00	55.67
4.5 hour	14.69	17.18	24.89	30.22	35.33	42.22	47.56
6 hour	12.32	14.53	21.50	26.50	31.33	37.67	42.83
9 hour	9.67	11.56	17.67	22.00	26.33	32.22	37.00
12 hour	8.16	9.83	15.25	19.25	23.33	28.83	33.25
18 hour	6.44	7.83	12.44	15.89	19.44	24.33	28.39
24 hour	5.46	6.67	10.71	13.75	16.96	21.38	25.04
30 hour	4.80	5.87	9.47	12.20	15.13	19.13	22.47
36 hour	4.31	5.25	8.56	11.03	13.69	17.36	20.44
48 hour	3.60	4.42	7.19	9.31	11.58	14.73	17.35
72 hour	2.78	3.40	5.54	7.18	8.96	11.38	13.40
96 hour	2.28	2.78	4.54	5.90	7.38	9.33	10.94
120 hour	1.94	2.37	3.87	5.04	6.31	7.98	9.33
144 hour	1.69	2.06	3.38	4.42	5.56	7.01	8.19
168 hour	1.49	1.83	3.02	3.96	5.01	6.31	7.38

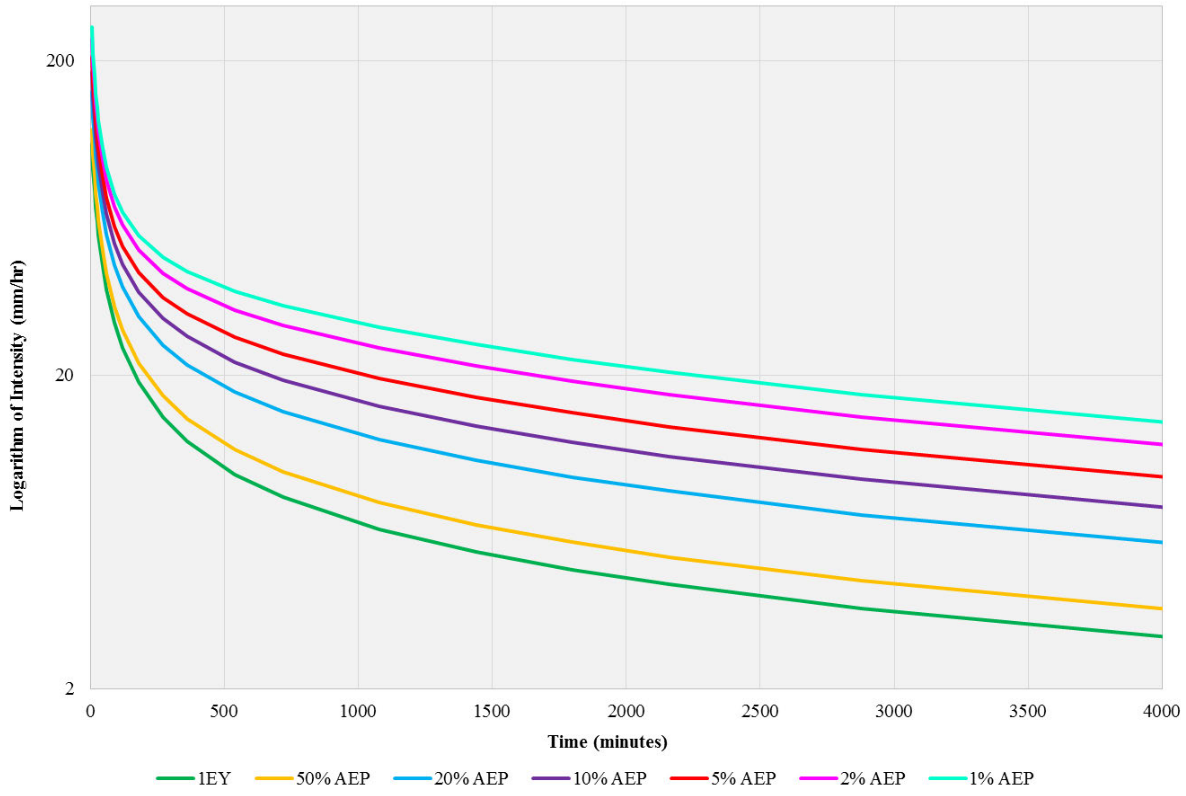


Figure C3 - Teemburra Creek Intensity Frequency Duration (IFD) Chart

Blacks Creek Catchment

Table C4 - Blacks Creek ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	106.68	119.52	158.40	182.40	205.20	232.80	253.20
10 min	88.80	99.60	130.20	150.00	168.60	191.40	207.60
15 min	76.40	85.20	111.60	128.40	144.00	163.20	177.20
20 min	67.20	75.00	98.10	112.80	126.60	143.40	155.70
25 min	60.24	67.20	88.08	101.52	113.76	128.88	139.92
30 min	54.80	61.20	80.40	92.40	103.80	117.80	128.00
45 min	43.60	48.80	64.67	74.67	84.00	95.73	104.27
1 hour	36.70	41.30	55.00	63.80	72.10	82.50	90.10
1.5 hour	28.40	32.20	43.73	51.13	58.13	67.33	74.00
2 hour	23.65	27.00	37.15	43.80	50.00	58.50	64.50
3 hour	18.20	21.00	29.63	35.33	41.00	48.33	53.67
4.5 hour	14.02	16.38	23.78	28.89	33.78	40.22	45.33
6 hour	11.70	13.77	20.33	25.00	29.50	35.67	40.33
9 hour	9.09	10.82	16.44	20.44	24.56	30.00	34.44
12 hour	7.63	9.17	14.17	17.75	21.50	26.58	30.67
18 hour	5.94	7.22	11.39	14.50	17.72	22.17	25.83
24 hour	5.00	6.08	9.71	12.46	15.33	19.33	22.63
30 hour	4.37	5.30	8.53	11.00	13.60	17.20	20.23
36 hour	3.92	4.75	7.67	9.89	12.28	15.58	18.36
48 hour	3.25	3.96	6.42	8.29	10.35	13.17	15.56
72 hour	2.49	3.03	4.90	6.38	7.97	10.15	12.00
96 hour	2.04	2.48	4.02	5.23	6.55	8.33	9.83
120 hour	1.73	2.10	3.43	4.47	5.61	7.13	8.42
144 hour	1.51	1.83	3.00	3.93	4.94	6.26	7.36
168 hour	1.33	1.63	2.69	3.53	4.46	5.63	6.61

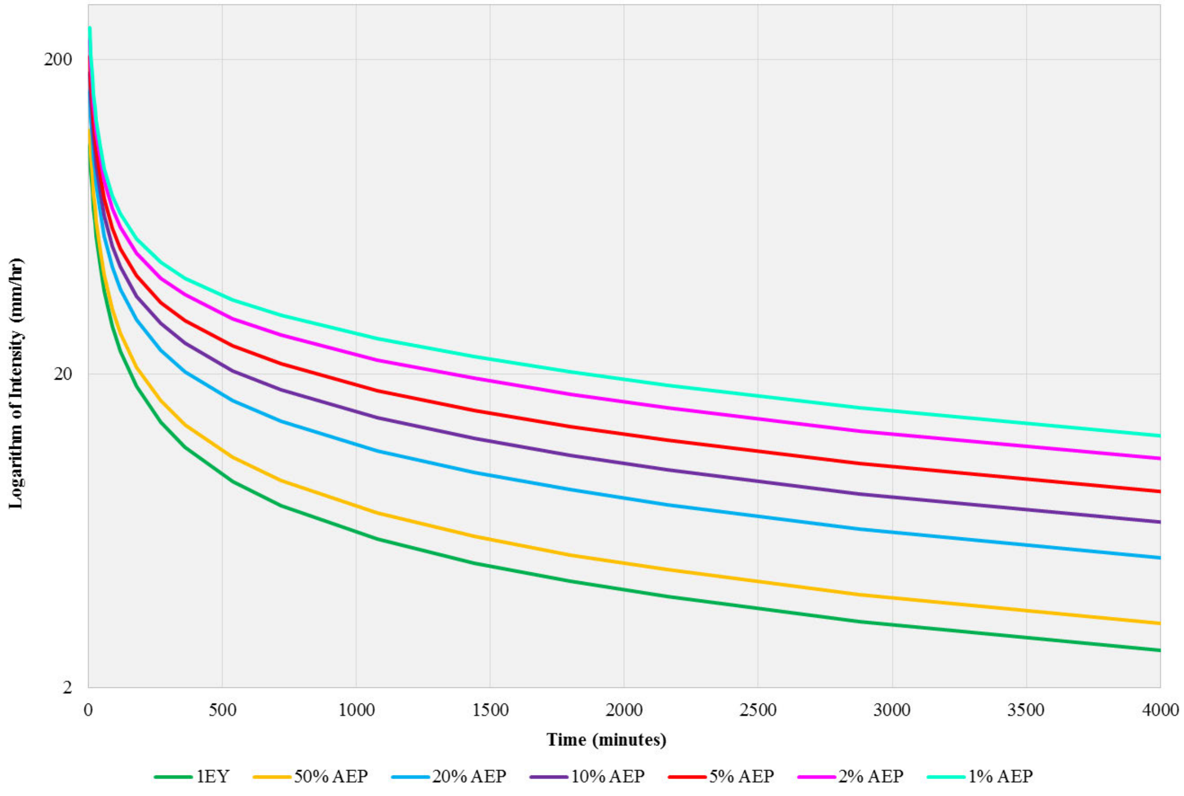


Figure C4 - Blacks Creek Intensity Frequency Duration (IFD) Chart

Stockmans Creek Catchment

Table C5 - Stockmans Creek ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	112.68	126.00	166.80	193.20	217.20	246.00	267.60
10 min	94.80	106.20	138.60	159.00	178.20	201.60	218.40
15 min	81.60	91.20	119.20	136.80	152.80	172.40	186.80
20 min	72.00	80.40	105.00	120.60	134.70	152.40	165.00
25 min	64.56	72.00	94.32	108.24	121.20	137.28	148.80
30 min	58.60	65.40	85.80	98.80	110.80	125.60	136.40
45 min	46.40	52.00	68.80	79.60	89.60	102.13	111.33
1 hour	38.70	43.60	58.30	67.70	76.50	87.80	95.90
1.5 hour	29.73	33.67	45.73	53.60	61.07	70.67	78.00
2 hour	24.50	27.90	38.45	45.35	52.00	60.50	67.00
3 hour	18.60	21.40	30.10	36.00	41.67	49.00	54.67
4.5 hour	14.16	16.44	23.56	28.67	33.33	39.78	44.89
6 hour	11.72	13.70	20.00	24.33	28.83	34.67	39.17
9 hour	9.02	10.64	15.89	19.67	23.44	28.56	32.67
12 hour	7.53	8.92	13.58	16.92	20.33	25.00	28.75
18 hour	5.89	7.00	10.83	13.67	16.61	20.72	24.06
24 hour	4.96	5.92	9.25	11.75	14.42	18.13	21.17
30 hour	4.33	5.20	8.17	10.43	12.87	16.27	19.10

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
36 hour	3.86	4.64	7.36	9.44	11.69	14.86	17.53
48 hour	3.25	3.90	6.21	8.02	10.00	12.77	15.15
72 hour	2.50	3.00	4.82	6.28	7.89	10.13	12.04
96 hour	2.04	2.47	3.98	5.19	6.55	8.42	10.02
120 hour	1.73	2.10	3.39	4.44	5.62	7.20	8.58
144 hour	1.51	1.83	2.97	3.88	4.91	6.28	7.43
168 hour	1.33	1.61	2.63	3.45	4.36	5.55	6.55

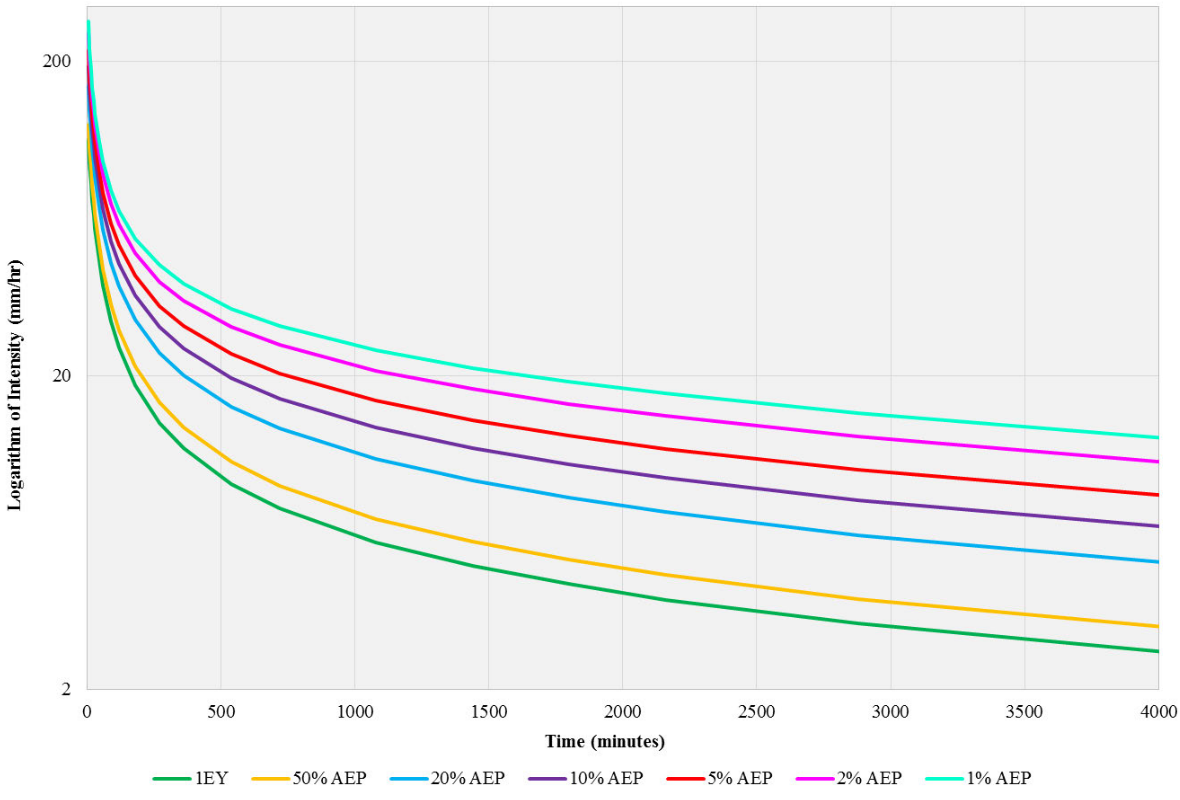


Figure C5 - Stockmans Creek Intensity Frequency Duration (IFD) Chart

Upper Pioneer River Catchment

Table C6 – Upper Pioneer River ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	119.52	133.20	176.40	202.80	228.00	259.20	282.00
10 min	100.20	111.60	145.80	167.40	187.20	211.20	229.20
15 min	86.40	96.00	125.20	143.20	160.40	181.20	196.00
20 min	76.20	84.60	110.40	126.60	141.60	159.90	173.40
25 min	68.16	76.08	99.36	114.00	127.68	144.48	156.72
30 min	62.00	69.20	90.60	104.20	116.80	132.40	143.80
45 min	49.33	55.33	73.07	84.40	95.07	108.40	118.13

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
1 hour	41.40	46.60	62.20	72.30	81.70	93.70	102.00
1.5 hour	32.07	36.33	49.40	57.93	66.00	76.00	84.00
2 hour	26.60	30.40	41.95	49.50	57.00	66.00	73.00
3 hour	20.47	23.60	33.33	40.00	46.00	54.33	60.67
4.5 hour	15.80	18.42	26.67	32.22	37.78	45.11	50.67
6 hour	13.20	15.53	22.83	28.00	33.00	39.67	44.83
9 hour	10.36	12.33	18.56	23.00	27.33	33.33	38.11
12 hour	8.75	10.50	16.08	20.00	24.08	29.58	34.00
18 hour	6.94	8.39	13.11	16.50	20.06	24.94	28.94
24 hour	5.96	7.17	11.29	14.33	17.54	22.00	25.71
30 hour	5.23	6.33	10.07	12.83	15.73	19.87	23.33
36 hour	4.72	5.72	9.11	11.67	14.36	18.22	21.44
48 hour	4.00	4.85	7.75	9.96	12.31	15.73	18.63
72 hour	3.10	3.76	6.04	7.81	9.71	12.46	14.86
96 hour	2.54	3.07	4.97	6.43	8.03	10.31	12.29
120 hour	2.14	2.60	4.21	5.47	6.84	8.75	10.42
144 hour	1.84	2.24	3.64	4.74	5.95	7.64	9.03
168 hour	1.61	1.96	3.20	4.18	5.25	6.67	7.92

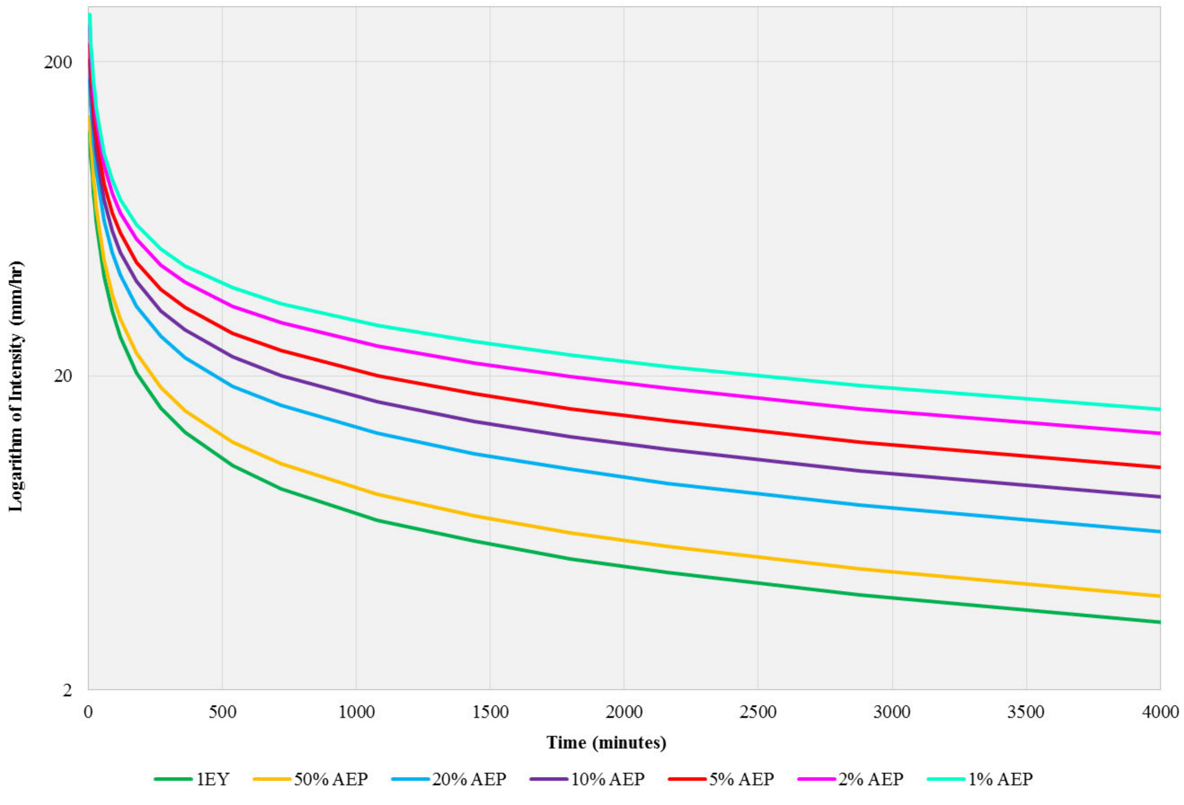


Figure C6 – Upper Pioneer River Intensity Frequency Duration (IFD) Chart

Middle Pioneer River Catchment

Table C7 – Middle Pioneer River ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	127.20	142.80	188.40	218.40	244.80	279.60	304.80
10 min	105.60	117.60	154.20	177.60	198.60	225.60	244.80
15 min	90.80	101.20	132.00	151.60	170.00	192.80	209.20
20 min	80.10	89.10	116.40	133.80	150.00	170.40	185.10
25 min	72.00	80.16	104.88	120.72	135.60	154.08	167.52
30 min	65.60	73.20	96.00	110.60	124.40	141.60	154.20
45 min	52.53	58.93	78.00	90.27	101.87	116.67	127.33
1 hour	44.60	50.10	66.90	77.90	88.20	101.00	111.00
1.5 hour	35.00	39.67	53.80	63.07	72.00	83.33	92.00
2 hour	29.40	33.50	46.05	54.50	62.50	72.50	80.50
3 hour	23.00	26.43	37.00	44.33	51.00	60.00	67.00
4.5 hour	18.04	20.93	30.00	36.22	42.22	50.22	56.22
6 hour	15.22	17.83	25.83	31.50	37.00	44.17	50.00
9 hour	12.00	14.11	21.11	25.89	30.67	37.22	42.33
12 hour	10.17	12.08	18.17	22.50	26.92	32.92	37.75
18 hour	8.06	9.61	14.78	18.50	22.28	27.67	32.00
24 hour	6.83	8.17	12.67	15.96	19.42	24.29	28.29
30 hour	5.97	7.17	11.20	14.20	17.33	21.83	25.60
36 hour	5.36	6.42	10.08	12.83	15.75	19.97	23.50
48 hour	4.48	5.38	8.50	10.88	13.44	17.15	20.29
72 hour	3.43	4.13	6.56	8.46	10.51	13.51	16.11
96 hour	2.80	3.38	5.39	6.97	8.70	11.15	13.33
120 hour	2.38	2.87	4.59	5.94	7.43	9.50	11.33
144 hour	2.06	2.49	4.01	5.19	6.49	8.33	9.86
168 hour	1.82	2.21	3.57	4.62	5.76	7.32	8.63

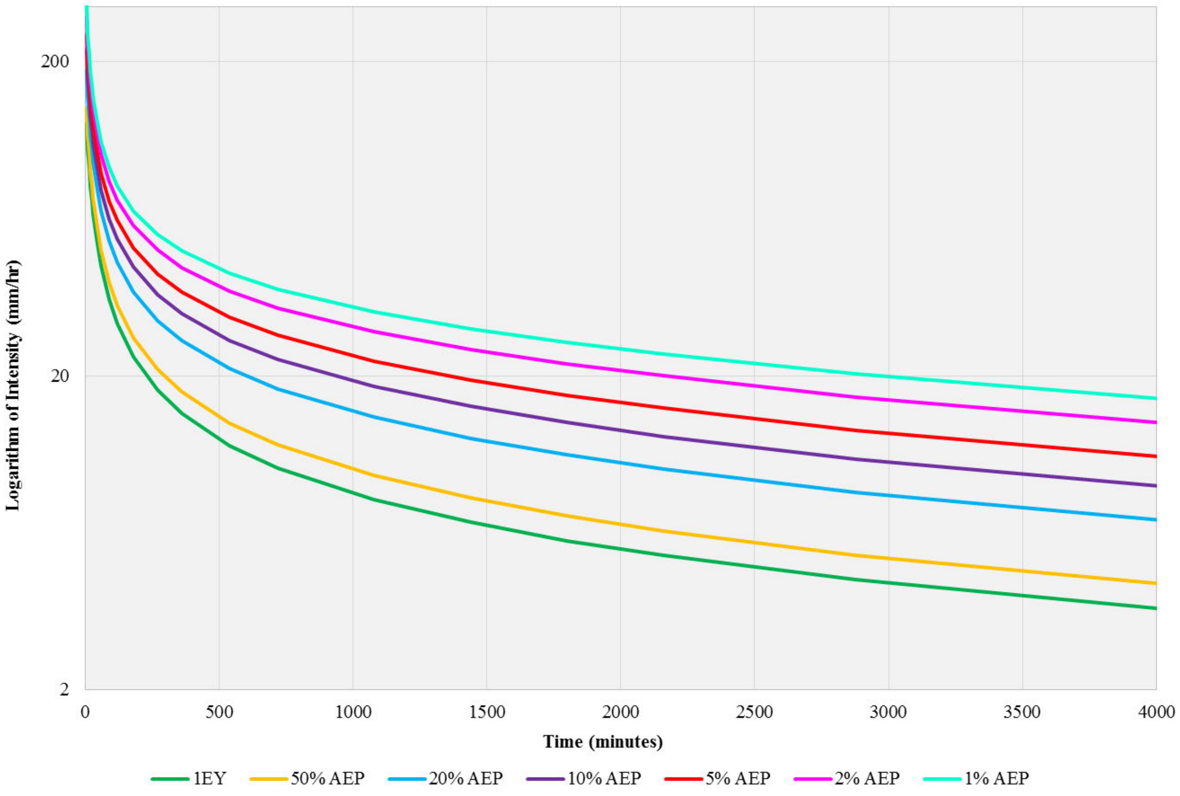


Figure C7 – Middle Pioneer River Intensity Frequency Duration (IFD) Chart

Lower Pioneer River Catchment

Table C8 – Lower Pioneer River ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	129.60	145.20	192.00	223.20	252.00	289.20	316.80
10 min	106.20	118.80	156.00	180.60	203.40	232.80	254.40
15 min	91.20	102.00	134.00	154.40	174.00	198.80	217.20
20 min	80.70	90.00	118.20	136.50	153.90	176.10	192.60
25 min	72.48	81.12	106.56	123.36	139.20	159.36	174.24
30 min	66.20	74.00	97.60	113.20	127.80	146.40	160.40
45 min	53.33	59.73	79.47	92.40	104.80	120.53	132.27
1 hour	45.30	50.90	68.20	79.70	90.60	105.00	115.00
1.5 hour	35.67	40.33	54.73	64.33	73.33	85.33	94.00
2 hour	29.95	34.05	46.75	55.00	63.50	74.00	82.00
3 hour	23.33	26.77	37.33	44.33	51.33	60.33	67.33
4.5 hour	18.20	21.02	30.00	36.00	41.78	49.56	55.56
6 hour	15.25	17.67	25.50	30.83	36.17	43.17	48.67
9 hour	11.89	14.00	20.44	25.00	29.56	35.67	40.44
12 hour	10.00	11.75	17.50	21.50	25.58	31.17	35.58
18 hour	7.83	9.22	13.94	17.33	20.83	25.72	29.67
24 hour	6.54	7.75	11.83	14.83	17.92	22.38	26.00
30 hour	5.70	6.77	10.40	13.07	15.90	20.00	23.40

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
36 hour	5.08	6.03	9.31	11.78	14.39	18.19	21.39
48 hour	4.21	5.02	7.79	9.92	12.21	15.56	18.42
72 hour	3.21	3.82	5.97	7.67	9.50	12.21	14.58
96 hour	2.61	3.13	4.91	6.30	7.82	10.08	11.98
120 hour	2.22	2.65	4.18	5.37	6.67	8.58	10.17
144 hour	1.93	2.31	3.65	4.69	5.80	7.43	8.82
168 hour	1.71	2.05	3.26	4.16	5.13	6.55	7.68

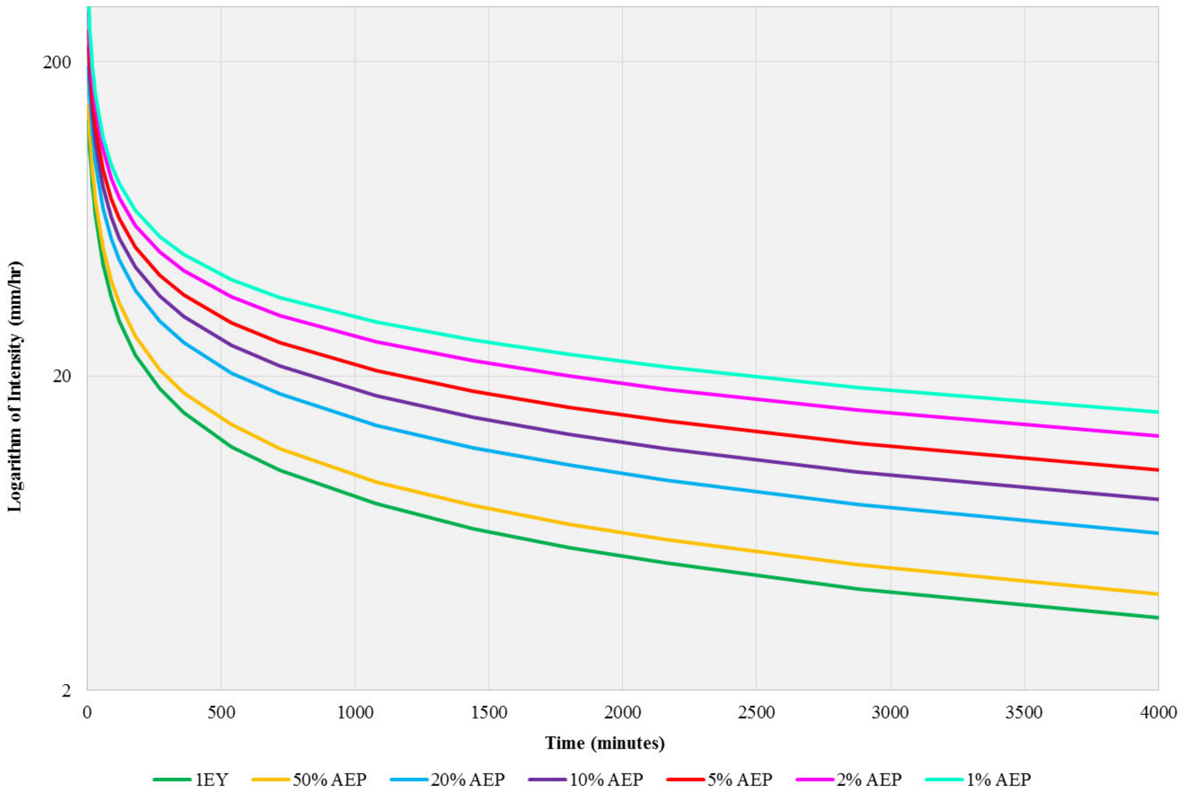


Figure C8 – Lower Pioneer River Intensity Frequency Duration (IFD) Chart

Pioneer River Outlet Catchment

Table C9 – Pioneer River Outlet ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	127.20	142.80	189.60	220.80	249.60	286.80	314.40
10 min	105.00	117.60	154.80	178.80	201.60	231.60	253.20
15 min	90.40	100.80	132.40	153.20	172.80	198.00	216.80
20 min	79.80	89.10	117.00	135.30	153.00	175.20	192.00
25 min	71.76	80.16	105.60	122.40	138.24	158.64	173.76
30 min	65.60	73.20	96.60	112.00	126.80	145.60	159.80

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
45 min	52.80	59.07	78.53	91.47	103.73	119.60	131.47
1 hour	44.80	50.30	67.40	78.70	89.60	104.00	114.00
1.5 hour	35.13	39.73	53.93	63.33	72.67	84.00	92.67
2 hour	29.45	33.45	45.90	54.00	62.00	72.50	80.50
3 hour	22.87	26.20	36.33	43.33	50.00	59.00	65.33
4.5 hour	17.73	20.47	29.11	34.89	40.44	48.00	53.56
6 hour	14.80	17.17	24.67	29.83	34.83	41.50	46.67
9 hour	11.44	13.44	19.67	23.89	28.11	34.00	38.56
12 hour	9.58	11.25	16.67	20.42	24.25	29.50	33.67
18 hour	7.44	8.78	13.17	16.33	19.56	24.17	27.89
24 hour	6.21	7.33	11.13	13.92	16.79	20.92	24.33
30 hour	5.37	6.37	9.73	12.23	14.83	18.63	21.80
36 hour	4.78	5.67	8.69	10.97	13.39	16.94	19.89
48 hour	3.96	4.69	7.25	9.23	11.33	14.44	17.08
72 hour	3.00	3.57	5.56	7.11	8.81	11.33	13.50
96 hour	2.45	2.92	4.56	5.85	7.27	9.36	11.15
120 hour	2.08	2.48	3.90	5.00	6.20	7.98	9.50
144 hour	1.82	2.17	3.42	4.38	5.41	6.93	8.19
168 hour	1.61	1.94	3.06	3.90	4.80	6.13	7.20

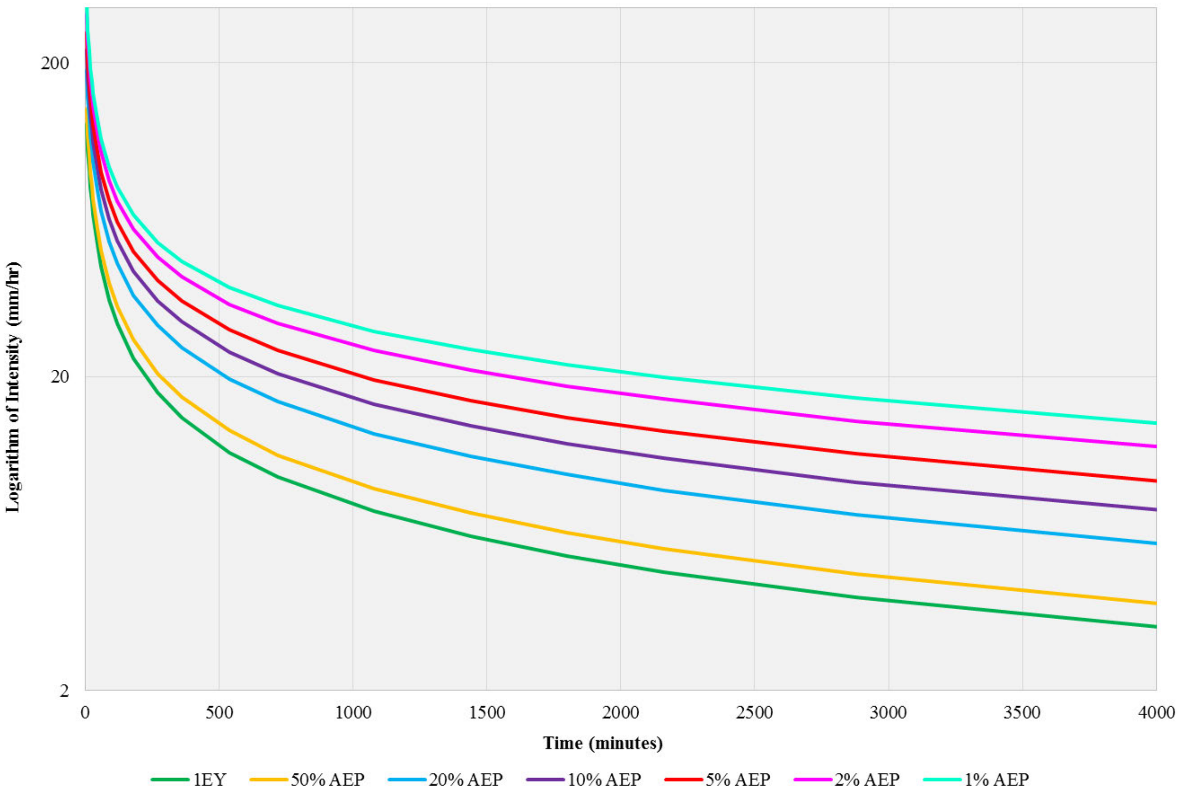


Figure C9 – Pioneer River Outlet Intensity Frequency Duration (IFD) Chart

Gooseponds Catchment

Table C10 - Gooseponds ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	127.20	142.80	189.60	220.80	249.60	286.80	314.40
10 min	105.00	117.60	154.80	179.40	202.20	231.60	253.80
15 min	90.40	100.80	132.80	153.60	173.20	198.40	216.80
20 min	79.80	89.10	117.30	135.60	153.00	175.50	192.00
25 min	71.76	80.16	105.60	122.40	138.24	158.64	173.76
30 min	65.60	73.20	96.80	112.20	126.80	145.60	159.80
45 min	52.67	59.07	78.67	91.47	103.87	119.60	131.33
1 hour	44.70	50.30	67.40	78.70	89.60	104.00	114.00
1.5 hour	35.13	39.73	53.93	63.40	72.67	84.00	92.67
2 hour	29.45	33.50	45.95	54.50	62.50	72.50	80.50
3 hour	22.93	26.27	36.67	43.67	50.33	59.33	66.00
4.5 hour	17.82	20.60	29.33	35.11	40.89	48.44	54.22
6 hour	14.92	17.33	25.00	30.17	35.17	42.00	47.33
9 hour	11.67	13.56	19.89	24.33	28.67	34.56	39.22
12 hour	9.75	11.42	17.00	20.83	24.75	30.17	34.42
18 hour	7.56	8.94	13.50	16.72	20.06	24.78	28.61
24 hour	6.33	7.50	11.42	14.29	17.25	21.50	25.00
30 hour	5.50	6.53	10.00	12.57	15.27	19.20	22.43
36 hour	4.89	5.81	8.94	11.31	13.78	17.42	20.47
48 hour	4.06	4.81	7.48	9.50	11.67	14.88	17.58
72 hour	3.08	3.67	5.72	7.32	9.07	11.65	13.88
96 hour	2.51	3.00	4.70	6.02	7.48	9.63	11.46
120 hour	2.13	2.55	4.01	5.14	6.38	8.20	9.75
144 hour	1.86	2.23	3.52	4.51	5.57	7.15	8.47
168 hour	1.65	1.99	3.14	4.02	4.94	6.31	7.38

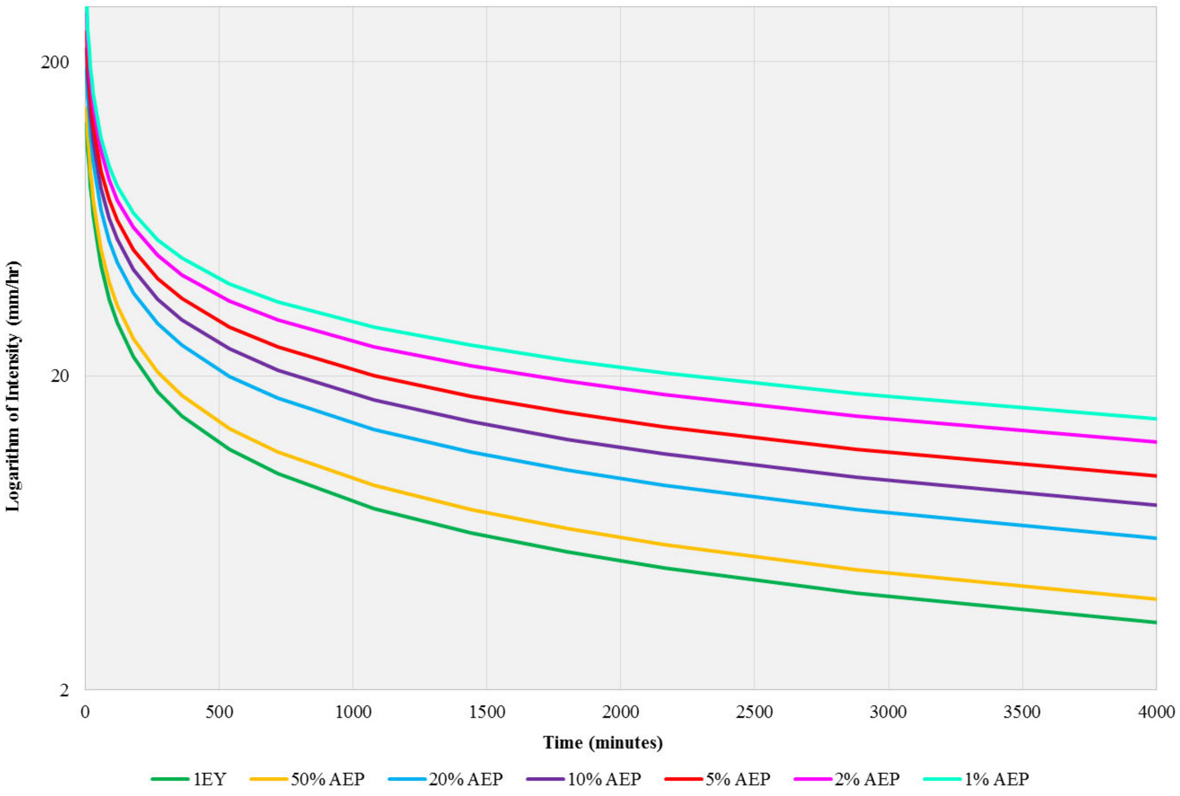


Figure C10 - Gooseponds Intensity Frequency Duration (IFD) Chart

Upper Bakers Creek Catchment

Table C11 – Upper Bakers Creek ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	130.80	146.40	193.20	224.40	253.20	289.20	316.80
10 min	107.40	119.40	157.20	181.20	203.40	231.60	252.60
15 min	92.00	102.80	134.40	154.80	174.00	198.00	215.60
20 min	81.30	90.60	118.80	136.80	153.90	175.20	191.10
25 min	73.20	81.60	107.28	123.60	139.20	158.88	173.28
30 min	66.80	74.60	98.20	113.40	127.80	146.00	159.40
45 min	53.87	60.27	80.00	92.93	105.07	120.67	132.00
1 hour	45.70	51.40	68.80	80.20	91.00	105.00	115.00
1.5 hour	36.00	40.73	55.33	64.93	74.00	86.00	94.67
2 hour	30.25	34.40	47.25	56.00	64.00	74.50	82.50
3 hour	23.60	27.07	38.00	45.00	52.00	61.33	68.33
4.5 hour	18.38	21.29	30.44	36.44	42.44	50.44	56.67
6 hour	15.42	18.00	26.00	31.50	36.83	44.17	49.67
9 hour	12.00	14.11	20.89	25.56	30.22	36.56	41.56
12 hour	10.17	11.92	17.83	22.00	26.17	32.00	36.67
18 hour	7.94	9.39	14.28	17.78	21.39	26.50	30.61
24 hour	6.67	7.92	12.13	15.25	18.50	23.08	26.92

