Seaspray Flood Study – Hydrology (R02)

March 2016

DOCUMENT STATUS

Version	Doc type	Reviewed by	Approved by	Distributed to	Date issued
V01	Report			WGCMA	30/01/2015
v02	Report			WGCMA	19/03/2015
v03	Final			WGCMA	03/03/2016

PROJECT DETAILS

Project Name	3569-03 Seaspray Flood Study
Client	West Gippsland Catchment Management Authority
Client Project Manager	
Water Technology Project Manager	
Report Authors	
Job Number	3569-03
Report Number	R02
Document Name	3569-03_R02v03b

Cover Photo: Floodway Regulator, Seaspray

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1. INTRODUCTION

Water Technology was commissioned by the West Gippsland CMA to undertake the Seaspray Flood Investigation. This project includes detailed hydrological and hydraulic modelling of Merriman Creek, taking into account the influences of ocean generated flooding and potential inflows from Lake Reeve, flood mapping of the Seaspray township and will provide recommendations for flood mitigation actions and works.

This report details the hydrological analysis undertaken for the project, and should be read in conjunction with the previous Data Review Report (Report 1).

2. STUDY AREA

Seaspray is a coastal town with 320 permanent residents which rises to more than 1,000 during summer. The township lies at the western end of the Ninety Mile Beach west of the Gippsland Lakes and approximately 200 km east of Melbourne. As detailed in the Urban Design Framework developed for the township by Wellington Shire Council (Wellington Shire Council, 2007), Seaspray is an important access point to the 90 Mile Beach Area and the extensive areas of small lot subdivision that extend easterly for 28km to Golden Beach and Paradise Beach.

The town is located at the downstream end of the Merriman Creek catchment, with the creek flowing along the western boundary of the township, before it passes through the 90 Mile Beach coastal dune system and enters the ocean. The study area is shown in Figure 2-1.

Figure 2-1 Seaspray Study Area

3. HYDROLOGY

3.1 Overview

The hydrological analysis for Seaspray consisted of a review of the hydrological context of the study area followed by flood frequency analysis and hydrologic modelling using RORB. The RORB model was calibrated on three gauged events and parameter sets for design were validated against flood frequency analysis and regional flow estimates. Due to uncertainty in gauged peak flow estimates, a review was undertaken of the rating curve for the Prospect Road gauge (227240) and changes to the rating curve were adopted for the 1993 flood event with a subsequent reduction of the peak flow estimate for that event.

3.2 Catchment Description

The Merriman Creek catchment has an area of 500 km^2 extending 50 km westward from the estuary mouth at Seaspray to a maximum elevation of 750 m AHD at Mount Tassie (Figure 3-1). The catchment consists of two main streams, Merriman Creek and Monkey Creek, whose confluence is just upstream of Giffard Road. A third minor catchment feeds a wetland swamp with significant storage east of Giffard Road. This wetland is unlikely to overflow into the Merriman Creek catchment in large events (up to at least the 2% AEP), with no outflow occurring in the RORB model (which has the storage explicitly included) for either the 1993 or 1995 calibration events.

The main stream length from the catchment divide to the mouth is 107 km with an equal area slope of 2.0 m/km.

The average annual rainfall at Seaspray is 620 mm. A steep rainfall gradient exists over the catchment with average annual rainfall reaching 1,500 mm in the headwaters, while at the catchment centroid the average annual rainfall is around 670 mm. Effectively the lower catchment is within the rain shadow of the Strzelecki Ranges.

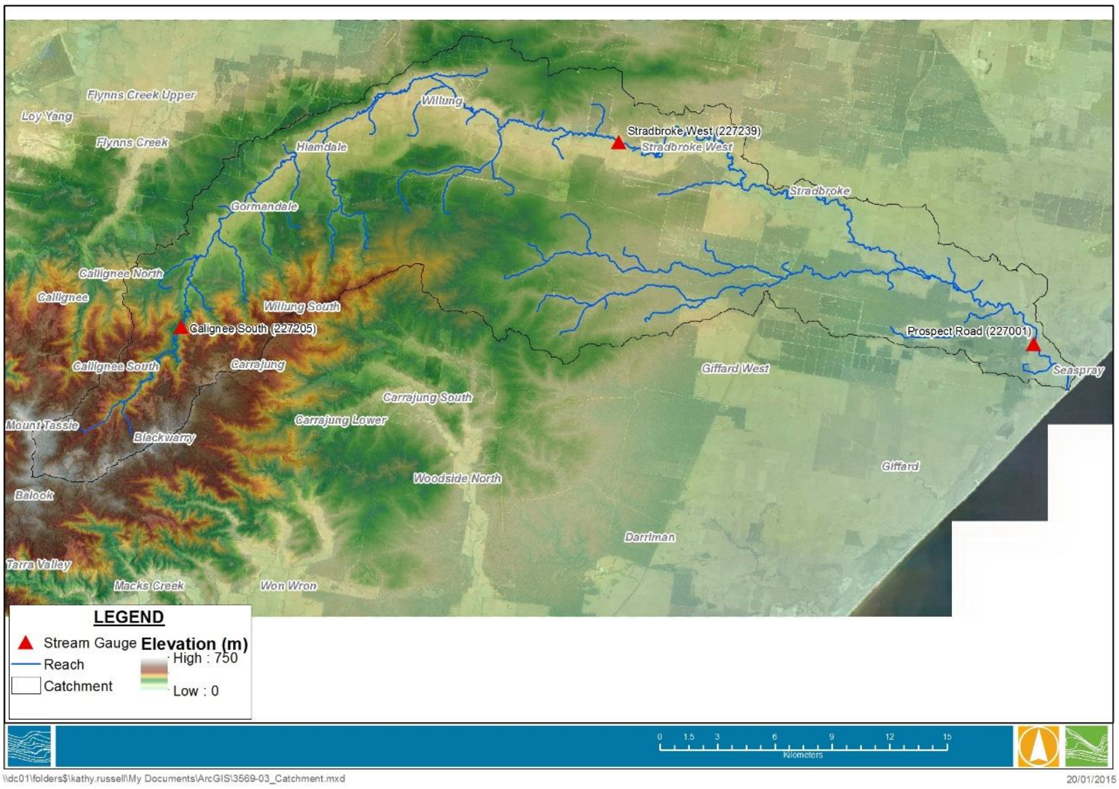

Figure 3-1 Merriman Creek Catchment

3.3 Design Rainfall Estimates

Design rainfalls were calculated for the 10%, 5%, 2%, 1% and 0.5% AEP events using the Intensity-Frequency-Duration analysis from AR&R (1987). The IFD parameters (Table 3-1) were obtained from the Bureau of Meteorology's IFD program website (www.bom.gov.au/ifd) for the catchment centroid.

Table 3-1 IFD parameters

2yr (50% AEP) 1hr rainfall intensity (mm/hr)	19.49
2yr (50% AEP) 12hr rainfall intensity (mm/hr)	4.42
2yr (50% AEP) 72hr rainfall intensity (mm/hr)	1.19
50yr (2% AEP) 1hr rainfall intensity (mm/hr)	40.02
50yr (2% AEP) 12hr rainfall intensity (mm/hr)	7.45
50yr (2% AEP) 72hr rainfall intensity (mm/hr)	2.49
Skew	0.36
F2	4.22
F50	15.22

Comparisons were made with the new IFDs developed for Project 1 of the Australian Rainfall and Runoff Revision Project (http://www.bom.gov.au/water/designRainfalls/revised-ifd/). On average the new IFD parameters were around 9% higher than ARR87 (for 1%-10% AEP events), with some AEP/Duration values up to 20% higher.

The 1987 IFD parameters were adopted to be consistent with existing methods for determining areal reduction factors, design losses and hydrologic parameter estimates, which were developed with the older IFD information.

Unfiltered temporal patterns from AR&R (1987) for Zone 1 were applied for the design events, as recommended in Hill et al. (1998) for use with their design losses. Rainfall depths were applied uniformly across the catchment.

Areal reduction factors from Siriwardena and Weinmann (1996) were applied to the point design rainfall estimates. These areal reduction factors are recommended for use in Victoria instead of the original AR&R values (Hill et al. 1998). These have now been recommended for adoption in the ARR Project 2 Stage 2 report (Engineers Australia, 2012).

3.4 Gauge Review

Three stream gauges exist within the Seaspray Catchment along Merriman Creek. Each of these gauges record instantaneous flow data over various time periods as listed below. Prospect Road Seaspray is the closest gauge to the Seaspray Flood Study mapping area, and the design flows determined at this location provide the design event inflows to the hydraulic model. Therefore the Prospect Road Gauge was the focus of the Flood Frequency Analysis (FFA) and for calibration of the hydrologic rainfall-runoff model. The Stradbroke West Gauge is located centrally within the catchment and was used in combination with the Prospect Road gauge to calibrate the hydrologic model. The Calignee South Gauge, as shown in Figure 3-1, is located in the upper reaches of the catchment and was not used specifically within the hydrologic assessment.

Table 3-2 Streamflow Gauges in Merriman Creek catchment

Gauge No.	Location	Period	Years	Catchment Area (km^2)
227240	Merriman Creek @ Prospect Road Seaspray	1983-2014	31	529
227001	Merriman Creek @ Seaspray (old Prospect Road site)	1966-1971	5	525
227239	Merriman Creek @ Stradbroke West	1983-2014	31	256
227205	Merriman Creek @ Calignee South	1946-1952 and 1965-2014	54	36

3.4.1 Merriman Creek at Prospect Road

The Merriman Creek at Prospect Road gauge (227240) has 31 years of instantaneous flow data. There were also 5 years of data from 1966-1971 at a previous gauging station (227001, referred to as Merriman Creek at Seaspray) located near the current Prospect Road gauge. Little is known about the reliability of the rating at the old gauge site, although numerous physical gauging measurements were undertaken for flows up to around 1,470 ML/d. The rating for the current gauge is officially considered reliable up to flows of 8,210 ML/d, however the small number of physical gauging measurements show considerable scatter across the whole range, particularly for low flows. Therefore flow estimates are likely to have a high degree of uncertainty.

Plotting of the published water levels and flows from the gauge clearly show a number of rating curves were used over the 32 year period (Figure 3-4). This is because the creek at this location has a sand bed which has been subject to significant instability over the record period. A changing water level – discharge relationship could be expected as the sand bed accretes and erodes.

The long term water level record (Figure 3-5) was also reviewed and it was found that following the 1993 event, gauged levels were elevated, even during low flow periods, for a number of years. The low flow water level, after suddenly increasing in 1993, slowly settled back to current levels over a period of around ten years. This is indicative of sand accretion in the channel after the 1993 event. The 1995 rating curve had a lower predicted flow for a given water level than the rating curve used in 1993, such that the recorded 1995 peak flow was lower than the 1993 peak, even though the water level was higher.

