LOWER FITZROY RIVER
INFRASTRUCTURE PROJECT

Appendix P3

Surface water resources supporting material

Part 3 Section 5 IQQM statistical modelling comparison

Part 3 Section 6 Hydrologic investigations and modelling

SunWater
MAKING WATER WORK

Gladstone Area Water Board

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5 IQQM statistical modelling comparison

5.1 Methodology

The modelling approach and outcomes detailed in the below documents were also reviewed with respect to the IQQM methodology and consistency in goals:

- Fitzroy Basin Draft Water Resource Plan Environmental Assessment – Stage 1 Background Report (DERM 2009)

  A summary of the basic water resource management profile, environmental provisions and the previous planning strategies and water monitoring programs for the Fitzroy Basin
- Fitzroy Basin Draft Water Resource Plan Environmental Assessment – Stage 2 Assessment Report (DERM 2010)
- A report detailing the outcomes of technical ecological assessments (including ecological risk assessments and climate change analyses) and reviews of the previous WRP and ROP.

Data were extracted from the IQQM-Project at the end of the system (Fitzroy WRP plan area Node 0, IQQM1 (Figure 3-1)), downstream of the Fitzroy River Barrage. Data extracted represented the base case (existing Eden Bann Weir) and three assumed development scenarios for assessment purposes:

- EB1: base case scenario (no development option) with existing yield
- EB2: an intermediate development scenario at Eden Bann Weir with a theoretical yield of 35,000 ML/a
- RW1+EB1: an intermediate development scenario at the Rookwood site with a theoretical yield of 54,000 ML/a
- RW2+EB3: upper limit development scenario with yield assessed capped at 76,000 ML/a and at predicted theoretical yield of 110,000 ML/a (Section 2).

Basic characteristics of the data sets were investigated and it was found that the flow data did not fit a normal distribution. Applying a $\log_{10}(x+1)$ transformation did not improve this distribution towards normality. Given the underlying assumptions of parametric statistics (e.g. normality and heteroscedasticity) were not able to be achieved, non-parametric statistics were deemed more appropriate for the dataset. Accordingly data was analysed using the multivariate statistical program Primer v. 6.0 (Clarke and Gorley, 2006). Data were subjected to non-metric multidimensional scaling (MDS) and one-way analysis of similarities (ANOSIM). ANOSIM was employed as this is a permutation based hypothesis testing tool, used to identify significant differences between defined factors.

The data sets comprised low flow events interspersed with a small number of very large flow events. In order to examine the relatively small changes between the low flow events, the $\log_{10}(x+1)$ transformation of the data was retained. This transformation resulted in the relative importance of the very large flow events being down-weighted in favour of the typical conditions. After the data was $\log_{10}(x+1)$ transformed, similarity matrices were produced based on Euclidean distances. Each matrix, calculated to display the similarity between pairs of samples, formed the basis of both the MDS and ANOSIM analysis.

A range of flow regimes were investigated based on the total annual flow under existing conditions (EB1). To identify representative flow regimes, annual flow was graphically represented in ascending order, with the lowest (1969) and highest (1918) flow years as shown on Figure 5-1 selected. Figure 5-2 presents untransformed data. Every tenth year was then identified for analysis, resulting in a total of 13 years that capture the range of flow regimes present in the data.

Table 5-1 provides the annual flow for each of the selected years under existing conditions (EB1).