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
30 hour	5.80	6.90	10.67	13.47	16.43	20.70	24.23
36 hour	5.17	6.17	9.58	12.14	14.89	18.83	22.17
48 hour	4.31	5.15	8.02	10.25	12.65	16.13	19.10
72 hour	3.28	3.92	6.17	7.93	9.86	12.68	15.14
96 hour	2.68	3.20	5.06	6.52	8.14	10.52	12.50
120 hour	2.27	2.72	4.31	5.55	6.93	8.92	10.58
144 hour	1.97	2.36	3.76	4.84	6.02	7.71	9.17
168 hour	1.74	2.10	3.33	4.29	5.32	6.79	7.98

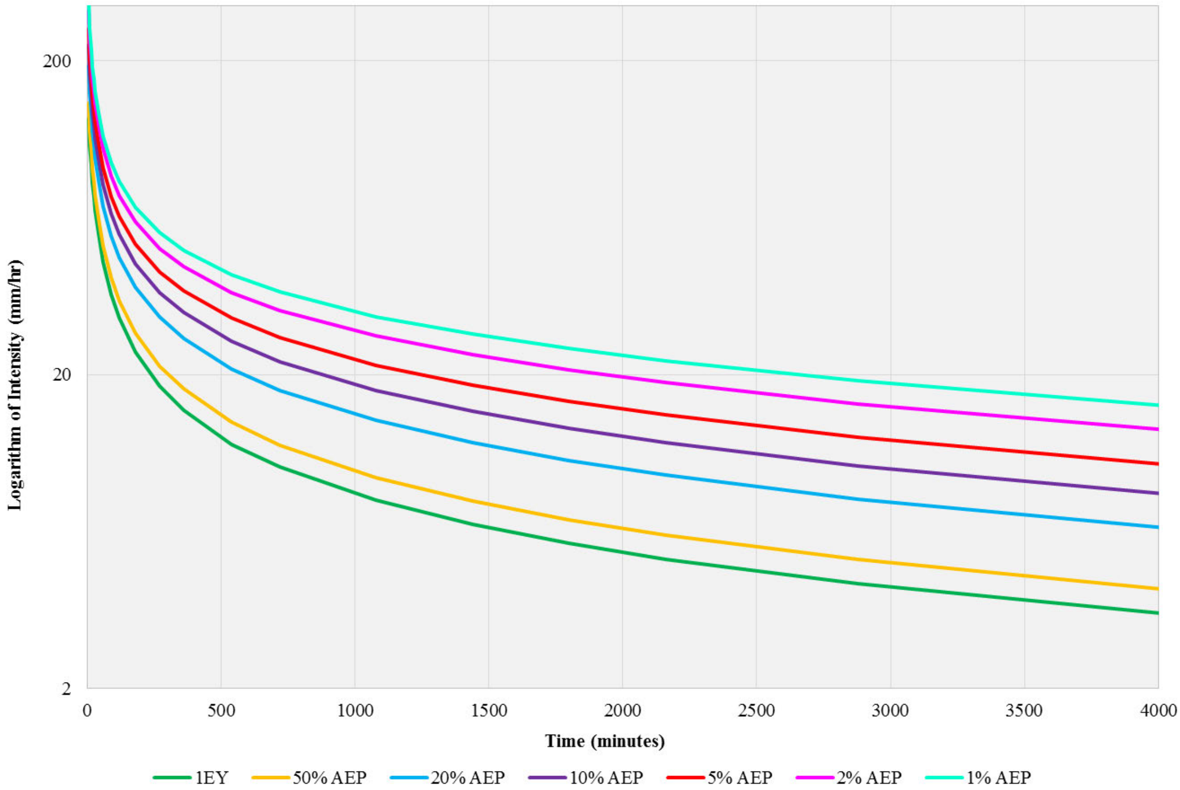


Figure C11 – Upper Bakers Creek Intensity Frequency Duration (IFD) Chart

Lower Bakers Creek Catchment

Table C12 - Lower Bakers Creek ARR16 Intensity Frequency Duration Table (BoM)

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
5 min	129.60	145.20	192.00	223.20	252.00	289.20	315.60
10 min	106.20	118.80	156.00	180.00	202.80	231.60	252.60
15 min	91.20	101.60	133.60	154.00	173.20	198.00	216.00
20 min	80.70	90.00	117.90	136.20	153.30	175.20	191.40
25 min	72.48	81.12	106.56	123.12	138.72	158.64	173.52
30 min	66.40	74.00	97.60	112.80	127.40	146.00	159.60

Duration	Rainfall Intensity (mm/hr) per AEP						
	1EY	50%	20%	10%	5%	2%	1%
45 min	53.47	59.87	79.47	92.40	104.67	120.27	131.87
1 hour	45.40	51.00	68.30	79.60	90.50	104.00	115.00
1.5 hour	35.73	40.40	54.73	64.27	73.33	85.33	94.00
2 hour	30.00	34.10	46.70	55.00	63.00	73.50	81.50
3 hour	23.37	26.77	37.33	44.33	51.00	60.00	66.67
4.5 hour	18.16	20.96	29.78	35.56	41.33	49.11	54.89
6 hour	15.18	17.67	25.33	30.50	35.67	42.67	48.00
9 hour	11.78	13.78	20.22	24.56	29.00	35.00	39.67
12 hour	9.92	11.58	17.17	21.08	25.00	30.50	34.83
18 hour	7.67	9.06	13.67	16.94	20.28	25.06	28.89
24 hour	6.42	7.58	11.54	14.42	17.42	21.75	25.29
30 hour	5.57	6.60	10.10	12.70	15.43	19.40	22.73
36 hour	4.97	5.89	9.06	11.44	13.97	17.67	20.75
48 hour	4.13	4.90	7.56	9.63	11.83	15.08	17.88
72 hour	3.13	3.72	5.81	7.43	9.21	11.86	14.17
96 hour	2.55	3.03	4.76	6.11	7.60	9.80	11.67
120 hour	2.16	2.58	4.06	5.22	6.48	8.33	9.92
144 hour	1.88	2.25	3.55	4.55	5.64	7.22	8.54
168 hour	1.66	2.00	3.16	4.04	4.98	6.37	7.50

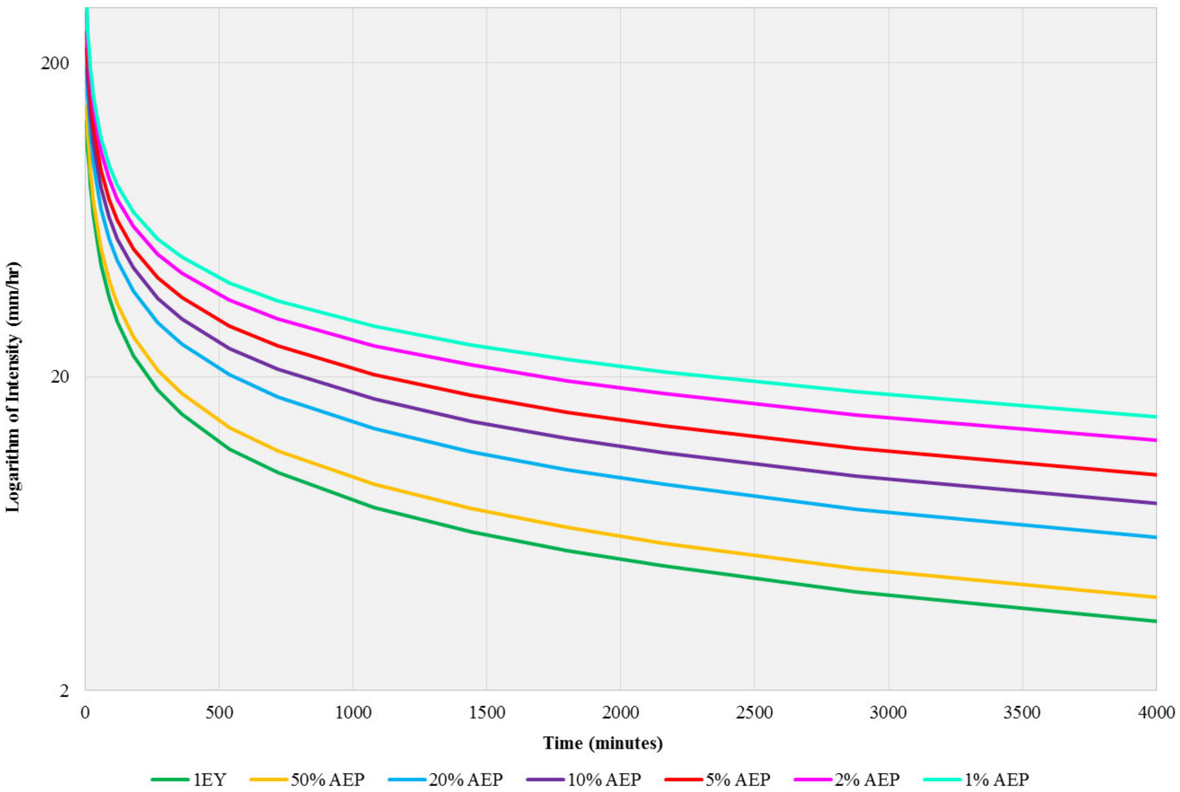


Figure C12 - Lower Bakers Creek Intensity Frequency Duration (IFD) Chart

Preburst Rainfall

The following sections contain ARR 2016 preburst rainfall depths and ratios for each rainfall catchment.

Upper Cattle Creek Catchment

Table C13 - Upper Cattle Creek ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	16.6 (0.367)	13.0 (0.217)	10.7 (0.154)	8.4 (0.107)	17.5 (0.196)	24.4 (0.25)
90 (1.5)	8.4 (0.156)	14.4 (0.198)	18.4 (0.216)	22.2 (0.23)	38.5 (0.346)	50.8 (0.416)
120 (2.0)	11.1 (0.181)	19.6 (0.234)	25.3 (0.257)	30.7 (0.273)	43.4 (0.332)	52.9 (0.367)
180 (3.0)	22.5 (0.309)	32.8 (0.321)	39.7 (0.325)	46.3 (0.328)	73.8 (0.445)	94.5 (0.512)
360 (6.0)	26.1 (0.266)	45.6 (0.314)	58.6 (0.329)	71.0 (0.337)	91.3 (0.36)	106.4 (0.371)
720 (12.0)	27.3 (0.206)	55.8 (0.271)	74.6 (0.288)	92.7 (0.296)	188.3 (0.487)	259.9 (0.583)
1080 (18.0)	18.9 (0.12)	47.4 (0.19)	66.4 (0.209)	84.5 (0.218)	158.9 (0.328)	214.7 (0.381)
1440 (24.0)	11.6 (0.066)	63.3 (0.223)	97.5 (0.268)	130.3 (0.292)	157.3 (0.281)	177.6 (0.271)
2160 (36.0)	9.8 (0.047)	50.3 (0.15)	77.1 (0.179)	102.9 (0.194)	115.5 (0.172)	124.9 (0.159)
2880 (48.0)	3.1 (0.014)	29.7 (0.08)	47.2 (0.099)	64.1 (0.108)	105.5 (0.141)	136.5 (0.155)
4320 (72.0)	0.0 (0.0)	19.7 (0.047)	32.7 (0.06)	45.2 (0.067)	51.3 (0.06)	55.8 (0.056)

Lower Cattle Creek Catchment

Table C14 - Lower Cattle Creek ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	17.7 (0.381)	14.0 (0.225)	11.5 (0.16)	9.1 (0.112)	34.2 (0.367)	53.0 (0.52)
90 (1.5)	13.0 (0.239)	25.6 (0.345)	33.9 (0.39)	41.9 (0.423)	56.3 (0.492)	67.1 (0.534)
120 (2.0)	22.8 (0.374)	29.2 (0.347)	33.5 (0.337)	37.5 (0.33)	48.6 (0.366)	56.8 (0.388)
180 (3.0)	24.1 (0.338)	33.9 (0.337)	40.5 (0.336)	46.7 (0.335)	59.9 (0.364)	69.7 (0.381)
360 (6.0)	28.1 (0.299)	46.4 (0.335)	58.6 (0.345)	70.3 (0.351)	92.7 (0.385)	109.6 (0.402)

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
720 (12.0)	27.2 (0.217)	54.6 (0.282)	72.8 (0.301)	90.2 (0.31)	160.3 (0.448)	212.9 (0.517)
1080 (18.0)	17.7 (0.118)	43.2 (0.184)	60.0 (0.202)	76.2 (0.211)	124.9 (0.278)	161.4 (0.309)
1440 (24.0)	11.1 (0.065)	48.3 (0.179)	73.0 (0.213)	96.6 (0.231)	145.8 (0.278)	182.6 (0.298)
2160 (36.0)	9.6 (0.048)	45.7 (0.142)	69.6 (0.169)	92.5 (0.182)	129.0 (0.201)	156.4 (0.207)
2880 (48.0)	1.5 (0.007)	25.2 (0.069)	40.8 (0.088)	55.9 (0.097)	100.1 (0.137)	133.3 (0.154)
4320 (72.0)	0.0 (0.0)	14.1 (0.034)	23.5 (0.043)	32.5 (0.048)	52.7 (0.061)	67.9 (0.066)

Teemburra Creek Catchment

Table C15 - Teemburra Creek ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.2 (0.194)	7.9 (0.142)	7.8 (0.12)	7.6 (0.104)	17.4 (0.207)	24.6 (0.27)
90 (1.5)	3.6 (0.073)	10.0 (0.149)	14.3 (0.182)	18.3 (0.206)	35.3 (0.343)	48.0 (0.425)
120 (2.0)	6.0 (0.107)	14.1 (0.184)	19.5 (0.216)	24.6 (0.239)	38.8 (0.325)	49.5 (0.375)
180 (3.0)	17.7 (0.27)	27.1 (0.294)	33.3 (0.303)	39.3 (0.309)	83.2 (0.555)	116.1 (0.696)
360 (6.0)	27.1 (0.311)	43.7 (0.338)	54.6 (0.345)	65.1 (0.347)	94.7 (0.418)	116.9 (0.456)
720 (12.0)	24.1 (0.205)	42.1 (0.23)	54.0 (0.234)	65.5 (0.234)	105.6 (0.305)	135.7 (0.34)
1080 (18.0)	16.3 (0.116)	38.1 (0.17)	52.5 (0.184)	66.4 (0.189)	103.5 (0.236)	131.4 (0.257)
1440 (24.0)	8.7 (0.054)	34.0 (0.132)	50.7 (0.154)	66.7 (0.164)	107.7 (0.21)	138.3 (0.23)
2160 (36.0)	2.2 (0.011)	39.6 (0.129)	64.3 (0.162)	88.1 (0.179)	123.9 (0.198)	150.7 (0.205)
2880 (48.0)	1.4 (0.007)	24.2 (0.07)	39.3 (0.088)	53.8 (0.097)	95.7 (0.135)	127.2 (0.153)
4320 (72.0)	0.0 (0.0)	12.1 (0.03)	20.1 (0.039)	27.8 (0.043)	51.9 (0.063)	70.0 (0.073)

Blacks Creek Catchment

Table C16 - Blacks Creek ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.2 (0.199)	8.2 (0.15)	8.3 (0.129)	8.3 (0.115)	16.8 (0.203)	23.2 (0.257)
90 (1.5)	2.6 (0.055)	8.9 (0.136)	13.0 (0.17)	17.0 (0.195)	35.0 (0.348)	48.5 (0.439)
120 (2.0)	1.8 (0.034)	10.5 (0.142)	16.3 (0.186)	21.9 (0.218)	36.6 (0.314)	47.6 (0.37)
180 (3.0)	7.7 (0.122)	14.6 (0.165)	19.2 (0.181)	23.7 (0.193)	52.4 (0.363)	74.0 (0.459)
360 (6.0)	27.1 (0.328)	41.8 (0.342)	51.6 (0.344)	61.0 (0.344)	77.3 (0.362)	89.6 (0.37)
720 (12.0)	17.8 (0.162)	36.2 (0.213)	48.5 (0.227)	60.2 (0.233)	83.5 (0.262)	101.1 (0.274)
1080 (18.0)	3.8 (0.029)	27.0 (0.132)	42.4 (0.163)	57.2 (0.179)	93.6 (0.234)	120.9 (0.26)
1440 (24.0)	3.5 (0.024)	28.9 (0.124)	45.8 (0.153)	62.0 (0.168)	100.7 (0.217)	129.7 (0.239)
2160 (36.0)	0.0 (0.0)	17.4 (0.063)	29.0 (0.081)	40.0 (0.091)	84.6 (0.151)	118.1 (0.179)
2880 (48.0)	0.0 (0.0)	13.2 (0.043)	21.9 (0.055)	30.3 (0.061)	74.9 (0.119)	108.4 (0.145)
4320 (72.0)	0.0 (0.0)	8.4 (0.024)	13.9 (0.03)	19.3 (0.034)	42.6 (0.058)	60.1 (0.07)

Stockmans Creek Catchment

Table C17 - Stockmans Creek ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	3.0 (0.069)	6.7 (0.115)	9.1 (0.134)	11.4 (0.149)	10.8 (0.123)	10.3 (0.108)
90 (1.5)	2.2 (0.043)	8.4 (0.122)	12.4 (0.155)	16.4 (0.179)	26.6 (0.251)	34.3 (0.294)
120 (2.0)	1.6 (0.029)	9.7 (0.126)	15.1 (0.166)	20.2 (0.194)	28.4 (0.235)	34.6 (0.259)
180 (3.0)	2.3 (0.036)	12.1 (0.134)	18.6 (0.173)	24.9 (0.2)	37.6 (0.256)	47.2 (0.288)
360 (6.0)	17.4 (0.212)	31.4 (0.261)	40.7 (0.278)	49.6 (0.287)	70.8 (0.341)	86.7 (0.369)
720 (12.0)	15.9 (0.149)	35.1 (0.216)	47.8 (0.236)	60.0 (0.246)	69.9 (0.233)	77.3 (0.224)
1080 (18.0)	6.2 (0.049)	24.8 (0.127)	37.1 (0.151)	48.9 (0.164)	86.8 (0.233)	115.2 (0.266)
1440 (24.0)	3.8 (0.026)	23.0 (0.104)	35.8 (0.127)	48.0 (0.139)	63.5 (0.146)	75.1 (0.148)

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
2160 (36.0)	0.0 (0.0)	17.7 (0.067)	29.5 (0.087)	40.8 (0.097)	83.2 (0.156)	114.9 (0.182)
2880 (48.0)	0.0 (0.0)	11.6 (0.039)	19.3 (0.05)	26.6 (0.055)	71.8 (0.117)	105.7 (0.145)
4320 (72.0)	0.0 (0.0)	9.1 (0.026)	15.1 (0.033)	20.9 (0.037)	45.2 (0.062)	63.4 (0.073)

Upper Pioneer River Catchment

Table C18 – Upper Pioneer River ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	17.3 (0.372)	13.7 (0.22)	11.3 (0.156)	9.0 (0.11)	21.2 (0.226)	30.4 (0.296)
90 (1.5)	10.0 (0.184)	17.5 (0.236)	22.4 (0.258)	27.1 (0.274)	46.9 (0.41)	61.8 (0.491)
120 (2.0)	8.7 (0.144)	17.9 (0.213)	24.0 (0.242)	29.8 (0.262)	40.1 (0.303)	47.9 (0.328)
180 (3.0)	18.2 (0.257)	28.0 (0.28)	34.5 (0.288)	40.7 (0.294)	84.8 (0.52)	117.9 (0.649)
360 (6.0)	27.3 (0.293)	43.8 (0.319)	54.7 (0.326)	65.2 (0.33)	84.6 (0.355)	99.0 (0.368)
720 (12.0)	21.4 (0.17)	44.6 (0.232)	60.0 (0.25)	74.7 (0.259)	106.4 (0.3)	130.1 (0.319)
1080 (18.0)	8.5 (0.056)	31.1 (0.132)	46.1 (0.155)	60.5 (0.168)	100.5 (0.224)	130.5 (0.251)
1440 (24.0)	6.8 (0.039)	33.9 (0.125)	51.8 (0.15)	69.0 (0.164)	101.1 (0.192)	125.1 (0.203)
2160 (36.0)	1.6 (0.008)	34.3 (0.105)	56.0 (0.133)	76.8 (0.149)	86.9 (0.133)	94.5 (0.122)
2880 (48.0)	1.4 (0.006)	22.8 (0.061)	37.0 (0.077)	50.6 (0.086)	90.3 (0.12)	120.0 (0.134)
4320 (72.0)	0.0 (0.0)	17.1 (0.039)	28.5 (0.051)	39.4 (0.056)	56.8 (0.063)	69.8 (0.065)

Middle Pioneer River Catchment

Table C19 – Middle Pioneer River ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.6 (0.172)	10.2 (0.152)	11.2 (0.144)	12.2 (0.138)	43.4 (0.428)	66.8 (0.601)
90 (1.5)	14.6 (0.246)	29.4 (0.365)	39.3 (0.415)	48.7 (0.451)	53.8 (0.43)	57.6 (0.419)
120 (2.0)	24.3 (0.363)	31.0 (0.337)	35.4 (0.326)	39.7 (0.319)	51.9 (0.357)	61.0 (0.38)
180	26.2	31.9	35.7	39.3	46.2	51.4

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
(3.0)	(0.33)	(0.286)	(0.269)	(0.256)	(0.256)	(0.256)
360 (6.0)	32.9 (0.309)	53.3 (0.343)	66.8 (0.354)	79.8 (0.36)	92.0 (0.347)	101.2 (0.338)
720 (12.0)	30.2 (0.209)	60.0 (0.275)	79.7 (0.295)	98.6 (0.305)	166.5 (0.421)	217.4 (0.48)
1080 (18.0)	30.2 (0.174)	52.9 (0.199)	68.0 (0.204)	82.4 (0.205)	159.6 (0.321)	217.5 (0.378)
1440 (24.0)	27.3 (0.139)	70.6 (0.232)	99.2 (0.259)	126.6 (0.272)	151.8 (0.261)	170.7 (0.251)
2160 (36.0)	14.7 (0.063)	50.2 (0.138)	73.7 (0.16)	96.3 (0.17)	123.2 (0.172)	143.4 (0.17)
2880 (48.0)	5.9 (0.023)	30.6 (0.075)	47.0 (0.09)	62.7 (0.097)	106.9 (0.13)	140.0 (0.144)
4320 (72.0)	0.0 (0.0)	19.6 (0.041)	32.6 (0.053)	45.0 (0.059)	55.9 (0.057)	64.1 (0.055)

<u>Lower Pioneer River Catchment</u>

Table C20 - Lower Pioneer River ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.8 (0.172)	11.1 (0.163)	12.6 (0.159)	14.1 (0.156)	45.3 (0.433)	68.7 (0.598)
90 (1.5)	14.0 (0.232)	25.6 (0.312)	33.3 (0.345)	40.6 (0.369)	45.9 (0.359)	49.9 (0.353)
120 (2.0)	21.5 (0.316)	29.5 (0.315)	34.7 (0.315)	39.8 (0.314)	52.7 (0.357)	62.4 (0.382)
180 (3.0)	26.0 (0.323)	36.1 (0.322)	42.8 (0.321)	49.2 (0.319)	54.2 (0.299)	57.9 (0.287)
360 (6.0)	28.2 (0.265)	49.6 (0.323)	63.7 (0.344)	77.3 (0.356)	100.7 (0.389)	118.3 (0.406)
720 (12.0)	27.5 (0.195)	55.7 (0.265)	74.3 (0.288)	92.1 (0.301)	116.9 (0.313)	135.4 (0.317)
1080 (18.0)	21.8 (0.131)	43.6 (0.174)	58.1 (0.186)	72.0 (0.192)	110.3 (0.238)	139.1 (0.26)
1440 (24.0)	11.5 (0.062)	39.0 (0.137)	57.2 (0.161)	74.7 (0.174)	94.7 (0.176)	109.6 (0.176)
2160 (36.0)	7.8 (0.036)	41.5 (0.124)	63.7 (0.15)	85.0 (0.164)	123.2 (0.188)	151.7 (0.197)
2880 (48.0)	1.7 (0.007)	23.5 (0.063)	38.0 (0.08)	51.8 (0.089)	98.6 (0.132)	133.7 (0.151)
4320 (72.0)	0.0 (0.0)	17.2 (0.04)	28.6 (0.052)	39.6 (0.058)	57.0 (0.065)	70.1 (0.067)

Pioneer River Outlet Catchment

Table C21 – Pioneer River Outlet ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.8 (0.172)	11.1 (0.163)	12.6 (0.159)	14.1 (0.156)	45.3 (0.433)	68.7 (0.598)
90 (1.5)	14.0 (0.232)	25.6 (0.312)	33.3 (0.345)	40.6 (0.369)	45.9 (0.359)	49.9 (0.353)
120 (2.0)	21.5 (0.316)	29.5 (0.315)	34.7 (0.315)	39.8 (0.314)	52.7 (0.357)	62.4 (0.382)
180 (3.0)	26.0 (0.323)	36.1 (0.322)	42.8 (0.321)	49.2 (0.319)	54.2 (0.299)	57.9 (0.287)
360 (6.0)	28.2 (0.265)	49.6 (0.323)	63.7 (0.344)	77.3 (0.356)	100.7 (0.389)	118.3 (0.406)
720 (12.0)	27.5 (0.195)	55.7 (0.265)	74.3 (0.288)	92.1 (0.301)	116.9 (0.313)	135.4 (0.317)
1080 (18.0)	21.8 (0.131)	43.6 (0.174)	58.1 (0.186)	72.0 (0.192)	110.3 (0.238)	139.1 (0.26)
1440 (24.0)	11.5 (0.062)	39.0 (0.137)	57.2 (0.161)	74.7 (0.174)	94.7 (0.176)	109.6 (0.176)
2160 (36.0)	7.8 (0.036)	41.5 (0.124)	63.7 (0.15)	85.0 (0.164)	123.2 (0.188)	151.7 (0.197)
2880 (48.0)	1.7 (0.007)	23.5 (0.063)	38.0 (0.08)	51.8 (0.089)	98.6 (0.132)	133.7 (0.151)
4320 (72.0)	0.0 (0.0)	17.2 (0.04)	28.6 (0.052)	39.6 (0.058)	57.0 (0.065)	70.1 (0.067)

Gooseponds Catchment

Table C22 - Gooseponds ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.6 (0.172)	10.0 (0.148)	10.9 (0.138)	11.8 (0.131)	41.0 (0.396)	62.8 (0.552)
90 (1.5)	16.1 (0.27)	27.4 (0.339)	34.9 (0.367)	42.1 (0.387)	46.2 (0.366)	49.3 (0.353)
120 (2.0)	24.3 (0.362)	29.0 (0.316)	32.2 (0.296)	35.2 (0.282)	49.8 (0.342)	60.7 (0.377)
180 (3.0)	26.0 (0.33)	37.5 (0.341)	45.1 (0.345)	52.4 (0.347)	59.0 (0.332)	63.9 (0.323)
360 (6.0)	29.8 (0.286)	50.6 (0.338)	64.3 (0.356)	77.5 (0.367)	101.1 (0.401)	118.8 (0.418)
720 (12.0)	29.2 (0.213)	61.0 (0.299)	82.0 (0.328)	102.2 (0.344)	131.7 (0.364)	153.8 (0.372)
1080 (18.0)	28.7 (0.178)	50.3 (0.207)	64.6 (0.214)	78.3 (0.217)	115.5 (0.259)	143.4 (0.279)
1440 (24.0)	17.6 (0.098)	57.3 (0.209)	83.6 (0.244)	108.8 (0.263)	103.2 (0.2)	99.0 (0.165)

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
2160 (36.0)	11.3 (0.054)	45.2 (0.14)	67.7 (0.166)	89.2 (0.18)	115.7 (0.184)	135.6 (0.184)
2880 (48.0)	1.7 (0.008)	24.2 (0.068)	39.1 (0.086)	53.4 (0.095)	101.6 (0.142)	137.7 (0.163)
4320 (72.0)	0.0 (0.0)	15.3 (0.037)	25.4 (0.048)	35.1 (0.054)	56.2 (0.067)	72.1 (0.072)

Upper Bakers Creek Catchment

Table C23 – Upper Bakers Creek ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.8 (0.171)	13.8 (0.201)	17.1 (0.213)	20.3 (0.223)	50.7 (0.483)	73.4 (0.638)
90 (1.5)	14.9 (0.243)	26.1 (0.315)	33.6 (0.345)	40.8 (0.367)	46.1 (0.357)	50.0 (0.352)
120 (2.0)	20.7 (0.301)	29.2 (0.309)	34.9 (0.312)	40.3 (0.315)	52.8 (0.354)	62.3 (0.377)
180 (3.0)	26.1 (0.321)	37.2 (0.327)	44.5 (0.329)	51.6 (0.33)	51.8 (0.282)	52.0 (0.254)
360 (6.0)	36.3 (0.337)	54.0 (0.346)	65.7 (0.348)	76.9 (0.348)	90.1 (0.341)	100.0 (0.335)
720 (12.0)	29.3 (0.204)	60.2 (0.281)	80.7 (0.305)	100.3 (0.319)	141.8 (0.369)	172.9 (0.393)
1080 (18.0)	28.8 (0.17)	55.4 (0.216)	73.1 (0.228)	90.0 (0.233)	121.9 (0.256)	145.9 (0.265)
1440 (24.0)	15.7 (0.083)	49.9 (0.171)	72.5 (0.198)	94.1 (0.212)	105.8 (0.191)	114.6 (0.177)
2160 (36.0)	9.8 (0.044)	44.6 (0.129)	67.7 (0.155)	89.8 (0.168)	126.3 (0.186)	153.7 (0.193)
2880 (48.0)	4.3 (0.018)	26.3 (0.068)	40.8 (0.083)	54.7 (0.09)	108.7 (0.14)	149.1 (0.163)
4320 (72.0)	0.0 (0.0)	13.1 (0.029)	21.8 (0.038)	30.1 (0.042)	55.0 (0.06)	73.7 (0.068)

Lower Bakers Creek Catchment

Table C24 – Lower Bakers Creek ARR16 Preburst Depths and Ratios

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
60 (1.0)	8.8 (0.172)	11.1 (0.162)	12.6 (0.159)	14.1 (0.156)	45.3 (0.434)	68.7 (0.599)
90 (1.5)	14.0 (0.231)	25.6 (0.312)	33.3 (0.345)	40.6 (0.369)	45.9 (0.36)	49.9 (0.354)
120 (2.0)	21.5 (0.316)	29.5 (0.316)	34.7 (0.315)	39.8 (0.315)	52.7 (0.358)	62.4 (0.383)
180	26.0	36.1	42.8	49.2	54.2	57.9

Duration min (hr)	Median Preburst Depth in mm (ratio)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
(3.0)	(0.323)	(0.323)	(0.322)	(0.321)	(0.301)	(0.289)
360 (6.0)	28.2 (0.267)	49.6 (0.326)	63.7 (0.347)	77.3 (0.361)	100.7 (0.394)	118.3 (0.411)
720 (12.0)	27.5 (0.198)	55.7 (0.27)	74.3 (0.294)	92.1 (0.307)	116.9 (0.32)	135.4 (0.324)
1080 (18.0)	21.8 (0.134)	43.6 (0.178)	58.1 (0.191)	72.0 (0.197)	110.3 (0.245)	139.1 (0.267)
1440 (24.0)	11.5 (0.063)	39.0 (0.141)	57.2 (0.165)	74.7 (0.179)	94.7 (0.181)	109.6 (0.181)
2160 (36.0)	7.8 (0.037)	41.5 (0.127)	63.7 (0.155)	85.0 (0.169)	123.2 (0.194)	151.7 (0.203)
2880 (48.0)	1.7 (0.007)	23.5 (0.065)	38.0 (0.082)	51.8 (0.091)	98.6 (0.136)	133.7 (0.156)
4320 (72.0)	0.0 (0.0)	17.2 (0.041)	28.6 (0.053)	39.6 (0.06)	57.0 (0.067)	70.1 (0.069)

Areal Reduction Factors

All catchments are situated within the East Coast North region for Areal Reduction Factors (ARF's). The equations for these factors as per ARR 2016 hydrology methods are as follows.

Figure C13 - East Coast North Region for ARF's

Long Duration ARF

$$ARF = Min\left\{1, \left[\begin{array}{l} 1 - a\left(Area^{b} - c\log_{10} Duration\right) Duration^{-d} \\ + eArea^{f} Duration^{g}\left(0.3 + \log_{10} AEP\right) + h10^{iArea\frac{Duration}{1440}}\left(0.3 + \log_{10} AEP\right) \end{array}\right]\right\}$$

Table C25 –East Coast North Long Duration ARF Parameters

Zone	a	b	c	d	e	f	g	h	i
East Coast North	0.327°	0.241	0.448	0.36	0.00096	0.48	-0.21	0.012	-0.0013

<u>Short Duration ARF</u>

$$ARF = Min\left\{1, \begin{bmatrix} 1-0.287\left(Area^{0.265}-0.439\log_{10}\left(Duration\right)\right).Duration^{-0.36} \\ +2.26\times10^{-3}\times Area^{0.226}.\ Duration^{0.125}\left(0.3+\log_{10}\left(AEP\right)\right) \\ +0.0141\times Area^{0.213}\times10^{-0.021\frac{\left(Duration-180\right)^2}{1440}}\left(0.3+\log_{10}\left(AEP\right)\right) \end{bmatrix}\right\}$$

As the critical duration for the Pioneer River catchment was determined to be 12 hours (720 minutes), the ARF for a short duration storm (≤12 hours) was calculated using the above equation for each rainfall catchment for each AEP. The results are shown in Table C26

Table C26 – Short Duration ARF's for Rainfall Catchments (12 Hour Storm)

Catchment	ARF (AEP %)					
	50%	20%	10%	5%	2%	1%
Upper Cattle	0.925	0.918	0.913	0.908	0.901	0.896
Lower Cattle	0.935	0.929	0.924	0.919	0.913	0.908
Teemburra	0.923	0.917	0.911	0.906	0.899	0.894
Stockmans	0.911	0.904	0.898	0.893	0.885	0.879
Blacks	0.919	0.912	0.907	0.901	0.894	0.889
Upper Pioneer	0.919	0.912	0.907	0.901	0.894	0.889
Middle Pioneer	0.939	0.933	0.929	0.924	0.918	0.914
Lower Pioneer	0.958	0.953	0.950	0.946	0.941	0.937
Pioneer Outlet	0.972	0.968	0.965	0.962	0.958	0.955
Goosepongs	0.963	0.959	0.955	0.952	0.947	0.944
Upper Bakers	0.937	0.931	0.926	0.921	0.915	0.911
Lower Bakers	0.953	0.948	0.944	0.940	0.935	0.931

Storm Losses

The initial and continuing loss data was extracted for each rainfall catchment from the ARR 2016 data hub. The loss values for each individual catchment are shown in Table C27

Table C27 – ARR 2016 Data Hub Loss Values

Catchments	Initial Loss (mm)	Continuing Loss (mm/hr)
Upper Cattle	60.0	5.2
Lower Cattle	54.0	5.0
Teemburra	48.0	4.3
Stockmans	58.0	3.6
Blacks	49.0	2.9
Upper Pioneer	54.0	4.3
Middle Pioneer	62.0	4.8
Lower Pioneer	67.0	4.8
Pioneer Outlet	66.0	4.7
Goosepnds	67.0	4.8
Lower Bakers	66.0	4.7
Upper Bakers	65.0	4.7

Ensemble Temporal Patterns

All catchments are situated within the Wet Tropics region (refer Figure C14) for the ARR 2016 Ensemble Temporal Patterns. The 10 patterns, as extracted from the ARR Data Hub with the design rainfall depth and Areal Reduction Factor (ARF) applied. The cumulative rainfall pattern for the critical duration of 12 hours (720 mins) for the 10 Ensemble patterns at each rainfall catchment is shown in the figures below. The ARR 1987 cumulative rainfall depths have also plotted on the graphs (in black) for comparison.