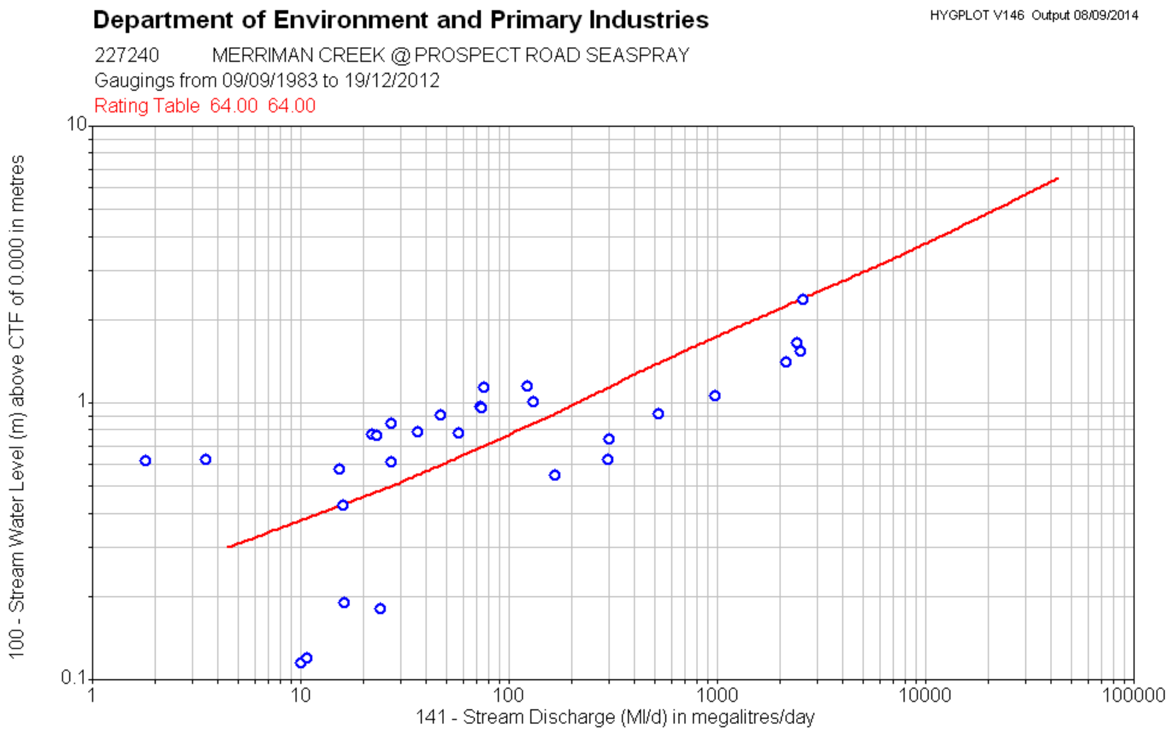

Figure 3-2 Rating Curve for 227240 Merriman Creek @ Prospect Road

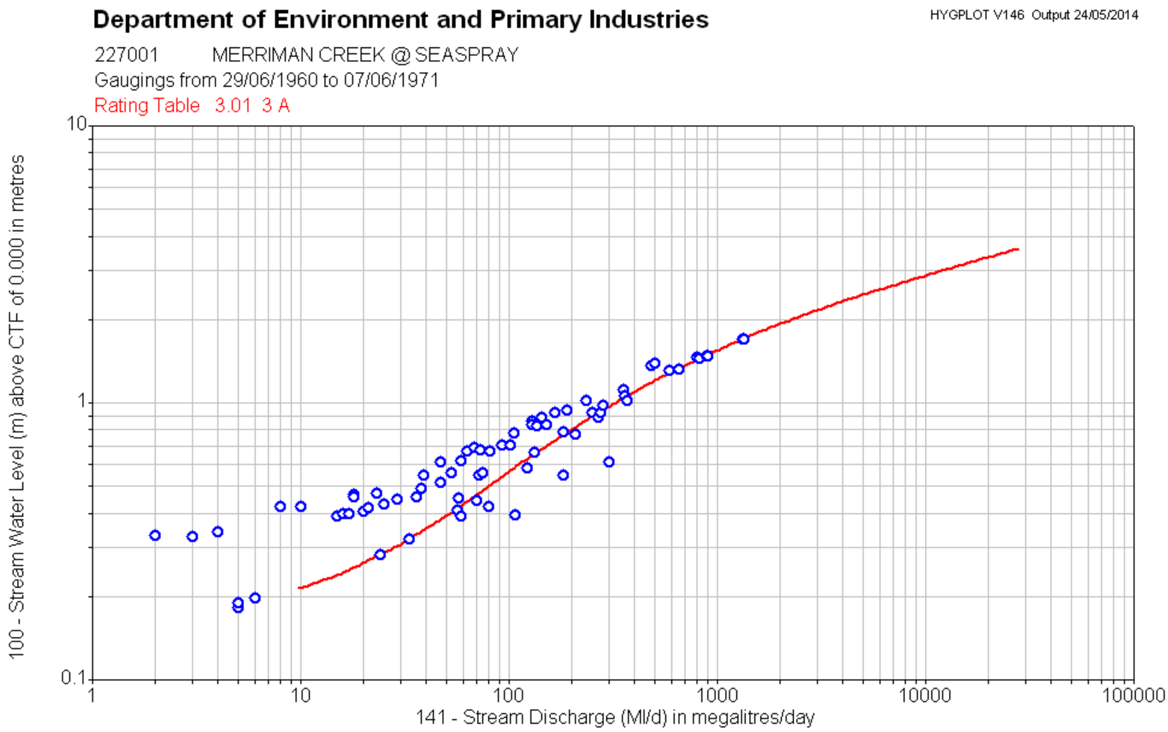

Figure 3-3 Rating Curve for 227001 Merriman Creek @ Seaspray

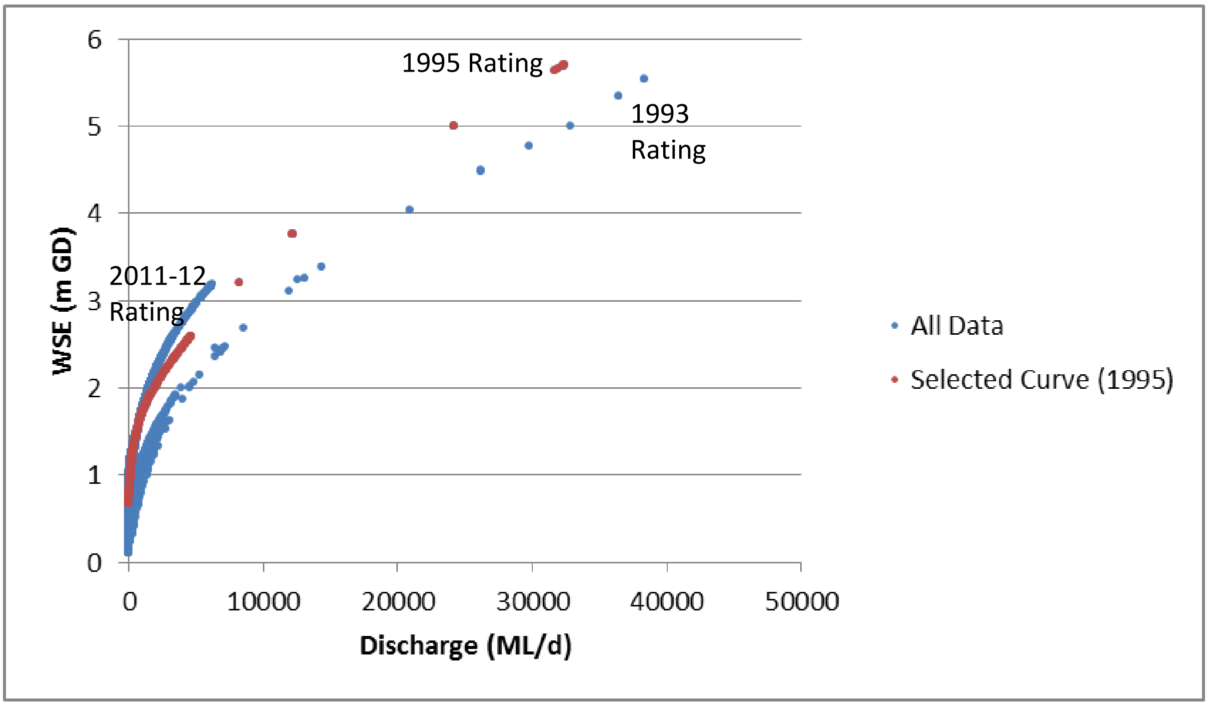

Figure 3-4 **Water Level vs. Discharge Plot for Prospect Road gauge (227240) showing variation in rating curves across period of record**

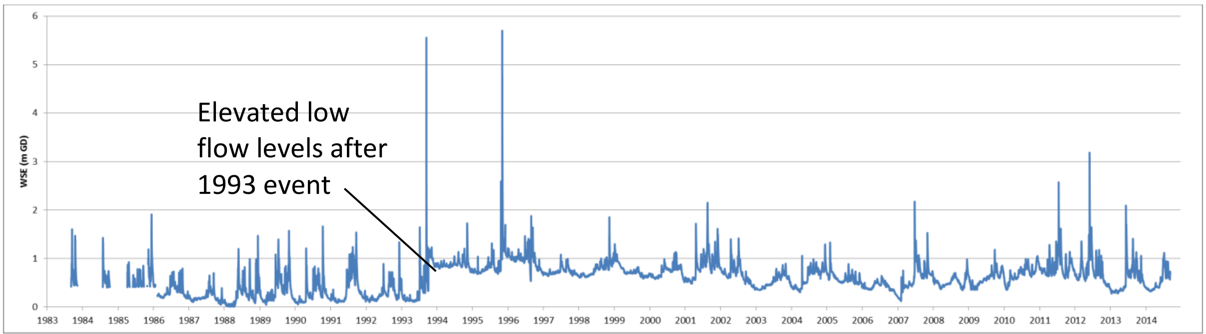

Figure 3-5 **Merriman Creek at Prospect Road Gauge Levels**

Two dimensional hydraulic modelling of the gauge reach was undertaken to verify the appropriateness of the rating curves for the calibration events. The 1995 event was run with the published flows and successfully reproduced the gauged water levels (Figure 3-6). Initial modelling of the 1993 event in the hydraulic model was not capable of reproducing the gauged levels with the gauged flows, with reasonable hydraulic parameters. The modelled water level over-predicted the measured peak water level by almost half a metre when published flows were used (Figure 3-7). A test was done with the use of the 1995 rating curve (i.e. reduced flows) to estimate a peak flow for the 1993 event, for which the model was able to reproduce the peak water levels. It is therefore recommended that due to likely changes that occurred in the physical characteristics of the gauge site during the 1993 event that the 1993 rating curve be applied before the peak, transitioning to the 1995 rating curve at and after the peak. The revised peak flow for 1993 using this approach is 30,240 ML/d, compared to the previous estimate of 38,360 ML/d.

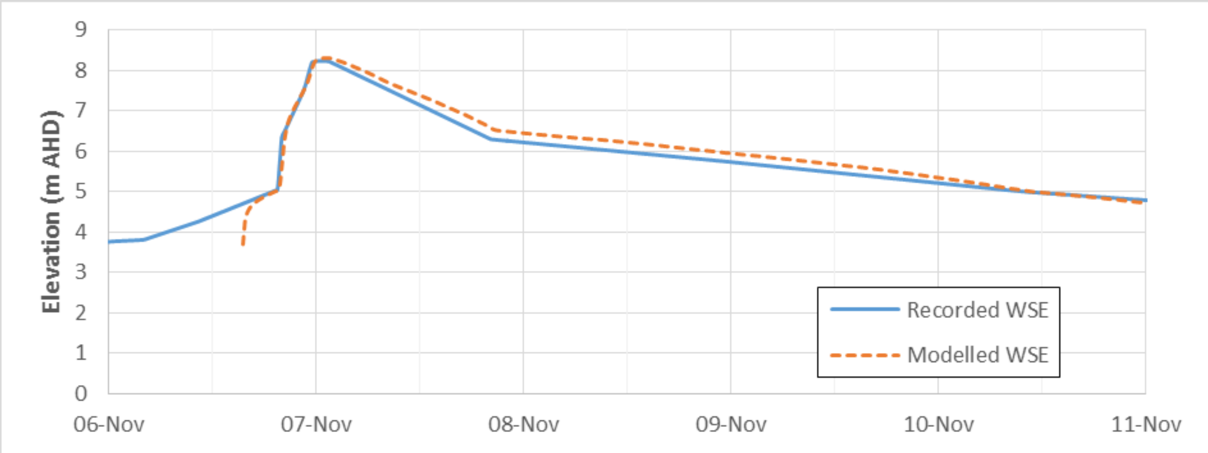

Figure 3-6 1995 Calibration Period Recorded and Modelled Water Surface Elevation at Gauge

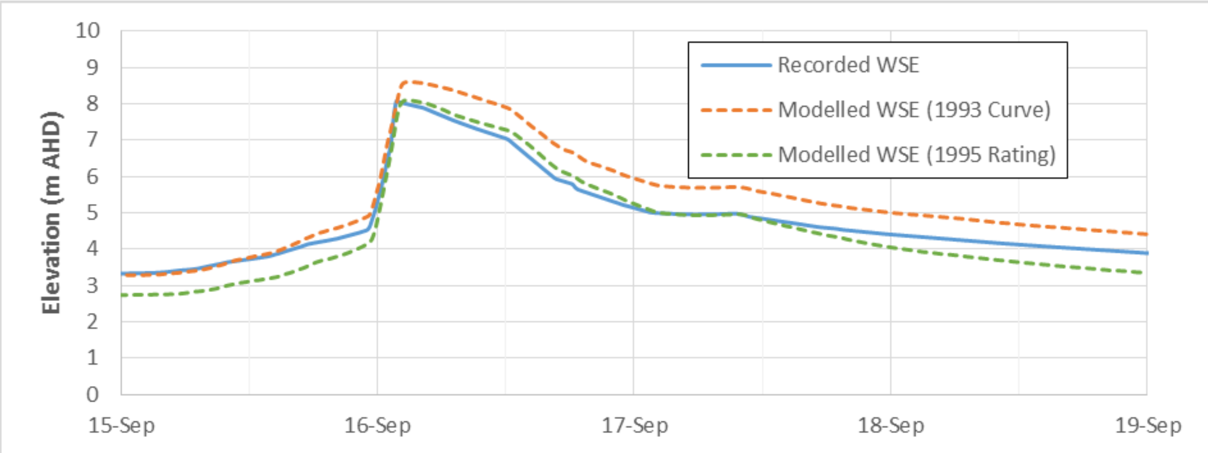

Figure 3-7 1993 Calibration Period Recorded and Modelled Water Surface Elevation at Gauge

3.4.2 Merriman Creek at Stradbroke West

The Merriman Creek at Stradbroke West gauge (227239) has 31 years of flow data from 1983-2014. The rating curve is based on a small number of high flow gauging measurements with very little scatter (Figure 3-8), and is therefore expected to be more reliable for estimating high flows than the Prospect Road gauge.

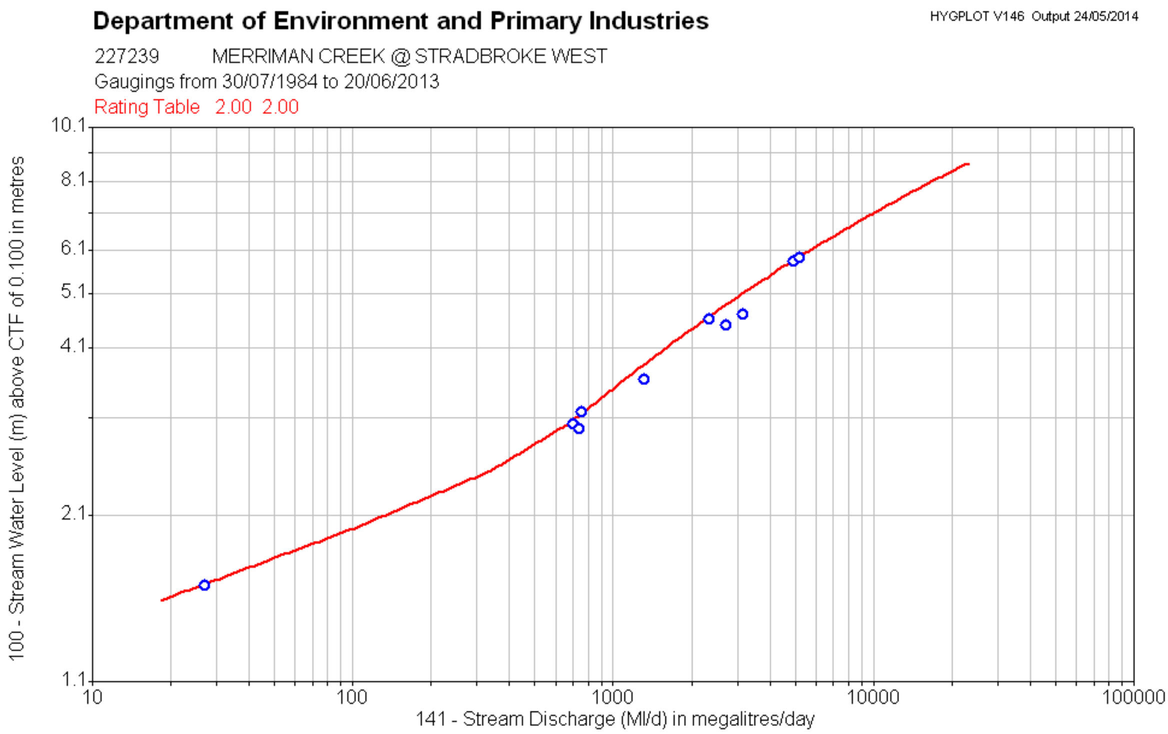

Figure 3-8 **Rating Curve for 227239 Merriman Creek at Stradbroke West Gauge**

3.5 Flood Frequency Analysis

3.5.1 Overview

Flood frequency analysis (FFA) allows the estimation of peak flows of selected Annual Exceedance Probability (AEP) events based on a statistical analysis. FFA was undertaken for Merriman Creek at the Prospect Road gauge (no. 227240) in order to provide peak flow estimates at Seaspray for verification of the RORB hydrologic model results.