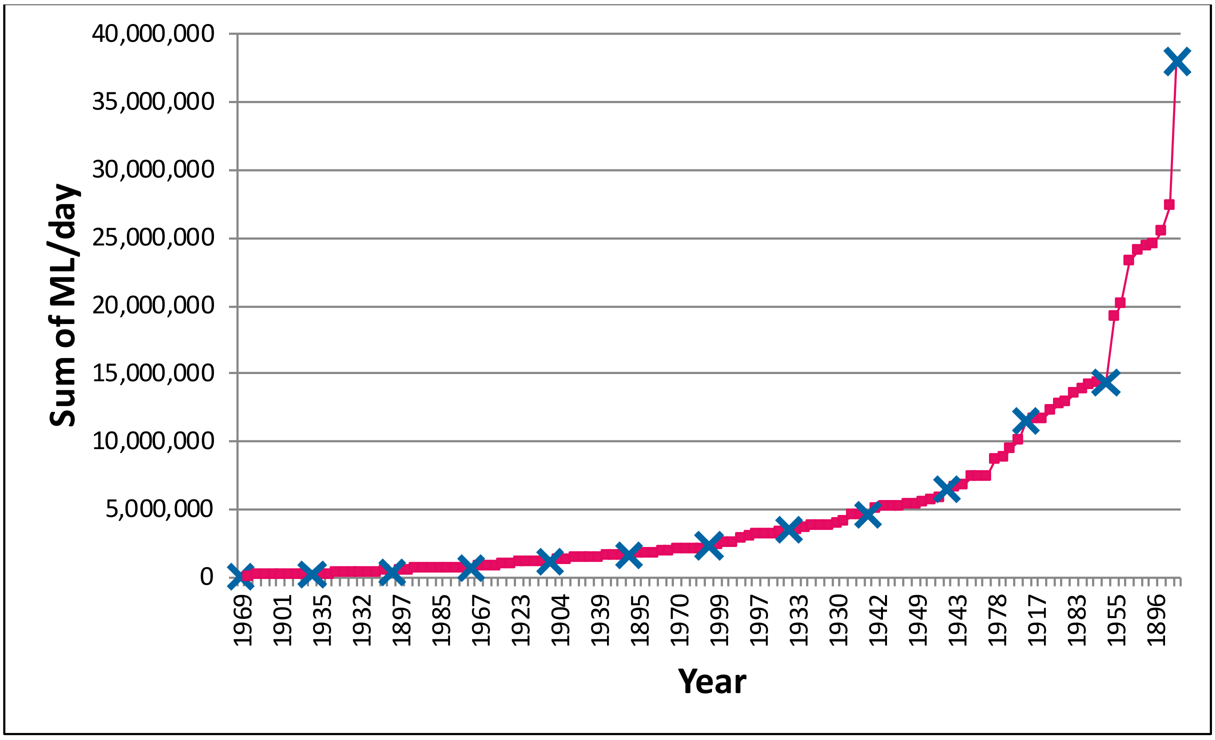
Table 5-1 Annual flow (EB1) for all analysed years

Year	Annual flow (ML)
1969	1,935
1965	156,816
1982	367,382
1952	650,791
2007	1,134,580
1909	1,630,359
1994	2,297,885
1913	3,431,637
1998	4,622,900
1988	6,490,384
1928	11,566,571
1976	14,304,898
1918	38,017,280

Non-metric MDS and ANOSIM were undertaken for each of the identified years, comparing each development scenario (EB2, RW1+EB1 and RW2+EB3) to the base case scenario (EB1). For these analyses data were pooled across months for each identified year under each flow scenario. Monthly flow data used in this context is considered to be the most suitable to allow for intra-annual variability in wet and dry seasons to be incorporated into the analysis. Annual data (e.g. a higher level of pooling) is not deemed suitable due to the inability to detect seasonal variation in flow. [1]