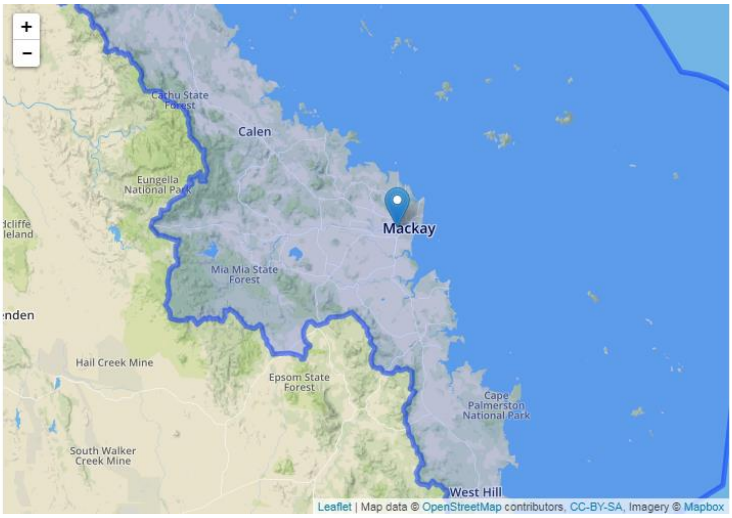


Figure C14 – Wet Tropics Region for ARR 2016 Ensemble Temporal Patterns

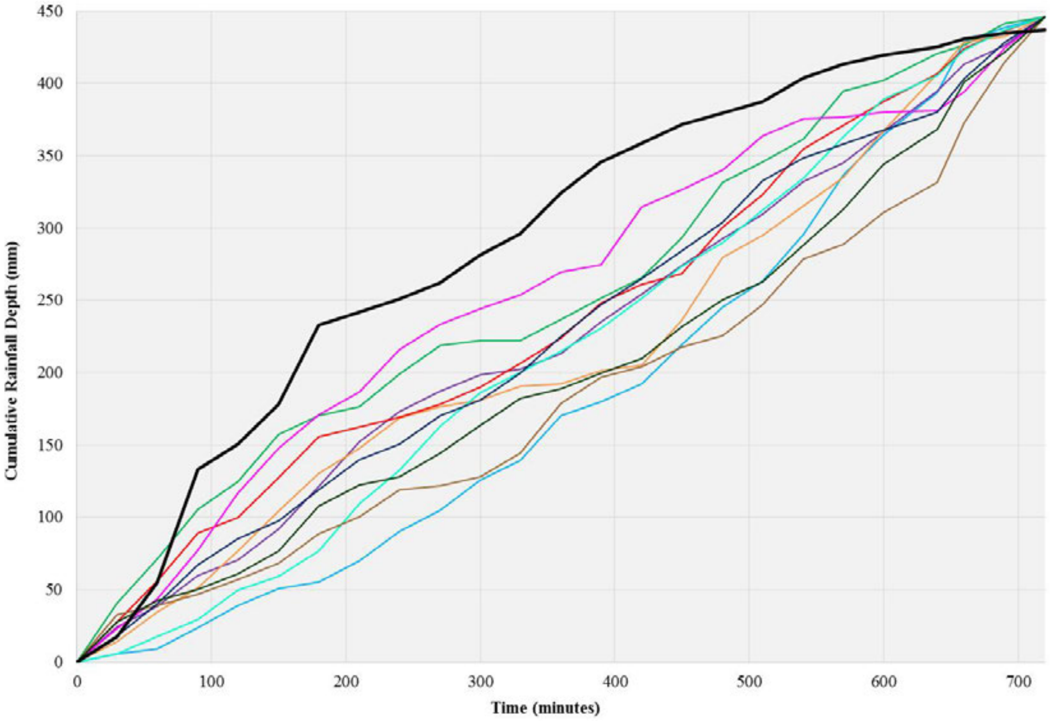


Figure C15 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Upper Cattle Creek

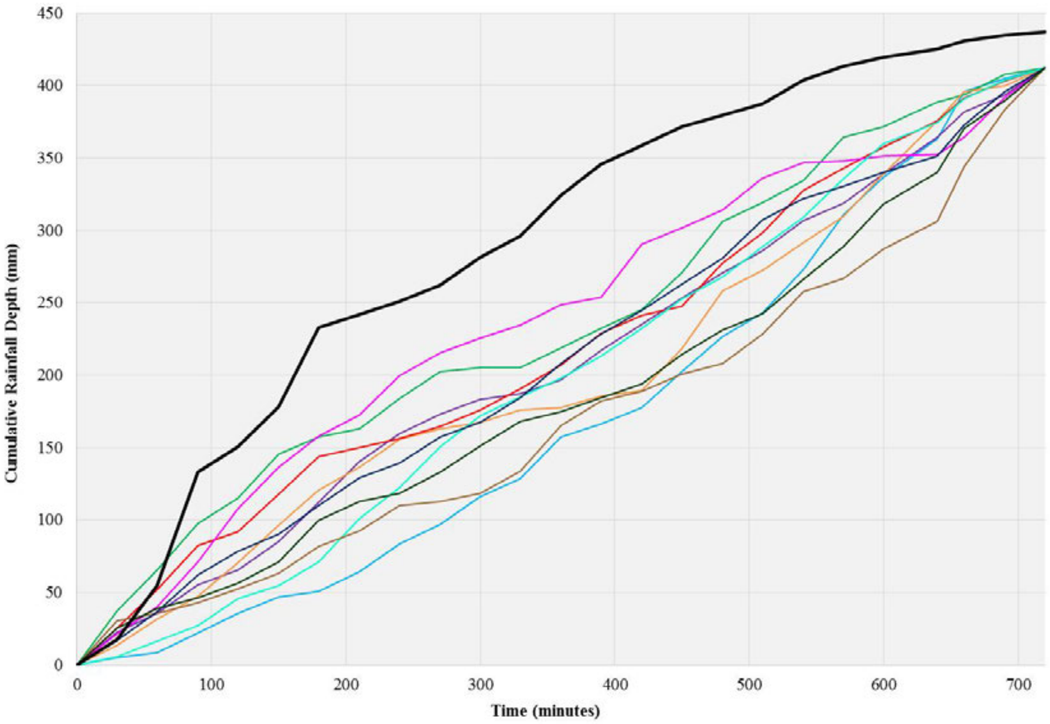


Figure C16 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Lower Cattle Creek

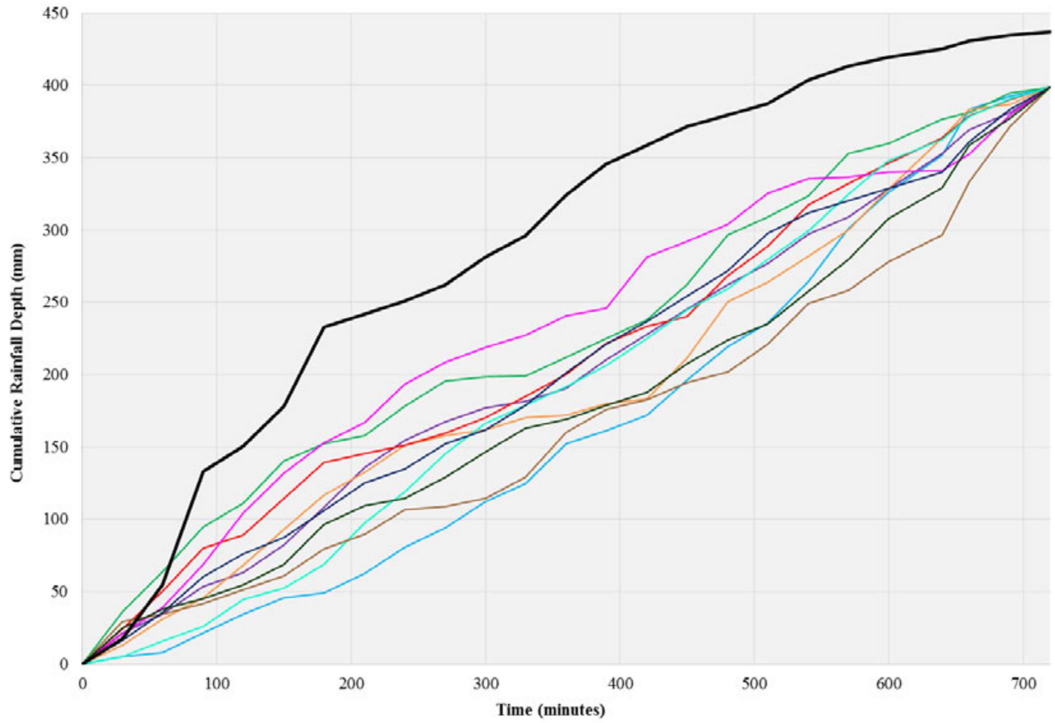


Figure C17 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Teemburra Creek

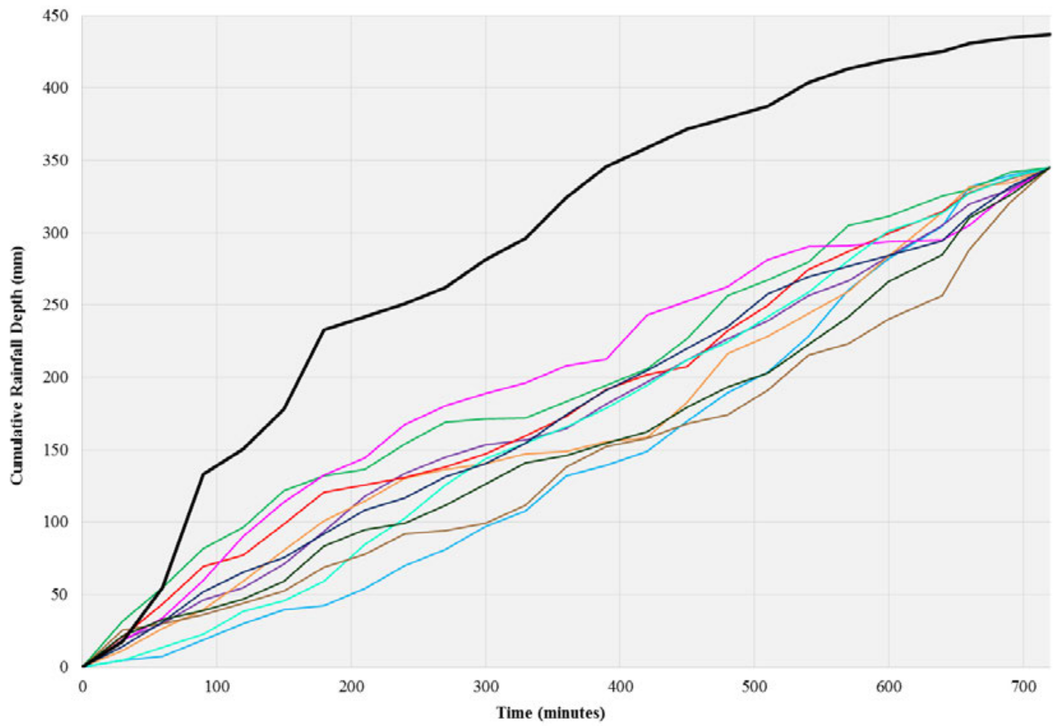


Figure C18 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Stockmans Creek

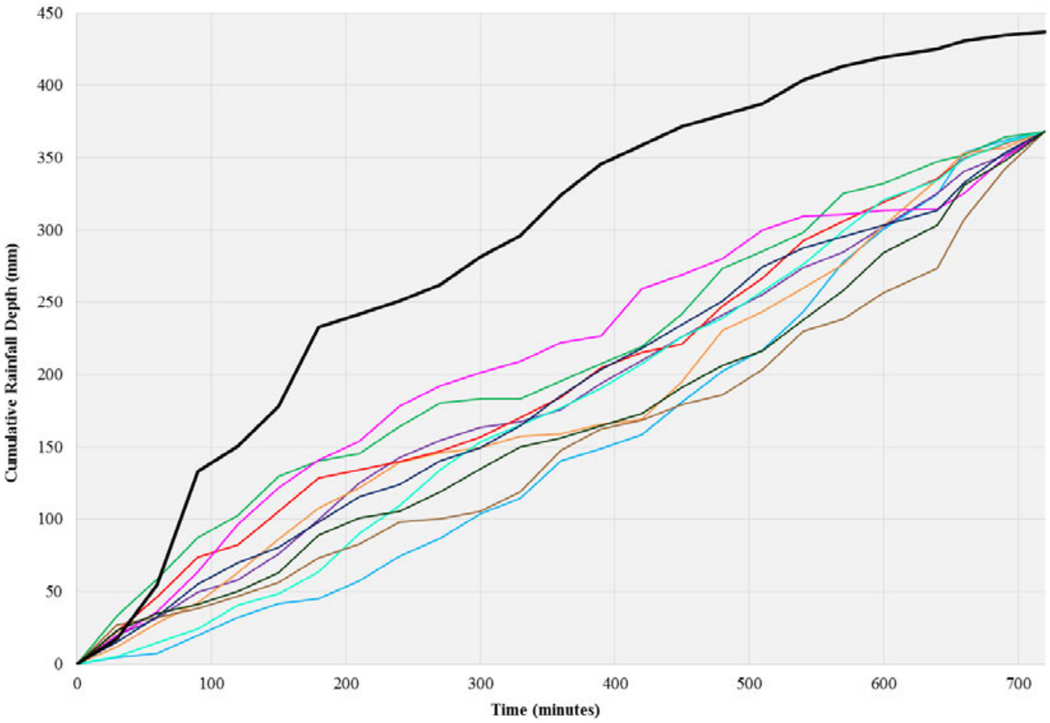


Figure C19 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Blacks Creek

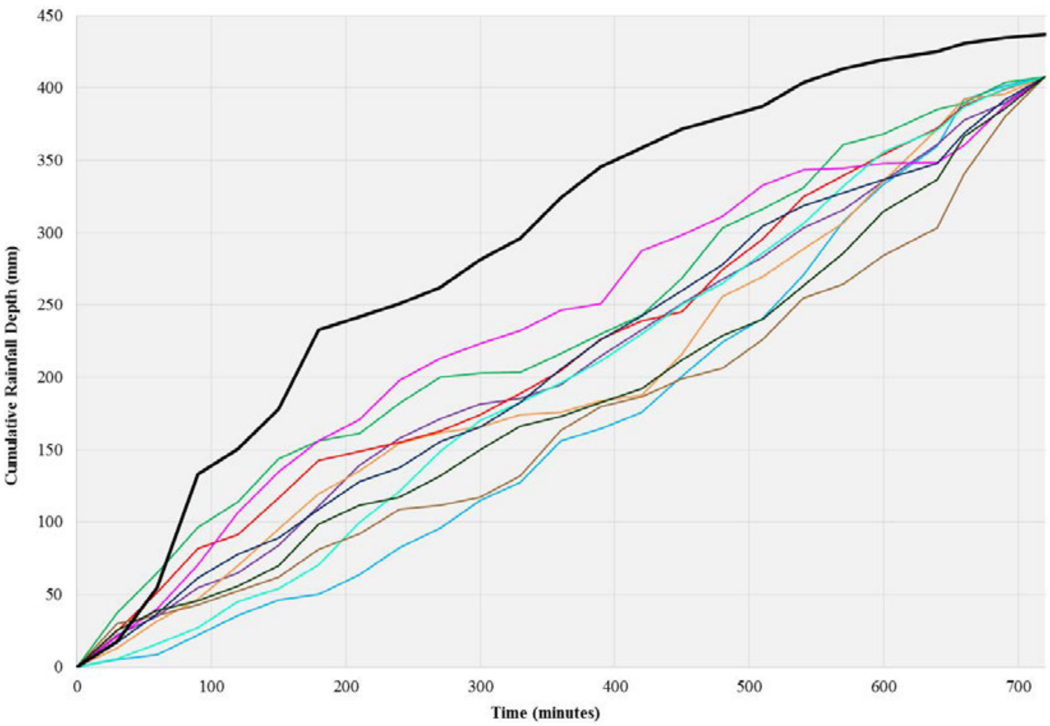


Figure C20 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Upper Pioneer River

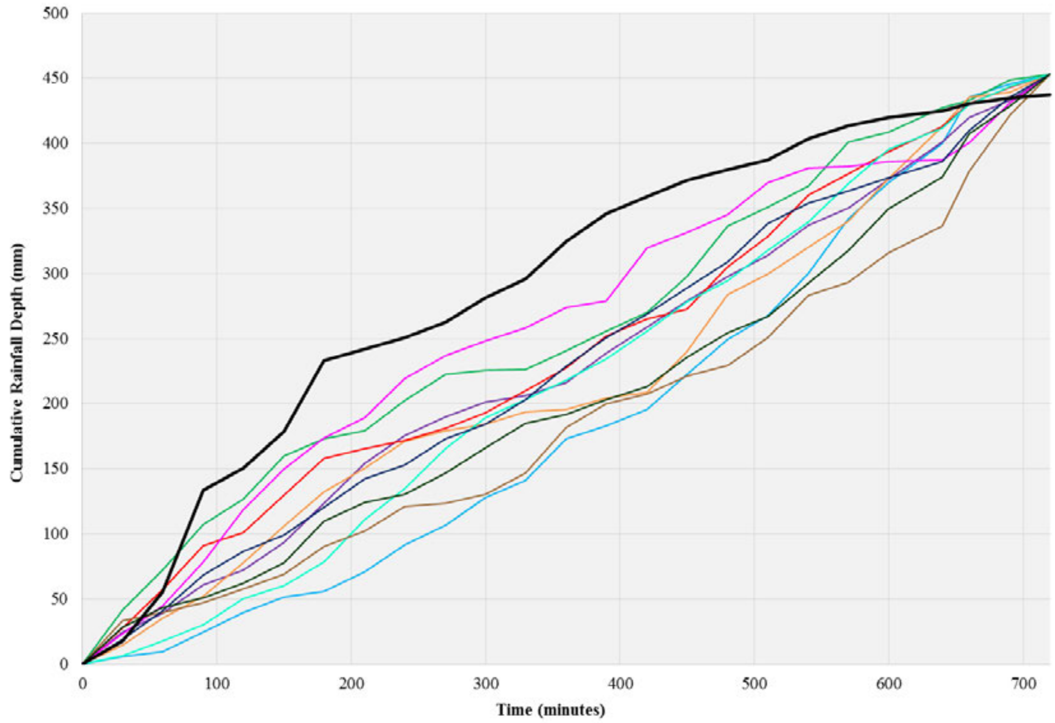


Figure C21 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Middle Pioneer River

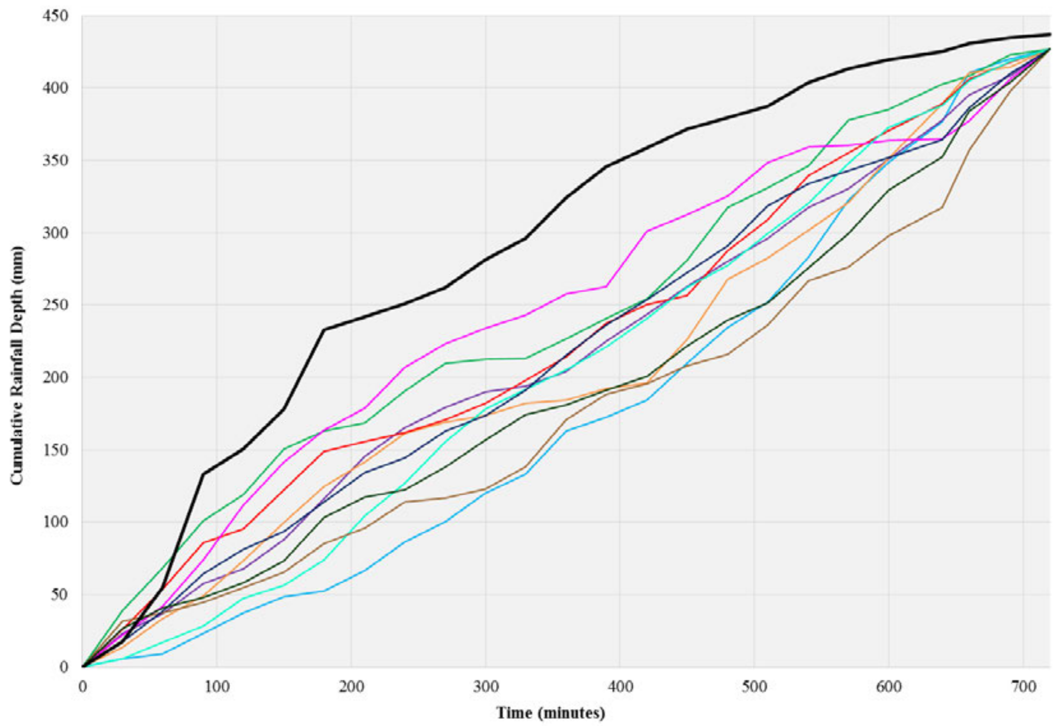


Figure C22 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Lower Pioneer River

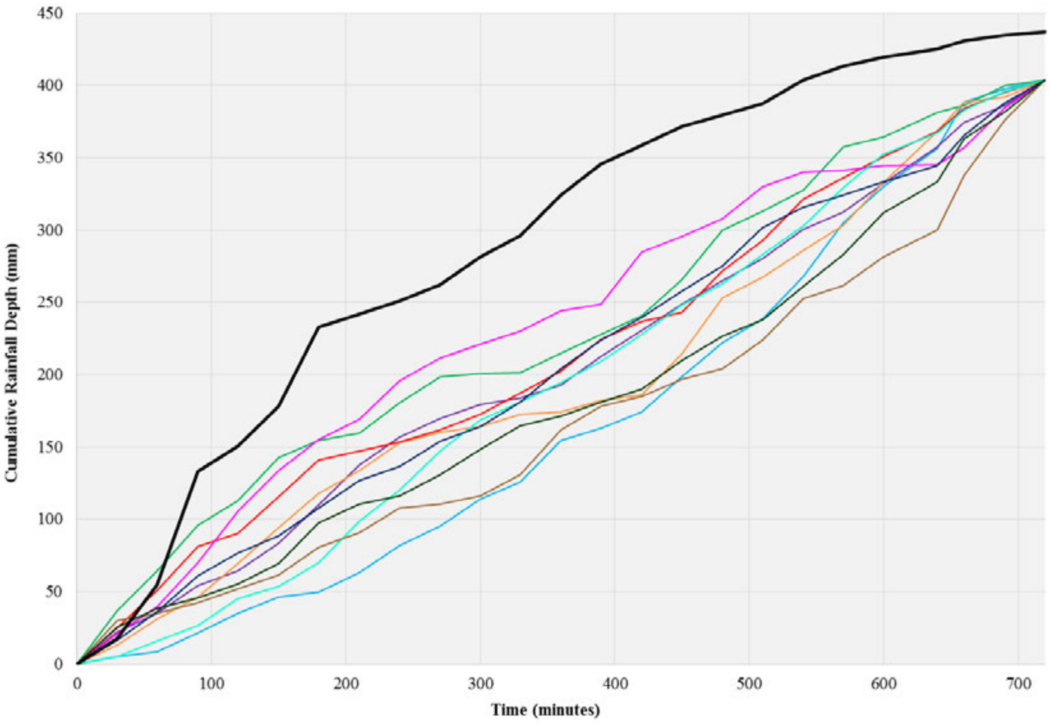


Figure C23 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Pioneer River Outlet

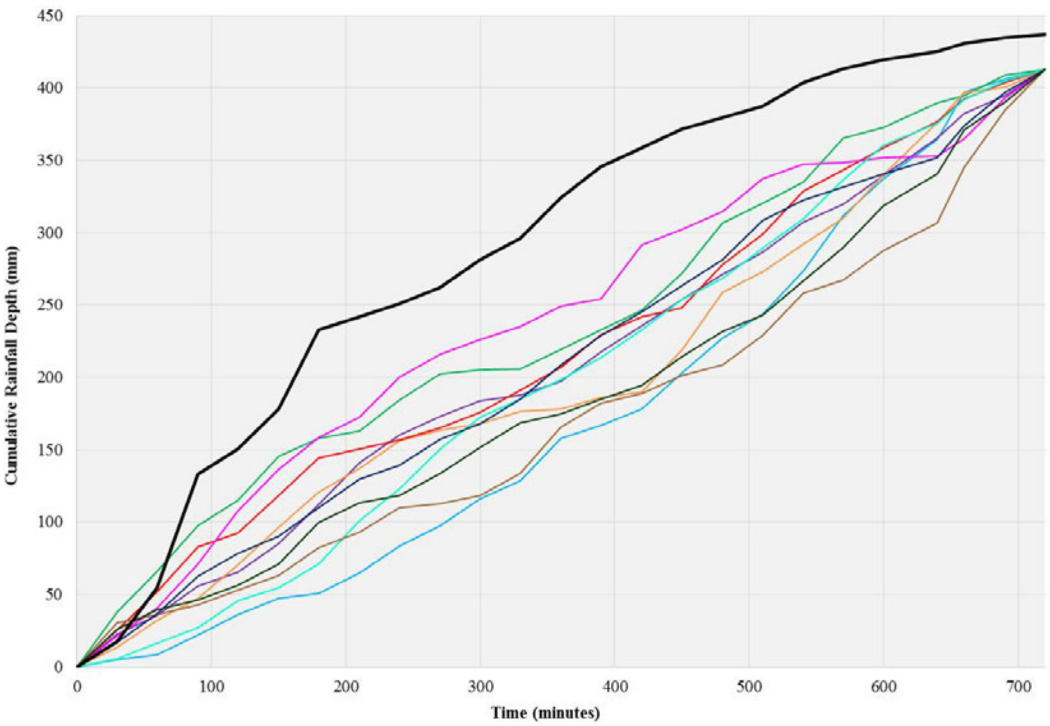


Figure C24 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Gooseponds Creek

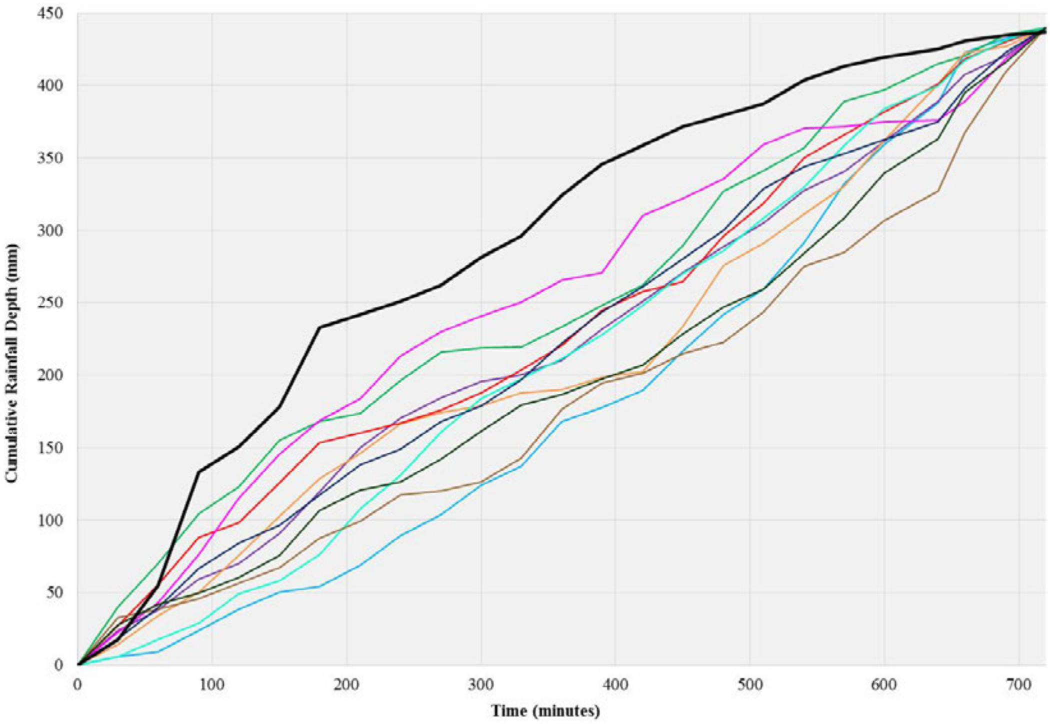


Figure C25 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Upper Bakers Creek

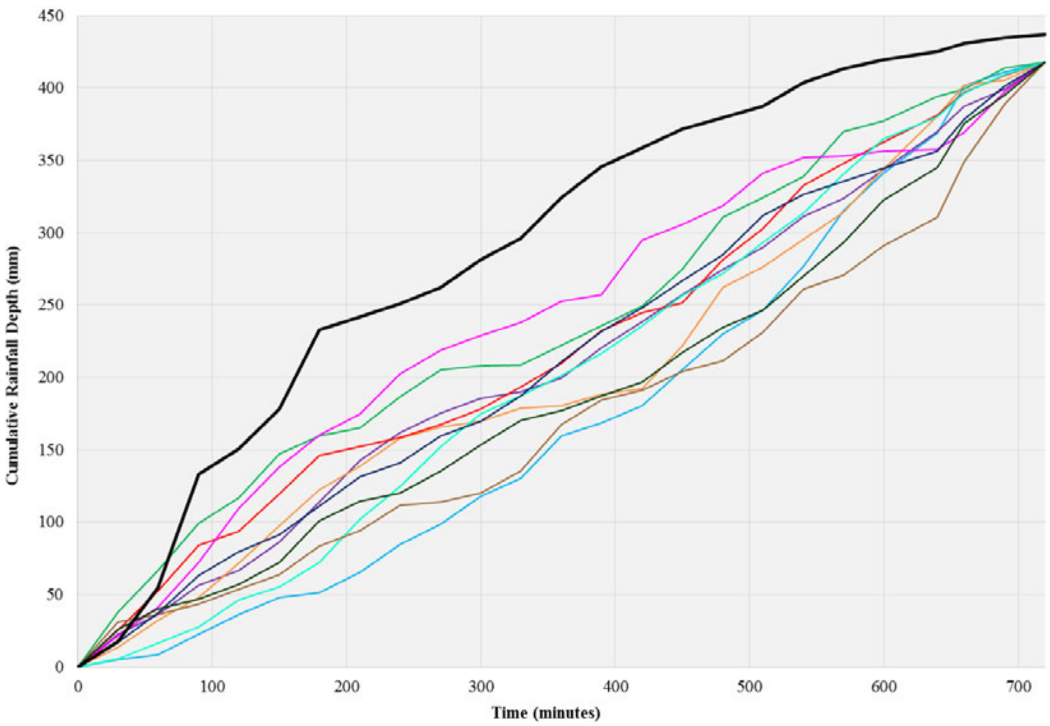


Figure C26 – Cumulative Rainfall Depth for Ensemble Temporal Patterns – Lower Bakers Creek

Selection of the Median Ensemble Temporal Pattern

The 10 Ensemble temporal patterns available from the ARR 2016 data hub were applied to the WBNM runoff routing model with an assessment being undertaken at key locations. The results of the assessment are shown in the figures below.

The Ensemble pattern that was found to produce the median peak discharge at that location (as per ARR guidelines) has been shown in a bold dashed line. However, as one pattern was required to be selected for the entire catchment an estimation of the most common median pattern (or found to be closest to the median pattern on most occasion) was undertaken. This found to return Ensemble pattern 4 as the pattern to be most representative of the median Ensemble temporal pattern for the entire Pioneer River catchment. This assessment was only undertaken for the 1% AEP event, with the resultant median pattern being used for all smaller events.

Pattern 4 has been shown as a solid bold line on all of the figures below.

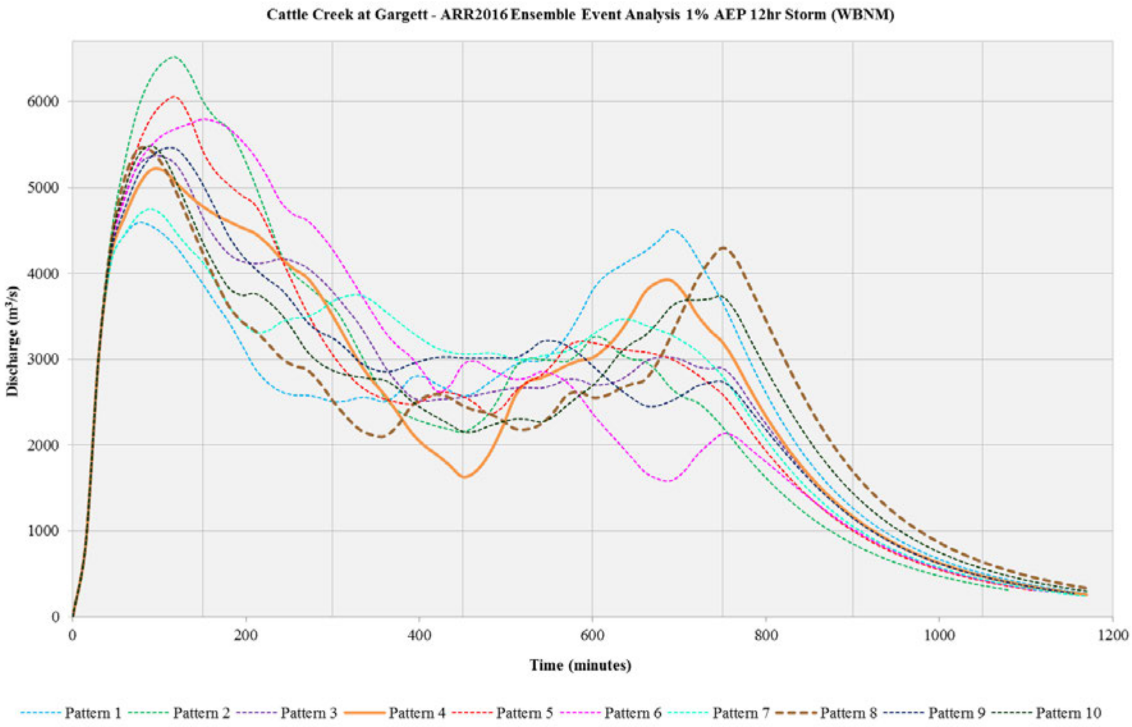


Figure C27 –Ensemble Temporal Patterns Analysis– Cattle Creek at Gargett

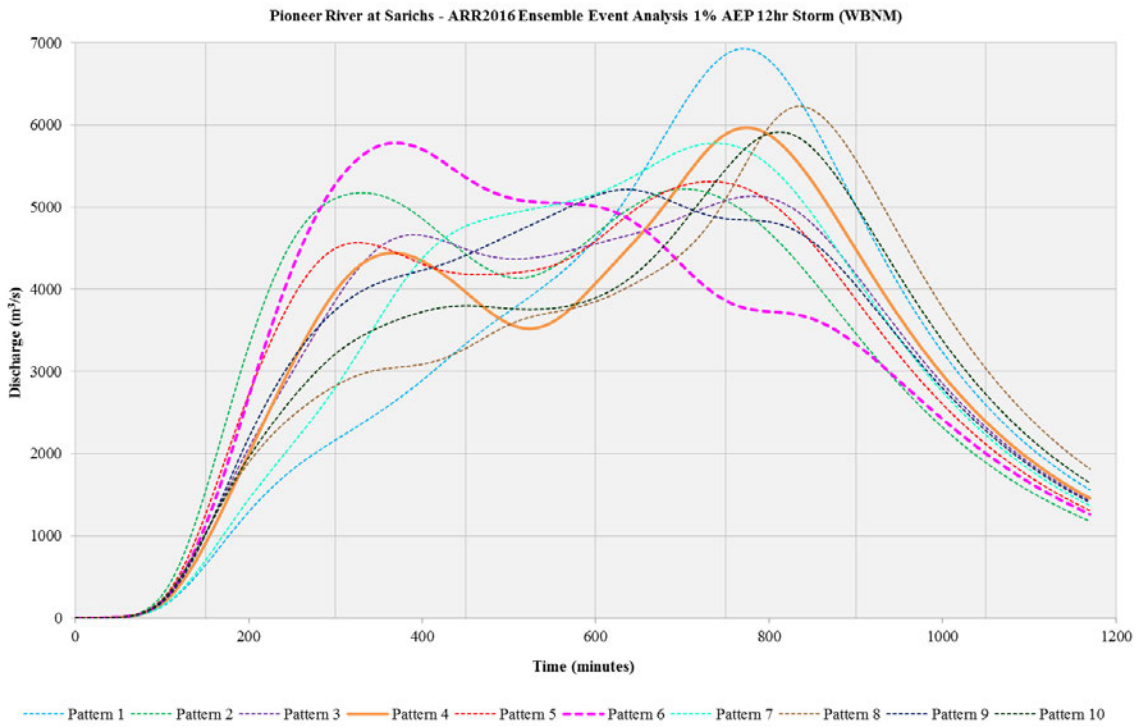


Figure C28 –Ensemble Temporal Patterns Analysis– Pioneer River at Sarichs

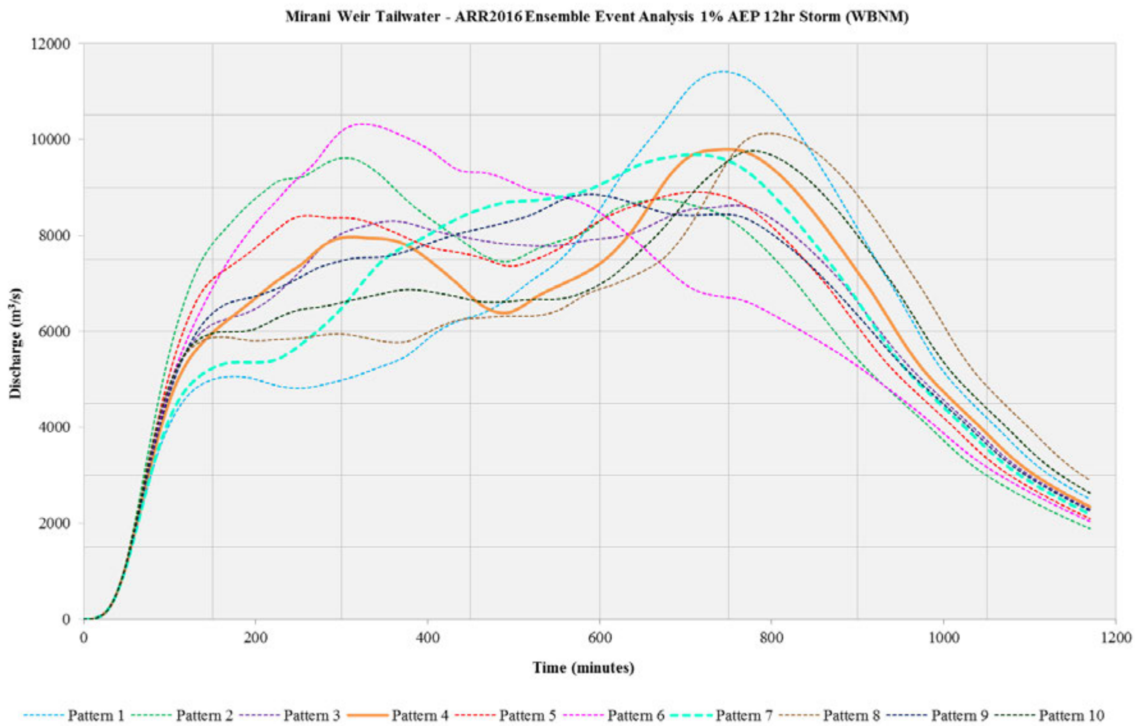


Figure C29 –Ensemble Temporal Patterns Analysis– Pioneer River at Mirani Weir Tailwater

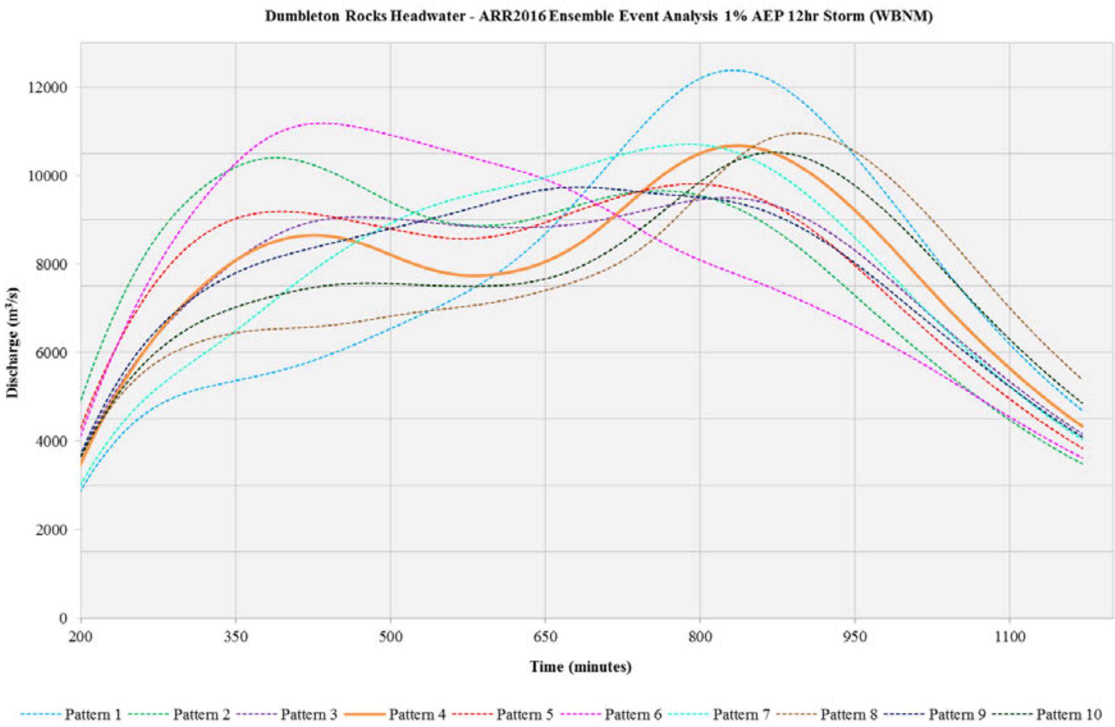


Figure C30 –Ensemble Temporal Patterns Analysis– Pioneer River at Dumbleton Rocks Headwater