The FFA was undertaken on the annual flow series from the gauge records using the analysis program FLIKE, as recommended in the updated ARR guidelines. FLIKE uses a Bayesian approach to parameter fitting (either the Global Probabilistic or Quasi-Newton fitting algorithms) to the records in order to assess the return period of different magnitude flows. There are a number of probability distributions which can be used to undertake an FFA, including the Log Pearson III, Log-Normal, Generalised Pareto, Generalised Extreme Value and Gumbel distributions and a selection of these are applied in the analysis, with the 'best fit' distribution adopted in the final assessment.

The flow series analysis assumed that the flow year was aligned with the calendar year which is appropriate given the typically wet winter and dry summer climate of the region.

3.5.2 Merriman Creek at Prospect Road

Annual maximum flows were extracted from the gauge record for Merriman Creek at Prospect Road gauge (227240) and are listed in Table 3-3. The 1993 peak flow was adjusted down from its gauged peak of 38,330 ML/d to 30,270 ML/d as per the rating curve review detailed in Section 3.4.

Table 3-3 Summary of annual peak flows for Merriman Creek at Prospect Road

Year	Max Flow (ML/d)	Year	Max Flow (ML/d)	Year	Max Flow (ML/d)
1952	16,760*	1989	2,150	2003	120
1966	800	1990	2,470	2004	360
1967	1,210	1991	2,030	2005	410
1968	1,410	1992	1,420	2006	40
1969	13,480	1993	30,270**	2007	1,950
1970	11,360	1994	1,050	2008	50
1971	28,150	1995	32,410	2009	290
1978	28,510*	1996	1,470	2010	130
1983	2,730	1997	200	2011	3,210
1984	2,080	1998	1,390	2012	6,190
1985	3,480	1999	310	2013	1,740
1986	370	2000	220	2014	290
1987	280	2001	1,880		
1988	1,810	2002	520		

**Estimated in 1980 flood study (Camp Scott Furphy 1980)*

***Reduced from 38,330 ML/d gauged peak to 30,270 ML/d as per gauge review*

Historic floods are known to have occurred in 1880, 1951, 1952, and 1978. These are floods that are known to have occurred within the catchment, but which have not been captured in the gauged record. The known information about these floods is shown in Table 3-4. The floods with known magnitude have been included in the annual series with the remaining years accounted for as peaks under and over a threshold as shown in Table 3-5. The 1880 flood is assumed to have been larger than the 1952 flood although this is not certain.

Table 3-4 Summary of historic flooding in Seaspray

Year	Comment
1880	Larger than 1951 flood
1951	Reported as the worst flood in 60 years
1952	Larger than 1951 flood. Unknown whether larger than 1880 flood. Discharge estimated at 16,760 ML/d (Camp Scott Furphy 1980)
1978	Reported in 1980 as the largest flood on record. Unknown whether larger than 1880 flood. Discharge estimated at 28,510 ML/d (Camp Scott Furphy 1980)

Table 3-5 Treatment of historic flood peak threshold

Threshold (ML/d)	Peaks above threshold	Peaks below threshold
16,760	1 (1880 flood. 1952 and 1978 peaks included explicitly in annual series)	94

The FFA was undertaken with (Figure 3-10) and without (Figure 3-9) historic information for comparison. The inclusion of historic events (1952 – 1983) produced a much narrower 90% confidence limit and a significantly flatter plot, but with a similar 10% AEP flow estimate. The FFA with historic information results in a 1% AEP flow estimate of 41,130 ML/d (476 m^3/s) with 5-95% confidence limits of 23,070 to 90,200 ML/d (267 to 1,044 m^3/s).

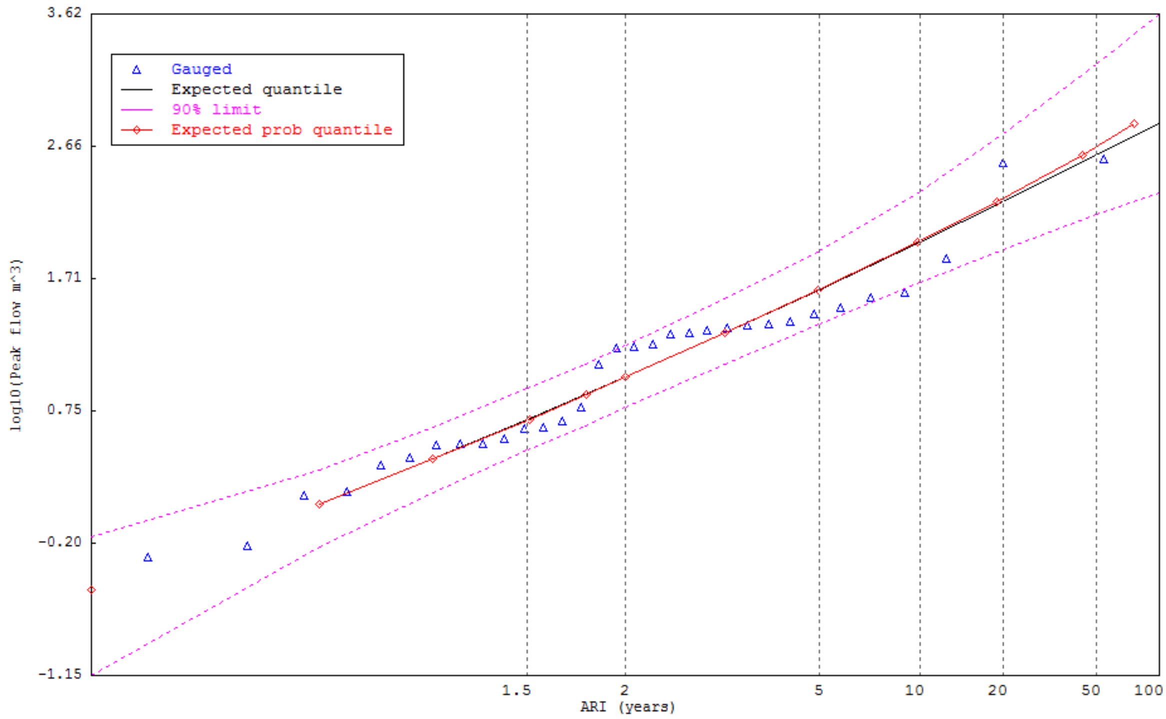

Figure 3-9 Log Pearson III distribution fitted to annual series for Merriman Creek at Prospect Road (gauge data only)

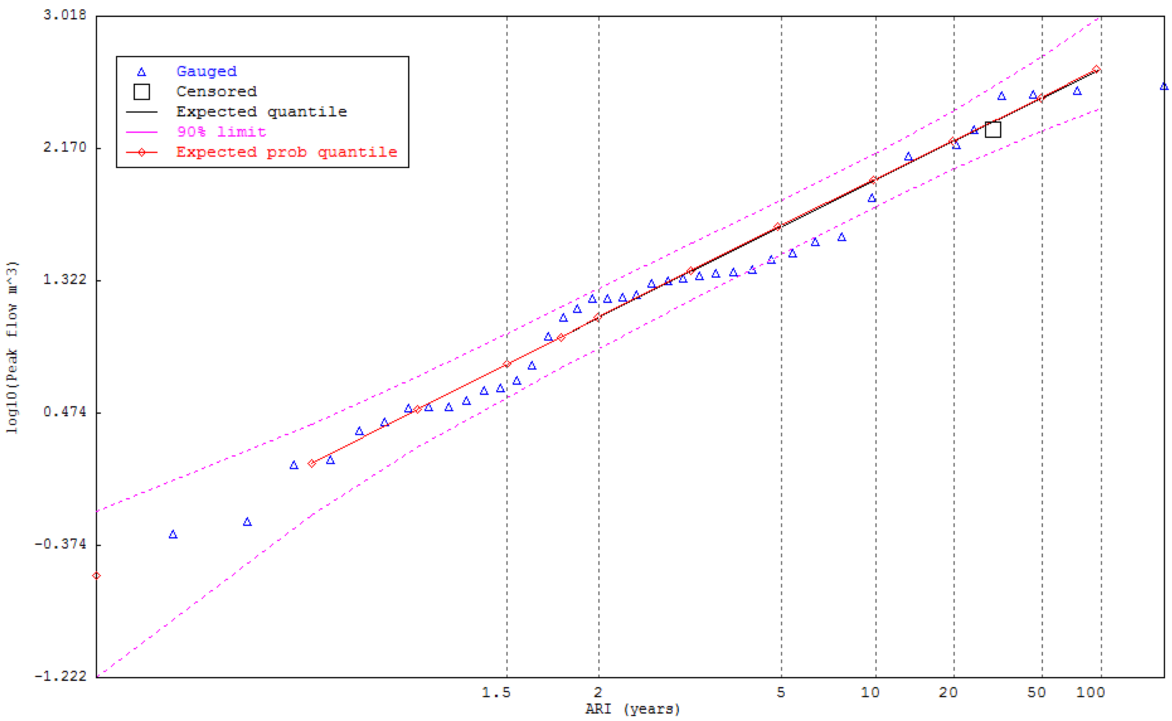

Figure 3-10 Log Pearson III distribution fitted to annual series for Merriman Creek at Prospect Road (with historic information included)

Table 3-6 FFA Peak Flow Estimates for Merriman Creek at Prospect Road

AEP	Peak Flow Estimate (ML/d)		Peak Flow Estimate (ML/d)	
	Log Pearson III (1983 – 2014)	5-95% Confidence Limits	Log Pearson III (with historic information)	5-95% Confidence Limits
10%	8,040	(4,150 - 18,840)	8,120	(5,440 - 11,840)
5%	15,720	(7,080 - 48,470)	14,260	(9,420 - 22,380)
2%	34,210	(12,700 - 15,290)	26,960	(16,500 - 50,110)
1%	58,150	(18,230 - 35,600)	41,130	(23,070 - 90,200)
0.5%	95,390	(25,140 - 78,110)	60,570	(30,150 - 160,190)

3.5.3 Comparison to Regional and Previous Estimates

Estimates of peak flows using the Australian Regional Flood Frequency (ARFF) method and from previous studies by Cardno (2010) and Camp Scott Furphy (1980) are shown in Table 3-7.

The Cardno (2010) estimate aligns closely to that of the FFA without historic data (i.e. only 1983 – 2014), while the Camp Scott Furphy (1980) estimate is much flatter (as it does not include the 1993 or 1995 floods) and aligns closely with the ARFF peak flow. The FFA estimate with historic information included lies between the two.

The current FFA with historic information included is broadly consistent with previous work, however it provides more credible estimates than the previous analyses, as it includes much more gauged data than the 1980 analysis, as well as historical information on flooding prior to gauge records (which was not included in the 2010 analysis).

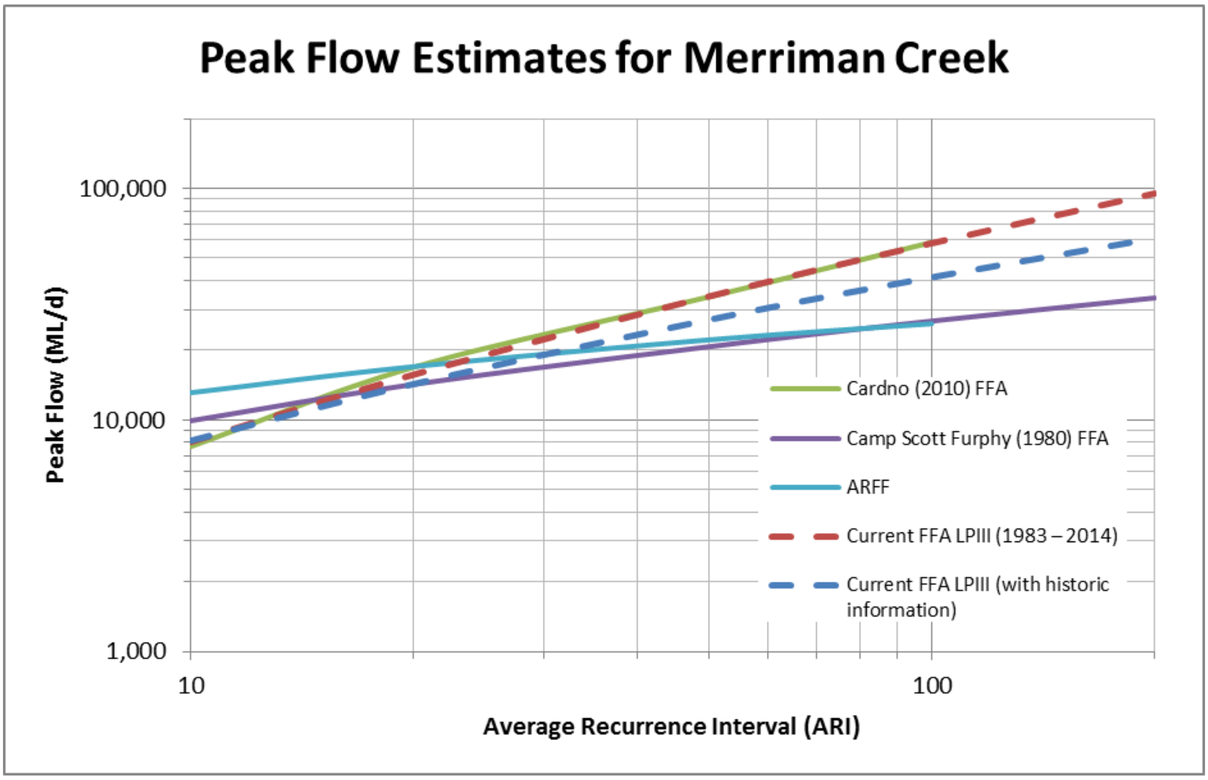

Figure 3-11 Comparison of Peak Flow Estimate Methods

Table 3-7 ARFF Flow Estimates and previous Flood Frequency Estimates for Prospect Road gauge

AEP	Peak Design Flow (ML/d) ARFF	Peak Design Flow (ML/d) (Cardno, 2010)	Peak Design Flow (ML/d) (Camp Scott Furphy, 1980)
10%	13,130	7,690	9,940
5%	17,020	16,930	14,260
2%	22,200	34,040	20,740
1%	26,090	58,580	26,780
0.2%	-	-	33,700

3.6 Hydrologic Modelling

3.6.1 Overview

A hydrological model of the catchment was developed for the purpose of extracting design flows to be used as boundary conditions in the Seaspray hydraulic model. The rainfall-runoff program, RORB (Version 6) was used for this study.