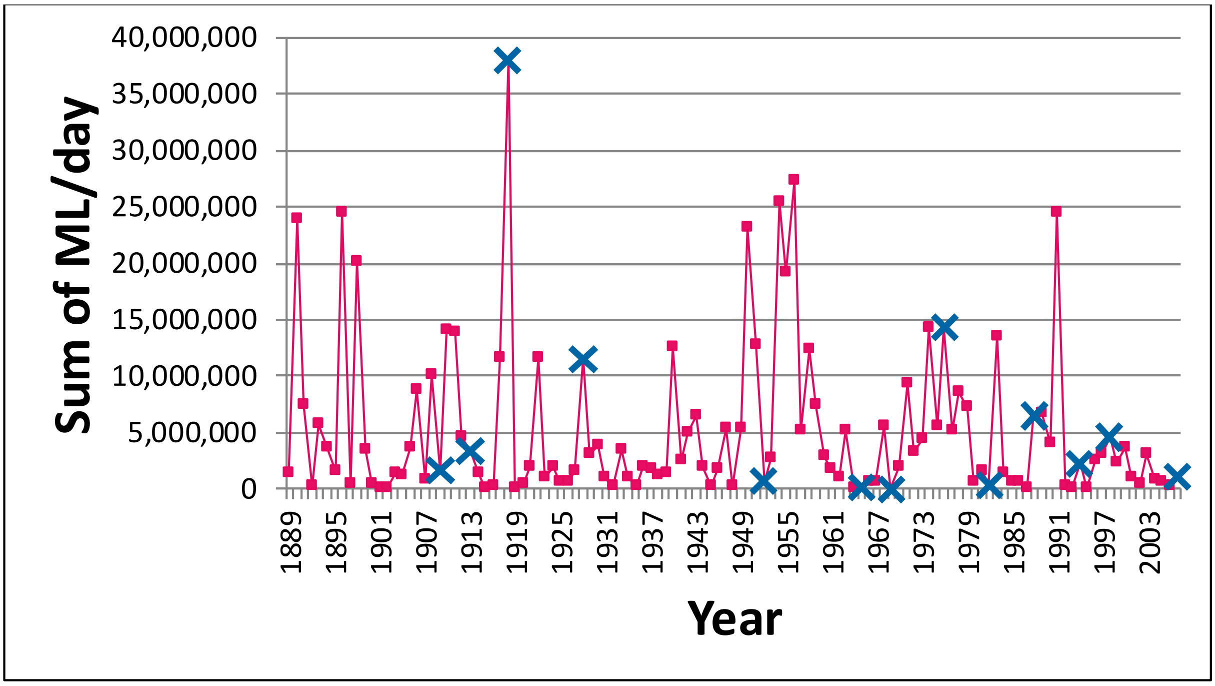
[1] For example, Annual Flow may represent one large flood event followed by low flow conditions, or may represent average flow conditions dependent on predicted seasonal trends. Analysing flow data on a smaller temporal scale (e.g. monthly data) increases the likelihood of detecting environmentally relevant differences between baseline and construction scenarios.

Figure 5-1 Total annual flow (ascending total flow)



Analysed years marked (**x**)

Figure 5-2 Total annual flow (time series)



Analysed years marked (**x**)

Daily data is considered unsuitable for use within the analysis due to the high degree of variability involved with data at this resolution, e.g. outliers and 'noise' present, and due to the likelihood that this scale would be inefficient at accurately capturing the dynamic flow conditions of the system.

The results of the ANOSIM were tested for three significance levels (*P* = 0.1, 0.05, and 0.01).

5.2 Results

A summary of the ANOSIM results for all tests is provided in Table 5-2. The MDS plots corresponding to the ANOSIM tests are presented in Appendix A. These plots provide a visual representation of the ANOSIM results and are presented in order of yearly outflows (Table 5-1). A summary is provided:

- EB1 versus EB2

  In all years, there were no significant differences (all significance levels) between the base case and development scenario.

- EB1 versus RW1+EB1

  In all years, there were no significant differences (all significance levels) between the base case and development scenario.

- EB1 versus RW2+EB3 (yield capped at 76,000 ML/a)

  With the exception of 1969, 1982 and 1994, there were no significant differences between the base case and development scenario at all three significance levels. This indicates that under the upper limit development scenario (with yield capped at 76,000 ML/a), minimal impacts on flow are expected to occur during years of high flow.

  Annual flow in 1969 was 1,935 ML, an extreme low flow year. Analysis of the 1969 data shows significance levels of *P* = 0.1 and 0.05 (*P* = 0.028) (Table 5-2). This result was due to the release of small volumes of water under the development scenario during months that had zero or very little flow under the base case scenario as shown in the hydrograph in Figure 5-3 and detailed in Table 5-3.