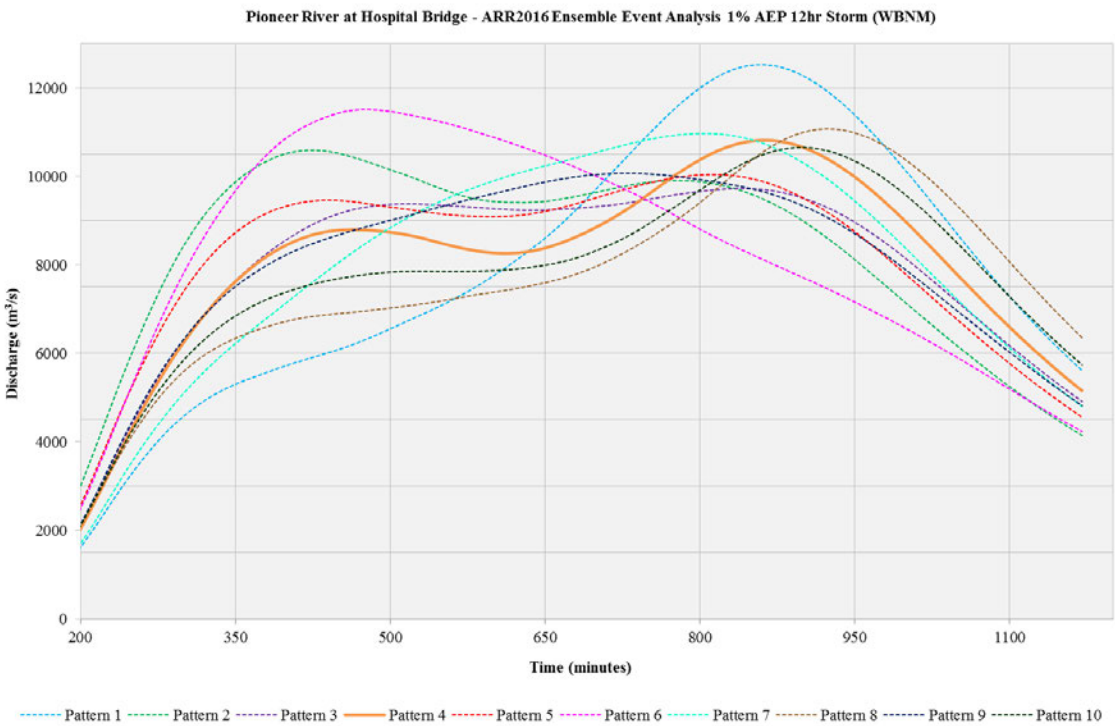


Figure C31 –Ensemble Temporal Patterns Analysis– Pioneer River at Hospital Bridge

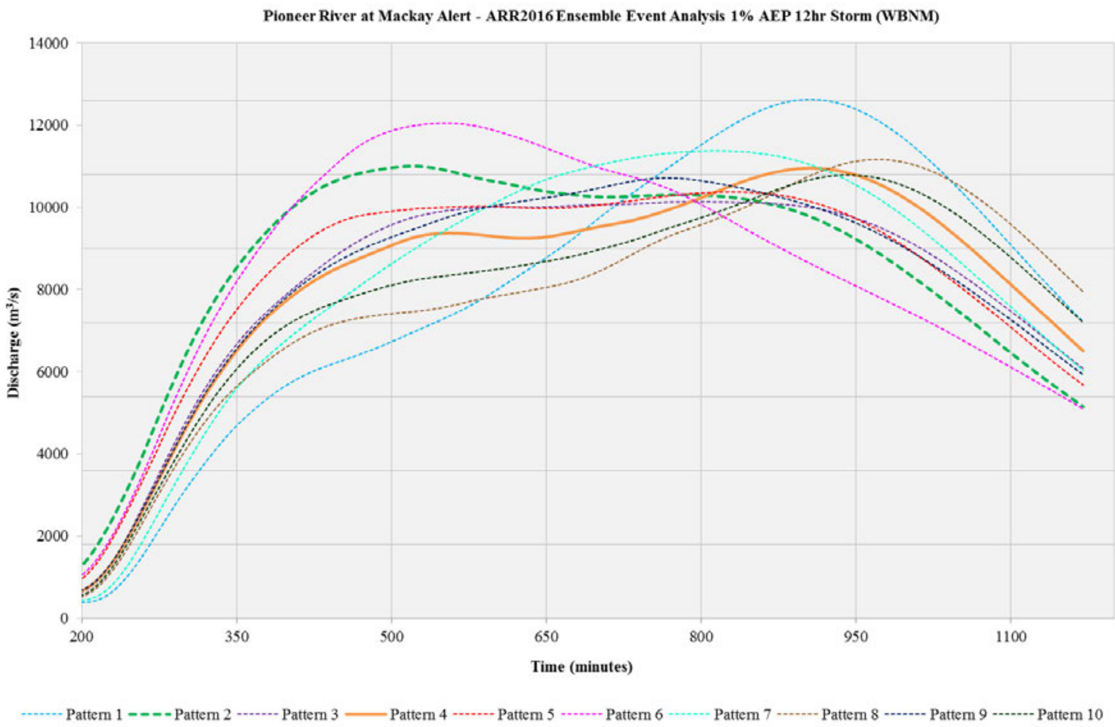


Figure C32 –Ensemble Temporal Patterns Analysis– Pioneer River at Mackay Alert

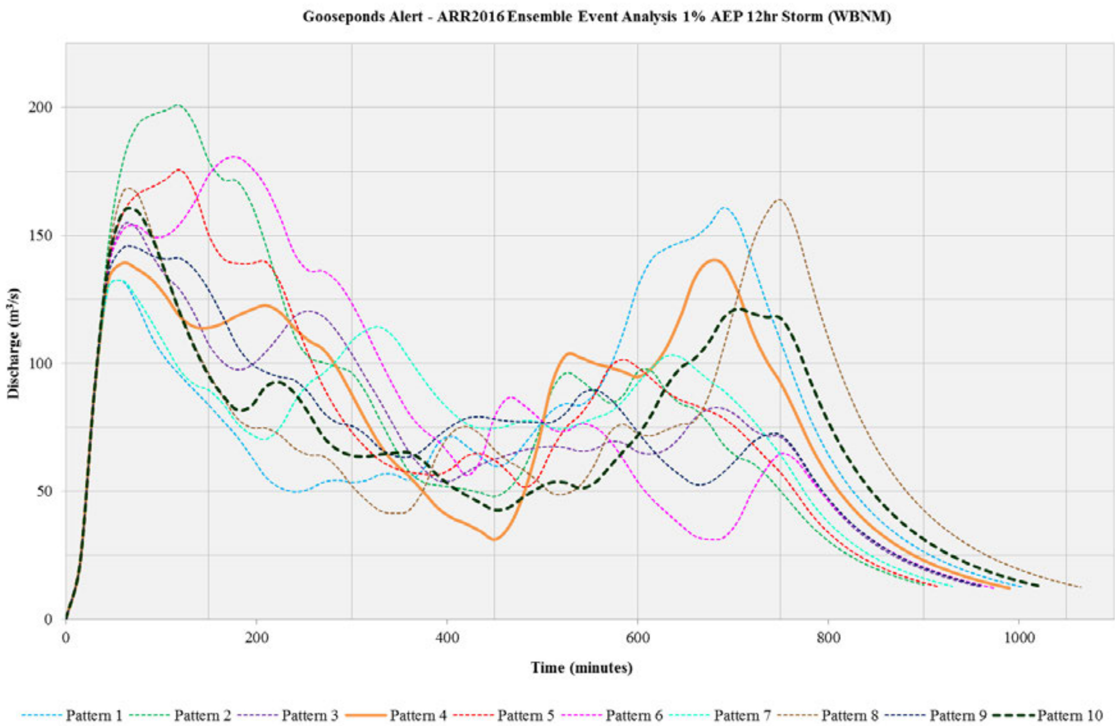


Figure C33 –Ensemble Temporal Patterns Analysis– Gooseponds Creek at Gooseponds Alert

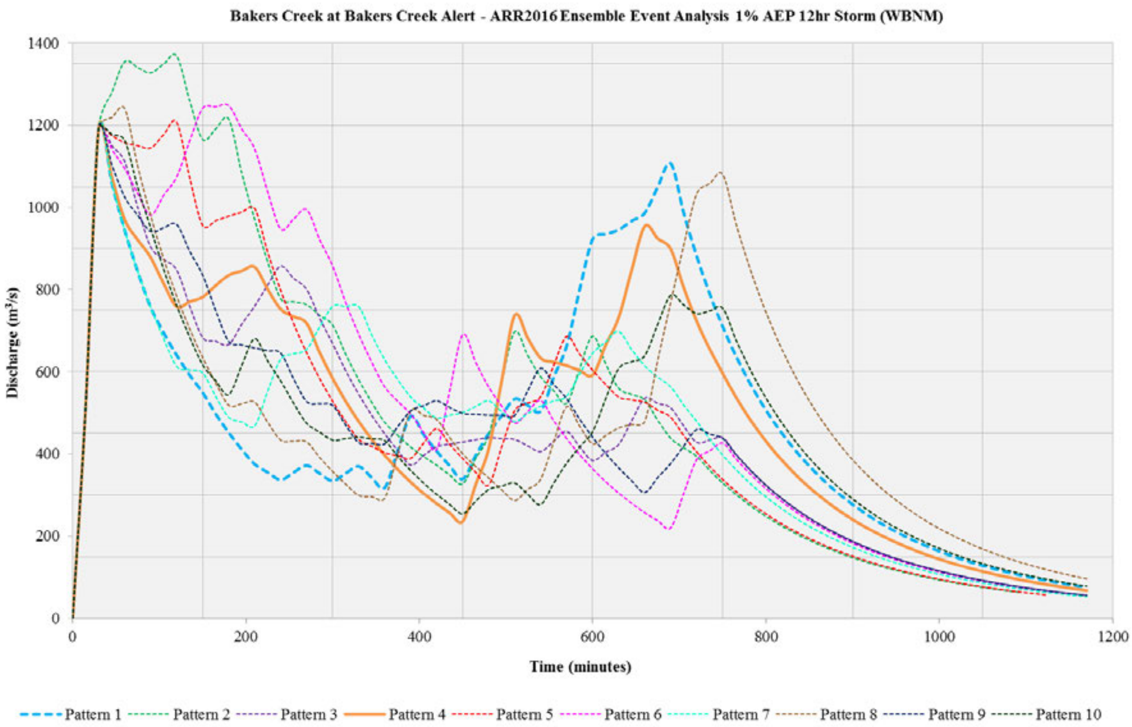


Figure C34 –Ensemble Temporal Patterns Analysis– Bakers Creek at Bakers Creek

APPENDIX D: WBNM MODEL SUBCATCHMENT CONFIGURATION

The WBNM Runoff Routing Model was developed using subcatchments delineated from the Bureau of Meteorology's GeoFabric database (refer methodology). These were then converted into GIS format to input into the WBNM model. The input parameters of the WBNM model are as per Table D1.

Table D1 – WBNM Subcatchment Parameters

SUBAREA NAME	D/S SUBAREA	AREA (ha)	CG EAST	CG NORTH	OUT EAST	OUT NORTH
BKRS_06	BKRS_07	2447.12	712561	7649907	715067.7	7650470
BKRS_07	BKRS_08	1234.83	717257.6	7650779	719995.5	7653050
BKRS_04	BKRS_05	1901.63	711590.7	7654562	713976.3	7654505
BKRS_05	BKRS_08	2006.77	716306.7	7654558	719582.1	7653794
BKRS_01	BKRS_02	2398.02	706697.6	7658726	709740.7	7659894
BKRS_02	BKRS_03	1536.74	712027.8	7658665	714238.6	7658657
BKRS_03	BKRS_08	1179.16	715825	7656380	719570	7653789
BKRS_08	BKRS_09	2185.65	721954.1	7652003	725022.6	7652851
BKRS_09	BKRS_10	2357.92	721815.6	7655502	725502.2	7653628
BKRS_10	SINK	1653.42	726019.9	7653789	727800.7	7652305
VINE_05	PION_10	360.27	729580	7662684	729124.6	7661722
VINE_02	VINE_03	529.18	721514.6	7665502	722426.4	7663137
VINE_01	VINE_03	1111.43	719888.7	7665500	721873	7663703
VINE_03	VINE_04	673.11	722321.2	7662407	724514.5	7662306
VINE_04	PION_10	1079.62	726590	7663449	728438.8	7661881
MACK_01	MACK_02	1160.81	726269.9	7657979	725369.9	7659664
MACK_02	PION_10	446.57	727076.9	7660000	725714.7	7660293
FURS_01	FURS_02	892.38	717194	7663562	717033.7	7661801
FURS_02	FURS_03	1177.35	718446	7661431	721066	7661552
FURS_03	PION_10	290.35	722817.8	7661106	724690.8	7661758
PION_08	PION_09	4195.07	711641.4	7664091	715366.9	7660938
PION_05	PION_06	1679	696600.6	7664183	698110.8	7661488
MCGR_04	MCGR_05	1784.07	689189	7665213	690041	7663245
MCGR_01	MCGR_02	2311.17	683938.7	7660932	687199.4	7661160
MCGR_02	MCGR_03	3810.43	683879.8	7665403	688400.6	7661446
MCGR_03	MCGR_05	1817.71	687132.8	7659754	690448.4	7661954

SUBAREA NAME	D/S SUBAREA	AREA (ha)	CG EAST	CG NORTH	OUT EAST	OUT NORTH
MCGR_05	PION_04	2503.99	692418.3	7661942	695099.8	7661748
CATT_13	CATT_14	2143.39	684610.5	7651552	685414.1	7655230
CATT_10	CATT_11	2383.36	675902	7658113	679626.3	7659794
CATT_09	CATT_11	4451.98	676880	7666728	680168.7	7660555
CATT_06	CATT_07	3990	669651.7	7668316	670399	7662228
CATT_03	CATT_04	5370	659698.5	7661666	663896.9	7662030
CATT_02	CATT_04	1231	661467.8	7666949	663298.9	7664921
CATT_01	CATT_04	3693	665947.1	7670256	663383.6	7664738
CATT_04	CATT_05	882	663652	7663440	664793.8	7661841
CATT_05	CATT_07	3775	666845	7661007	670035.2	7661845
CATT_07	CATT_08	699.23	670917	7661987	671549.9	7662215
CATT_08	CATT_11	5092.19	675008.7	7663751	678964.9	7661858
CATT_11	CATT_12	1633.1	678727.6	7657053	681002.1	7657314
CATT_12	CATT_14	3383.82	681140	7654601	683648	7655349
CATT_14	PION_03	1154.51	686339	7656120	689700.5	7657101
STOC_07	STOC_08	1190	699241.4	7639940	698508.1	7637672
STOC_06	STOC_08	3077	701595.4	7636573	698349.4	7637535
STOC_08	STOC_09	2838	695386.9	7639503	692004.7	7641956
STOC_04	STOC_05	1483	690068	7634288	692975.7	7635040
STOC_02	STOC_03	2357	688521.4	7630370	691435.8	7631743
STOC_01	STOC_03	3944	694175	7628040	691610.4	7631463
STOC_03	STOC_05	2669	694554.4	7633081	693113.3	7635035
STOC_05	STOC_09	1993	692117.3	7638004	691800.9	7641215
STOC_09	PION_01	632	691866	7641352	691300.9	7642914
BLAC_11	BLAC_12	2326.57	682534.6	7636474	684791.2	7640383
BLAC_09	BLAC_10	2228	678928.1	7639900	682185.1	7642141
BLAC_07	BLAC_08	1810	681621.1	7647314	682275.1	7643343
BLAC_03	BLAC_04	6261	669447.4	7638706	673602.9	7643109
BLAC_02	BLAC_04	2191	668173	7642049	672675.8	7643534
BLAC_01	BLAC_04	4934	664962	7641128	672794.4	7643688
BLAC_04	BLAC_05	273	672915.4	7644207	673812.5	7643307
BLAC_05	BLAC_06	2663	675571.2	7643276	679198.9	7646217
TEEMB_07	TEEMB_08	760.09	674735.5	7653576	674285.4	7651738
TEEMB_06	TEEMB_08	6521	666004.7	7647190	673248.2	7650638
TEEMB_03	TEEMB_04	2601	667165.1	7653705	671101.1	7654109

SUBAREA NAME	D/S SUBAREA	AREA (ha)	CG EAST	CG NORTH	OUT EAST	OUT NORTH
TEEMB_01	TEEMB_02	491.78	671743.9	7657897	672850.2	7656241
TEEMB_02	TEEMB_04	3075	667868.9	7656025	672467	7654440
TEEMB_04	TEEMB_05	316.07	672516.7	7653563	672867.2	7652740
TEEMB_05	TEEMB_08	2372	667953.7	7651018	673098.7	7650861
TEEMB_08	TEEMB_09	117	673736.2	7651030	674306.6	7651548
TEEMB_09	BLAC_06	4325	676323	7649769	679513.5	7646240
BLAC_06	BLAC_08	1439	679851.4	7644236	682156.9	7643440
BLAC_08	BLAC_10	155	682683.9	7642823	682301.5	7642169
BLAC_10	BLAC_12	2009.43	682571.2	7640902	684807.1	7640700
BLAC_12	BLAC_13	3521	687222.7	7638424	690081.1	7640437
BLAC_13	PION_01	728	688940.6	7641711	690899.8	7642833
PION_01	PION_02	3817.85	689556.3	7645471	688459.2	7647444
PION_02	PION_03	4206.28	687783.8	7651400	689639.9	7656939
PION_03	PION_04	829.78	692060.7	7659142	695200.9	7661437
PION_04	PION_06	775.82	695853.9	7660716	698126.6	7661335
PION_06	PION_07	3266.64	702110.5	7662942	705598.5	7661928
PION_07	PION_09	1568.52	709002.5	7662943	714800.6	7660912
PION_09	PION_10	2529.3	719083.7	7659372	724796.6	7661509
PION_10	SINK	634.39	726814.2	7661142	729073.8	7660858

APPENDIX E: CRITICAL DURATION ASSESSMENT RESULTS

ARR 1987 Hydrology Critical Duration Assessment

Figure E1 through Figure E8 show the results from a critical duration assessment undertaken at a variety of locations using the WBNM runoff routing model with ARR 1987 hydrologic inputs.

A variety of storm durations were assessed, including the 12, 24, 36 and 48 hour storms. The critical duration assessment was only undertaken for the 1% AEP event, this duration was then taken as critical for all smaller events.

The 24 hour (1440 minute) storm was found to produce the highest peak discharge at all assessment locations and therefore was taken forward as the critical duration storm for the Pioneer River catchment (when applying ARR 1987 inputs).

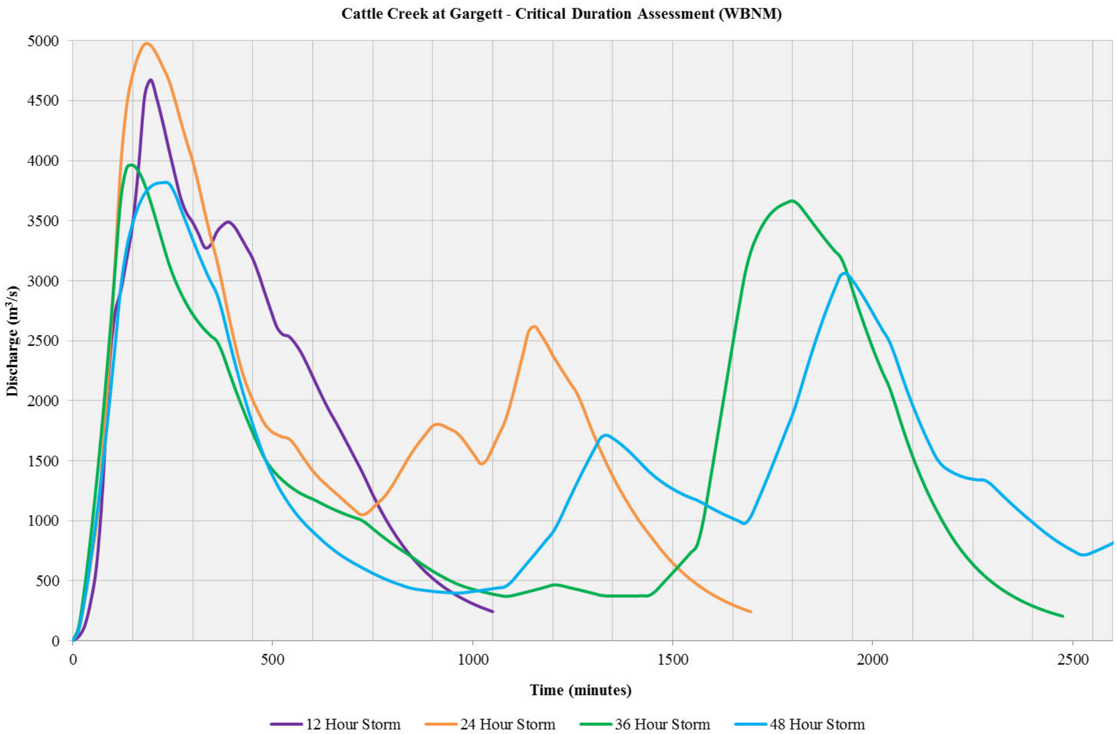


Figure E1 – ARR 1987 Critical Duration Assessment: Cattle Creek at Gargett

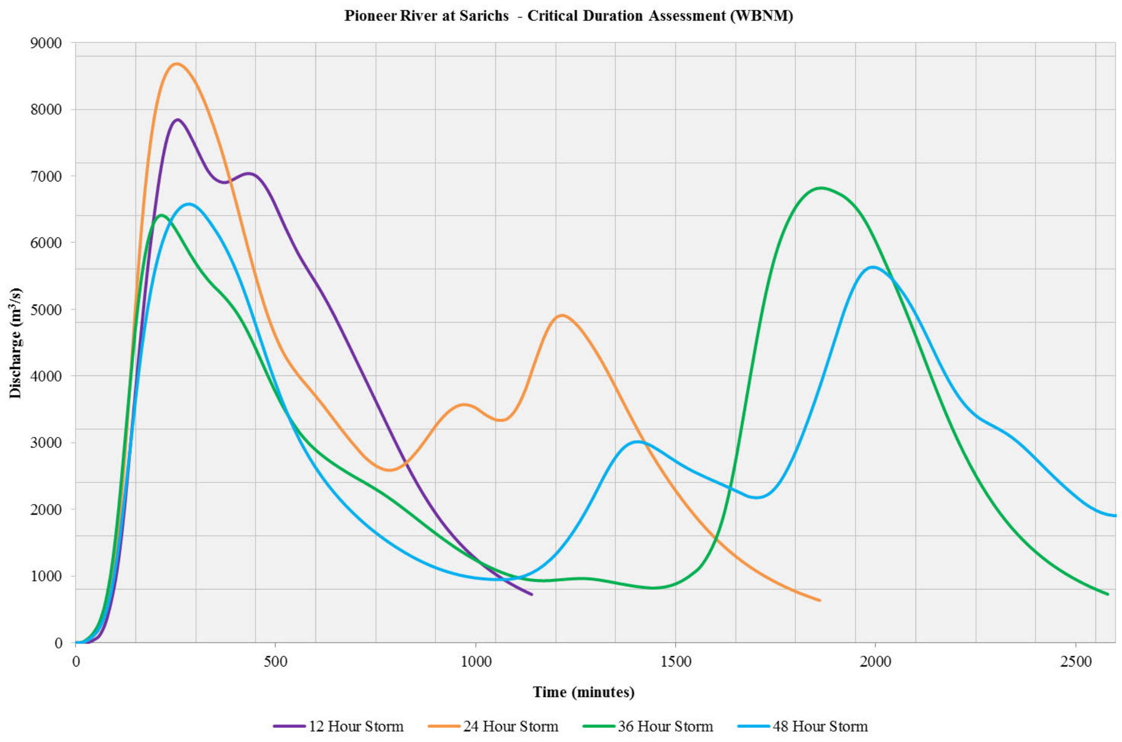


Figure E2 – ARR 1987 Critical Duration Assessment: Pioneer River at Sarichs

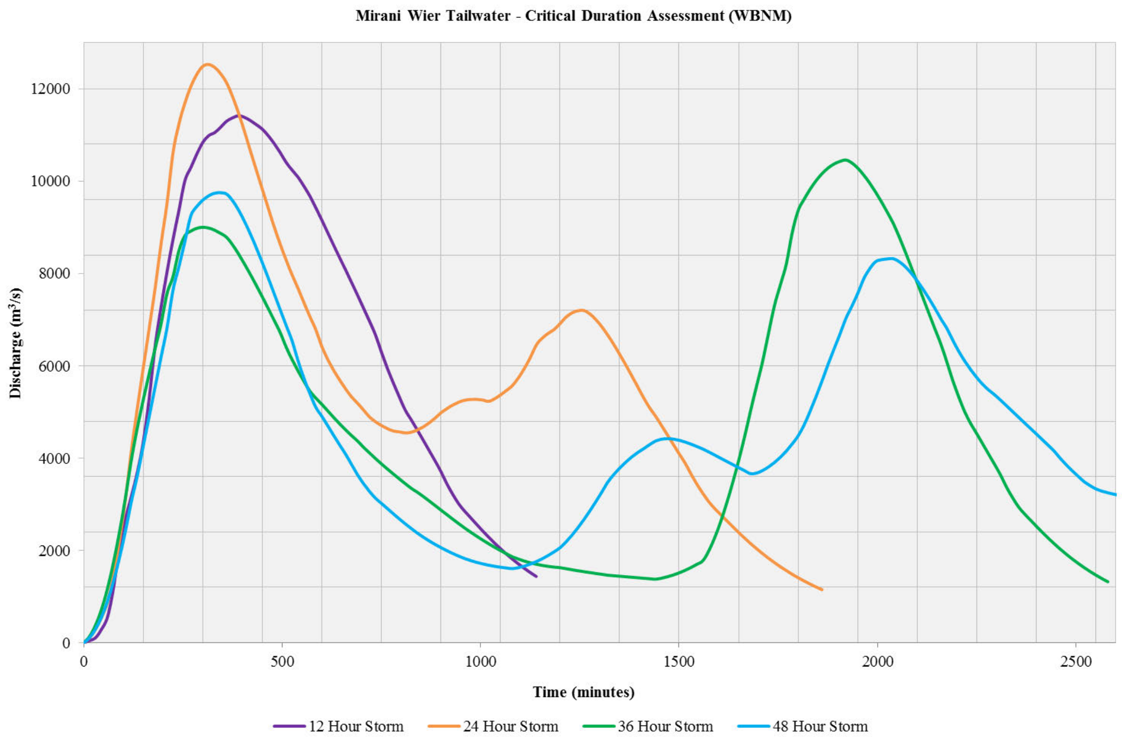


Figure E3 – ARR 1987 Critical Duration Assessment: Mirani Weir Tailwater

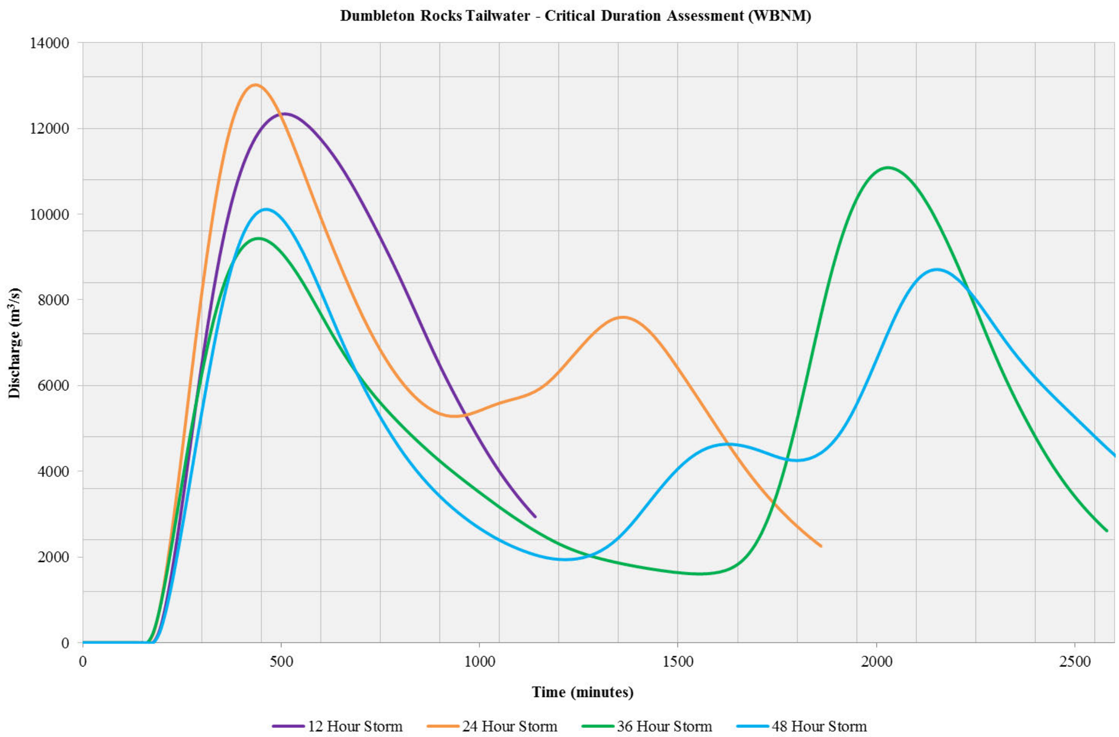


Figure E4 – ARR 1987 Critical Duration Assessment: Pioneer River at Dumbleton Rocks Tailwater

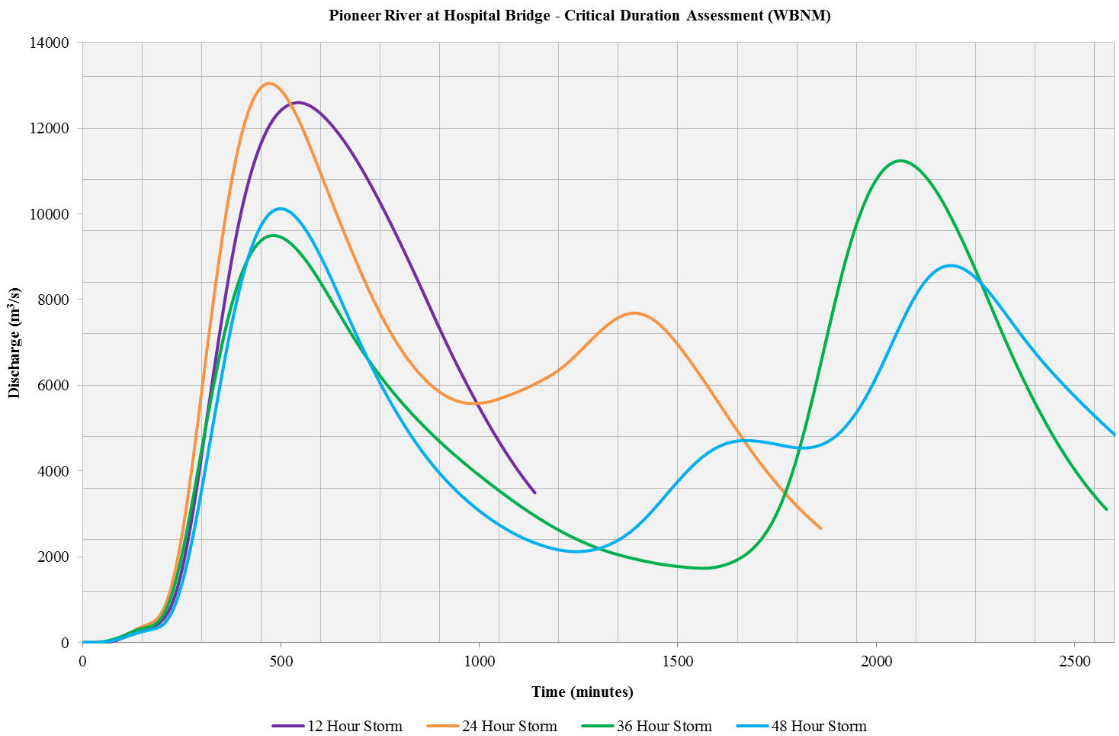


Figure E5 – ARR 1987 Critical Duration Assessment: Pioneer River at Hospital Bridge

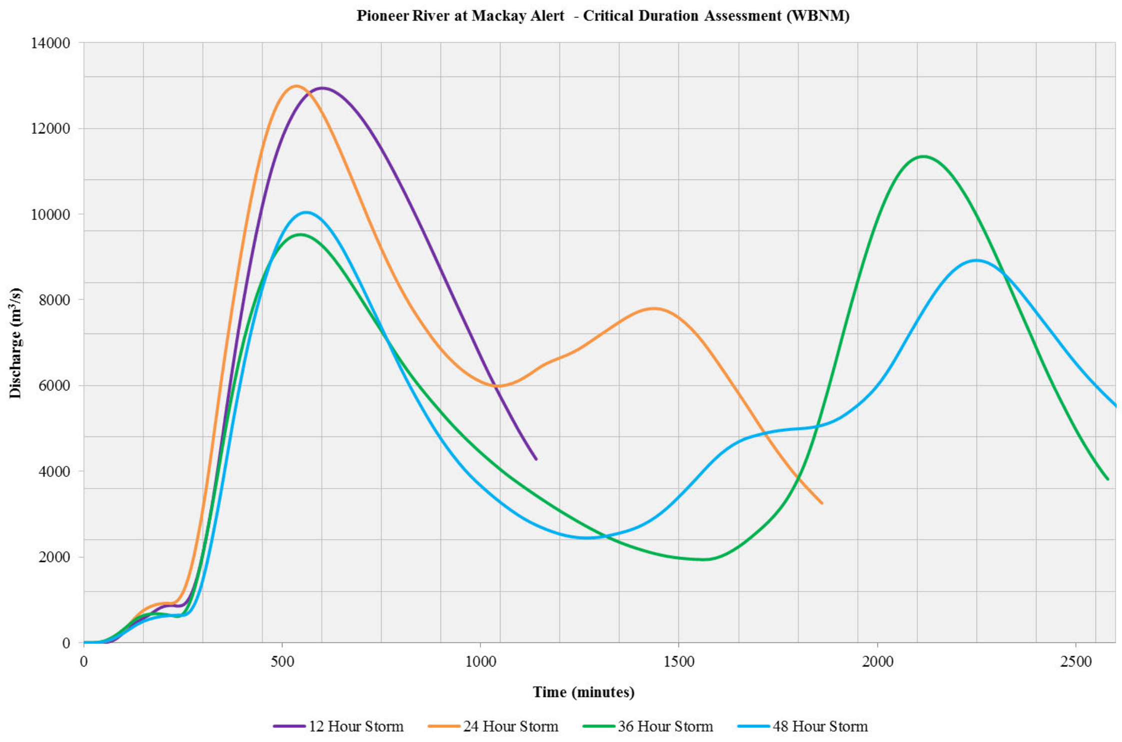


Figure E6 – ARR 1987 Critical Duration Assessment: Pioneer River at Mackay Alert

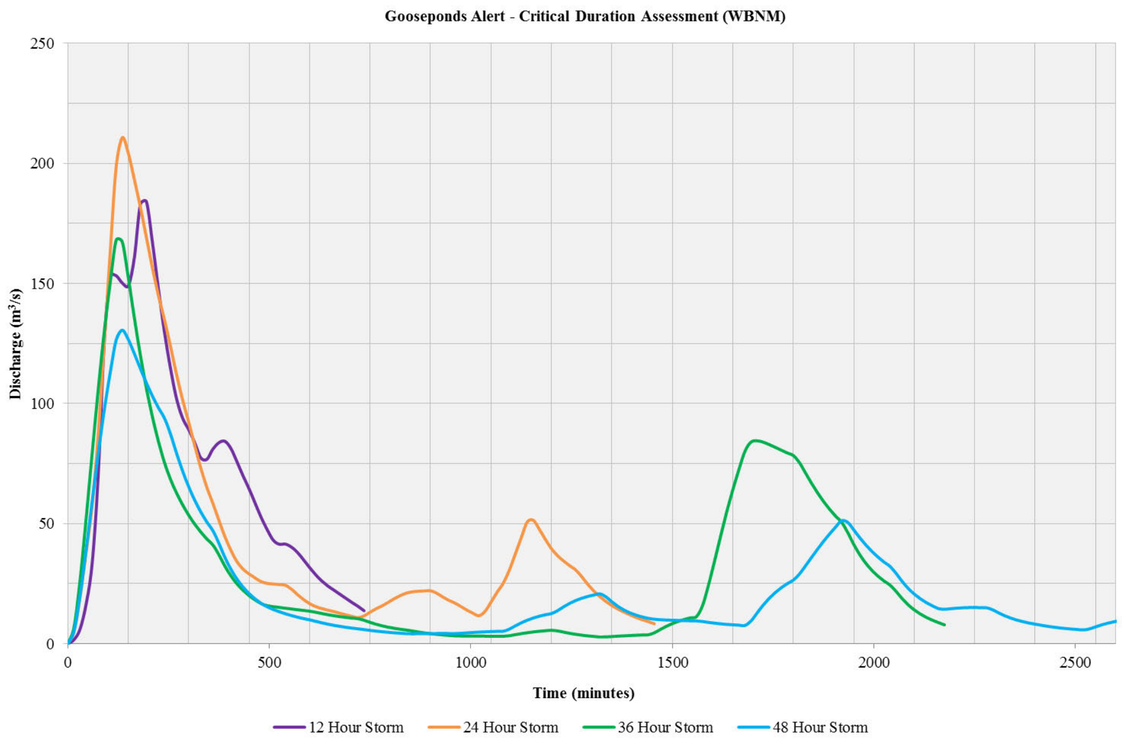


Figure E7 – ARR 1987 Critical Duration Assessment: Gooseponds Creek at Gooseponds Alert

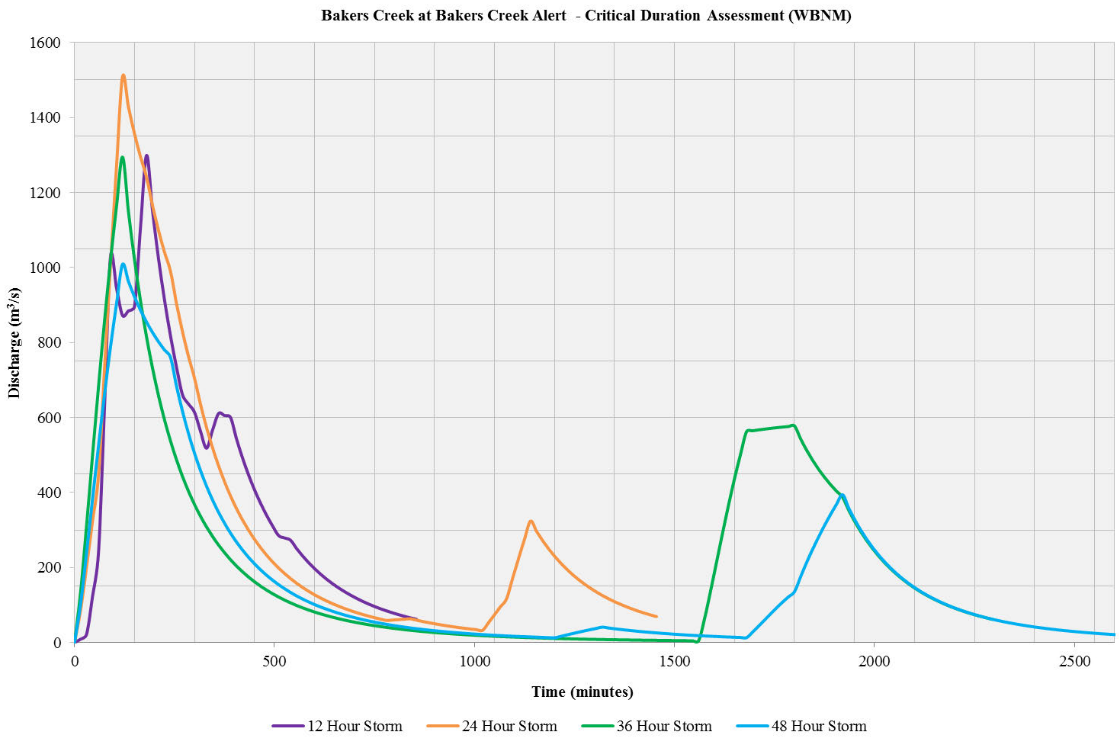


Figure E8 – ARR 1987 Critical Duration Assessment: Bakers Creek at Bakers Creek Alert

ARR 2016 Hydrology Critical Duration Assessment

Figure E9 through Figure E16 show the results from a critical duration assessment undertaken at a variety of locations using the WBNM runoff routing model with ARR 2016 (Ensemble event) hydrologic inputs. All events were assessed using the Ensemble temporal pattern that was found to represent the median peak discharge for the majority of the Pioneer River catchment (i.e. the mode of the results at all assessment locations)

A variety of storm durations were assessed, including the 3, 6, 12, 18 and 24 hour storms. The critical duration assessment was only undertaken for the 1% AEP event, this duration was then taken as critical for all smaller events.