RORB is a non-linear rainfall runoff and streamflow routing model which is used for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas, connected by a series of conceptual reach storages. Design storm rainfall is input to the centroid of each subarea. Specified losses are then deducted, and the excess routed through the reach network.

There are two streamflow gauges within the catchment which were used to calibrate the RORB model. The Merriman Creek gauge at Prospect Road is located only 9 km from Seaspray and has 31 years of flow records from 1983-2014, covering the 1993, 1995 and 2012 floods and other smaller floods. As noted in Section 3.4 the accuracy of the rating curve for the Prospect Road gauge is likely to be relatively low, particularly for low flows. The Merriman Creek gauge at Stradbroke West has the same record period, and a more reliable rating curve. Parameter selection was based on calibration to both gauges and comparison to accepted regional methods, and the design flows were validated against the flood frequency analysis. The approach for the selection of routing and loss parameters is outlined in the following sections.

3.6.2 RORB Model Development

The ArcHydro tool was used to determine the initial delineation of catchments, sub-catchments and drainage paths of Merriman Creek for input to the hydrologic model (Figure 3-12). The catchment was split into 49 sub-catchments with areas ranging from 5.5 to 17.5 km^2. A RORB model was then constructed of the catchment using miRORB (MapInfo RORB tools).

The RORB model was at a resolution adequate to resolve the main drainage paths and sub-catchments, and to provide smooth hydrographs for the hydraulic model.

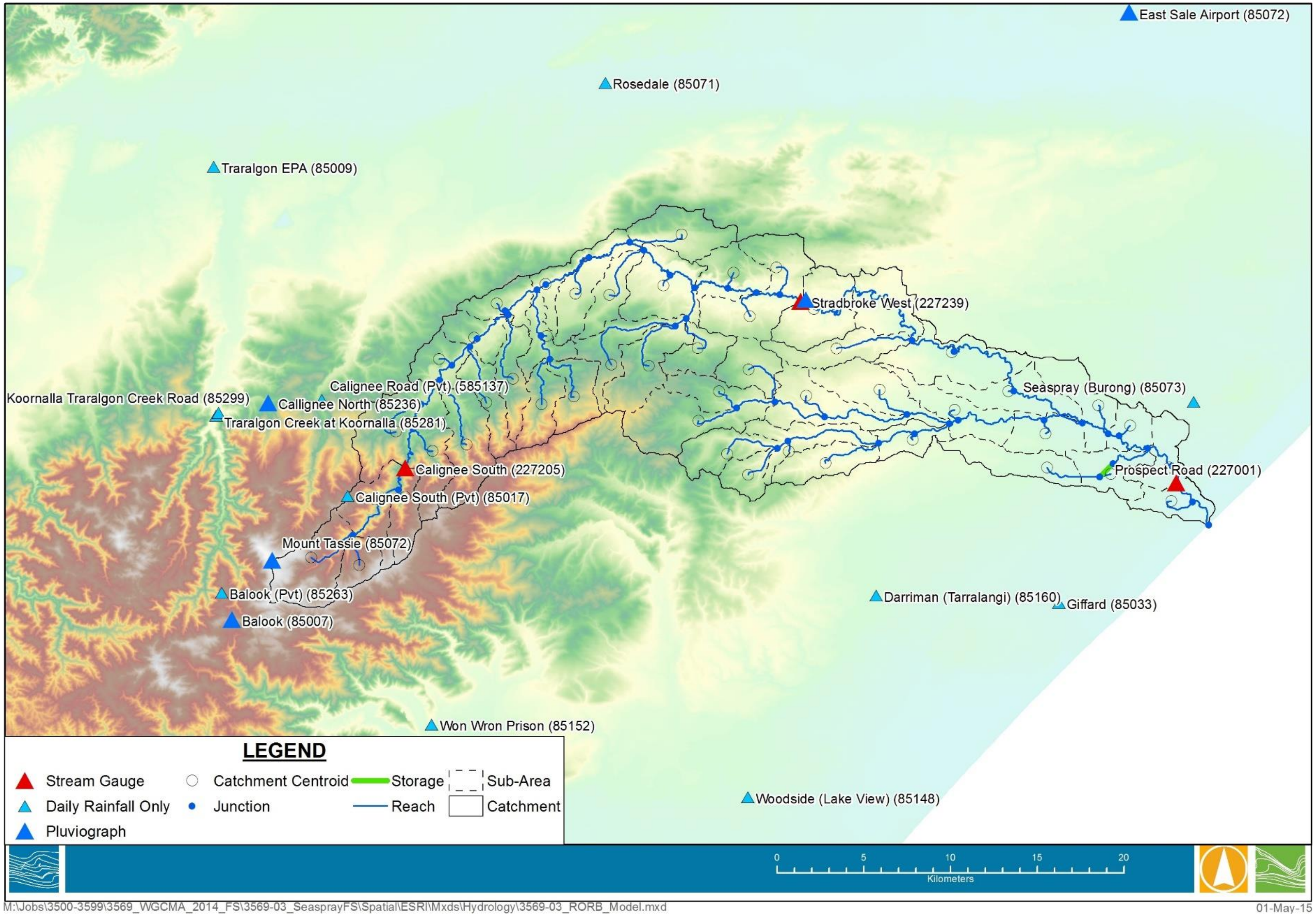

Figure 3-12 RORB model structure

3.6.3 Fraction Impervious

The fraction impervious (FI) was included in the hydrologic model by applying a representative FI for each planning zone, as listed in Table 3-8. The planning zones used for designation of FIs are shown in Figure 3-13. The values chosen to represent each planning zone were adopted from Melbourne Water's MUSIC guidelines (2010).

Table 3-8 Fraction Impervious Designation for Planning Zones

Zone Code	Zone Description	Fraction Impervious (FI)
B1Z – B5Z	Business (1 – 5) Zone	0.9
FZ	Farming Zone	0.1
FZ	Farming Zone - Forestry*	0.1
IN1Z – IN2Z	Industrial (1 – 2) Zone	0.9
LDRZ	Low Density Residential Zone	0.2
MUZ	Mixed Use Zone	0.7
PCRZ	Public Conservation and Resource Zone	0
PPRZ	Public Park and Recreation Zone	0.1
PUZ1	Public Use Zone - Service and Utility	0.05
PUZ2	Public Use Zone - Education	0.7
PUZ3	Public Use Zone - Health and Community	0.7
PUZ4	Public Use Zone - Transport	0.7
PUZ5	Public Use Zone - Cemetery/Crematorium	0.6
PUZ6	Public Use Zone - Local Government	0.7
PUZ7	Public Use Zone - Other Public Use	0.6
R1Z	Residential 1 Zone	0.45
R2Z	Residential 2 Zone	0.6
RCZ, RCZ1, RCZ2	Rural Conservation Zone	0
RDZ1	Road Zone - Category 1	0.7
RDZ2	Road Zone - Category 2	0.6
RLZ, RLZ2, RLZ4, RLZ6	Rural Living Zone	0.2
SUZ1, SUZ2, SUZ10	Special Use Zone (1, 2, 10)	0.6
TZ	Township Zone	0.55

**Farming Zone was split into farming and forestry areas to allow sensitivity to imperviousness caused by bushfire to be tested.*

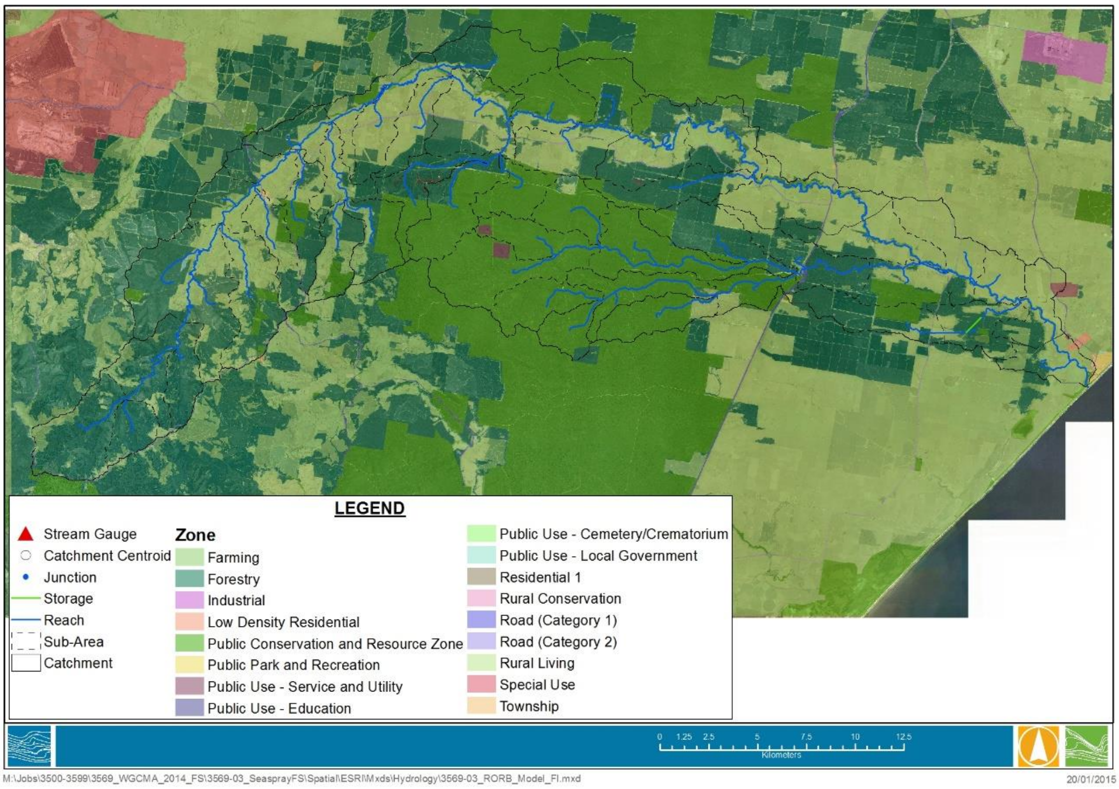

Figure 3-13 Planning Overlay used for Fraction Impervious

3.6.4 Calibration Events

Three recorded events were used for the calibration of the RORB hydrologic model; namely the September 1993, November 1995, and June 2012 floods. Each of these events was represented with a unique temporal and aerial rainfall pattern generated from local rainfall gauge records. The outflow hydrographs from the RORB model were then compared to stream gauges at two locations; Prospect Road, and Stradbroke West. A pluviograph was installed at Stradbroke West in 2006 and has been incorporated into the June 2012 calibration event.

The 1993 and 1995 events were large events with AEP between 1% and 2%, while the 2012 was a moderate event with AEP of around 10%. The 2012 event exhibited quite different behaviour to the other two, with a longer lag between the Stradbroke West and Prospect Road gauges and an attenuation rather than an increase in flow between the two gauges.

Figure 3-14, Figure 3-16, and Figure 3-18 illustrate the aerial rainfall patterns used in each of the calibration runs. Figure 3-15, Figure 3-17, and Figure 3-19 show the rainfall hyetographs and recorded flows at Prospect Road and Stradbroke West. The 1993 Prospect Road flow has been adjusted as detailed in Section 3.4.

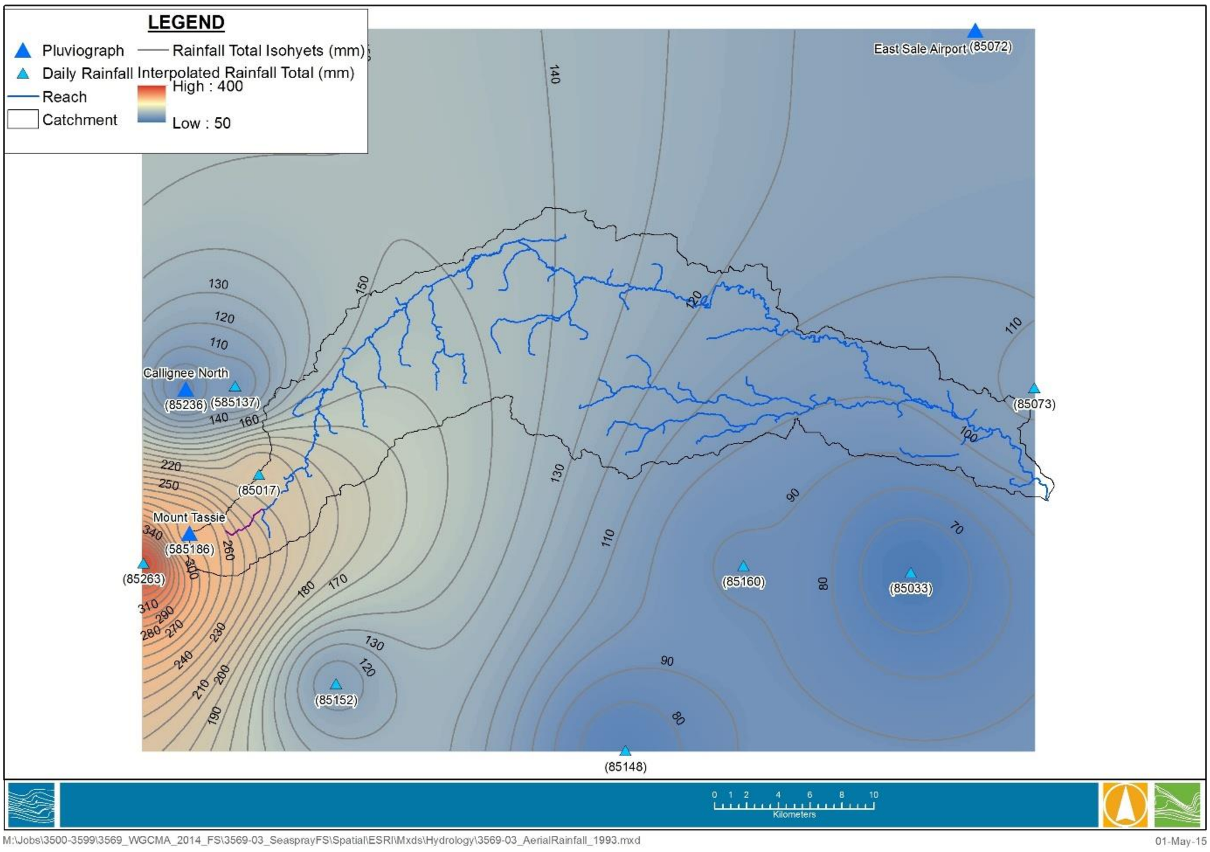

Figure 3-14 Total rainfall across the catchment for the September 1993 calibration event