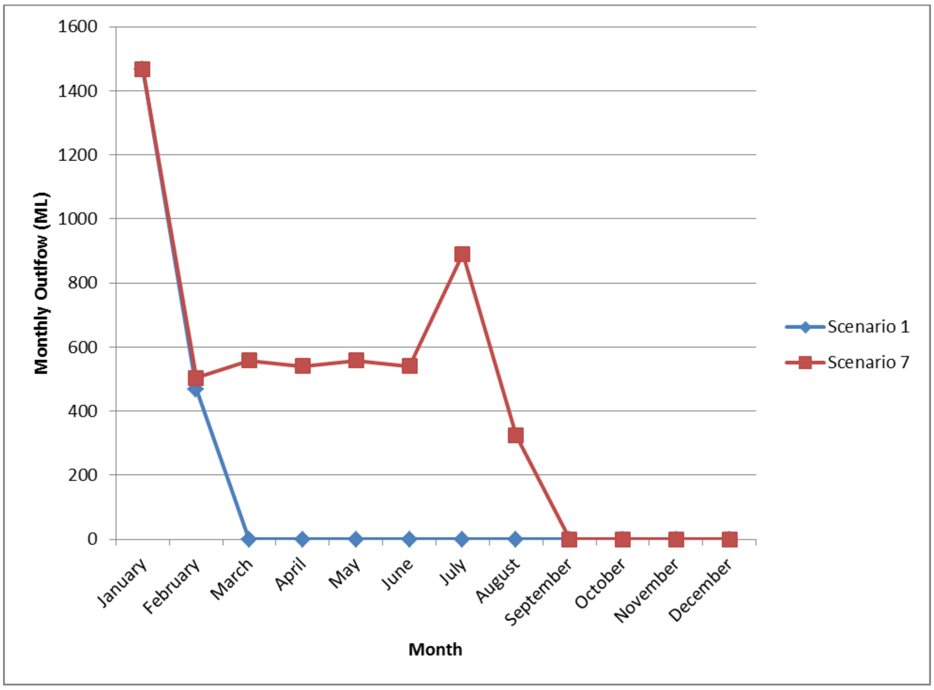
  Annual flow in 1982 (367,382 ML) and 1994 (2,297,885 ML) was low and moderate, respectively. Analysis of the 1982 data shows significance levels at *P* = 0.1 (*P* = 0.073) (Table 5-2). For 1994, significance levels of *P* = 0.1 and *P* = 0.05 (*P* = 0.022) were achieved (Table 5-2). Examination of the base case hydrographs and outflow data for these years (Figure 5-4; Table 5-4 and Figure 5-5; Table 5-5, respectively) identified that the majority of the flows occurred in March (i.e. a large outflow event in an otherwise dry year). The significant differences between the base case and development scenario were due to an initial reduction in flow during the outflow event followed by the release of small volumes of water under the development scenario during months that had zero or very little flow under the base case scenario.

Table 5-2 ANOSIM results summary for all development scenarios at all years analysed

Year[1]	Scenario EB1 vs EB2					Scenario EB1 vs RW1+EB1					Scenario EB1 vs RW2+EB3[2]				
	ANOSIM		Significance level			ANOSIM		Significance level			ANOSIM		Significance level		
	Global R	P-Value	0.1	0.05	0.001	Global-R	P-Value	0.1	0.05	0.001	Global-R	P-Value	0.1	0.05	0.001
1969	-0.022	0.859	ns	ns	ns	-0.022	0.861	ns	ns	ns	0.2	0.026	*	*	ns
1965	-0.076	0.100	ns	ns	ns	-0.066	0.996	ns	ns	ns	-0.038	0.754	ns	ns	ns
1982	0.005	0.411	ns	ns	ns	0.025	0.289	ns	ns	ns	0.101	0.073	*	ns	ns
1952	-0.076	0.992	ns	ns	ns	-0.077	1.0	ns	ns	ns	-0.059	0.890	ns	ns	ns
2007	-0.07	0.990	ns	ns	ns	-0.071	0.99	ns	ns	ns	-0.049	0.880	ns	ns	ns
1909	-0.062	0.981	ns	ns	ns	-0.045	0.858	ns	ns	ns	-0.043	0.821	ns	ns	ns
1994	-0.056	0.932	ns	ns	ns	-0.054	0.920	ns	ns	ns	0.162	0.022	*	*	ns
1913	-0.077	0.996	ns	ns	ns	-0.078	0.999	ns	ns	ns	-0.073	0.990	ns	ns	ns
1998	-0.078	1.0	ns	ns	ns	-0.065	0.952	ns	ns	ns	-0.056	0.898	ns	ns	ns
1988	-0.078	0.998	ns	ns	ns	-0.068	0.997	ns	ns	ns	-0.075	0.994	ns	ns	ns
1928	-0.081	1.0	ns	ns	ns	-0.080	1.0	ns	ns	ns	-0.08	0.998	ns	ns	ns
1976	-0.083	1.0	ns	ns	ns	-0.082	1.0	ns	ns	ns	-0.082	0.999	ns	ns	ns
1918	-0.057	0.891	ns	ns	ns	-0.043	0.765	ns	ns	ns	0.002	0.404	ns	ns	ns