The 12 hour (720 minute) storm was found to produce the highest peak discharge at the majority of assessment locations and therefore was taken forward as the critical duration storm for the Pioneer River catchment (when applying ARR 2016 inputs).

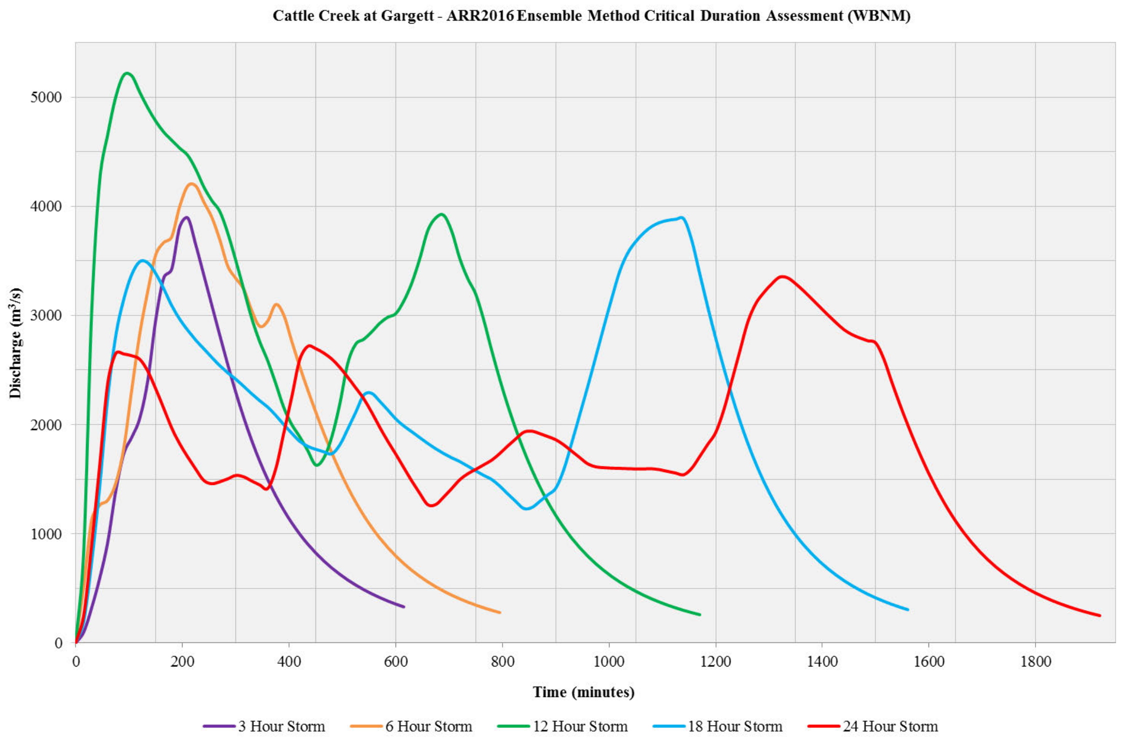


Figure E9 – ARR 2016 Critical Duration Assessment: Cattle Creek at Gargett

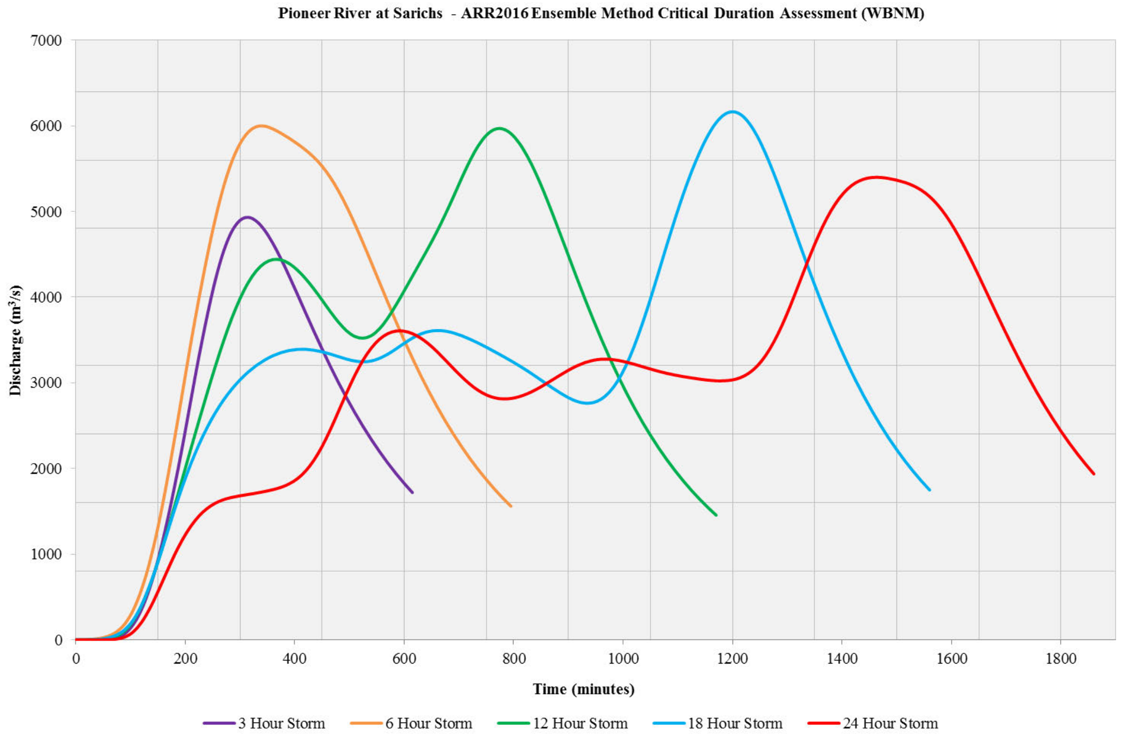


Figure E10 – ARR 2016 Critical Duration Assessment: Pioneer River at Sarichs

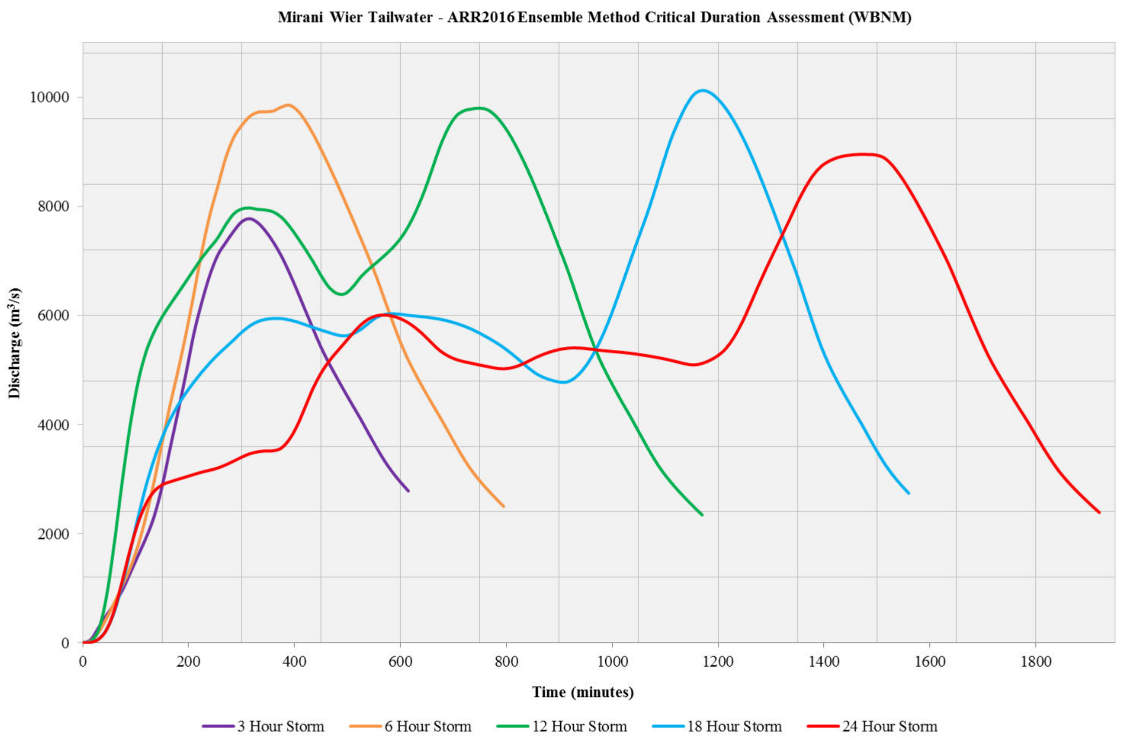


Figure E11 – ARR 2016 Critical Duration Assessment: Pioneer River at Mirani Weir Tailwater

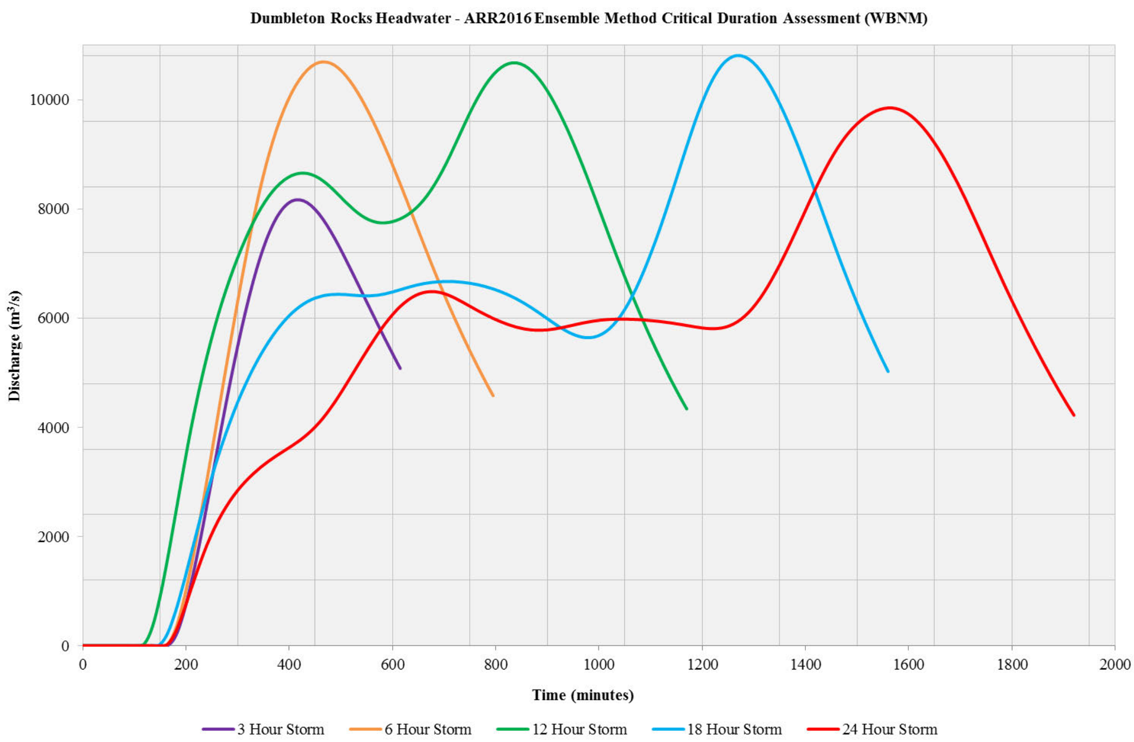


Figure E12 – ARR 2016 Critical Duration Assessment: Pioneer River at Dumbleton Rocks Headwater

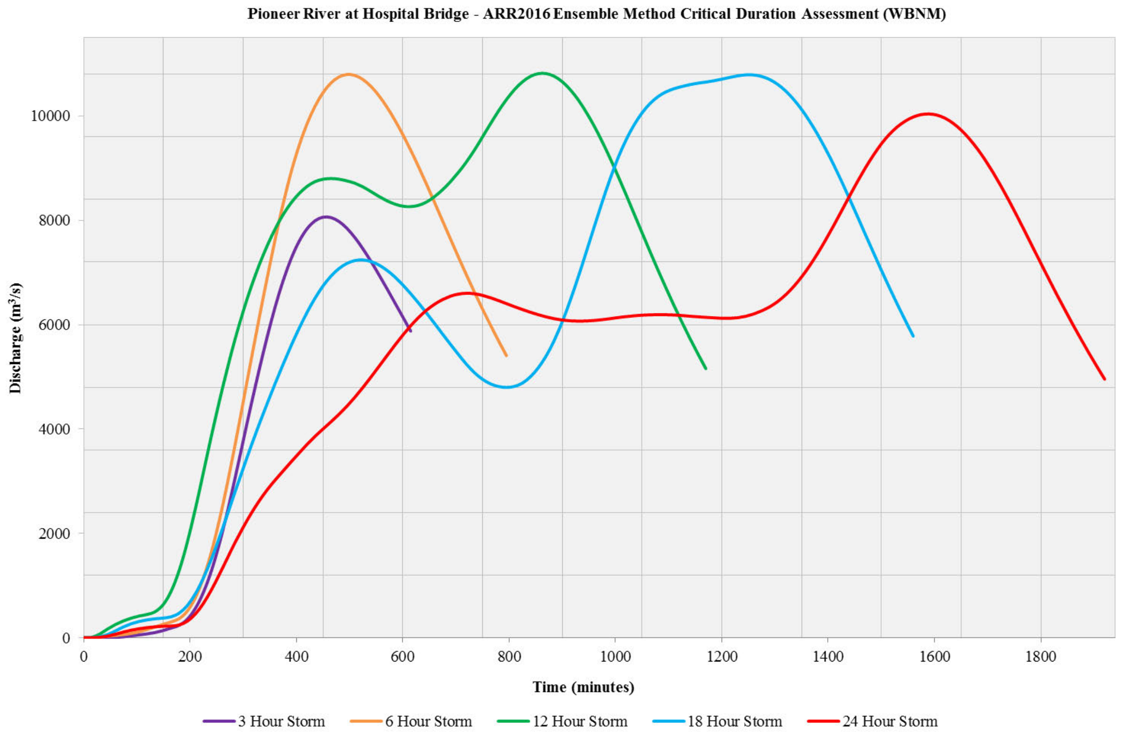


Figure E13 – ARR 2016 Critical Duration Assessment: Pioneer River at Hospital Bridge

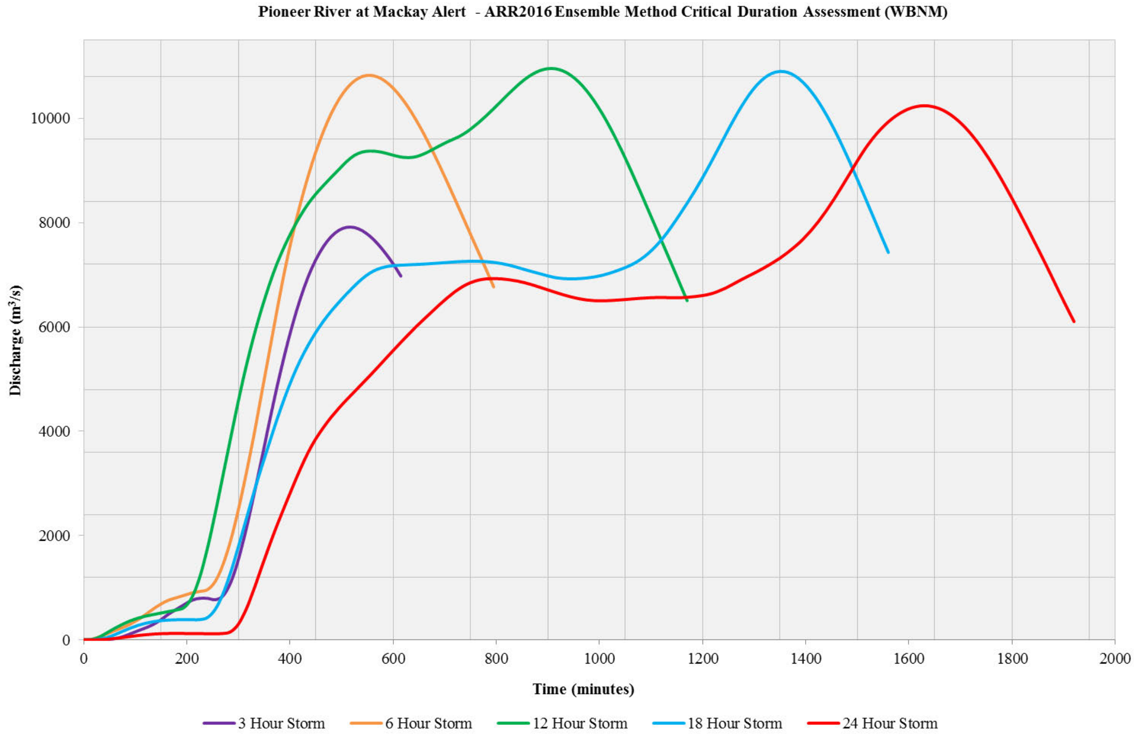


Figure E14 – ARR 2016 Critical Duration Assessment: Pioneer River at Mackay Alert

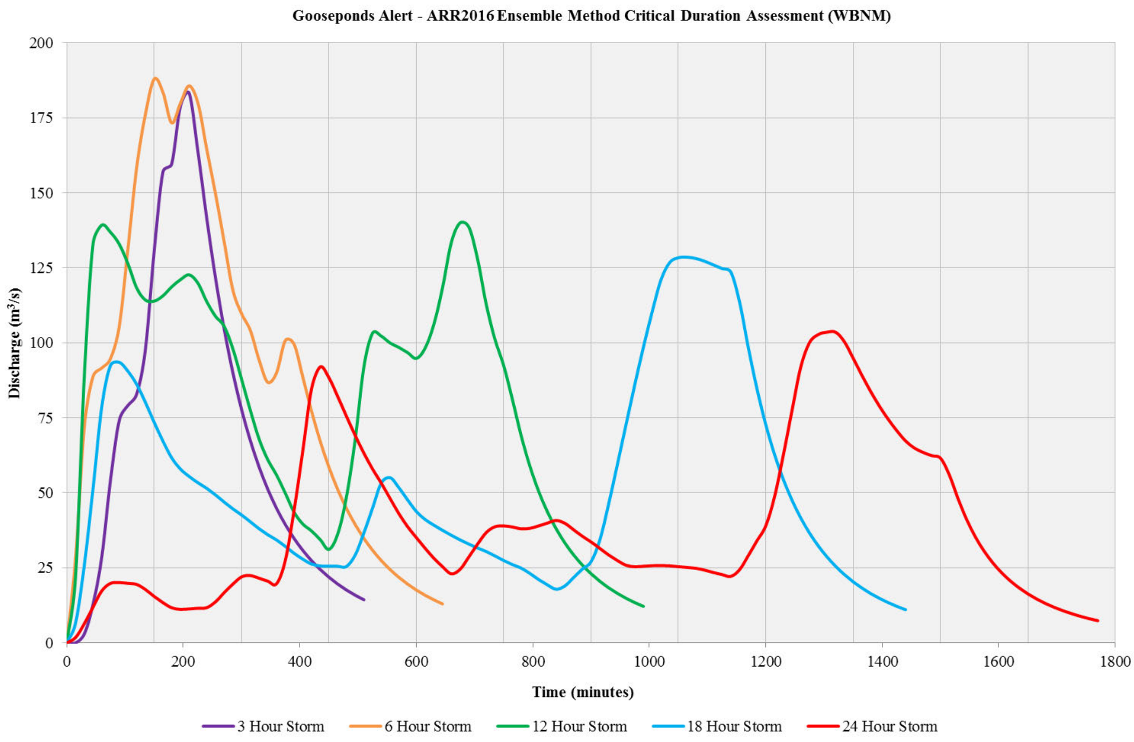


Figure E15 – ARR 2016 Critical Duration Assessment: Gooseponds Creek at Gooseponds Alert

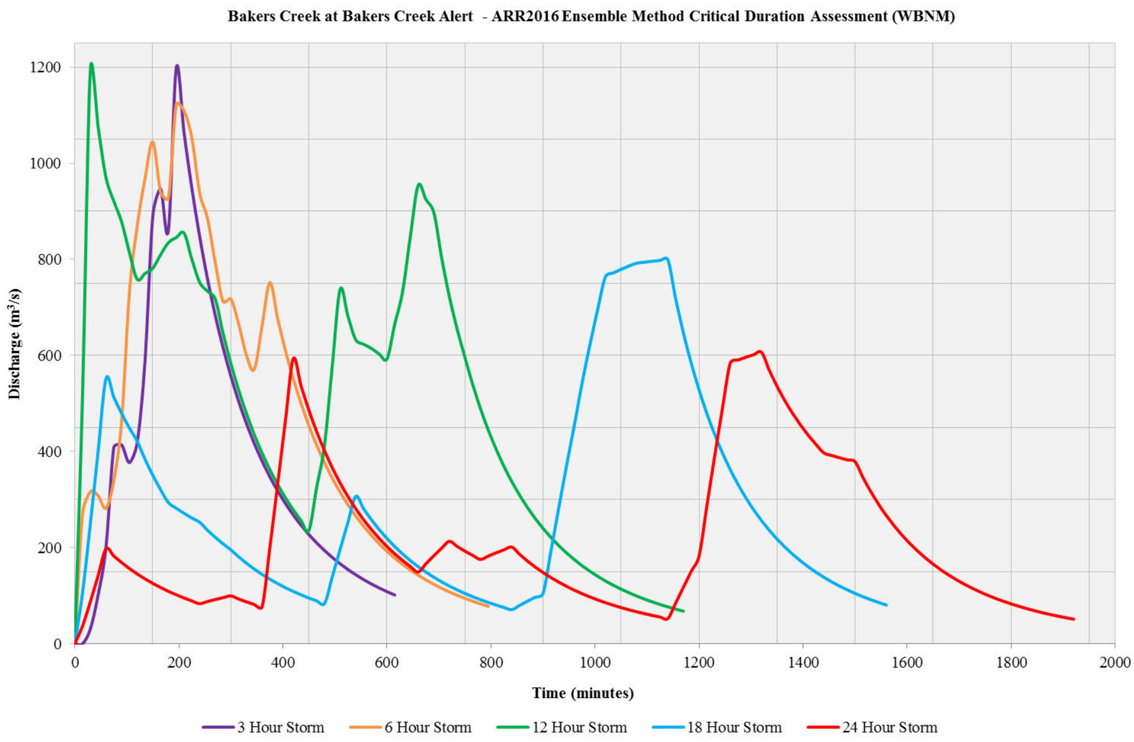


Figure E16 – ARR 2016 Critical Duration Assessment: Bakers Creek at Bakers Creek Alert

APPENDIX F: TUFLOW MODEL DEVELOPMENT AND RESULTS MAPPING

An integrated one-dimensional (1D) / two-dimensional (2D) numerical hydraulic model was developed to simulate flood behaviour within the study area, through solving of the depth averaged two-dimensional momentum and continuity equations for free-surface flow. TUFLOW Software version Build 2016-03-AD-iSP has been used for model simulations.

An overview of the model setup and key parameters is provided in Table F1.

Table F1 – TUFLOW Model Setup Overview

Parameter	Information
Completion Date	September 2017
AEP's Assessed	50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP
Hydrologic Modelling Approach	Discrete Inflow from WBNM Model
IFD Input Parameters	Based on ARR, both 2016 and 1987 versions, refer Methodology
Hydraulic Modelling Approach	TUFLOW version 2016-03-AD-iSP
Model Extent	Refer Methodology
Grid Size	20 m
DEM (year flown)	SRTM (2000)
Roughness	Spatially varying and depth varying standard values compliant with both ARR guidelines and MRC Flood modelling guidelines
Eddy Viscosity	SMAGORINSKY (default)
Model Calibration	2008 and 2007 flood events
Downstream Model Boundary	Height/Time varying boundary along Coral Sea using heights from Mackay Outer Harbour Tidal Plane
Hydraulic Model Timesteps	5 seconds (2D) and 2 seconds (1D)
Hydraulic Model Wetting and Drying Depths	Cell centre set at 0.0002 m Cell side set at 0.0001 m
Sensitivity Analyses	Not Applicable