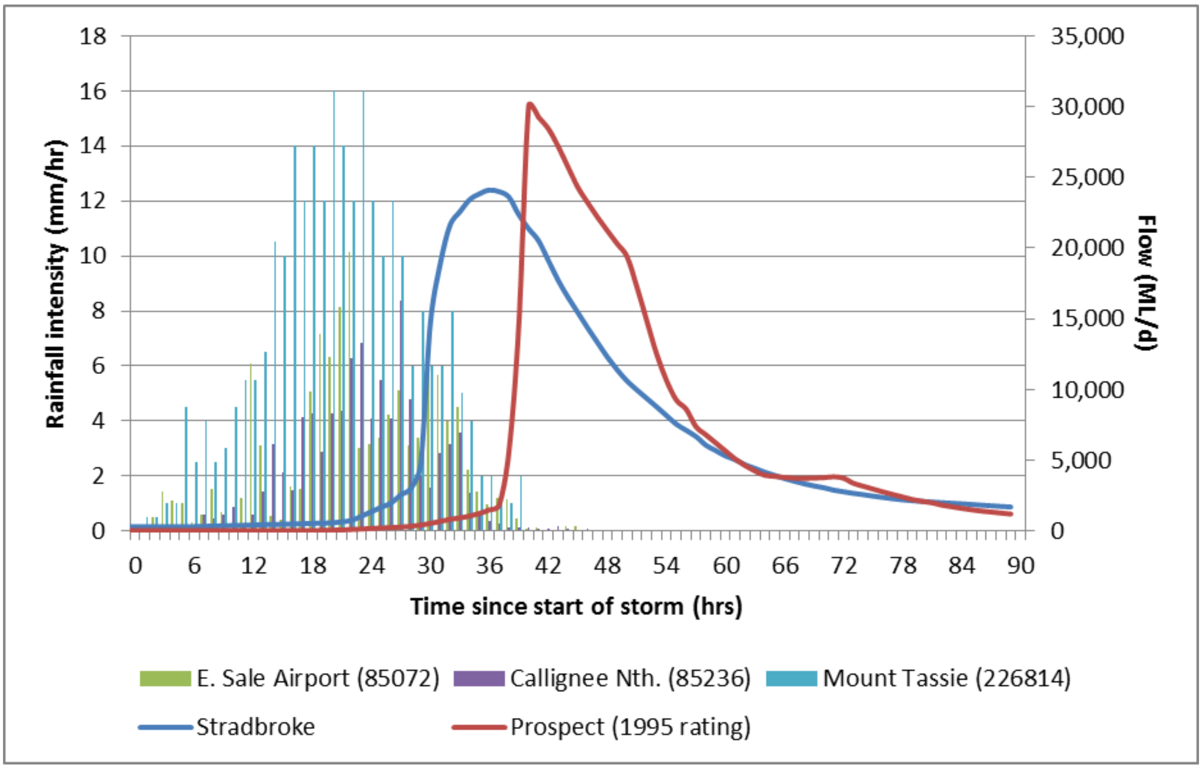

Figure 3-15 Calibration event hyetographs and instantaneous flow hydrographs for the September 1993 calibration event

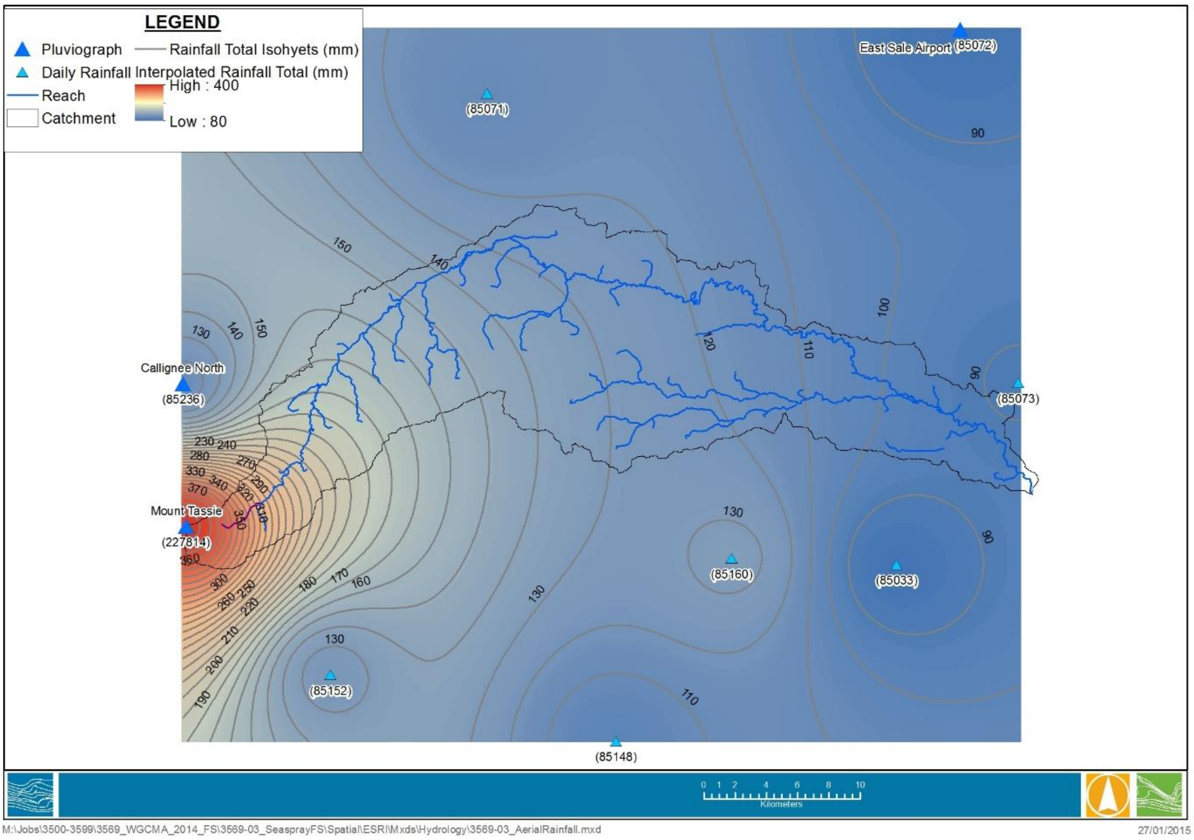

Figure 3-16 Total rainfall across the catchment for the November 1995 calibration event

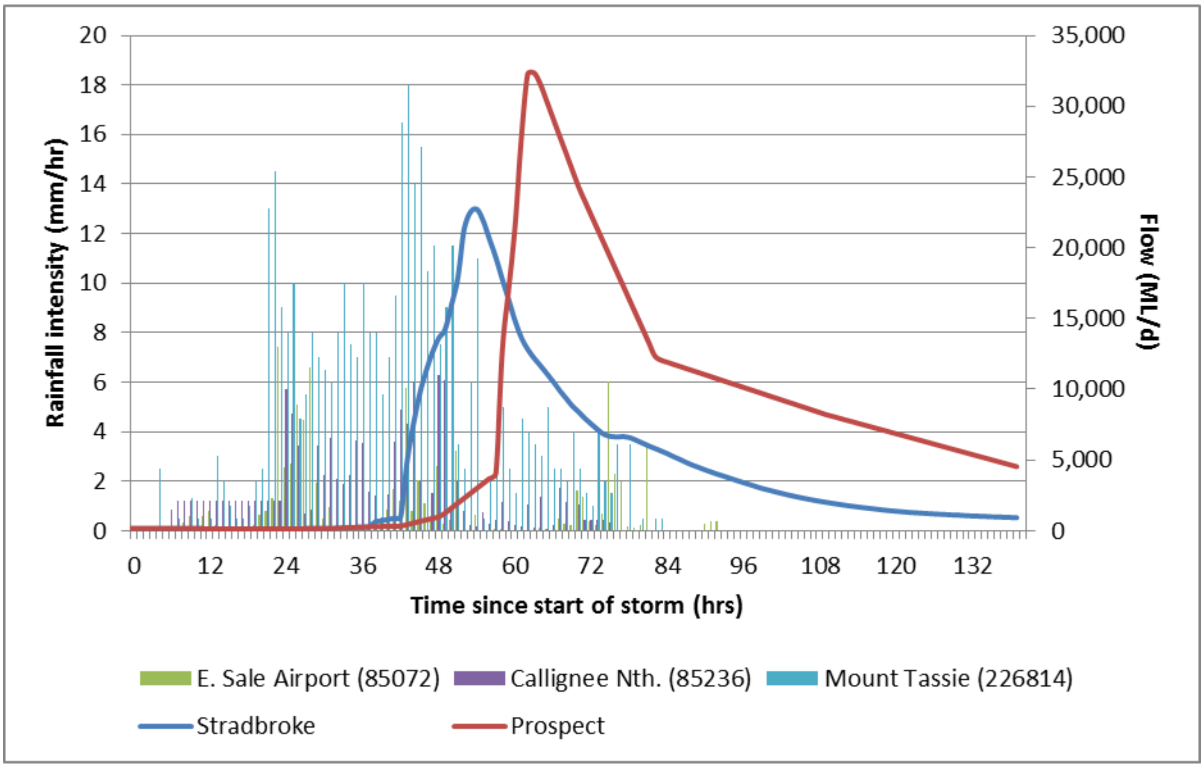

Figure 3-17 Calibration event hyetographs and instantaneous flow hydrographs for the November 1995 calibration event

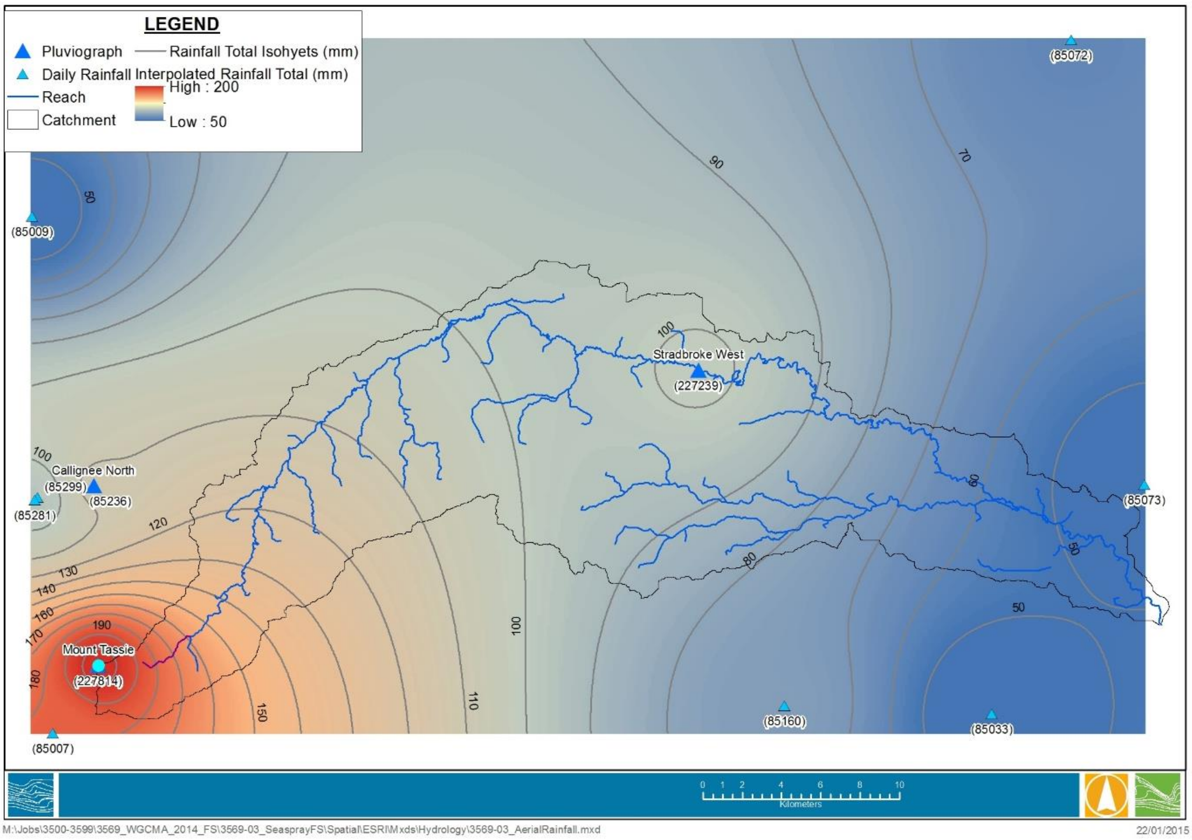

Figure 3-18 Total rainfall across the catchment for the June 2012 calibration event

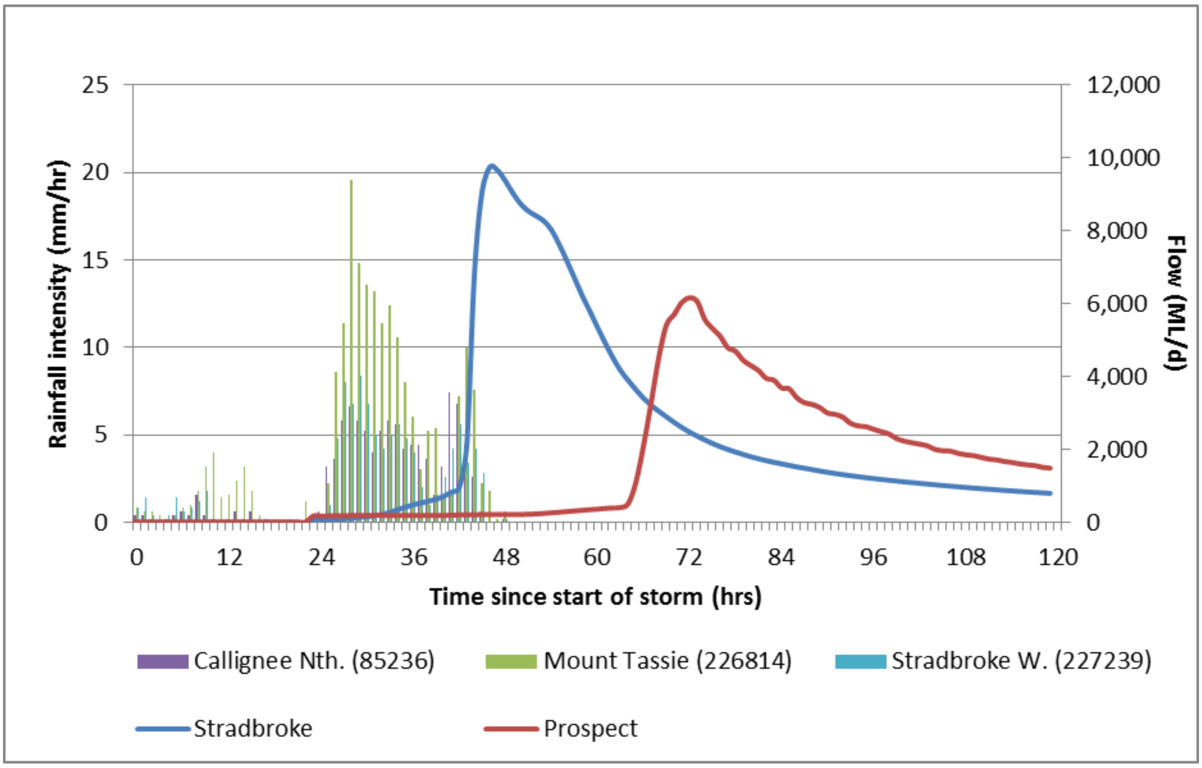

Figure 3-19 Calibration event hyetographs and instantaneous flow hydrographs for the June 2012 calibration event

3.6.5 Calibration of RORB Parameters

The RORB model was run using the initial loss/continuing loss model. The RORB storage and loss parameters – k_c, m, initial loss (IL) and continuing loss (CL) – were fitted for each calibration event to give the best fit to the observed hydrographs. Priority was given to fitting the hydrograph peak and shape, with the volume and timing fitted as a lower priority. The routing parameter k_c was set using a combination of calibration and regional equations. The exponent m was set to 0.8 as per RORB default.