[1]The table is presented in ascending order of total annual flows; [2] Yield capped at 76,000 ML/a; ns = not (statistically) significant; * (statistically) significant

Figure 5-3 1969 hydrograph

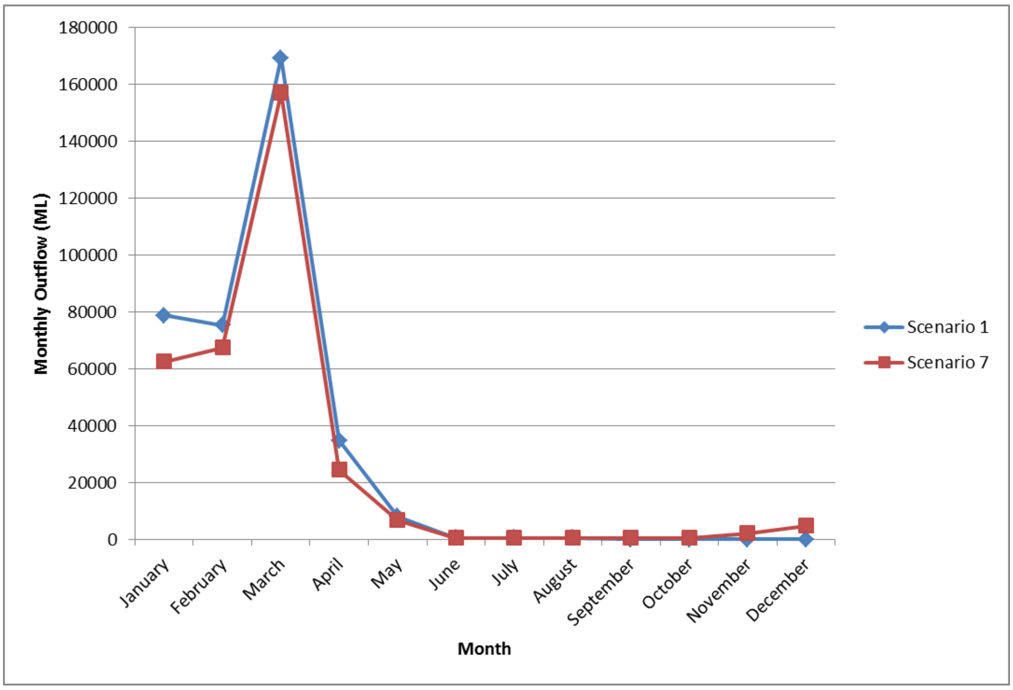


* Scenario 1 = EB1 (base case); Scenario 7 = RW2+EB3 (with yield capped at 76,000 ML/a).

Table 5-3 1969 monthly outflows

Month	Monthly flow (ML)	
	Scenario EB1 (base case)	Scenario RW2+EB3 (yield capped at 76,000 ML/a)
January	1467.1	1467.1
February	468	504
March	0	558
April	0	540
May	0	558
June	0	540
July	0	890
August	0	324
September	0	0
October	0	0
November	0	0
December	0	0