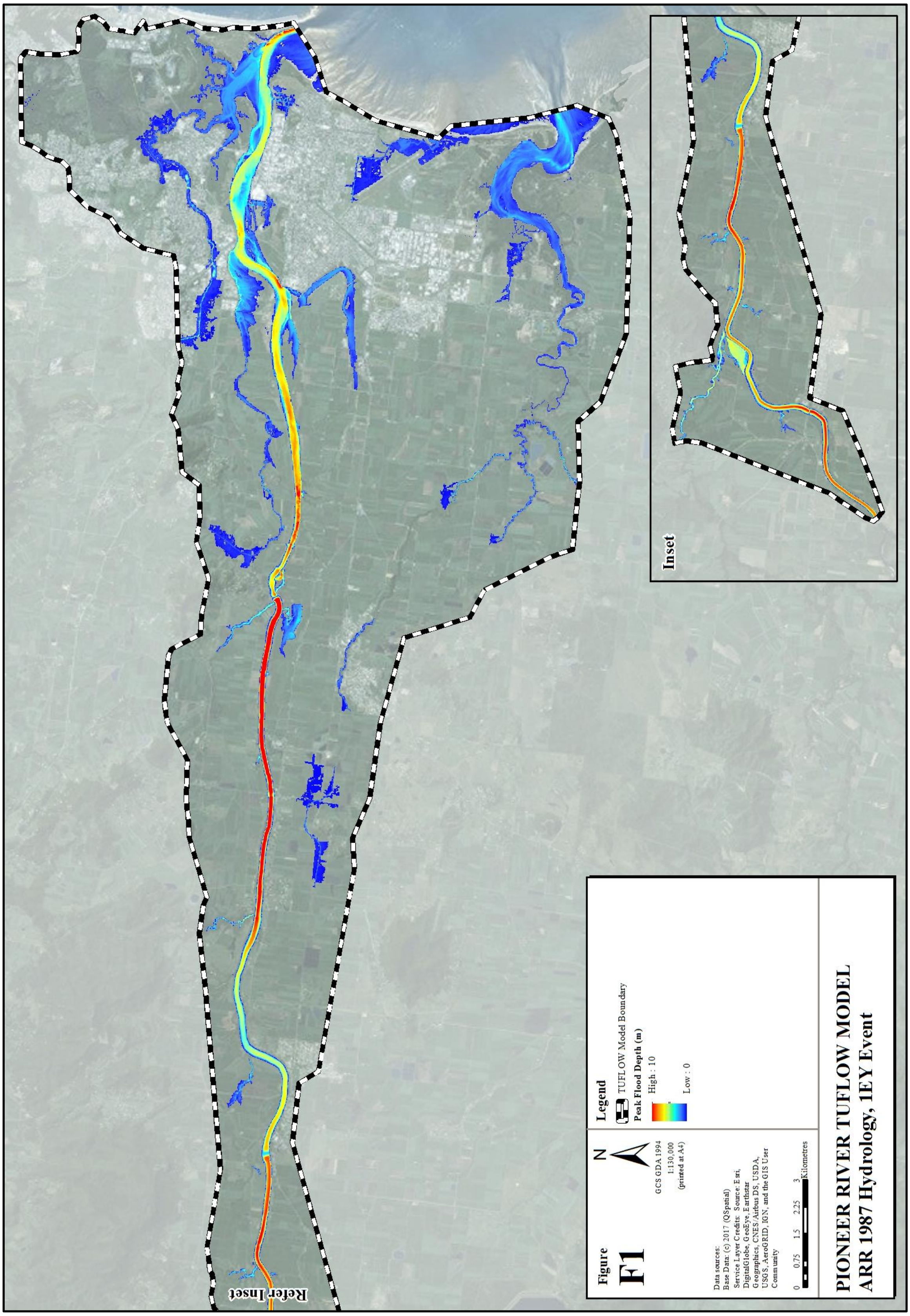


Figure F1 – TUFLOW Model Results Mapping: ARR 1987 Hydrology, 1EY Event

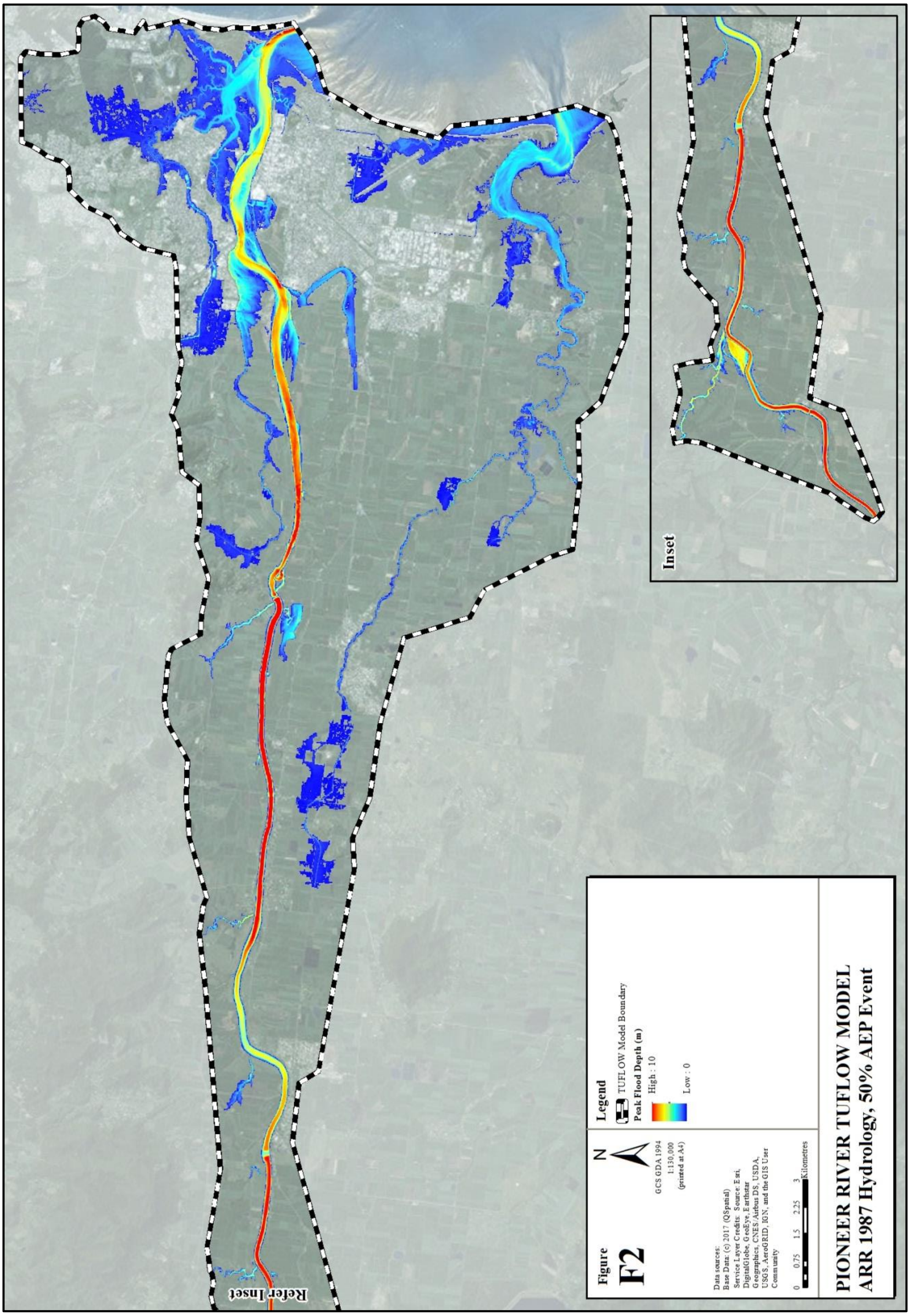


Figure F2 – TUFLOW Model Results Mapping: ARR 1987 Hydrology, 50% AEP Event

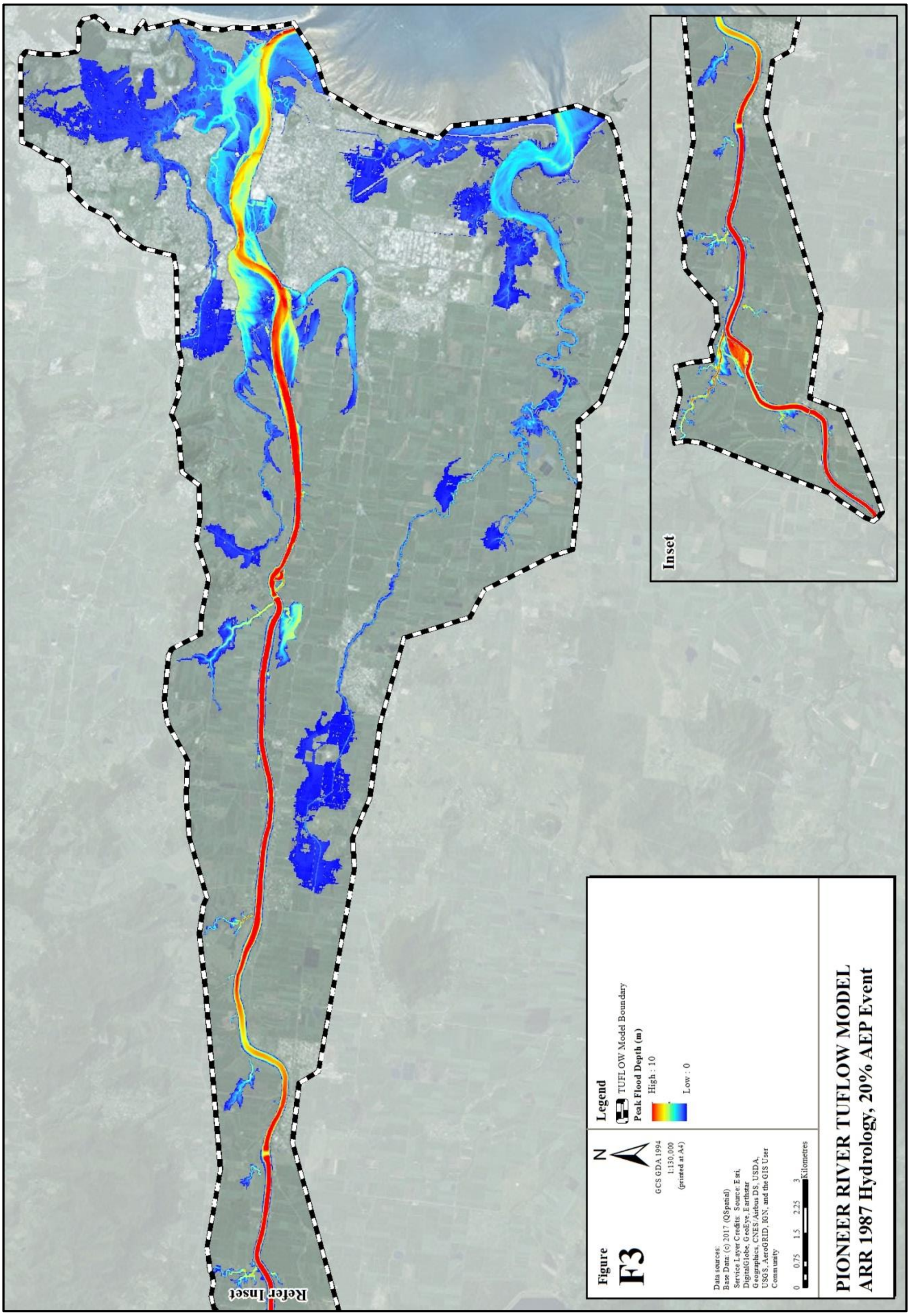


Figure F3 – TUFLOW Model Results Mapping: ARR 1987 Hydrology, 20% AEP Event

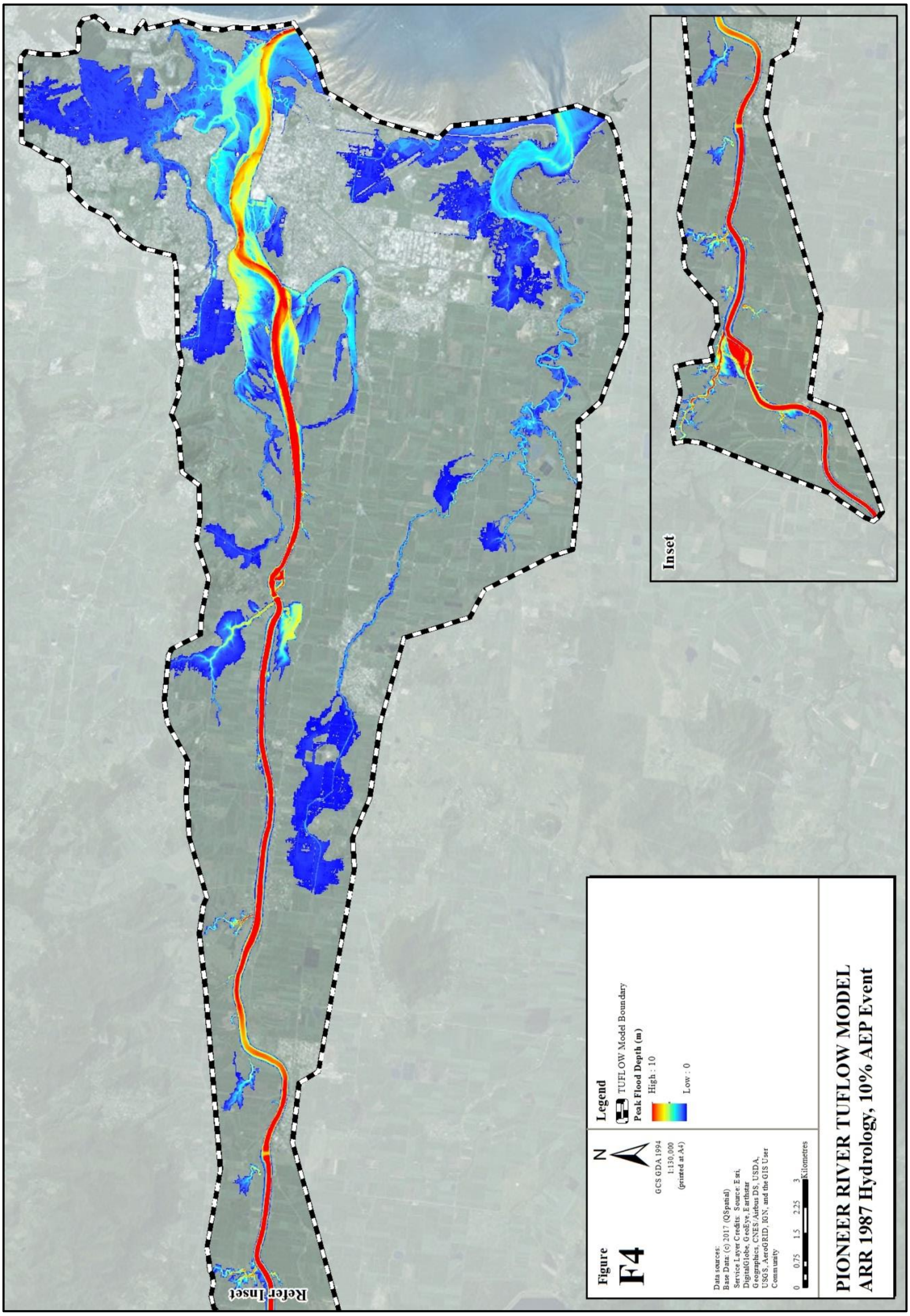


Figure F4 – TUFLOW Model Results Mapping: ARR 1987 Hydrology, 10% AEP Event

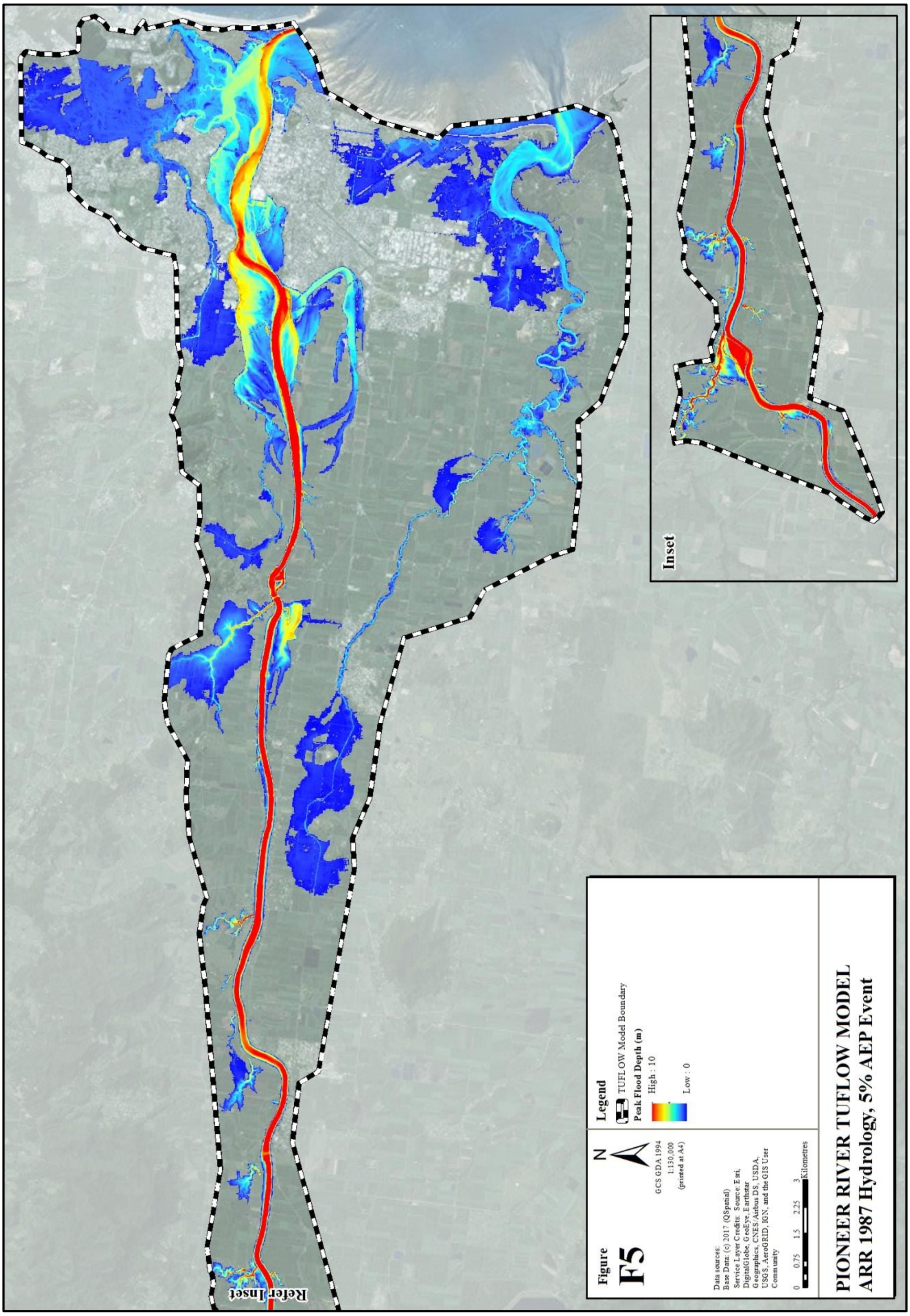


Figure F5 – TUFLOW Model Results Mapping: ARR 1987 Hydrology, 5% AEP Event

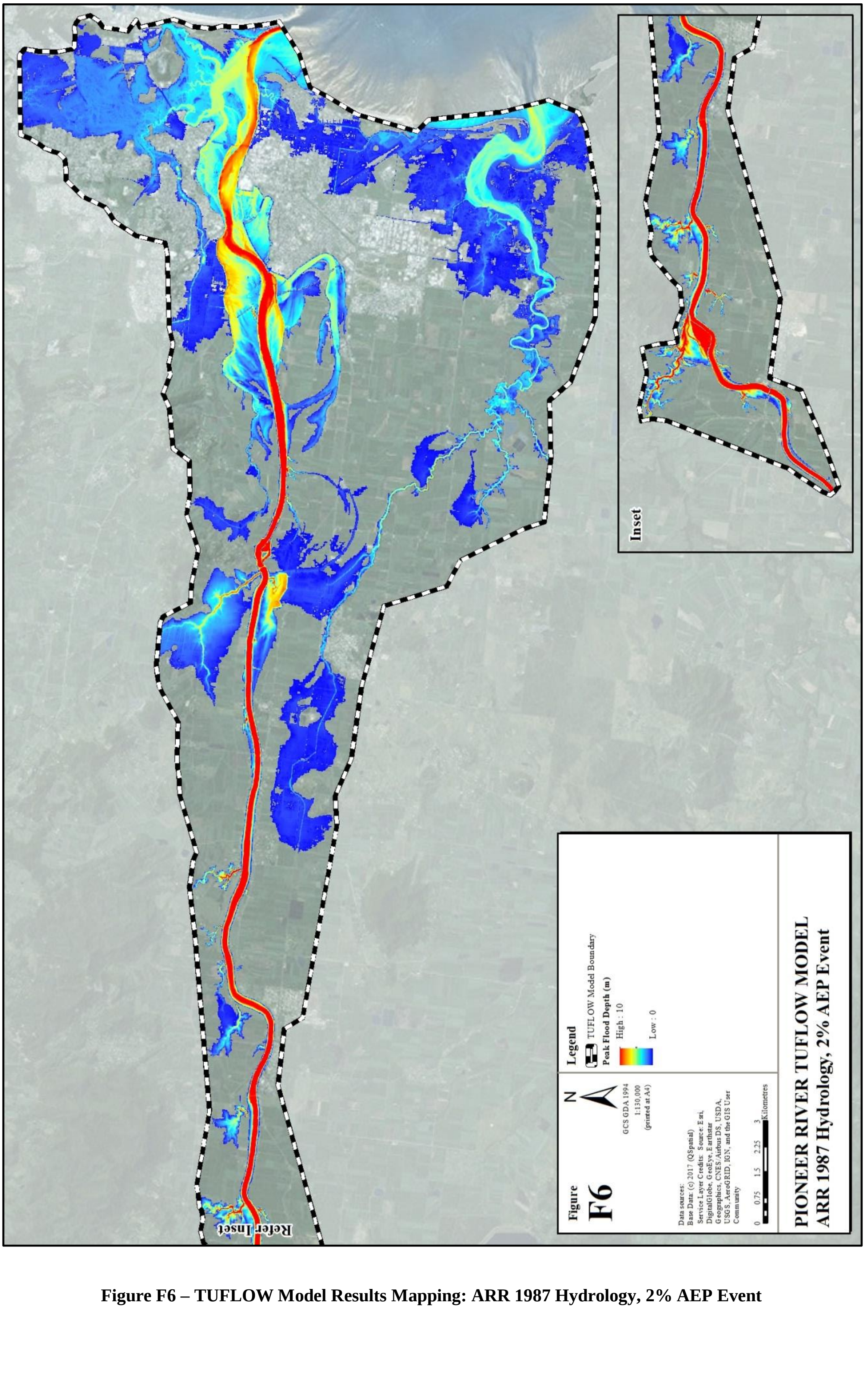


Figure F6 – TUFLOW Model Results Mapping: ARR 1987 Hydrology, 2% AEP Event

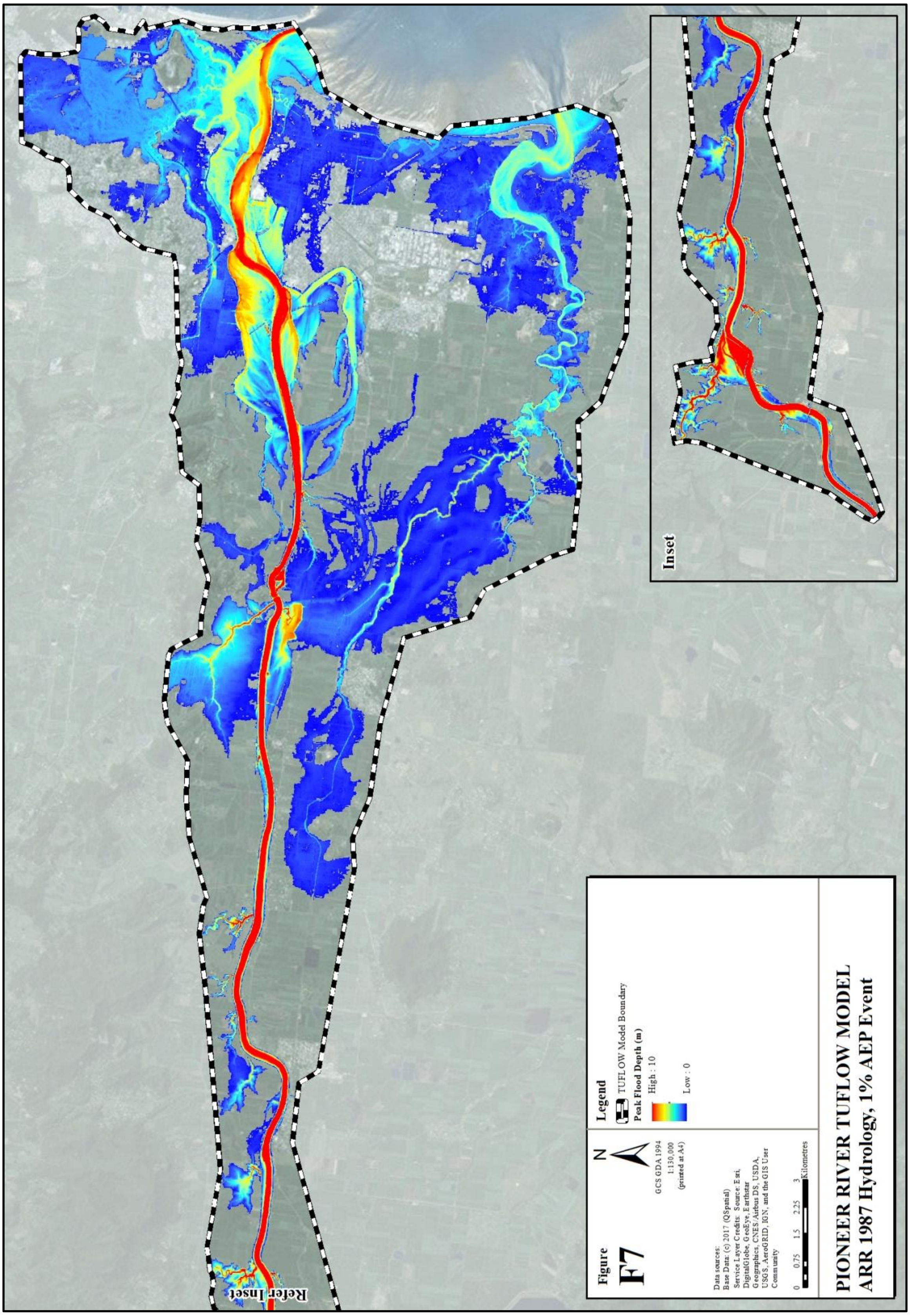


Figure F7 – TUFLOW Model Results Mapping: ARR 1987 Hydrology, 1% AEP Event

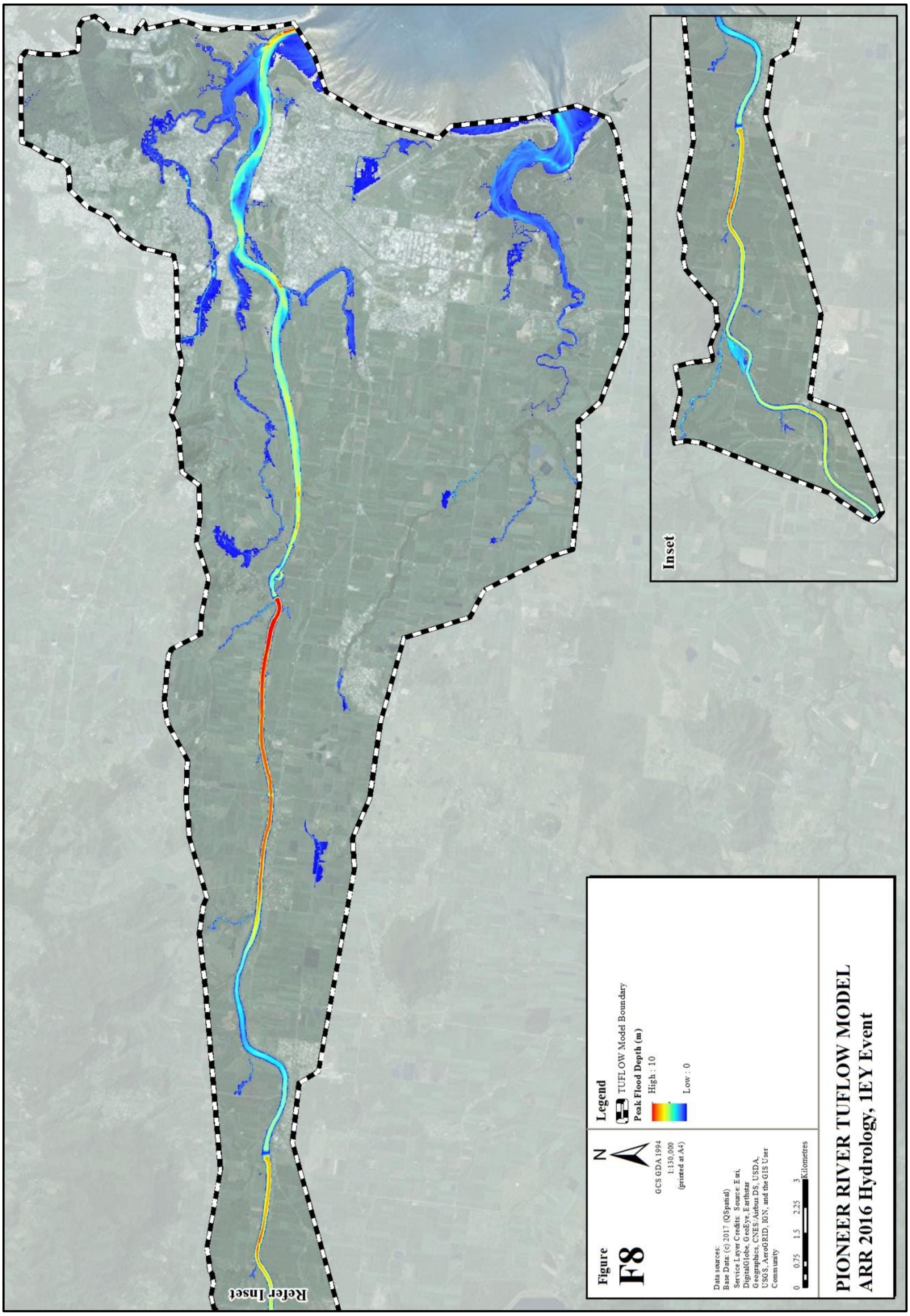


Figure F8 – TUFLOW Model Results Mapping: ARR 2016 Hydrology, 1EY Event

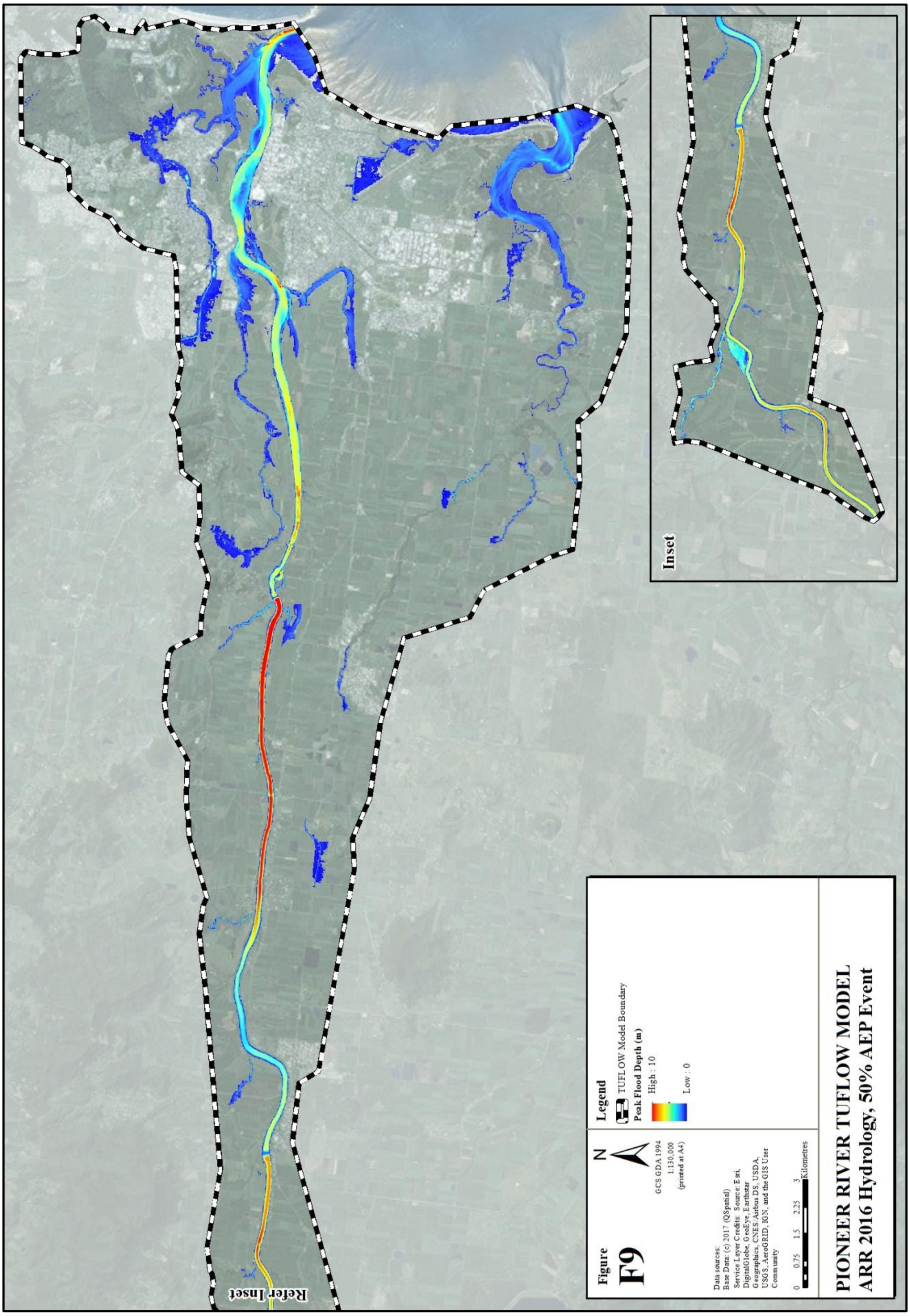


Figure F9 – TUFLOW Model Results Mapping: ARR 2016 Hydrology, 50% AEP Event

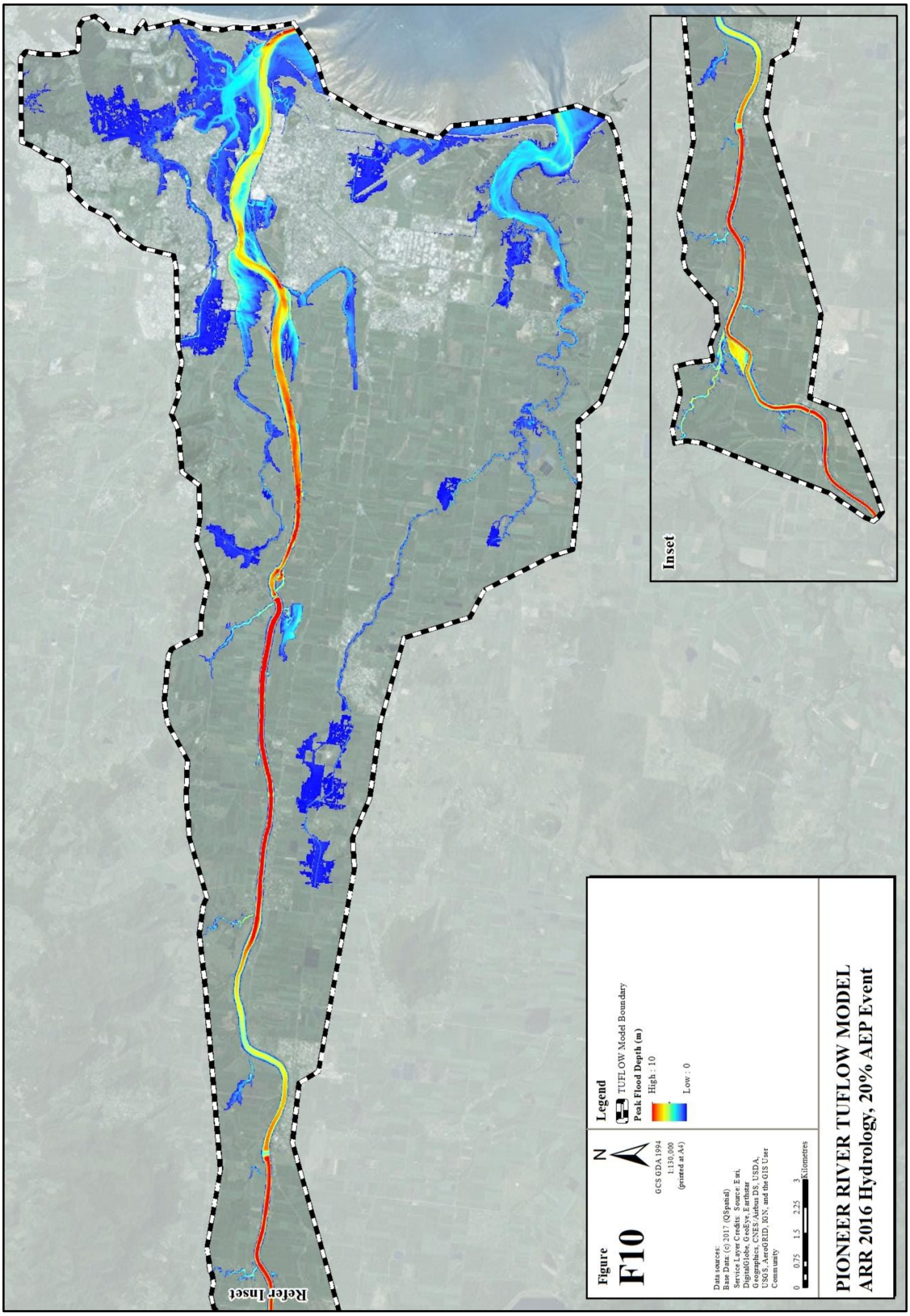


Figure F10 – TUFLOW Model Results Mapping: ARR 2016 Hydrology, 20% AEP Event

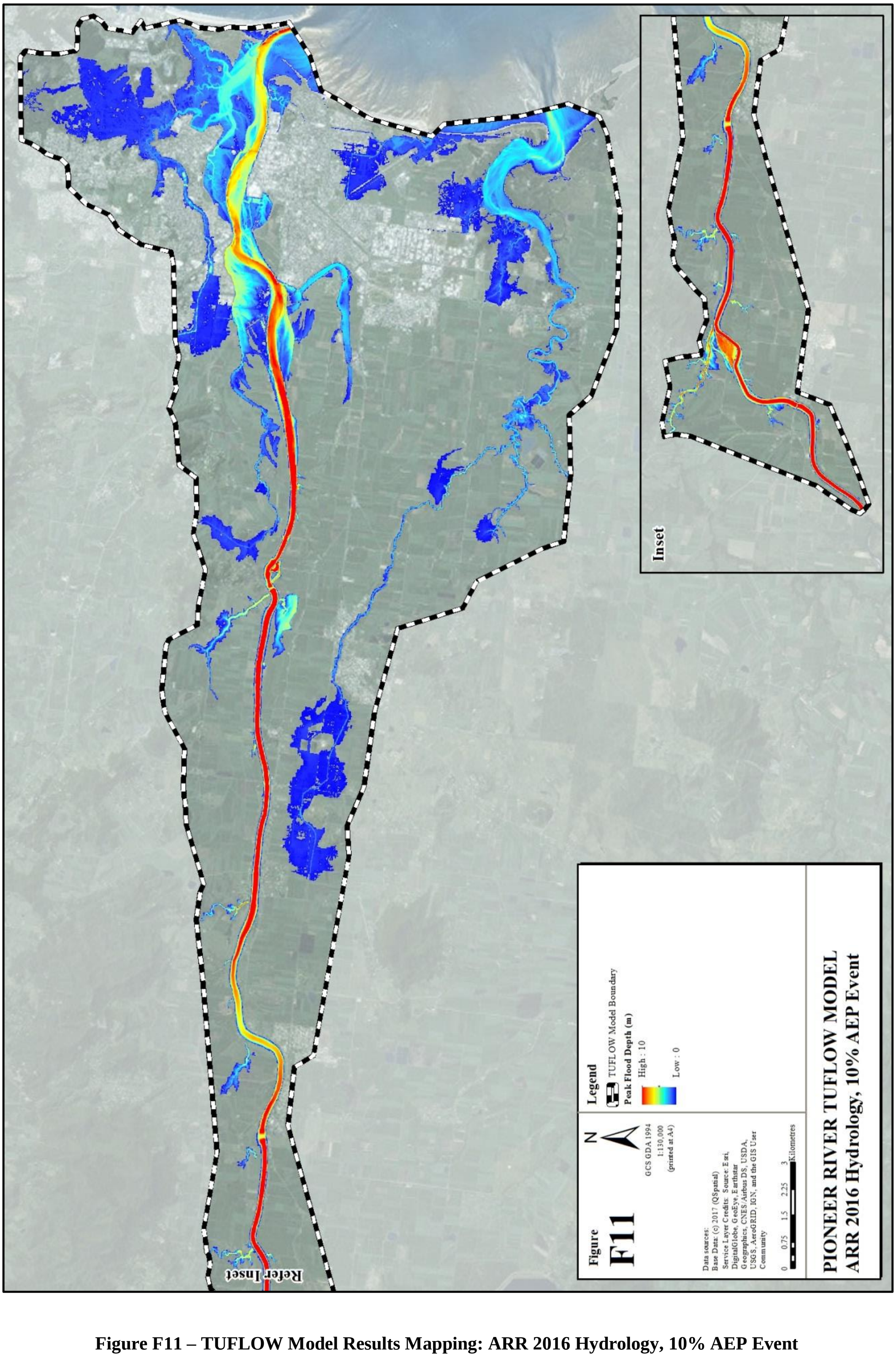


Figure F11 – TUFLOW Model Results Mapping: ARR 2016 Hydrology, 10% AEP Event

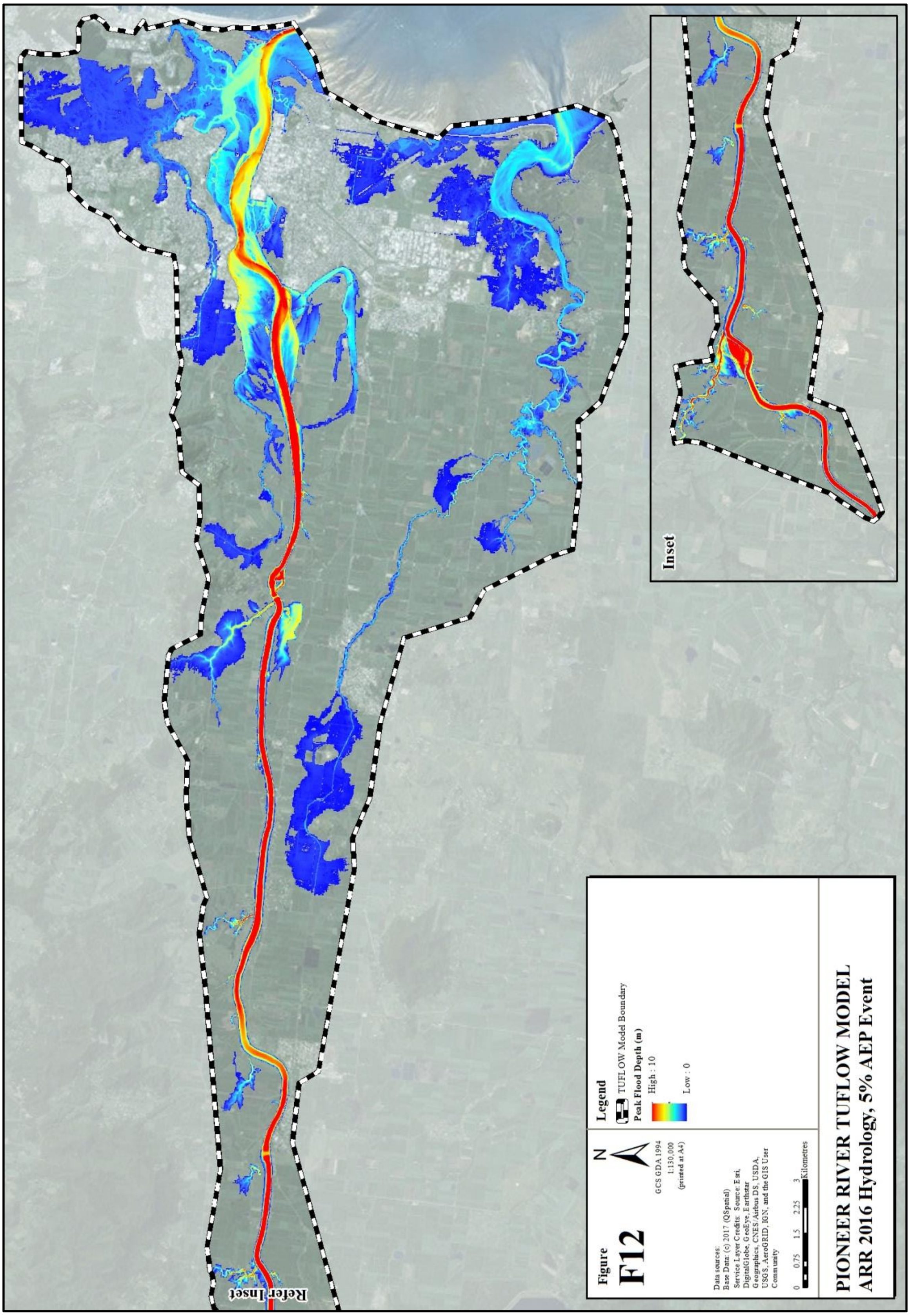


Figure F12 – TUFLOW Model Results Mapping: ARR 2016 Hydrology, 5% AEP Event

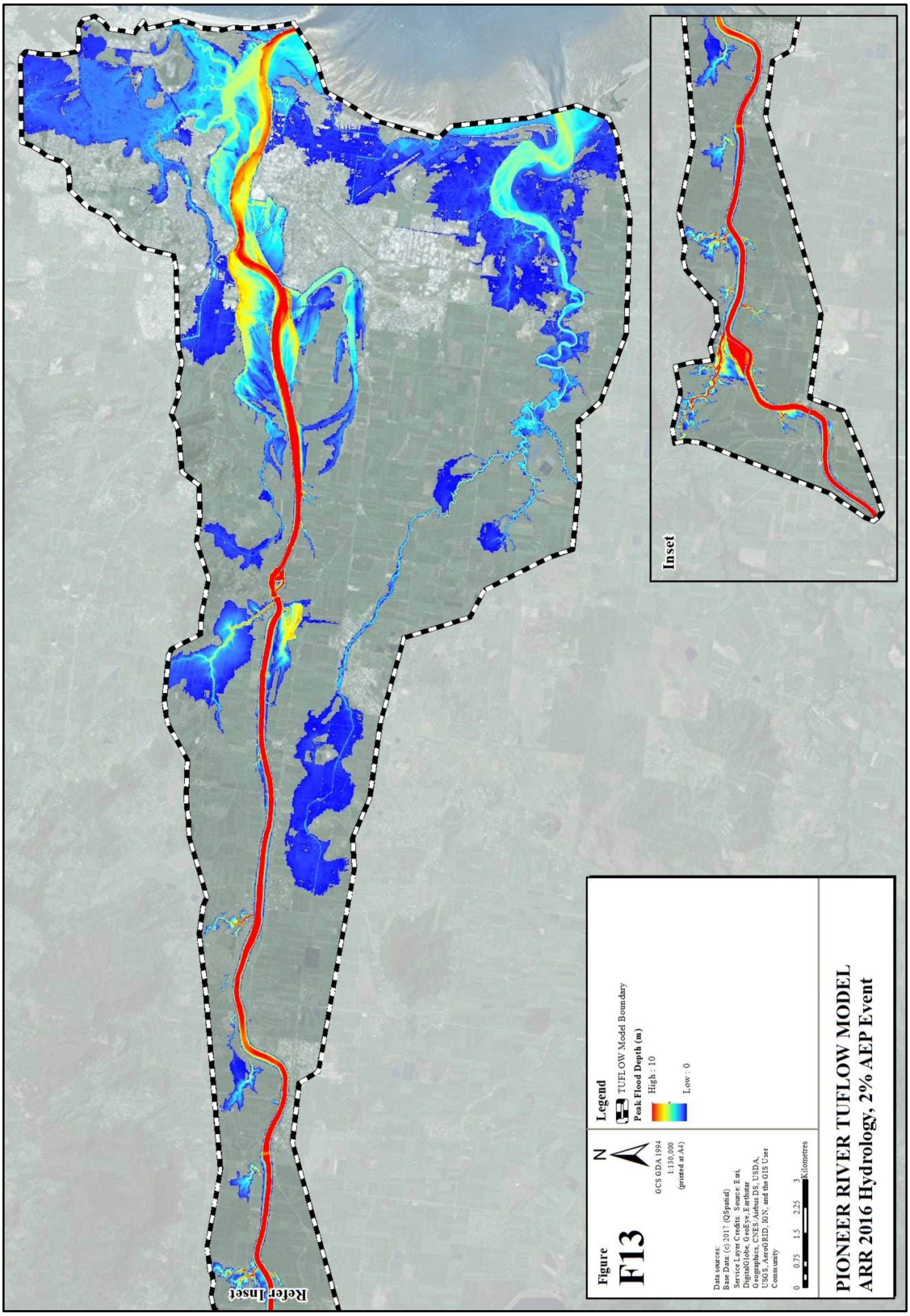


Figure F13 – TUFLOW Model Results Mapping: ARR 2016 Hydrology, 2% AEP Event

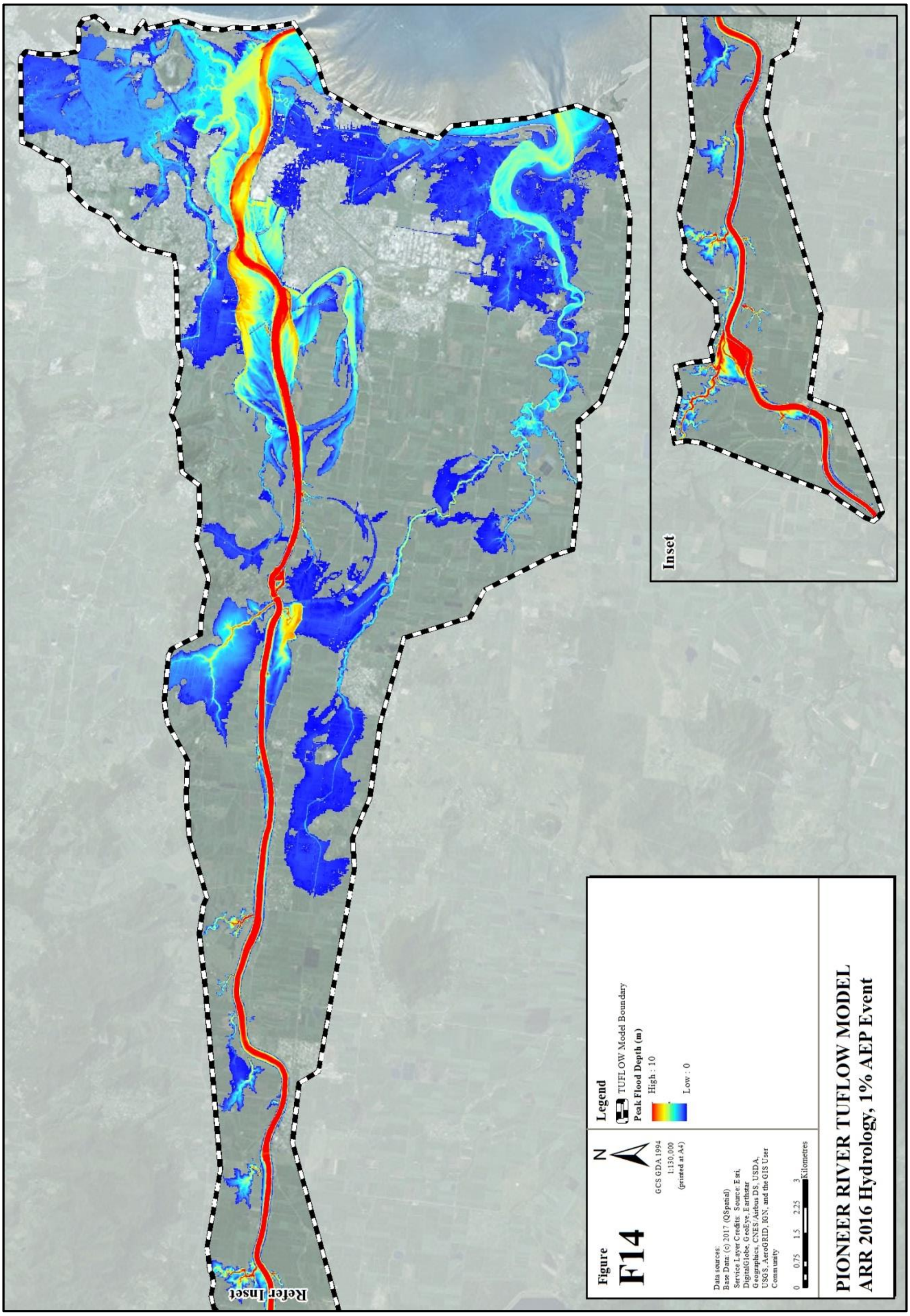


Figure F14 – TUFLOW Model Results Mapping: ARR 2016 Hydrology, 1% AEP Event

<u>APPENDIX G:</u> RFFE MODEL RESULTS

<u>Cattle Creek at Highams Bridge</u>

Table G1 – RFFE Model Input Data – Cattle Creek at Highams Bridge

INPUT DATA	
Latitude (Outlet)	-21.133
Longitude (Outlet)	148.651
Latitude (Centroid)	-21.098
Longitude (Centroid)	148.574
Catchment Area (km^2)	196.4
Distance to Nearest Gauged Catchment (km)	3.36
50% AEP 6 Hour Rainfall Intensity (mm/hr)	15.990398
2% AEP 6 Hour Rainfall Intensity (mm/hr)	41.454156
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	0.63
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.358