A number of regional equations for the calculation of k_c that are recommended in ARR87 and the RORB manual were used to guide the selection of this parameter (Table 3-9).

Table 3-9 Regional equations for the calculation of k_c

Source	k_c
Aust wide Yu (1989) (Pearse et al 2002)	53
Vic MAR>800 mm - Eq 3.21 ARR87 (Book V)	42
Vic MAR<800 mm - Eq 3.22 ARR87 (Book V)	28
Aust wide Dyer (1994) (Pearse et al 2002)	62
Victoria data (Pearse et al, 2002)	69
Default RORB	49

The final calibrated model parameters for each event are summarised in Table 3-10. The fit of the RORB modelled hydrographs to the gauged hydrographs at the Stradbroke West and Prospect Road gauges for each calibration is shown in Figure 3-20, Figure 3-21, and Figure 3-22.

Table 3-10 RORB Calibration Parameters

Event	k_c	m	IL (mm)	CL (mm/hr)
1993	40	0.8	70	2.3
1995	28	0.8	40	1.0
2012	50	0.8	50	3.8

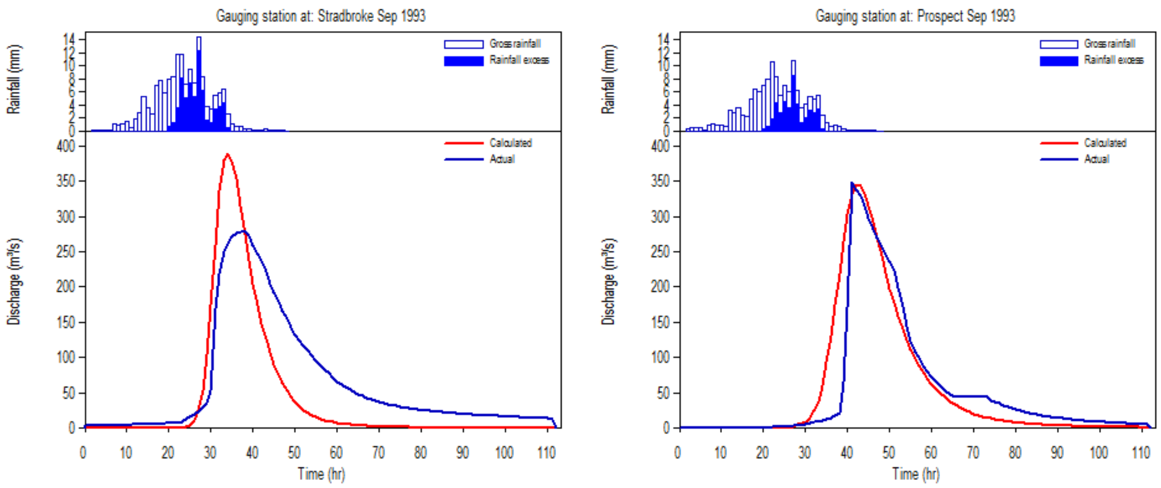

Figure 3-20 Calibration plots for September 1993

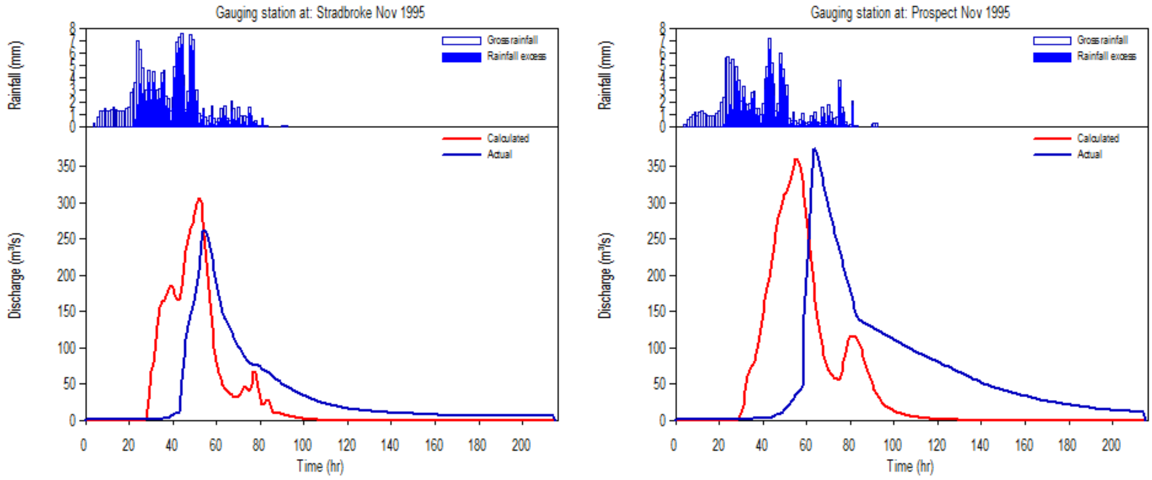

Figure 3-21 Calibration plots for November 1995

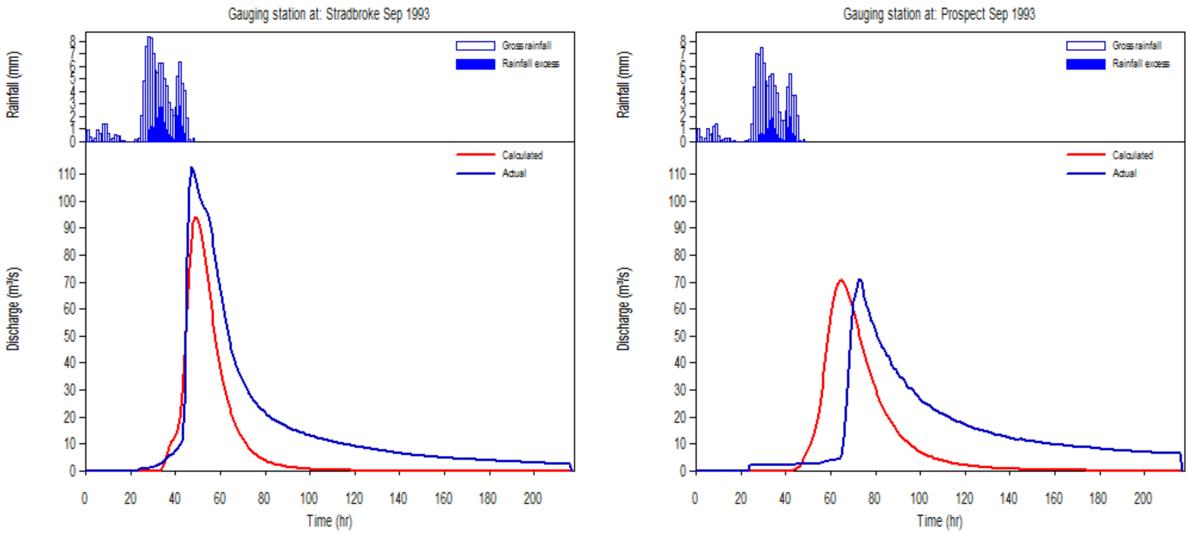

Figure 3-22 Calibration plots for June 2012

The shape and volume of the fitted hydrographs at Prospect Road agrees well with gauged data. The 1993 event was particularly well-matched in shape, timing and volume. The volume at Stradbroke West was a little low while the peak was too high, indicating that the routing parameters applied for the whole model may not be appropriate for prediction at Stradbroke west. This could be improved by use of an interstation area at the Stradbroke West gauge, which would allow RORB to use separate routing parameters upstream and downstream of that point. This issue is common in RORB modelling; however the aim of this model is to provide a reasonable flow estimate as a boundary to our hydraulic model, therefore matching the closest gauge to the study area (Prospect Road) is the main focus of calibration. The mismatch may also indicate that the spatial distribution of rainfall across the catchment was not well represented. The spatial variability across the gauges in and around the catchment is extremely high; therefore it is reasonable that there may have been additional variation within the catchment that was not picked up by the rain gauge network.

In two of the events (1995 and 2012) the RORB model predicted the peak of the hydrograph approximately ten hours early at the Prospect Road Gauge. One possible reason for significant variation in the timing of the hydrograph peak is poor capture of the actual temporal patterns within the catchment by the pluviographs, only two of which were located in the catchment. There were no pluviographs in the lower catchment for the 1995 event. For both, the timing at Stradbroke West was well matched.

The k_c values fitted for the calibration events are within the range of those provided by regional equations in Table 3-9. The initial losses tend to be higher than design values and continuing losses tend to be on the low side for the 1993 and 1995 events but reasonable for the 2012 event.

3.6.6 Design Loss Estimates

Design losses were estimated by the design loss prediction equations developed by Hill et al (1998) and (Hill et al. 2014). The losses recommended in ARR87 consistently overestimated peak flows. The Hill et al (1998) losses, in combination with the new areal reduction factors from Siriwardena and Weinmann (1996), produced peaks that were more consistent with the results of flood frequency analysis.

The initial loss is calculated by first calculating the storm initial loss using Equation 1, then the burst initial loss (Equation 2). The burst initial loss varies with storm duration and accounts for the embedded nature of AR&R design rainfalls (Hill et al 1998). The continuing loss is estimated using Equation 3.

Storm initial loss:

$$IL_s = -25.8BFI + 33.8 \quad (1)$$

Burst initial loss:

$$IL_b = IL_s \left\{1 - \frac{1}{1+142\frac{\sqrt{duration}}{MAR}}\right\} \quad (2)$$

Continuing loss:

$$CL = 7.97BFI + 0.00659PET - 6.00 \quad (3)$$

The ARR Project 6 Stage 3 report (Hill et al. 2014) contains new recommended initial and continuing loss equations, as shown below. No recommendation has been made for the conversion of storm initial loss to burst initial loss.

Storm initial loss:

$$IL_s = 16.7 + 0.141P_{24h}^{2\%} - 0.291MedianAPI \quad (4)$$

Continuing loss:

$$CL = 3.0\ mm/hr \quad (5)$$

The values of the input parameters to the previous equations are as follows:

- Median API 34 mm
- BFI 0.52
- PET 1,007 mm
- MAR 670 mm
- $P_{24h}^{2\%}$ 144 mm

The design loss estimates produced by both methods are given in Table 3-11. The design initial losses tend to be lower than those calibrated in RORB, while the continuing losses tend to be higher. The Project 6 losses (Hill et al. 2014) are the most consistent with calibration out of the two sets.

Table 3-11 Design loss parameter estimates

Source	IL_s (mm)	IL_b (mm)	CL (mm/hr)
Hill et al (1996)	20	8-13 (for 9-72 hour events)	4.8
Project 6 Stage 3 Report (Hill et al. 2014)	27	-	3

3.6.7 Design Flood Validation

Various k_c and loss values were compared to the FFA at Prospect Road (including historical events) to determine the best fit for the design hydrology. The ARR Project 6 Stage 3 Report (Hill et al. 2014) did not find evidence of variation of loss values with AEP, a finding which is consistent with a range of previous studies. Therefore, rather than varying losses to fit the design flows exactly to the FFA, the validation process was aimed at selecting a single set of loss parameters which provided a reasonable fit across the whole range. This approach was justified given the FFA was based on a dataset with 38 years of historical gauged data, at a gauge with known rating curve uncertainties, reducing the confidence in the design flood estimates at high flows (for events larger than 2% AEP).

Initial loss values ranging from 13 mm to 70 mm were tested with a preference for values closer to the lower end of the calibration results (40 mm) and higher end of the design loss equation estimates (27 mm). The k_c values tested ranged from 30 to 50 and a value of 30 was found to produce the hydrograph most consistent with measured data. Continuing losses ranged from 1 to 10 mm/hr with a preference to values within the range used for calibration (1.0 – 3.8 mm/hr). Selected results are shown in Figure 3-23 below.

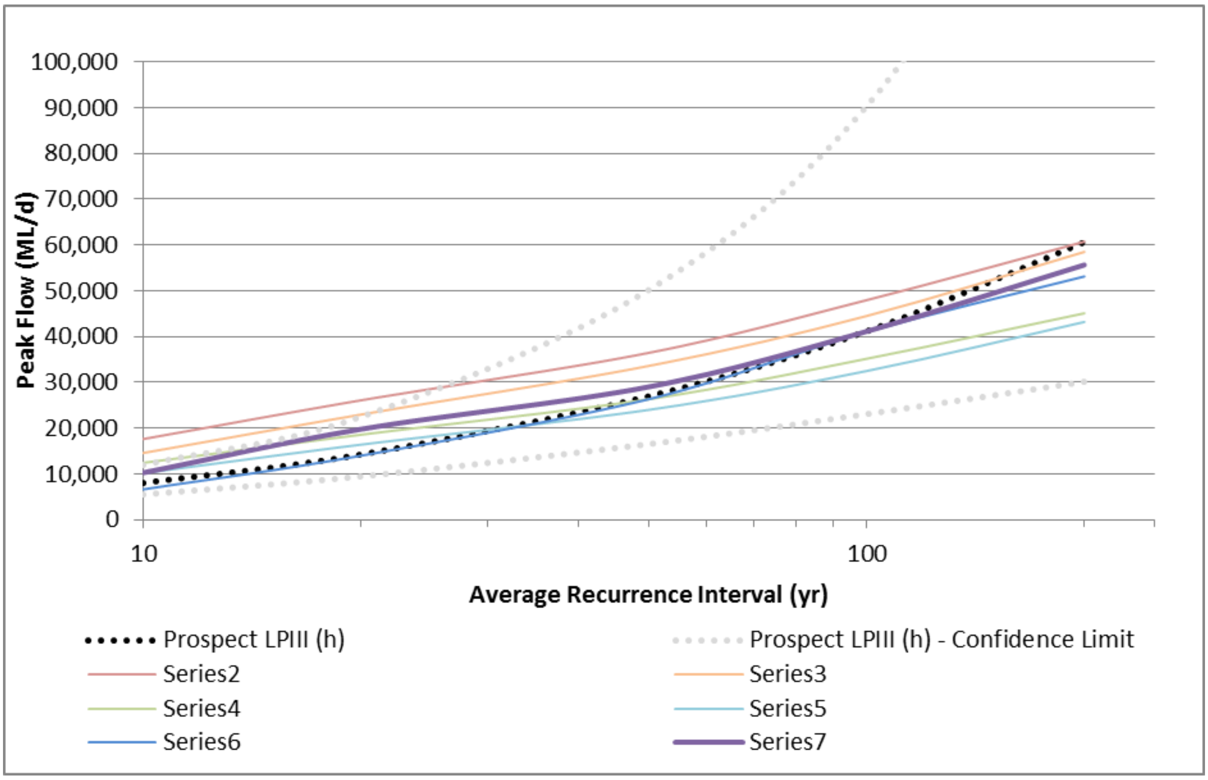

Figure 3-23 RORB peak flow results for various k_c and IL values

The shape of the hydrographs for selected parameter sets was compared to the 1993 gauged hydrograph to check that the selected k_c value matched produced a hydrograph shape consistent with actual events (Figure 3-24). The comparison shows that a k_c of 30 provides a better fit to the hydrograph shape than a k_c of 40, regardless of the loss values.

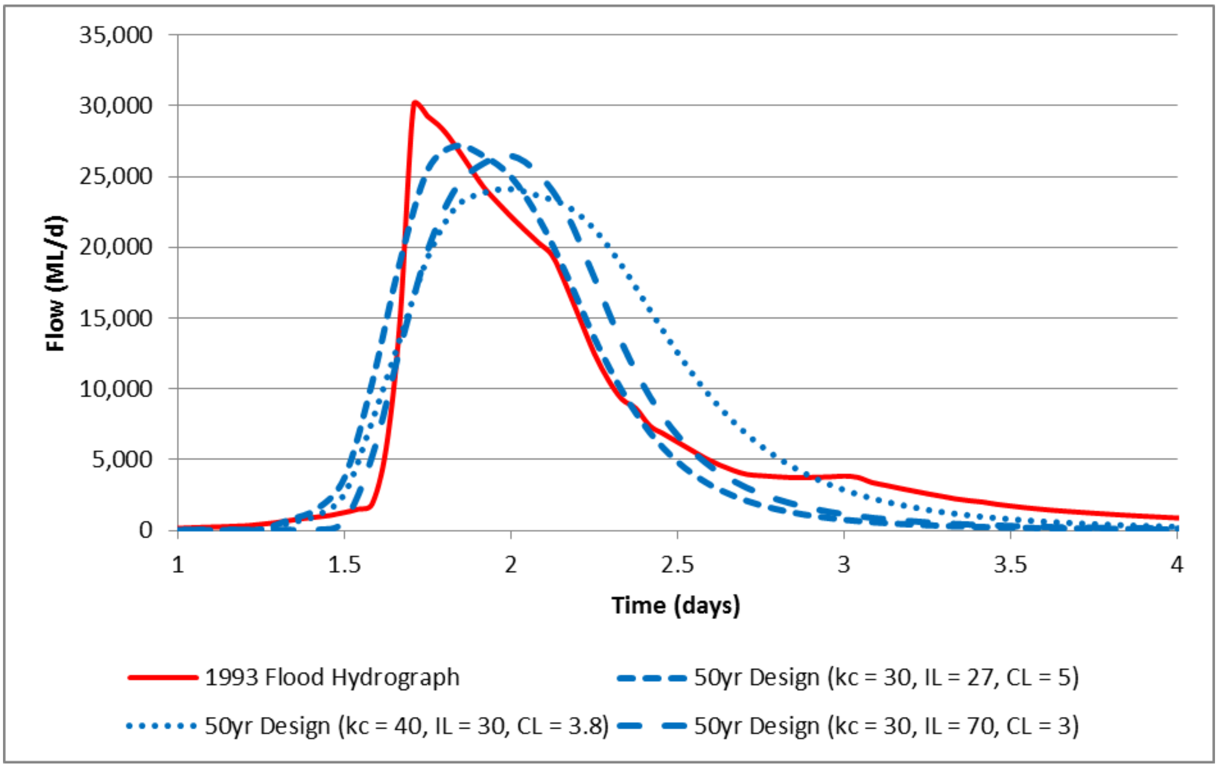

Figure 3-24 Comparison of shape of design hydrographs for selected parameter sets

The parameter set with k_c of 30, initial loss of 50 mm and continuing loss of 3.8 mm/hr was selected as the most reasonable fit to the FFA. These parameter values provide a reasonable fit to both high and low frequency events. The estimates are above, but within the confidence limits of the FFA for the 10%, 5%, and 2% AEP events, and are approximately equal to the FFA for the 1% AEP. The 0.5% AEP peak flow is around 8% lower than the FFA, but well within its confidence limits.

3.7 Design Flows and Hydrographs

The routing and loss parameters summarised in Table 3-12 were found to produce a reasonable fit to the FFA while also lying within the range of calibration values and were thus adopted for design event modelling. The resulting hydrographs are shown in Figure 3-25. Note that for this project the RORB model design event flows were given equal confidence with the flood frequency analysis due to the issues with the gauge data, rating curve and small period of record. Therefore the design flows have adopted the RORB results and have not adjusted the flows to the flood frequency analysis.

Table 3-12 Recommended design parameters

k_c	m	IL (mm)	CL (mm/hr)
30	0.8	50	3.8

Table 3-13 Recommended design peak flows

AEP	RORB Design Flow (ML/d)	Critical Duration
10%	10,280	48hr
5%	19,790	48hr
2%	29,030	36hr
1%	41,130	48hr
0.5%	55,640	48hr

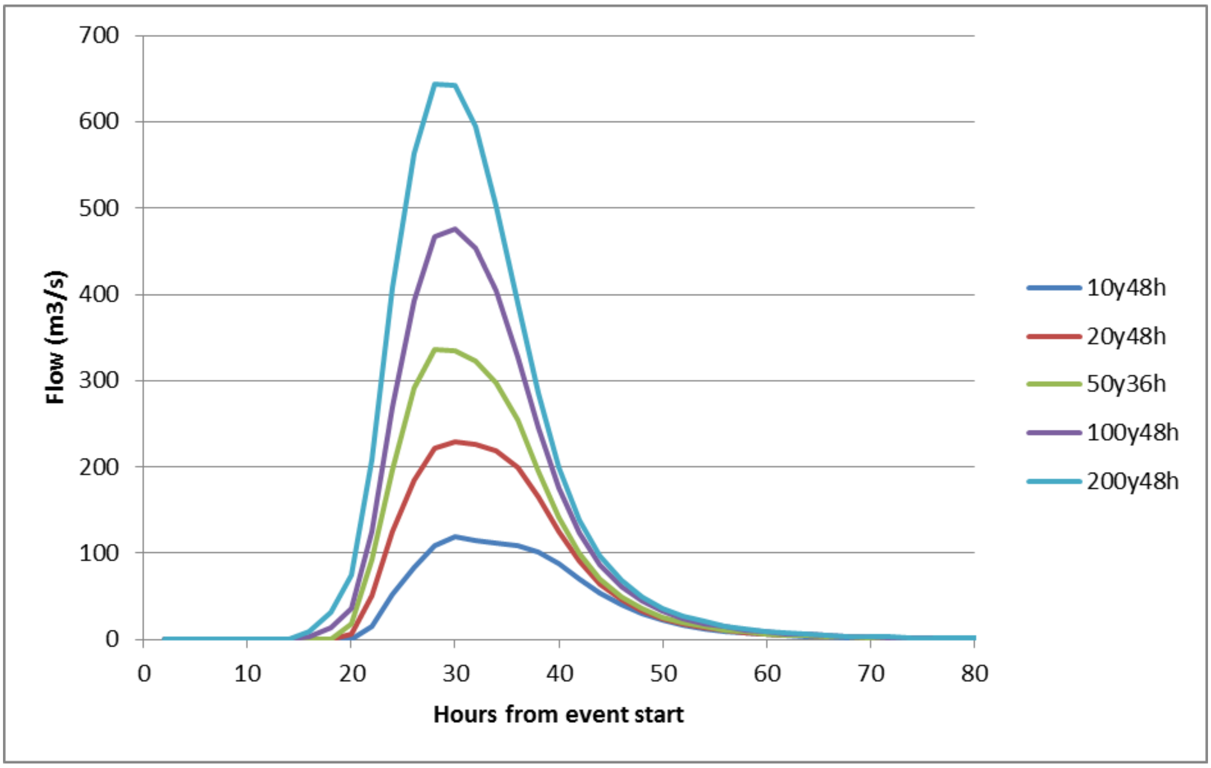

Figure 3-25 Merriman Creek at Seaspray design flood hydrographs

3.8 Sensitivity Analysis

3.8.1 Climate Change

The impacts of climate change were tested by increasing the rainfall intensity by 5% per degree of warming, in line with latest guidance from Australian Rainfall and Runoff (Engineers Australia 2014). A scenario of 2°C of warming (i.e. 10% increase in rainfall intensity) was adopted for this sensitivity test. This is consistent with 'Climate Change in Australia Projections' (CSIRO 2015) report which suggests for an intermediate climate scenario a temperature increase of between 1.1 °C to 2.0 °C is likely for the Southern Slopes region, which includes Merriman Creek catchment.

The 10% increase in rainfall intensity was applied to the burst rainfall depth in the RORB storm files for each of the design events for the critical duration (36 or 48 hr). The results of the climate change sensitivity test at the catchment outlet are summarised in Table 3-14 and shown in Figure 3-26. Due to the hydrologic response of the catchment, the proportional increases in catchment flows are significantly larger than the applied percentage increase in rainfall. This is most notable in the more frequent events where the 10% AEP design flood event produces a 46% increase in flow. As the magnitude of the design event increases, the percentage increases in flow are reduced to a value more closely aligned to the percentage increase in rainfall.

Table 3-14 Comparison of peak flows for increases in rainfall intensity due to climate change

AEP	Current Conditions		Scenario of 2°C of Warming		Increase in Flow (%)
	Burst Rainfall Depth (mm)	RORB Design Flow (ML/d)	Burst Rainfall Depth (mm)	RORB Design Flow (ML/d)	
10%	107.5	10,250	118.3	14,970	46
5%	126.6	20,040	139.3	26,010	30
2%	135.0	28,390	148.5	35,700	26
1%	175.1	41,080	192.6	52,490	28
0.5%	198.3	55,720	218.1	66,330	19

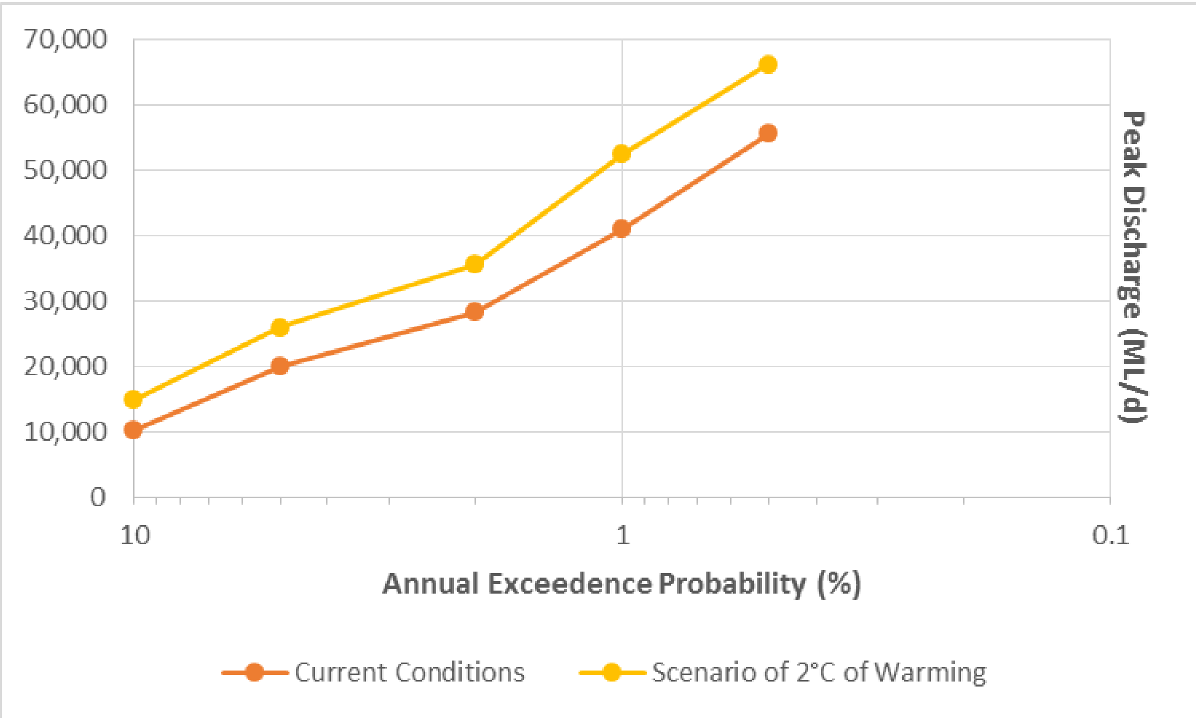

Figure 3-26 Climate change sensitivity comparison for Merriman Creek at Seaspray

3.8.2 Bushfire

The impacts of bushfires were tested by adjusting the fraction impervious values of the Merriman Creek sub-catchments to reflect an agreed severity of bushfire. For example, Blackham et al (2012) provides values of equivalent percentage impervious for different levels of burn severity, based on BAER (2009) (Table 3-15).