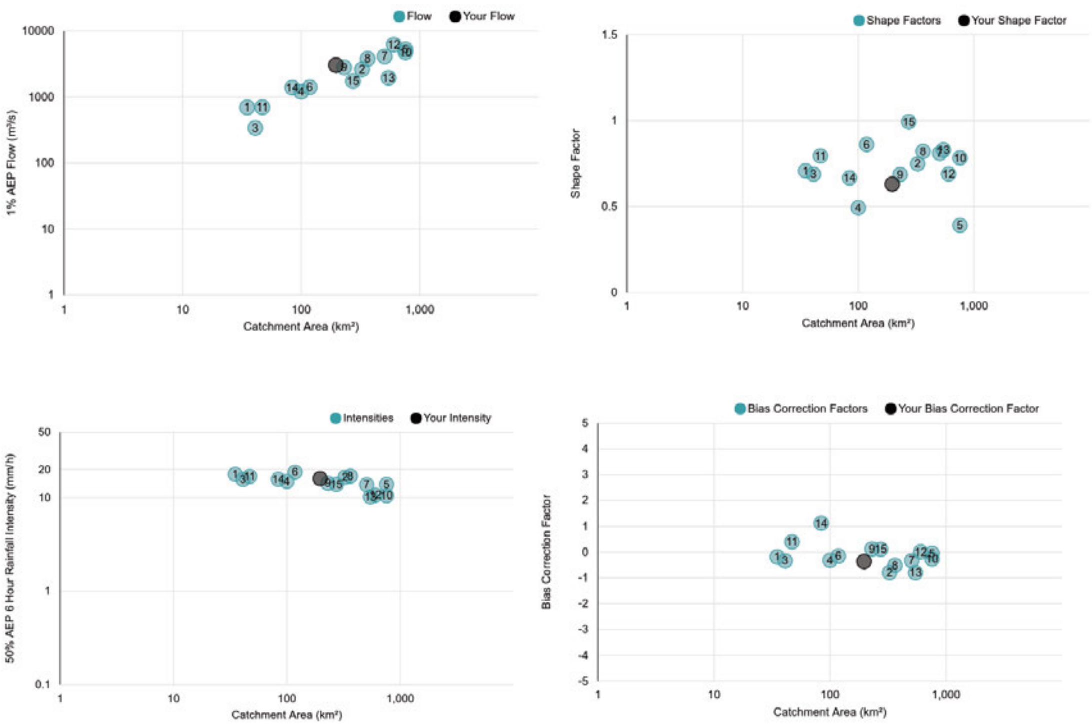


Figure G1 – RFFE Model Comparison Graphs – Cattle Creek at Highams Bridge

Cattle Creek at Gargett

Table G2 – RFFE Model Input Data – Cattle Creek at Gargett

INPUT DATA	
Latitude (Outlet)	-21.177
Longitude (Outlet)	148.742
Latitude (Centroid)	-21.109
Longitude (Centroid)	148.617
Catchment Area (km^2)	332.0
Distance to Nearest Gauged Catchment (km)	0.39
50% AEP 6 Hour Rainfall Intensity (mm/hr)	16.628553
2% AEP 6 Hour Rainfall Intensity (mm/hr)	43.084976
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	0.82
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.751

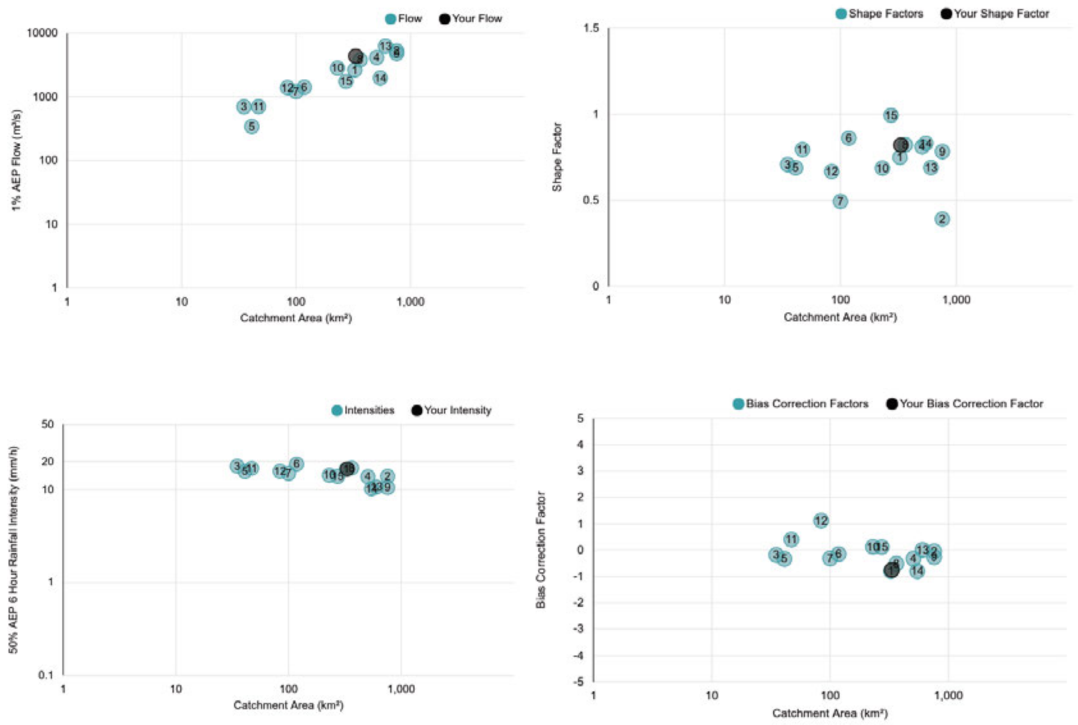


Figure G2 – RFFE Model Comparison Graphs – Cattle Creek at Gargett

Blacks Creek at Whitefords

Table G3 – RFFE Model Input Data – Blacks Creek at Whitefords

INPUT DATA	
Latitude (Outlet)	-21.316
Longitude (Outlet)	148.837
Latitude (Centroid)	-21.227
Longitude (Centroid)	148.687
Catchment Area (km^2)	511.18
Distance to Nearest Gauged Catchment (km)	1.72
50% AEP 6 Hour Rainfall Intensity (mm/hr)	14.977832
2% AEP 6 Hour Rainfall Intensity (mm/hr)	39.060531
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	0.81
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.178

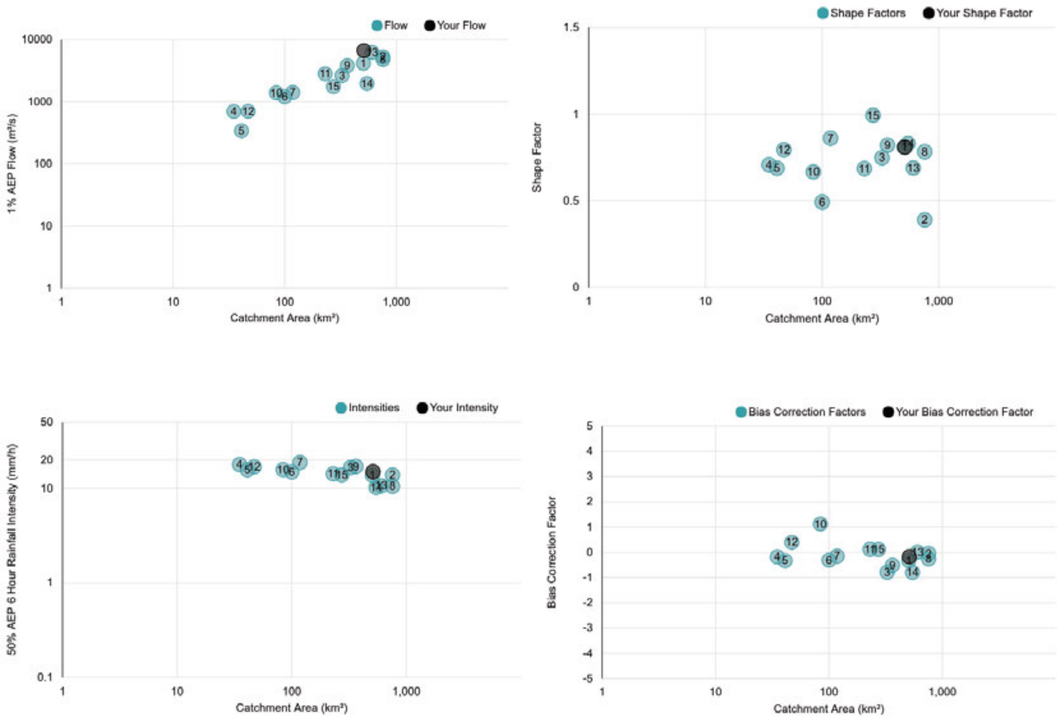


Figure G3 – RFFE Model Comparison Graphs – Blacks Creek at Whitefords

Pioneer River at Sarichs

Table G4 – RFFE Model Input Data – Pioneer River at Sarichs

INPUT DATA	
Latitude (Outlet)	-21.267
Longitude (Outlet)	148.818
Latitude (Centroid)	-21.316
Longitude (Centroid)	148.761
Catchment Area (km^2)	751.19
Distance to Nearest Gauged Catchment (km)	0.39
50% AEP 6 Hour Rainfall Intensity (mm/hr)	13.539285
2% AEP 6 Hour Rainfall Intensity (mm/hr)	34.719347
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	**0.29***
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.062

**The catchment has unusual shape. Results have lower accuracy and may not be directly applicable in practice.*

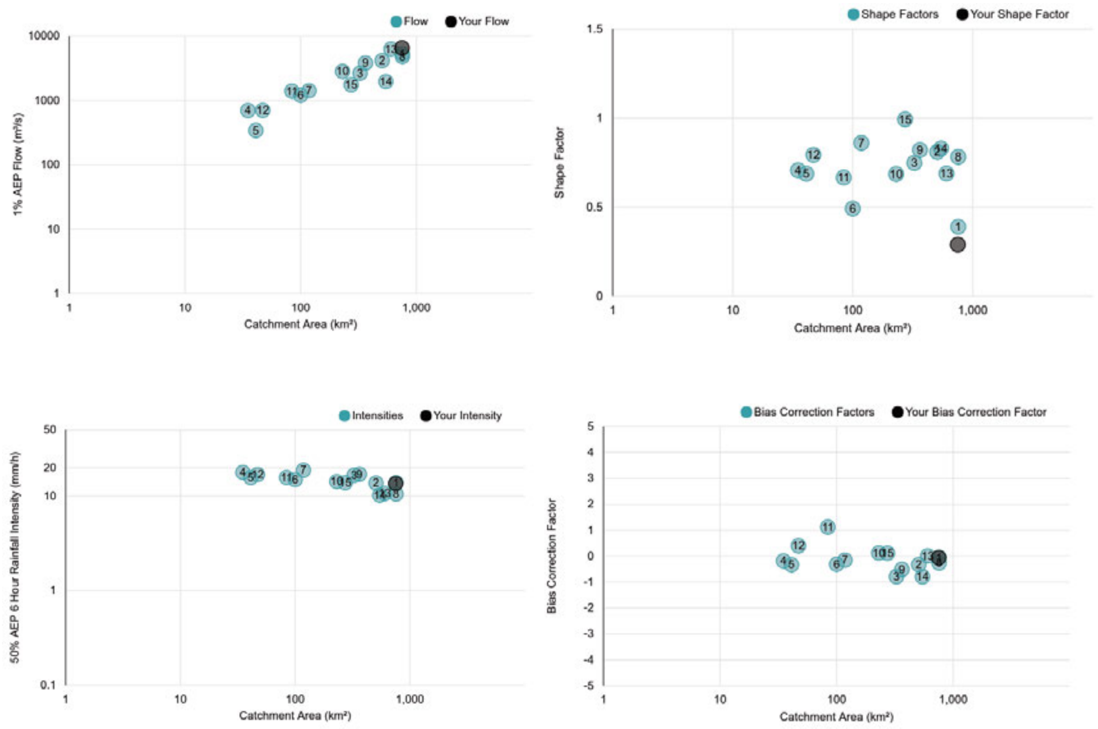


Figure G4 – RFFE Model Comparison Graphs – Pioneer River at Sarichs

Pioneer River at Mirani Weir Tailwater

Table G-5 – RFFE Model Input Data – Pioneer River at Mirani Weir Tailwater

INPUT DATA	
Latitude (Outlet)	-21.179
Longitude (Outlet)	148.829
Latitude (Centroid)	-21.239
Longitude (Centroid)	148.738
Catchment Area (km^2)	**1192.03***
Distance to Nearest Gauged Catchment (km)	9.23
50% AEP 6 Hour Rainfall Intensity (mm/hr)	15.050463
2% AEP 6 Hour Rainfall Intensity (mm/hr)	39.103875
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	**0.33****
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.061

**The catchment is outside the recommended catchment size of 0.5 to 1,000 km . Results have lower accuracy and may not be directly applicable in practice*

***The catchment has unusual shape. Results have lower accuracy and may not be directly applicable in practice.*

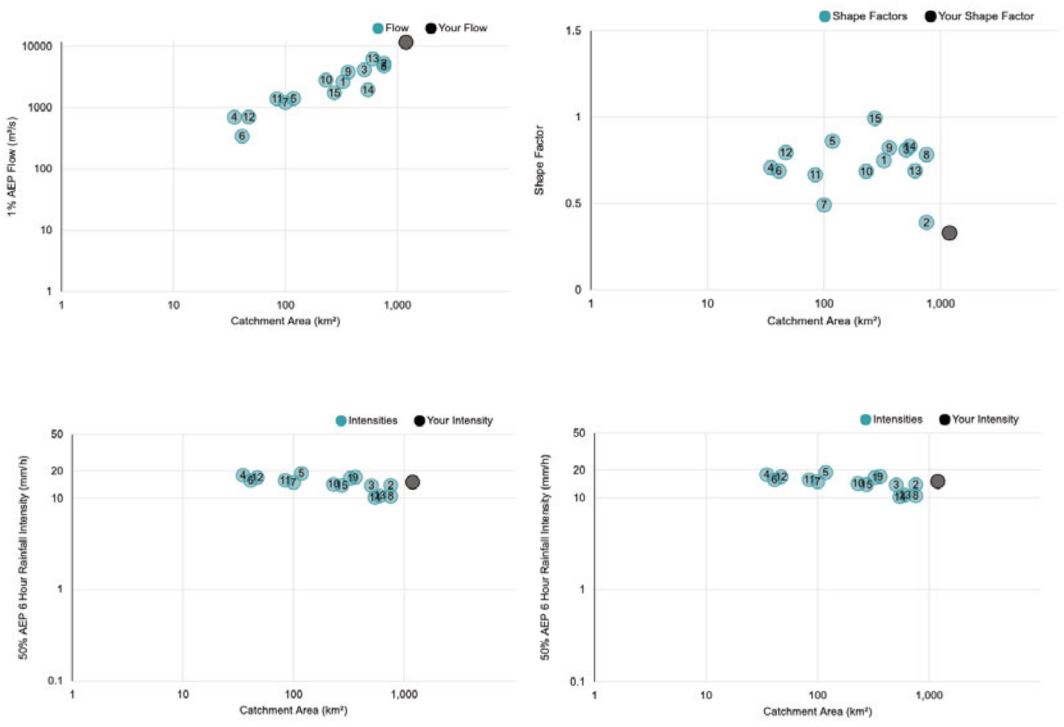


Figure G5 – RFFE Model Comparison Graphs – Pioneer River at Mirani Weir Tailwater

Pioneer River at Dumbleton Weir Tailwater

Table G6 – RFFE Model Input Data – Pioneer River at Dumbleton Weir Tailwater

INPUT DATA	
Latitude (Outlet)	-21.141
Longitude (Outlet)	149.069
Latitude (Centroid)	-21.24
Longitude (Centroid)	148.75
Catchment Area (km^2)	**1395.49***
Distance to Nearest Gauged Catchment (km)	29.53
50% AEP 6 Hour Rainfall Intensity (mm/hr)	15.050463
2% AEP 6 Hour Rainfall Intensity (mm/hr)	39.103875
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	0.93
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.255

**The catchment is outside the recommended catchment size of 0.5 to 1,000 km . Results have lower accuracy and may not be directly applicable in practice*

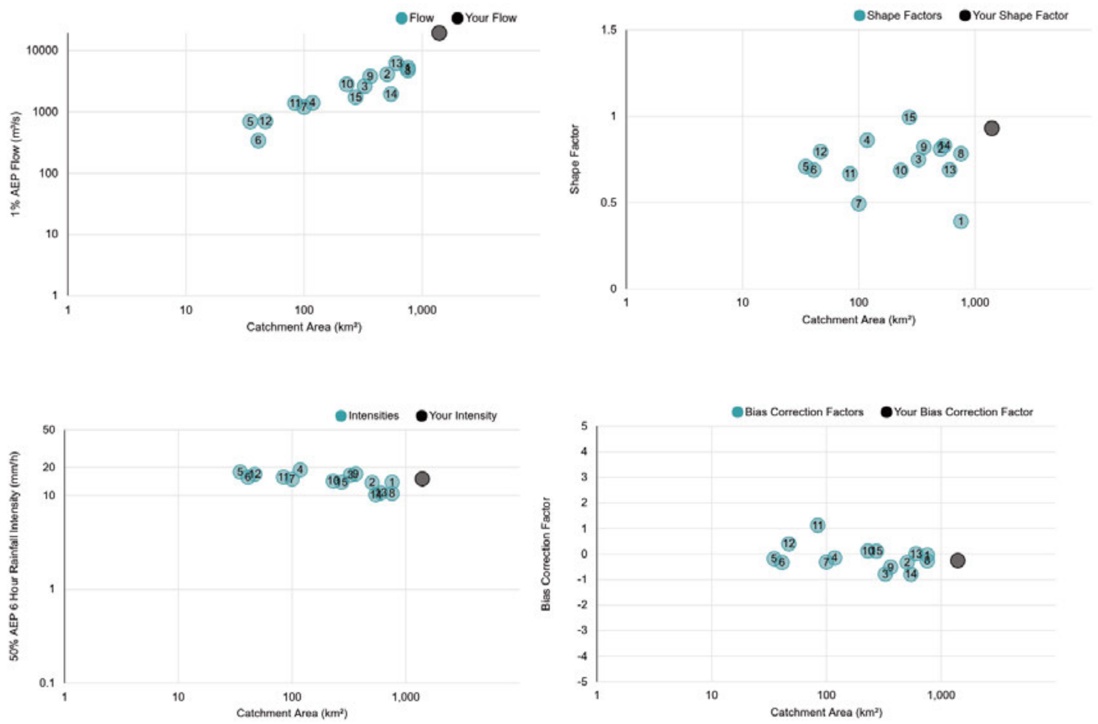


Figure G6 – RFFE Model Comparison Graphs – Pioneer River at Dumbleton Weir Tailwater

Pioneer River at Hospital Bridge Alert

Table G-7 – RFFE Model Input Data – Pioneer River at Hospital Bridge Alert

INPUT DATA	
Latitude (Outlet)	-21.151
Longitude (Outlet)	149.156
Latitude (Centroid)	-21.232
Longitude (Centroid)	148.789
Catchment Area (km^2)	**1486.33***
Distance to Nearest Gauged Catchment (km)	37.26
50% AEP 6 Hour Rainfall Intensity (mm/hr)	14.674436
2% AEP 6 Hour Rainfall Intensity (mm/hr)	37.704864
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	1.01
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.232

**The catchment is outside the recommended catchment size of 0.5 to 1,000 km . Results have lower accuracy and may not be directly applicable in practice*

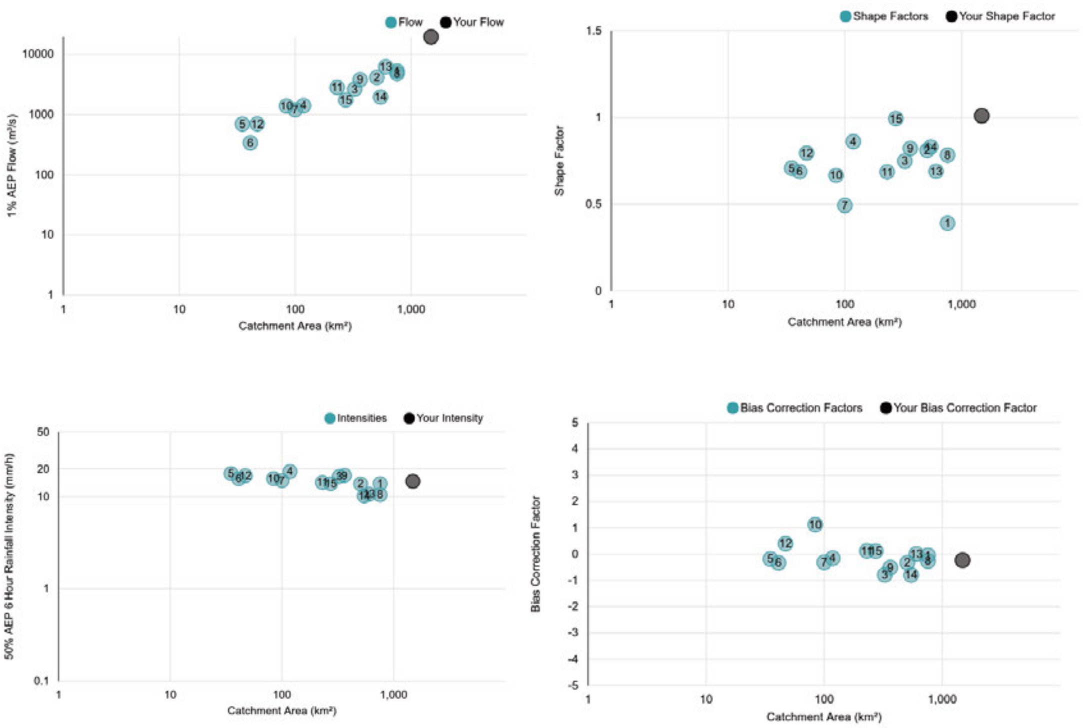


Figure G7 – RFFE Model Comparison Graphs – Pioneer River at Hospital Bridge Alert

Pioneer River at Mackay Alert

Table G8 – RFFE Model Input Data – Pioneer River at Mackay Alert

INPUT DATA	
Latitude (Outlet)	-21.139
Longitude (Outlet)	149.206
Latitude (Centroid)	-21.232
Longitude (Centroid)	148.803
Catchment Area (km^2)	**1507.75***
Distance to Nearest Gauged Catchment (km)	42.58
50% AEP 6 Hour Rainfall Intensity (mm/hr)	14.709592
2% AEP 6 Hour Rainfall Intensity (mm/hr)	37.602133
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	**1.11****
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.219

**The catchment is outside the recommended catchment size of 0.5 to 1,000 km . Results have lower accuracy and may not be directly applicable in practice*

***The catchment has unusual shape. Results have lower accuracy and may not be directly applicable in practice.*

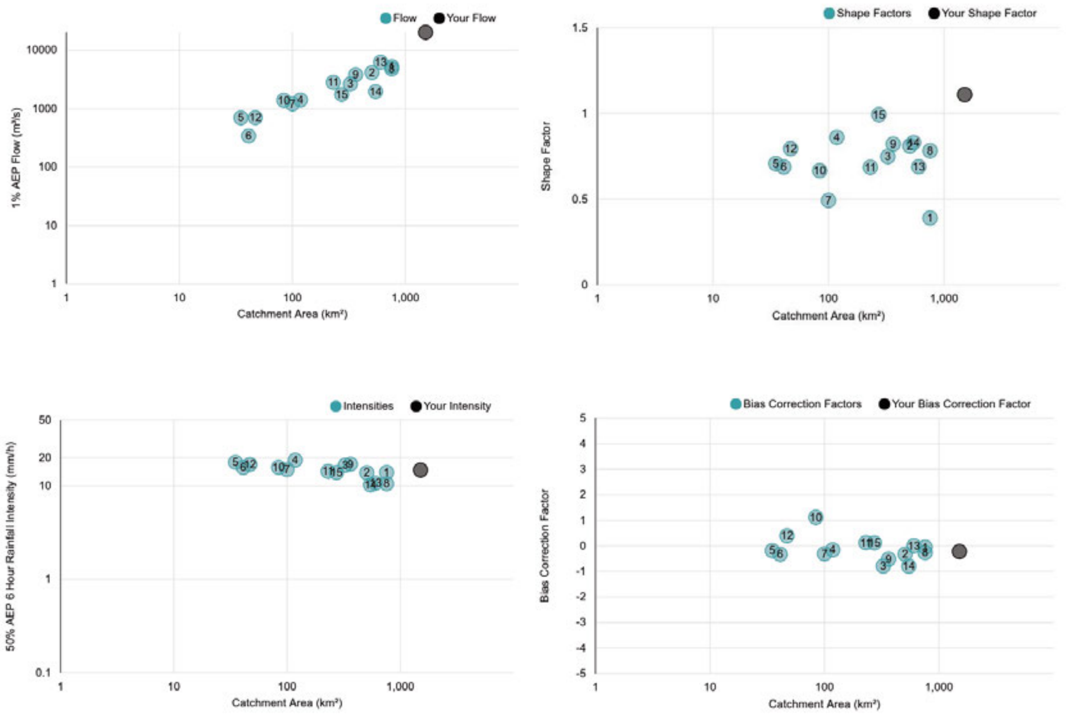


Figure G8 – RFFE Model Comparison Graphs – Pioneer River at Mackay Alert

Gooseponds Creek at Gooseponds Alert

Table G9 – RFFE Model Input Data – Gooseponds Creek at Gooseponds Alert

INPUT DATA	
Latitude (Outlet)	-21.127
Longitude (Outlet)	149.162
Latitude (Centroid)	-21.106
Longitude (Centroid)	149.128
Catchment Area (km^2)	23.14
Distance to Nearest Gauged Catchment (km)	38.86
50% AEP 6 Hour Rainfall Intensity (mm/hr)	17.540745
2% AEP 6 Hour Rainfall Intensity (mm/hr)	42.578876
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	0.88
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.232

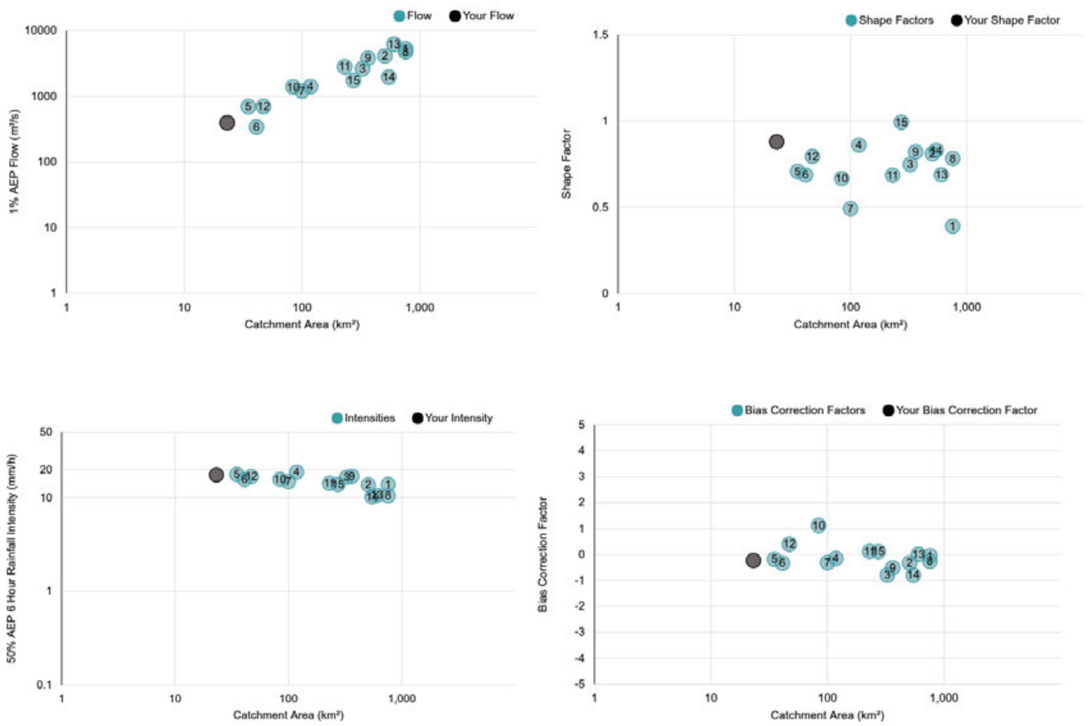


Figure G9 – RFFE Model Comparison Graphs – Gooseponds Creek at Gooseponds Alert

Bakers Creek at Bakers Creek Alert

Table G10 – RFFE Model Input Data – Bakers Creek at Bakers Creek Alert

INPUT DATA	
Latitude (Outlet)	-21.212
Longitude (Outlet)	149.167
Latitude (Centroid)	-21.201
Longitude (Centroid)	149.063
Catchment Area (km^2)	36.54
Distance to Nearest Gauged Catchment (km)	148.9
50% AEP 6 Hour Rainfall Intensity (mm/hr)	17.895669
2% AEP 6 Hour Rainfall Intensity (mm/hr)	43.700649
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 V1
Region Source (User/Auto)	Auto
Shape Factor	0.89
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.223

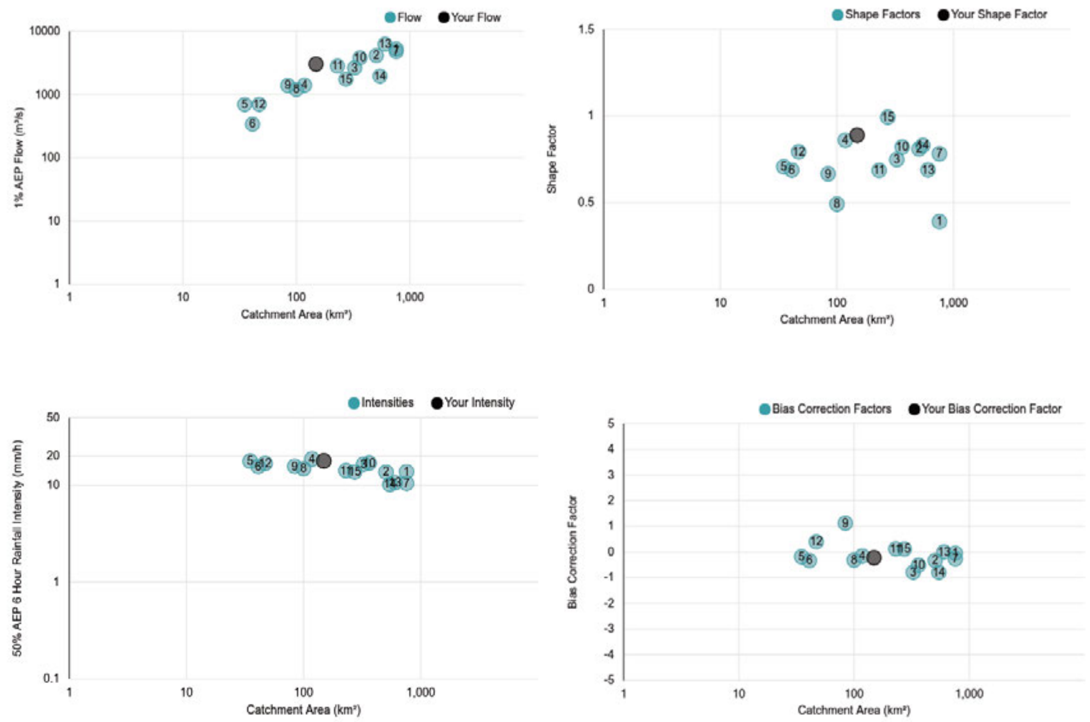


Figure G10 – RFFE Model Comparison Graphs – Bakers Creek at Bakers Creek Alert

APPENDIX H: RATIONAL METHOD CALCULATIONS

Cattle Creek at Highams Bridge

Table H1 – Rational Method Data and Results: Cattle Creek at Highams Bridge

RATIONAL METHOD DATA	
Area of Catchment (km^2)	196.4
Length of Flowpath (km)	25.5
Time of Concentration (min)	155.9
Runoff Coefficient, 2% AEP (C_{50})	0.834
Discharge Estimate, 1EY (Q_1)	839.26
Discharge Estimate, 50% AEP (Q_2)	1097.89
Discharge Estimate, 20% AEP (Q_5)	1449.02
Discharge Estimate, 10% AEP (Q_{10})	1670.48
Discharge Estimate, 5% AEP (Q_{20})	2204.66
Discharge Estimate, 2% AEP (Q_{50})	2950.37
Discharge Estimate, 1% AEP (Q_{100})	3509.13

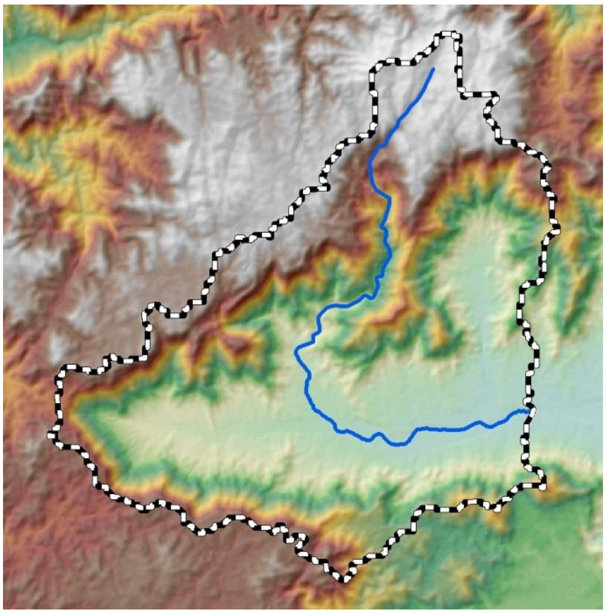


Figure H1 – Rational Method Catchment and Flowpath: Cattle Creek at Highams Bridge

Cattle Creek at Gargett

Table H2 – Rational Method Data and Results: Cattle Creek at Gargett

RATIONAL METHOD DATA	
Area of Catchment (km^2)	332.0
Length of Flowpath (km)	39.0
Time of Concentration (min)	207.1
Runoff Coefficient, 2% AEP (C_{50})	0.861
Discharge Estimate, 1EY (Q_1)	1140.31
Discharge Estimate, 50% AEP (Q_2)	1494.65
Discharge Estimate, 20% AEP (Q_5)	1986.91
Discharge Estimate, 10% AEP (Q_{10})	2303.28

RATIONAL METHOD DATA	
Discharge Estimate, 5% AEP (Q_{20})	3050.03
Discharge Estimate, 2% AEP (Q_{50})	4117.50
Discharge Estimate, 1% AEP (Q_{100})	4884.79

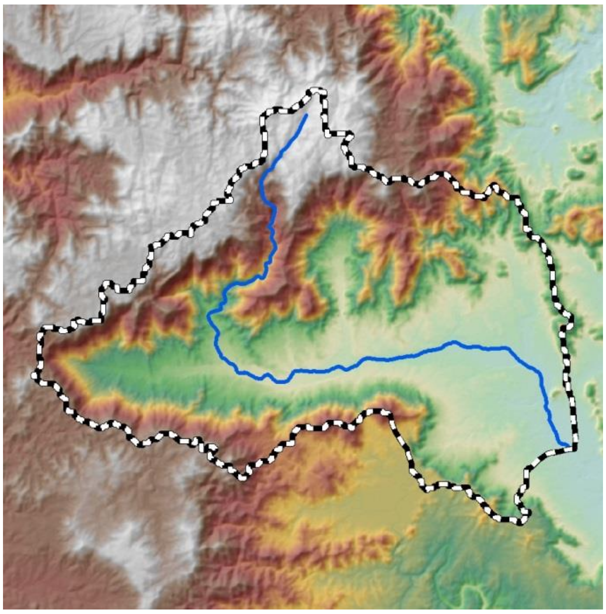


Figure H2 – Rational Method Catchment and Flowpath: Cattle Creek at Gargett

Blacks Creek at Whitefords

Table H3 – Rational Method Data and Results: Blacks Creek at Whitefords

RATIONAL METHOD DATA	
Area of Catchment (km^2)	511.2
Length of Flowpath (km)	49.0
Time of Concentration (min)	214.1
Runoff Coefficient, 2% AEP (C_{50})	0.803
Discharge Estimate, 1EY (Q_1)	1712.54
Discharge Estimate, 50% AEP (Q_2)	2243.06
Discharge Estimate, 20% AEP (Q_5)	2985.74
Discharge Estimate, 10% AEP (Q_{10})	3461.56
Discharge Estimate, 5% AEP (Q_{20})	4586.17
Discharge Estimate, 2% AEP (Q_{50})	6175.63
Discharge Estimate, 1% AEP (Q_{100})	7354.40

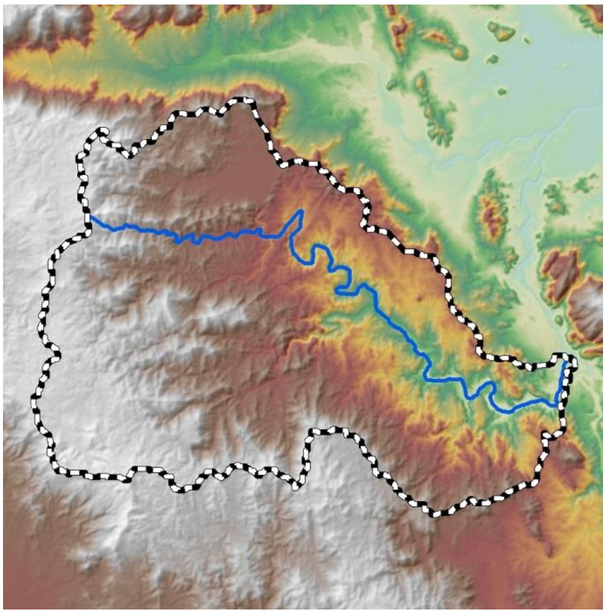


Figure H3 – Rational Method Catchment and Flowpath: Blacks Creek at Whitefords

Pioneer River at Sarichs

Table H4 – Rational Method Data and Results: Pioneer River at Sarichs

RATIONAL METHOD DATA	
Area of Catchment (km^2)	751.2
Length of Flowpath (km)	55.0
Time of Concentration (min)	307.6
Runoff Coefficient, 2% AEP (C_{50})	0.772
Discharge Estimate, 1EY (Q_1)	1906.64
Discharge Estimate, 50% AEP (Q_2)	2515.34
Discharge Estimate, 20% AEP (Q_5)	3381.72
Discharge Estimate, 10% AEP (Q_{10})	3942.13
Discharge Estimate, 5% AEP (Q_{20})	5251.96
Discharge Estimate, 2% AEP (Q_{50})	7077.52
Discharge Estimate, 1% AEP (Q_{100})	8513.52

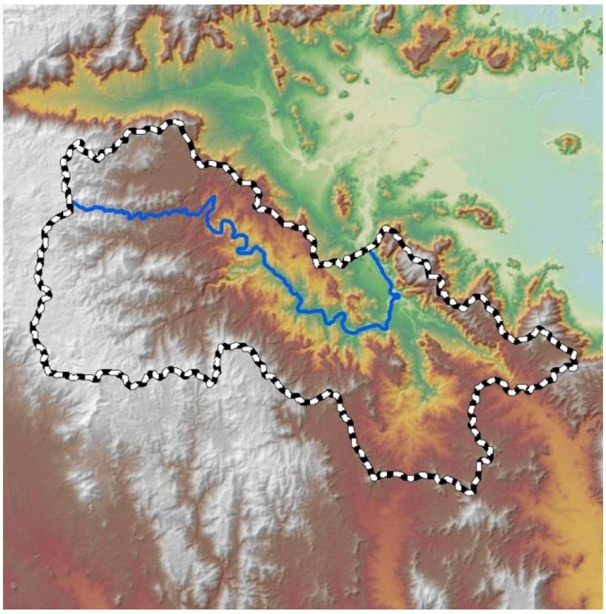


Figure H4 – Rational Method Catchment and Flowpath: Pioneer River at Sarichs

Pioneer River at Mirani Weir Tailwater

Table H5 – Rational Method Data and Results: Pioneer River at Mirani Weir Tailwater

RATIONAL METHOD DATA	
Area of Catchment (km^2)	1192.0
Length of Flowpath (km)	70.4
Time of Concentration (min)	366.1
Runoff Coefficient, 2% AEP (C_{50})	0.759
Discharge Estimate, 1EY (Q_1)	2668.60
Discharge Estimate, 50% AEP (Q_2)	3507.22
Discharge Estimate, 20% AEP (Q_5)	4744.02
Discharge Estimate, 10% AEP (Q_{10})	5540.38
Discharge Estimate, 5% AEP (Q_{20})	7400.35
Discharge Estimate, 2% AEP (Q_{50})	9997.44
Discharge Estimate, 1% AEP (Q_{100})	11991.31

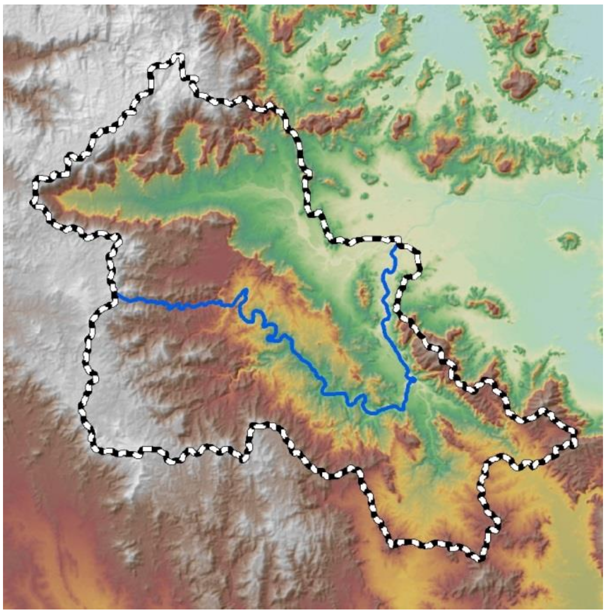


Figure H5 – Rational Method Catchment and Flowpath: Pioneer River at Mirani Weir Tailwater

Pioneer River at Dumbleton Weir Tailwater

Table H6 – Rational Method Data and Results: Pioneer River at Dumbleton Weir Tailwater

RATIONAL METHOD DATA	
Area of Catchment (km^2)	1395.5
Length of Flowpath (km)	95.0
Time of Concentration (min)	467.1
Runoff Coefficient, 2% AEP (C_{50})	0.744
Discharge Estimate, 1EY (Q_1)	2610.71
Discharge Estimate, 50% AEP (Q_2)	3438.11
Discharge Estimate, 20% AEP (Q_5)	4698.94
Discharge Estimate, 10% AEP (Q_{10})	5493.35
Discharge Estimate, 5% AEP (Q_{20})	7364.84
Discharge Estimate, 2% AEP (Q_{50})	10002.33
Discharge Estimate, 1% AEP (Q_{100})	12012.37

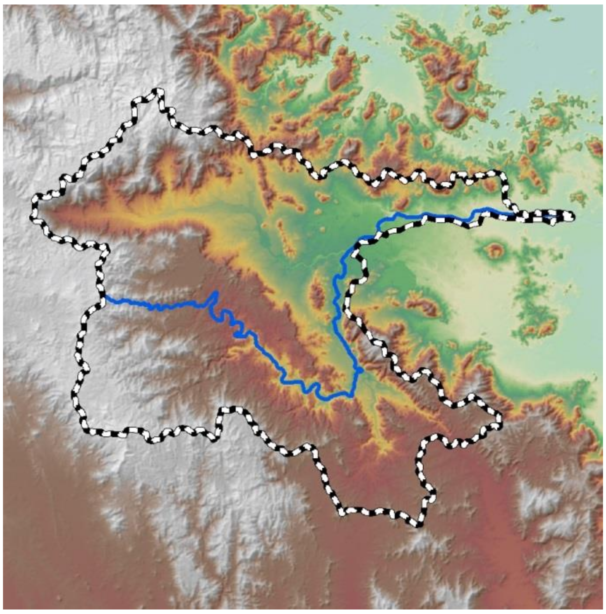


Figure H6 – Rational Method Catchment and Flowpath: Pioneer River at Dumbleton Weir Tailwater

Pioneer River at Hospital Bridge Alert

Table H7 – Rational Method Data and Results: Pioneer River at Hospital Bridge Alert

RATIONAL METHOD DATA	
Area of Catchment (km^2)	1486.3
Length of Flowpath (km)	110.3
Time of Concentration (min)	468.5
Runoff Coefficient, 2% AEP (C_{50})	0.744
Discharge Estimate, 1EY (Q_1)	2774.05
Discharge Estimate, 50% AEP (Q_2)	3652.72
Discharge Estimate, 20% AEP (Q_5)	4993.51
Discharge Estimate, 10% AEP (Q_{10})	5838.23
Discharge Estimate, 5% AEP (Q_{20})	7827.64
Discharge Estimate, 2% AEP (Q_{50})	10630.77
Discharge Estimate, 1% AEP (Q_{100})	12767.02

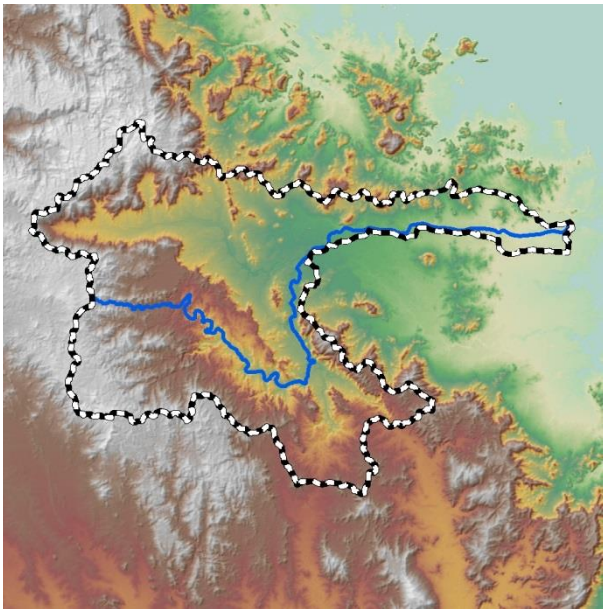


Figure H7 – Rational Method Catchment and Flowpath: Pioneer River at Hospital Bridge Alert

Pioneer River at Mackay Alert

Table H8 – Rational Method Data and Results: Pioneer River at Mackay Alert

RATIONAL METHOD DATA	
Area of Catchment (km^2)	1508.8
Length of Flowpath (km)	111.2
Time of Concentration (min)	472.3
Runoff Coefficient, 2% AEP (C_{50})	0.743
Discharge Estimate, 1EY (Q_1)	2798.62
Discharge Estimate, 50% AEP (Q_2)	3683.78
Discharge Estimate, 20% AEP (Q_5)	5039.28
Discharge Estimate, 10% AEP (Q_{10})	5893.10
Discharge Estimate, 5% AEP (Q_{20})	7902.33
Discharge Estimate, 2% AEP (Q_{50})	10732.01
Discharge Estimate, 1% AEP (Q_{100})	12888.36

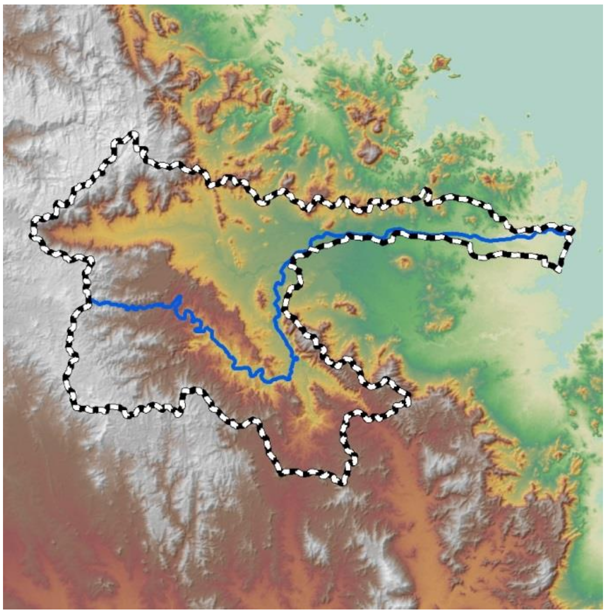


Figure H8 – Rational Method Catchment and Flowpath: Pioneer River at Mackay Alert

Gooseponds Creek at Gooseponds Alert

Table H9 – Rational Method Data and Results: Gooseponds Creek at Gooseponds Alert

RATIONAL METHOD DATA	
Area of Catchment (km^2)	23.1
Length of Flowpath (km)	9.5
Time of Concentration (min)	237.8
Runoff Coefficient, 2% AEP (C_{50})	0.766
Discharge Estimate, 1EY (Q_1)	55.55
Discharge Estimate, 50% AEP (Q_2)	73.13
Discharge Estimate, 20% AEP (Q_5)	98.59
Discharge Estimate, 10% AEP (Q_{10})	115.03
Discharge Estimate, 5% AEP (Q_{20})	153.42
Discharge Estimate, 2% AEP (Q_{50})	206.93
Discharge Estimate, 1% AEP (Q_{100})	248.56

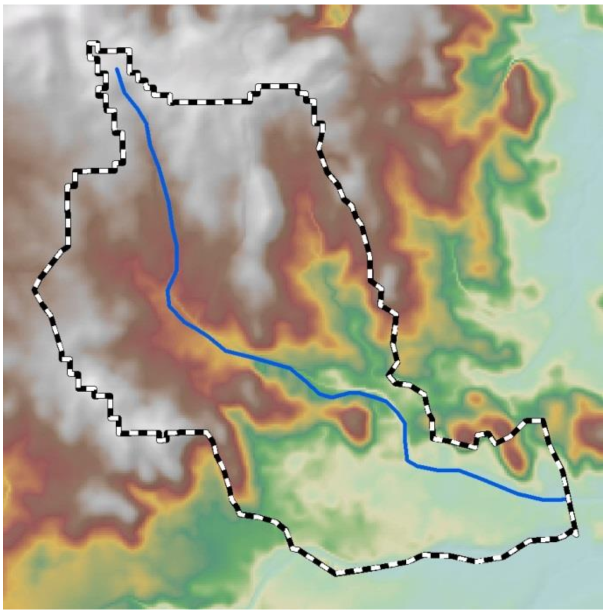


Figure H9 – Rational Method Catchment and Flowpath: Gooseponds Creek at Gooseponds Alert

Bakers Creek at Bakers Creek Alert

Table H10 – Rational Method Data and Results: Bakers Creek at Bakers Creek Alert

RATIONAL METHOD DATA	
Area of Catchment (km^2)	148.9
Length of Flowpath (km)	30.0
Time of Concentration (min)	407.2
Runoff Coefficient, 2% AEP (C_{50})	0.753
Discharge Estimate, 1EY (Q_1)	308.21
Discharge Estimate, 50% AEP (Q_2)	406.91
Discharge Estimate, 20% AEP (Q_5)	551.50
Discharge Estimate, 10% AEP (Q_{10})	643.27
Discharge Estimate, 5% AEP (Q_{20})	860.54
Discharge Estimate, 2% AEP (Q_{50})	1167.35
Discharge Estimate, 1% AEP (Q_{100})	1401.72

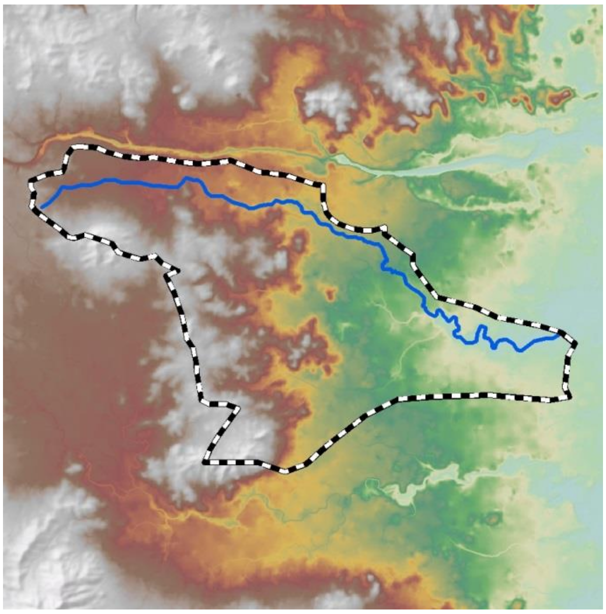


Figure H10 – Rational Method Catchment and Flowpath: Bakers Creek at Bakers Creek Alert

APPENDIX J: DNRM STREAM GAUGE STATION INFORMATION

125009A – Cattle Creek at Highams Bridge

Table J1 – Site Information: Cattle Creek at Highams Bridge (DNRM, 2017)

SITE INFORMATION	
Site	125009A Cattle_Ck Higham Bge
Site Name	Cattle Creek at Higham's Bridge
Commencement Date	19/06/2002
Cease Date	-
Grid Ref. Zone	Zone 55
Easting	671274.5
Northing	7662284.8
Grid Datum	MGA94 Map Grid of Australia 1994
Latitude	-21.1326805 21°07'57.6"S
Longitude	148.6492333 148°38'57.2"E
Lat/Long Datum	GDA94 Geodetic Datum of Australia 1994

Table J2 – Station Information: Cattle Creek at Highams Bridge (DNRM, 2017)

STATION INFORMATION	
Stream Distance	25 km from station to mouth
Zero Gauge	79.09
Datum	AHD Aust. Height Datum
Control	Rock Bar
Max Gauged Stage	1.998
Max Gauge Date	22/03/2012
Downstream from Dam	False
Min Peak Discharge	20
Time Between Peaks	1440 Mins
Catchment Area	198.000

Upstream

Downstream

Control | Site

Figure J1 - Site Photos: Cattle Creek at Highams Bridge (DNRM, 2017)

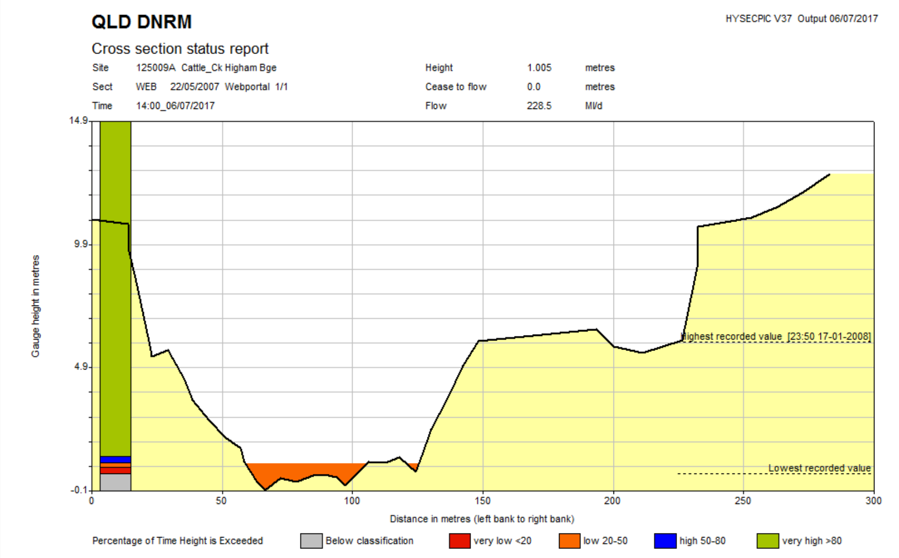


Figure J2 – Cross Section Status Report: Cattle Creek at Highams Bridge (DNRM, 2017)

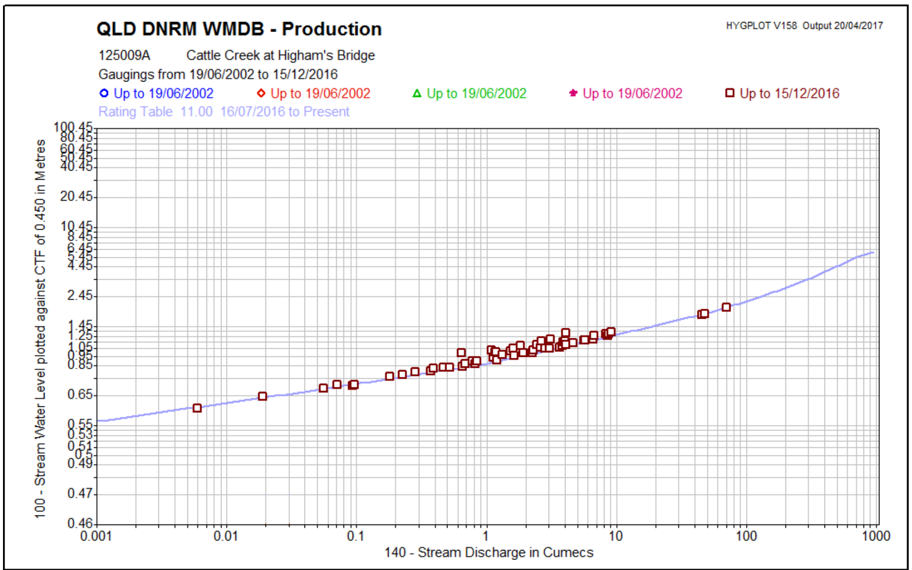


Figure J3 – Latest Rating Curve: Cattle Creek at Highams Bridge (DNRM, 2017)

125004B – Cattle Creek at Gargett

Table J3 – Site Information: Cattle Creek at Gargett (DNRM, 2017)

SITE INFORMATION	
Site	125004B Cattle_Ck Gargett
Site Name	Cattle Creek at Gargett
Commencement Date	03/07/1986
Cease Date	-
Grid Ref. Zone	Zone 55
Easting	681010.1
Northing	7657126.8
Grid Datum	MGA94 Map Grid of Australia 1994
Latitude	-21.178275 21°10'41.8"S
Longitude	148.7435527 148°44'36.8"E
Lat/Long Datum	GDA94Geodetic Datum of Australia 1994
Elevation	53

Table J4 – Station Information: Cattle Creek at Gargett (DNRM, 2017)

STATION INFORMATION	
Stream Distance	11 km from station to mouth
Zero Gauge	52.526
Datum	AHD Aust. Height Datum
Control	Control Weir
Max Gauged Stage	7.12
Max Gauge Date	04/04/1989
Downstream from Dam	False
Min Peak Discharge	20
Time Between Peaks	1440 Mins
Catchment Area	326.000


Upstream


Downstream

Control

Figure J4 - Site Photos: Cattle Creek at Gargett (DNRM, 2017)

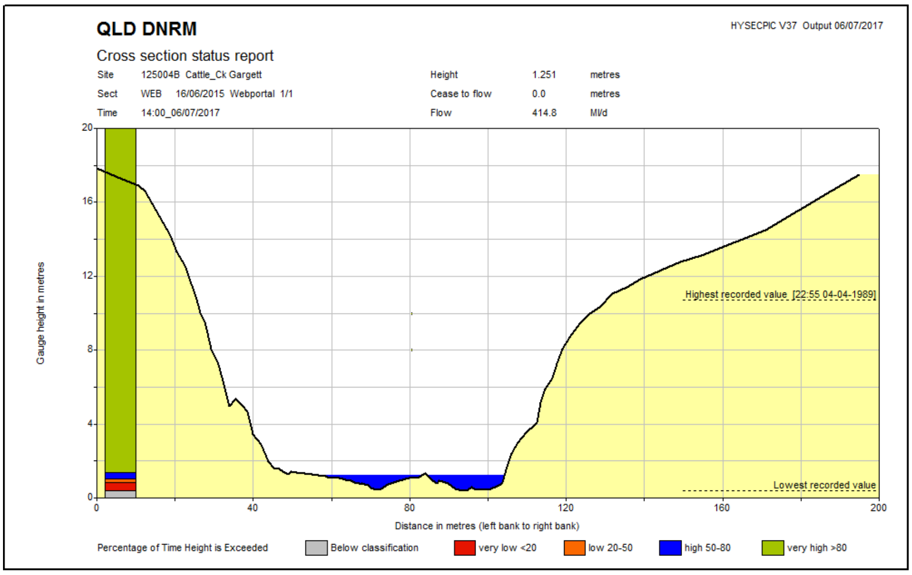


Figure J5 – Cross Section Status Report: Cattle Creek at Gargett (DNRM, 2017)

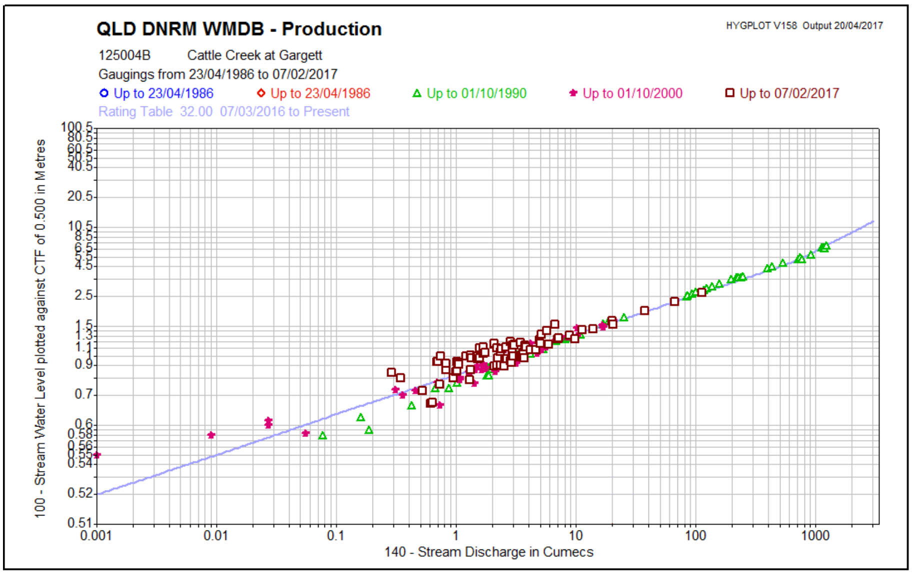


Figure J6 – Latest Rating Curve: Cattle Creek at Gargett (DNRM, 2017)

125005A – Blacks Creek at Whitefords

Table J5 – Site Information: Blacks Creek at Whitefords (DNRM, 2017)

SITE INFORMATION	
Site	125005A Blacks_Ck Whitefords
Site Name	Blacks Creek at Whitefords
Commencement Date	12/12/1973
Cease Date	-
Grid Ref. Zone	Zone 55
Easting	690333.2
Northing	7641009.8
Grid Datum	MGA94 Map Grid of Australia 1994
Latitude	-21.3228 21°19'22.1"S
Longitude	148.8351167 148°50'06.4"E
Lat/Long Datum	GDA94Geodetic Datum of Australia 1994
Elevation	58

Table J6 – Station Information: Blacks Creek at Whitefords (DNRM, 2017)

STATION INFORMATION	
Stream Distance	64.9 km from station to mouth
Zero Gauge	57.702
Datum	AHD Aust. Height Datum
Control	Sand Gravel
Max Gauged Stage	7.102
Max Gauge Date	23/02/2000
Downstream from Dam	False
Min Peak Discharge	20
Time Between Peaks	1440 Mins
Bed Slope	0.001
Catchment Area	509.400


Upstream


Downstream

Figure J7 - Site Photos: Blacks Creek at Whitefords (DNRM, 2017)

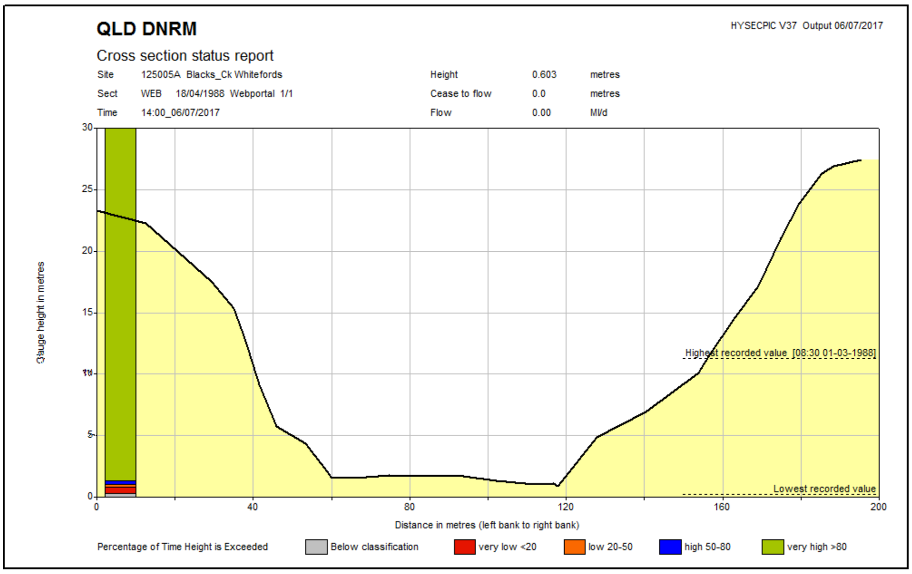


Figure J8 – Cross Section Status Report: Blacks Creek at Whitefords (DNRM, 2017)

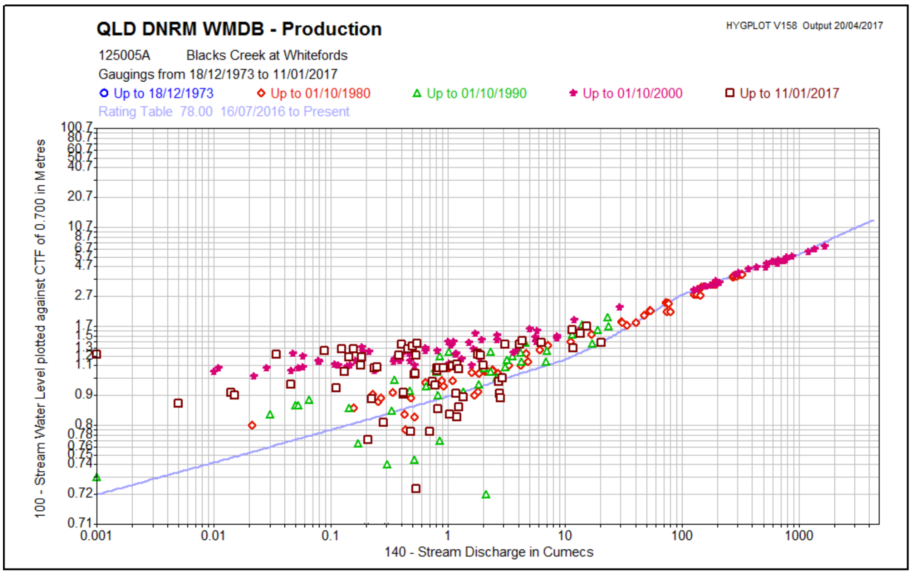


Figure J9 – Latest Rating Curve: Blacks Creek at Whitefords (DNRM, 2017)

125002C – Pioneer River at Sarichs

Table J7 – Site Information: Pioneer River at Sarichs (DNRM, 2017)

SITE INFORMATION	
Site	125002C Pioneer_R Sarich's
Site Name	125002C Pioneer_R Sarich's
Commencement Date	17/02/1958
Cease Date	-
Grid Ref. Zone	Zone 55
Easting	688860.5
Northing	7646611.1
Grid Datum	AMG84 Australian Map Grid 1984
Latitude	-21.27237 21°16'20.5"S
Longitude	148.8203 148°49'13.1"E
Lat/Long Datum	GDA94Geodetic Datum of Australia 1994
Elevation	48

Table J8 – Station Information: Pioneer River at Sarichs (DNRM, 2017)

STATION INFORMATION	
Stream Distance	57.7 km from station to mouth
Zero Gauge	48.013
Datum	AHD Aust. Height Datum
Control	Sand & Rock
Max Gauged Stage	9.626
Max Gauge Date	16/02/1968
Downstream from Dam	False
Min Peak Discharge	20
Time Between Peaks	1440 Mins
Bed Slope	0.0018
Catchment Area	757.000

Upstream

Downstream

Site

Figure J10 - Site Photos: Pioneer River at Sarichs (DNRM, 2017)

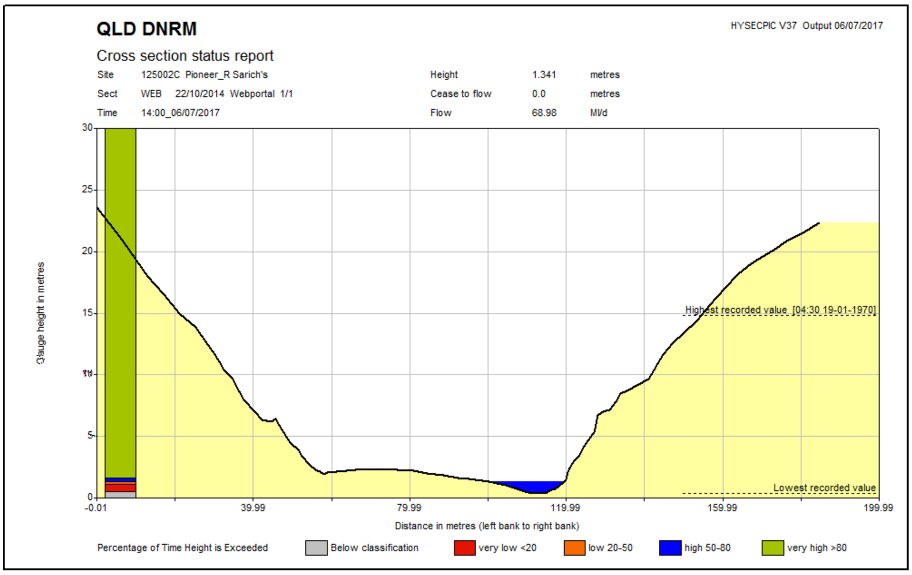


Figure J11 – Cross Section Status Report: Pioneer River at Sarichs (DNRM, 2017)

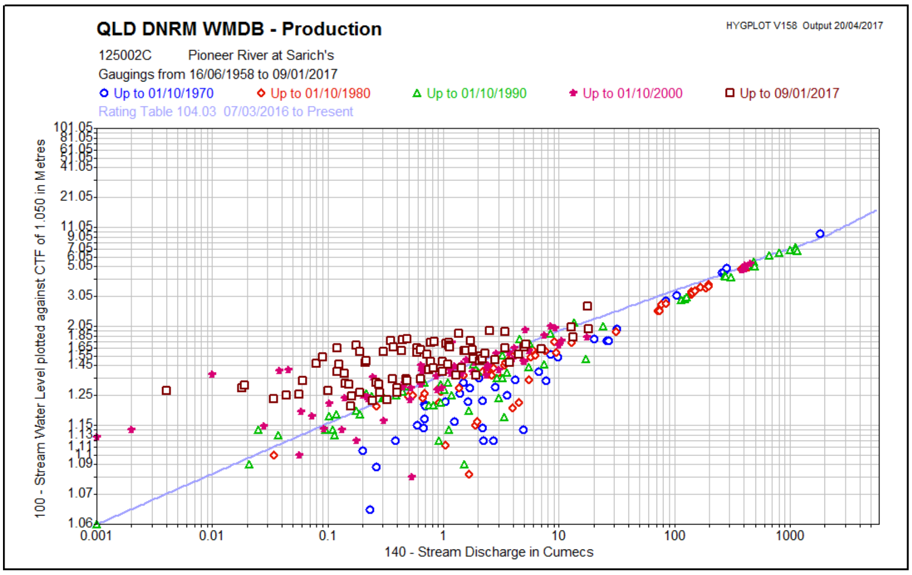


Figure J12 – Latest Rating Curve: Pioneer River at Sarichs (DNRM, 2017)

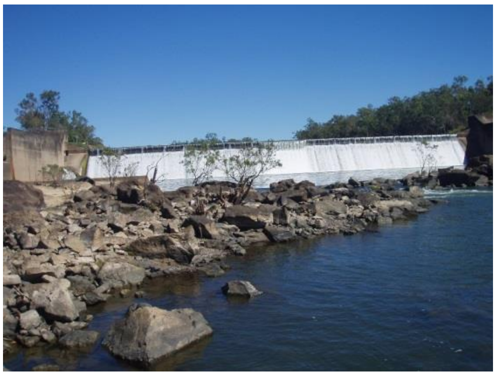
125007A – Pioneer River at Mirani Weir Tailwater

Table J9 – Site Information: Pioneer River at Mirani Weir Tailwater (DNRM, 2017)

SITE INFORMATION	
Site	125007A Mirani Weir TW
Site Name	Pioneer River at Mirani Weir Tailwater
Commencement Date	09/11/1977
Cease Date	-
Grid Ref. Zone	Zone 55
Easting	690101.3
Northing	7657144.1
Grid Datum	AMG84 Australian Map Grid 1984
Latitude	-21.17712 21°10'37.6"S
Longitude	148.83108 148°49'51.9"E
Lat/Long Datum	GDA94Geodetic Datum of Australia 1994
Elevation	34

Table J10 – Station Information: Pioneer River at Mirani Weir Tailwater (DNRM, 2017)

STATION INFORMATION	
Stream Distance	45.7 km from station to mouth
Zero Gauge	34.267
Datum	AHD Aust. Height Datum
Control	Control Weir
Max Gauged Stage	10.79
Max Gauge Date	06/02/1979
Downstream from Dam	False
Min Peak Discharge	20
Time Between Peaks	1440 Mins
Bed Slope	0.0013
Catchment Area	1211.000



Upstream

Downstream

Figure J13 - Site Photos: Pioneer River at Mirani Weir Tailwater (DNRM, 2017)

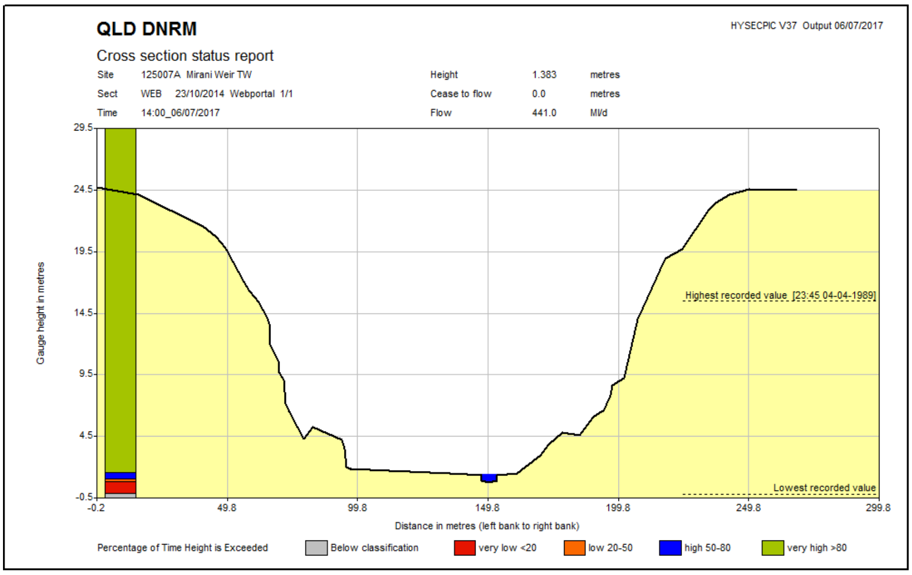


Figure J14 – Cross Section Status Report: Pioneer River at Mirani Weir Tailwater (DNRM, 2017)

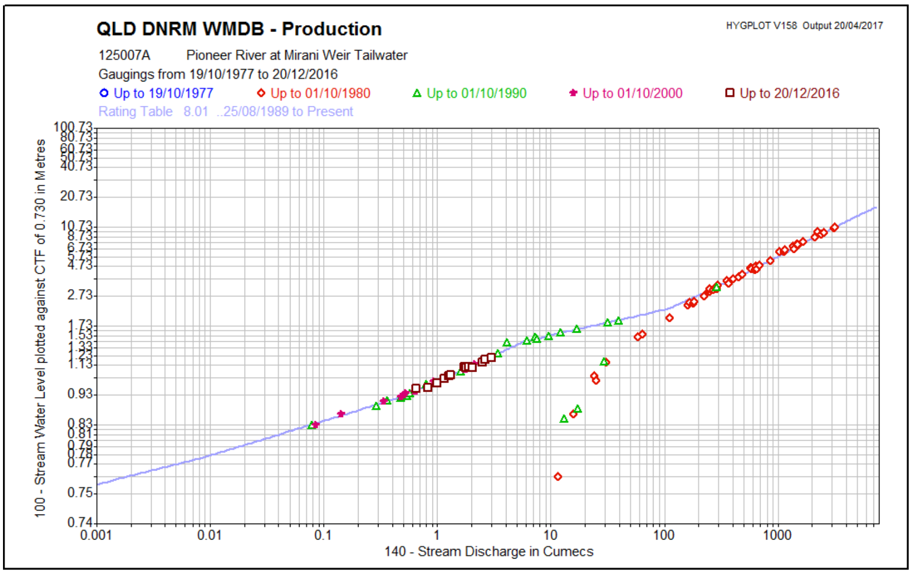


Figure J15 – Latest Rating Curve: Pioneer River at Mirani Weir Tailwater (DNRM, 2017)

125016A – Pioneer River at Dumbleton Weir Tailwater

Table J11 – Site Information: Pioneer River at Dumbleton Weir Tailwater (DNRM, 2017)

SITE INFORMATION	
Site	125016A Dumbleton T/W
Site Name	Pioneer River at Dumbleton Weir T/W
Commencement Date	22/12/2005
Cease Date	-
Grid Ref. Zone	Zone 55
Easting	715595.4
Northing	7660735.2
Grid Datum	MGA94 Map Grid of Australia 1994
Latitude	-21.1419305 21°08'30.9"S
Longitude	149.0760833 149°04'33.9"E
Lat/Long Datum	GDA94Geodetic Datum of Australia 1994
Elevation	10

Table J12 – Station Information: Pioneer River at Dumbleton Weir Tailwater (DNRM, 2017)

STATION INFORMATION	
Stream Distance	16.6 km from station to mouth
Datum	AHD Aust. Height Datum
Control	Rock
Max Gauged Stage	13.8
Max Gauge Date	02/02/2007
Downstream from Dam	False
Catchment Area	1488.000

Upstream

Downstream

Control

Figure J16 - Site Photos: Pioneer River at Dumbleton Weir Tailwater (DNRM, 2017)

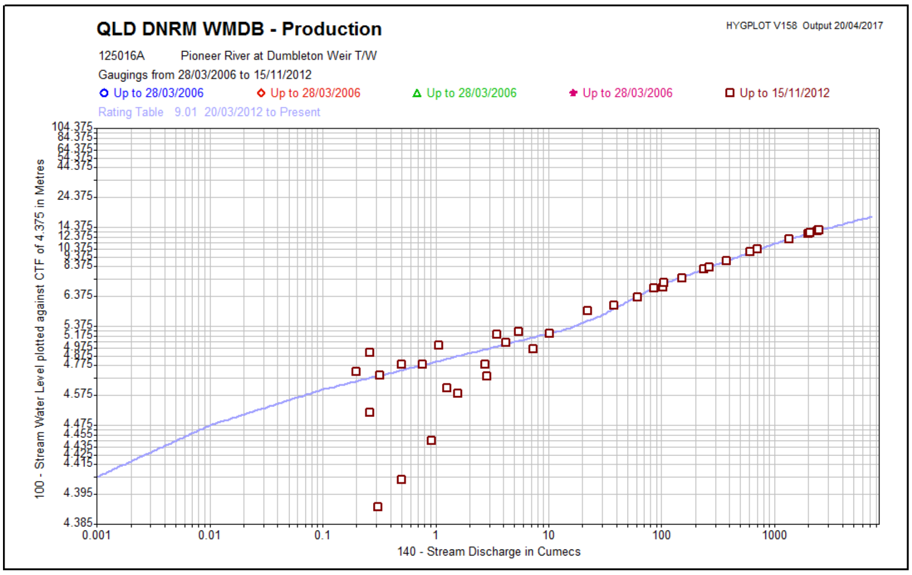


Figure J17 – Latest Rating Curve: Pioneer River at Dumbleton Weir Tailwater (DNRM, 2017)