Table 3-15 Equivalent percentage impervious for different levels of burn severity

Burn severity	Equivalent percentage impervious
Unburned	0.1
Low	0.3
Moderate	0.7
High	0.9

The impervious fraction for all Farming Zone (Forestry), and Public Conservation and Resource Zone areas across the catchment have been increased to 0.3, 0.7, and 0.9, to represent low, moderate and high severity burns across the catchment. The resulting fraction impervious for each of the sub-catchments was recalculated and the RORB model was rerun for a range of design events.

As expected, the increase of impervious fraction due to bushfire had a significant impact on the peak flows generated from the catchment at the catchment outlet.

Table 3-16 Comparison of peak flows for increases in impervious fraction due to bushfire

AEP	Peak Catchment Discharge (ML/d)						
	Unburned Catchment	Low Intensity Bushfire	Increase in Flow (%)	Moderate Intensity Bushfire	Increase in Flow (%)	High Intensity Bushfire	Increase in Flow (%)
10 %	10,280	17,880	74%	29,810	190%	36,720	257%
5 %	20,040	26,520	32%	39,660	98%	47,000	135%
2 %	28,430	37,930	33%	52,790	86%	61,260	115%
1 %	41,130	50,280	22%	66,360	61%	74,820	82%
0.5 %	55,730	64,110	15%	81,390	46%	89,600	61%

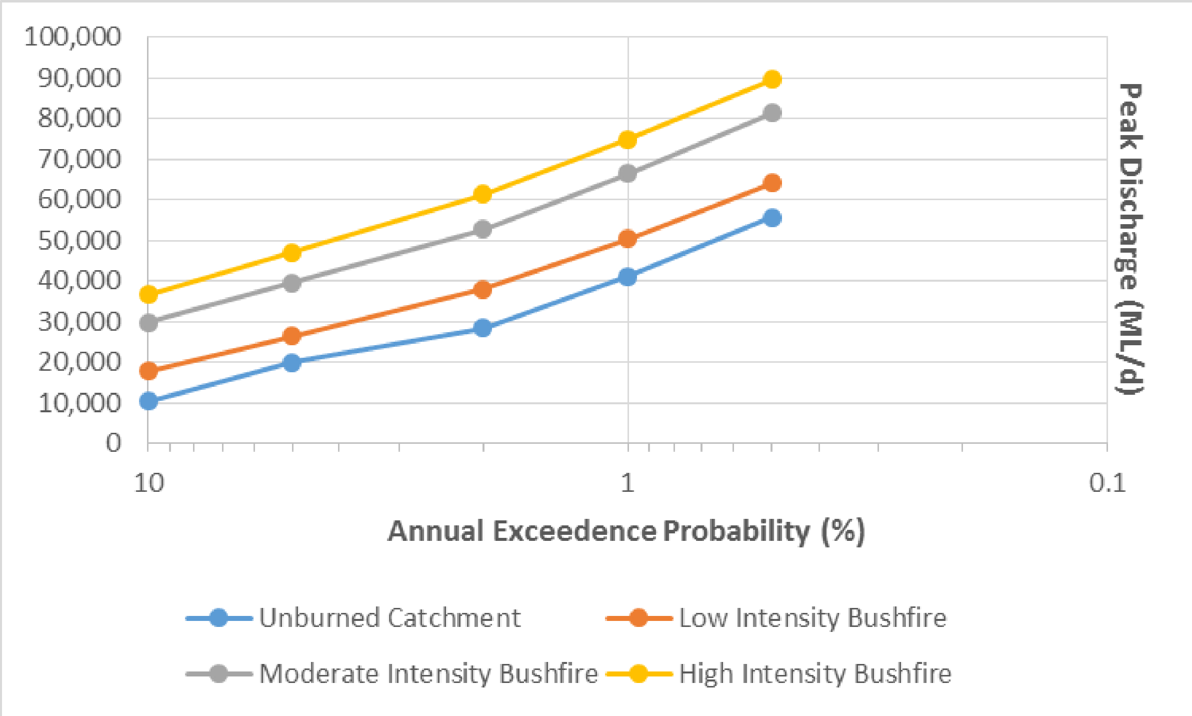

Figure 3-27 **Bushfire sensitivity comparison for Merriman Creek at Seaspray**

3.9 PMF Estimation

The Probable Maximum Flood (PMF) is the flow generated from the theoretical maximum precipitation for a given duration under current climate conditions. A PMF Estimate for Merriman Creek at Seaspray (at the lagoon mouth) was prepared using the Quick Method of Nathan et al. (1994). This method applies a set of empirical equations to compute a triangular PMF hydrograph. The equations are:

$$Q_p = 129.1\ A^{0.616}$$

$$V = 497.7\ A^{0.984}$$

$$T_p = 0.0001062\ A^{-1.057}\ V^{1.446}$$

$$T_r = V / (1.8\ Q_p)$$

Where:

Q_p is peak flow (m^3/s)

A is catchment area (km^2)

V is hydrograph volume (ML)

T_p is time to peak of hydrograph (h)

And T_r is base length of hydrograph (h)

The equations are applicable to southeast Australian catchments from 1 to 10,000 km^2 that do not have large lakes or storages. As the Merriman Creek catchment contains a reasonably large swamp storage (around 35 ha in area increasing to around 90 ha when full), a correction to the catchment area is required before the equations are applied. The catchment area used was 484 km^2 (the total catchment area minus the swamp catchment). The resulting PMF peak flow, volume, time to peak and hydrograph length are:

A	484 km^2
Q_p	502,680 ML/d (5,818 m^3/s)
V	218,200 ML
T_p	8.1 h
T_r	20.8 h

The resulting triangle hydrograph is shown in Figure 3-28.

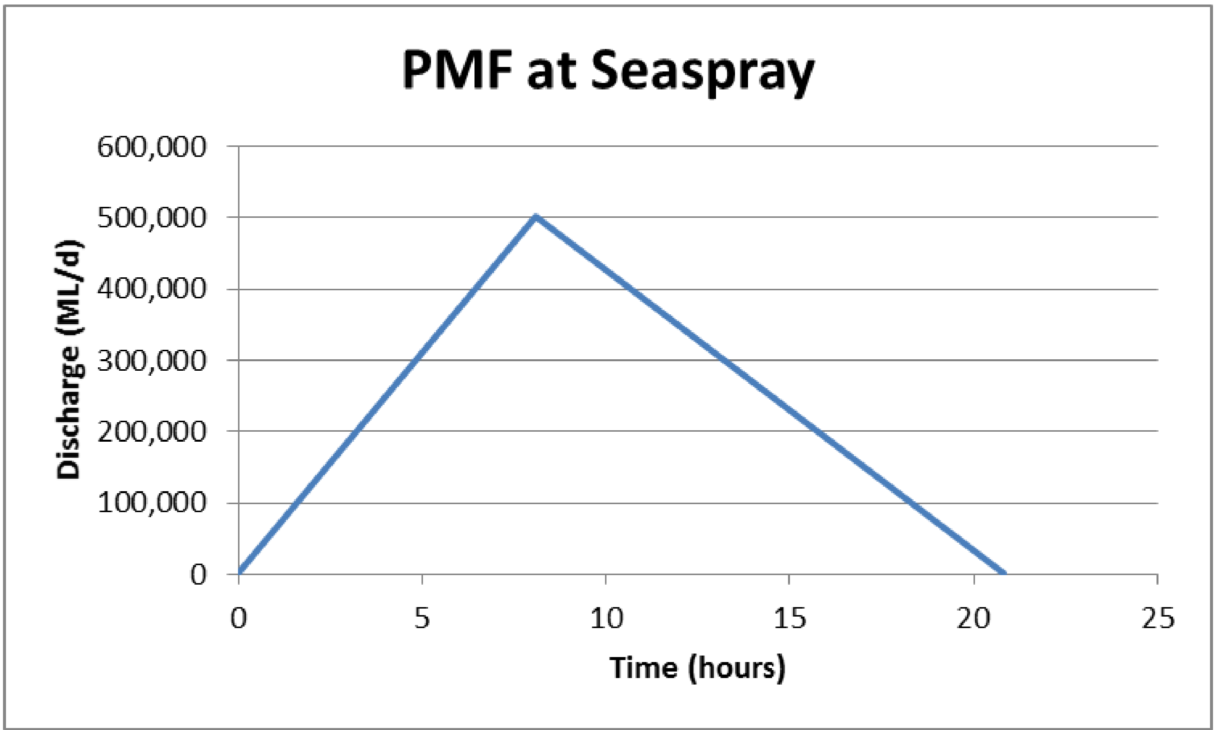

Figure 3-28 PMF hydrograph for Merriman Creek at Seaspray

4. CONCLUSION

Through a process of flood frequency analysis, hydrologic modelling and application of various regional methods, a set of design flood hydrographs have been developed for Merriman Creek at Seaspray. While the hydrological analysis has been based on robust methods and extensive cross-validation, there are limitations due to issues with the gauge at Prospect Road and difficulties in calibration of the RORB model. However, validation of the design peak flows against flood frequency analysis and comparison of the design hydrographs against gauged hydrographs gives confidence that they are representative of actual flood risk and appropriate for design.

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APPENDIX A RATING CURVE CHECK WITH HYDRAULIC MODEL

The hydraulic model developed in this project was used to verify the appropriateness of the rating curves for the 1993 and 1995 calibration events.

Boundary

A discharge boundary a little over one kilometre north of Prospect Road was used to apply flows to the hydraulic model. Flows were calculated from gauged water levels and the rating curves from 1993 and 1995.

Resolution

In the vicinity of the gauge, a 5 metre quadrilateral mesh was used in order to sufficiently represent the topography in the area and the modelled flood extent. Upstream and downstream of this area the main river channel was made up of quadrilateral elements 3 to 6 metres in width and with varying lengths. To the sides of the main river channel across the floodplain triangular elements increased in size to approximately 2,000 m^2.

Roughness

A roughness map was used in this early version of the model with the vegetated river channel upstream of Prospect Road modelled with a Manning's n roughness of 0.100 and the surrounding area was given a Manning's n roughness of 0.045. The roughness map was refined and modified for the calibration and validation simulations.

Results

Water levels were extracted from the model at the gauge site and compared directly to the gauge record, and the flood extents around the gauge were compared with those reported in the flood summary report (HydroTechnology 1995).

Modelling of the 1995 event produced a close fit of water levels with the recorded levels at the gauge, as shown in **Error! Reference source not found.** below. Both the rising and falling limbs, as ell as the peak, produced a close match. It is therefore considered that the 1995 rating curve was appropriate for the 1995 event.

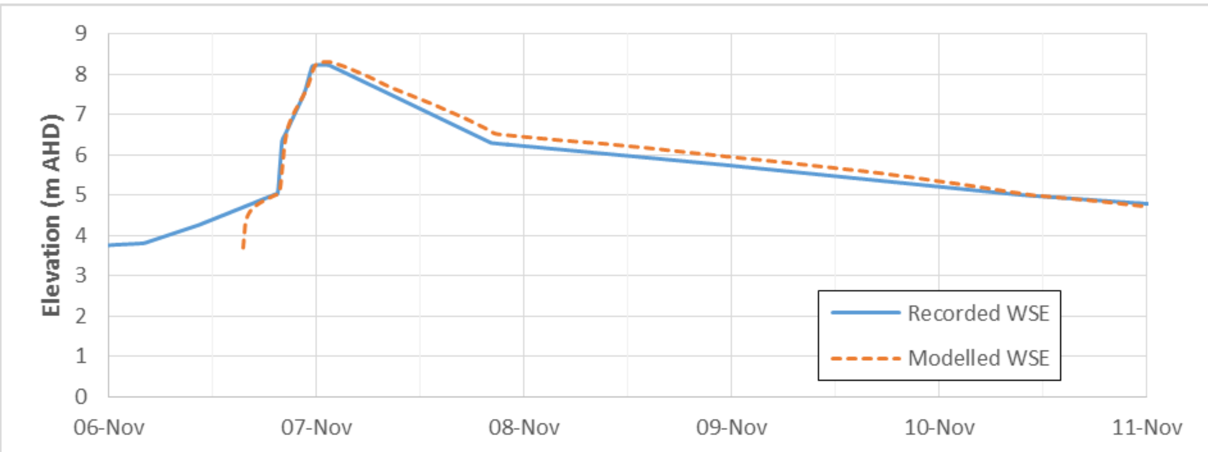

Figure A-1 1995 Calibration Period Recorded and Modelled WSE at Gauge

Initial modelling of the 1993 event with the 1993 rating curve produced a water level at the gauge that matched in the rising limb, overshot the peak, then maintained an elevated position on the falling limb. Adjustment of the flow rates with the application of the 1995 rating curve produced a curve that under-predicted the antecedent water level but had a close fit for the rise, peak, and retreat of the water. A comparison of these two simulations and the recorded water level at the gauge are illustrated in Figure A-2 below.

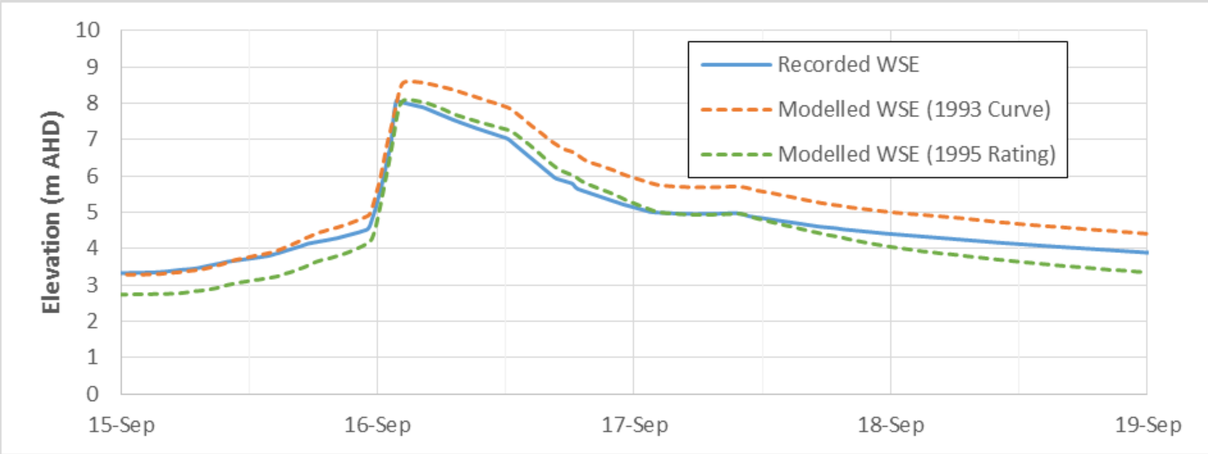

Figure A-2 1993 Calibration Period Recorded and Modelled WSE at Gauge