Figure 5-4 1982 hydrograph

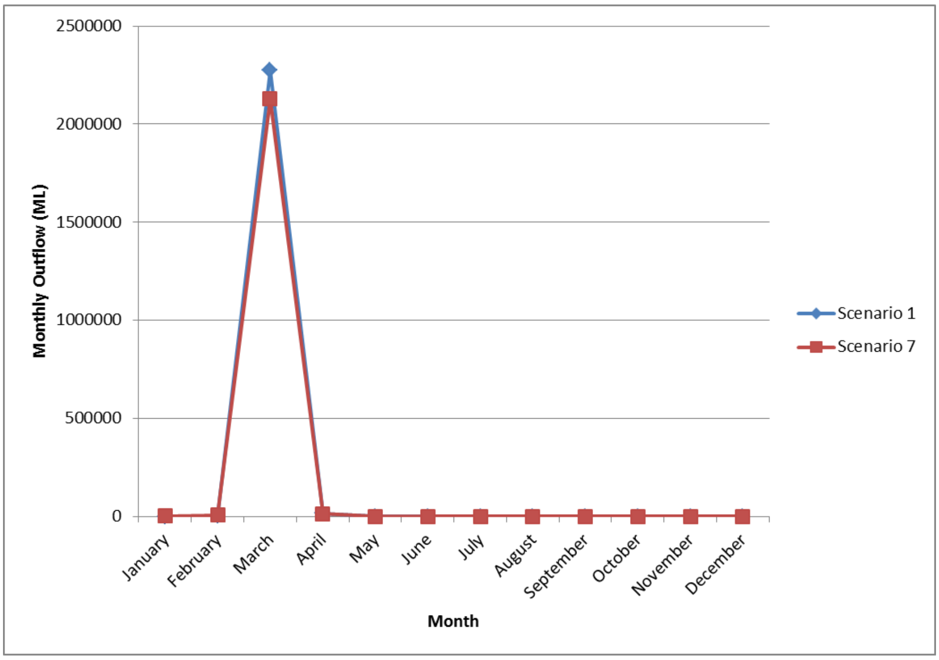


* Scenario 1 = EB1 (base case); Scenario 7 = RW2+EB3 (with yield capped at 76,000 ML/a).

Table 5-4 1982 monthly outflows

Month	Monthly flow (ML)	
	Scenario EB1 (base case)	Scenario RW2+EB3 (yield capped at 76,000 ML/a)
January	78671.3	62475
February	75224.3	67404.6
March	169063.5	156826.8
April	34786.7	24468.5
May	8052.6	6717.4
June	540	540
July	558	558
August	486	558
September	0	540
October	0	558
November	0	2234.1
December	0	4708.9

Figure 5-5 1994 hydrograph



* Scenario 1 = EB1 (base case); Scenario 7 = RW2+EB3 (with yield capped at 76,000 ML/a).

Table 5-5 1994 monthly outflows

Month	Monthly flow (ML)	
	Scenario EB1 (base case)	Scenario RW2+EB3 (yield capped at 76,000 ML/a)
January	0	3058.3
February	3500	6480
March	2275157.1	2127912.6
April	17427.7	14112.9
May	558	558
June	540	540
July	558	558
August	144	558
September	0	540
October	0	558
November	0	540
December	0	523.5

Further investigation of years with similar hydrograph patterns (flood events in dry years) was undertaken to determine if the 1982 and 1994 findings were consistent across the model. Twelve years, with a range of total annual flows, were selected based on hydrograph patterns (Figure 5-6 and Figure 5-7). For all analysed years, there were no significant differences between the base case scenario and the upper limit development scenario (RW2+EB3). This indicates that the results of 1982 and 1994 are not consistent with hydrographs that expressed the same general pattern. This, therefore, suggests that the proposed maximum construction scenario is likely to have minimal impacts on flow levels below the Fitzroy River Barrage.

- EB1 versus RW2+EB3 (theoretical yield of 110,000 ML/a)

  With the exception of 1965, 1982 and 1994, there were no significant differences between the base case and development scenario at all three significance levels as presented in Table 5-6. This indicates under the development scenario (and with a theoretical yield of 110,000 ML/a) minimal impacts on flow are expected to occur during years of high flow. Appendix B shows MDS plots for all analysed years.

  Annual flow in 1965 was 156,816 ML, and in 1982 was 367,382 ML; thus these years are considered to be low flow years. Analysis of the 1965 data found significance levels of P = 0.1 (P = 0.066; Table 5-6) between the base case and the development scenario. Analysis of the 1982 data shows significance levels at P = 0.1 (P = 0.088, Table 5-6). For 1994, a moderate flow year, significance levels of P = 0.1 and P = 0.05 (P = 0.025; Table 5-6) are defined.

The base case hydrographs for these years identified that the majority of the flows occurred during a single month (December in 1965; March in 1983 and 1994; Figure 5-8 to Figure 5-10). These years can therefore be described as having a large outflow event in an otherwise dry year. The significant differences between scenario 1 and scenario 7 (theoretical yield of 110,000 ML/a) were due to an initial reduction in flow during the outflow event followed by the release of small volumes of water under scenario 7 during months that had zero or very little flow under the base case scenario (Figure 5-8 to Figure 5-9 and Table 5-7 to Table 5-9).

Figure 5-6 Hydrographs of selected years 1889, 1906, 1908, 1910, 1911 and 1942

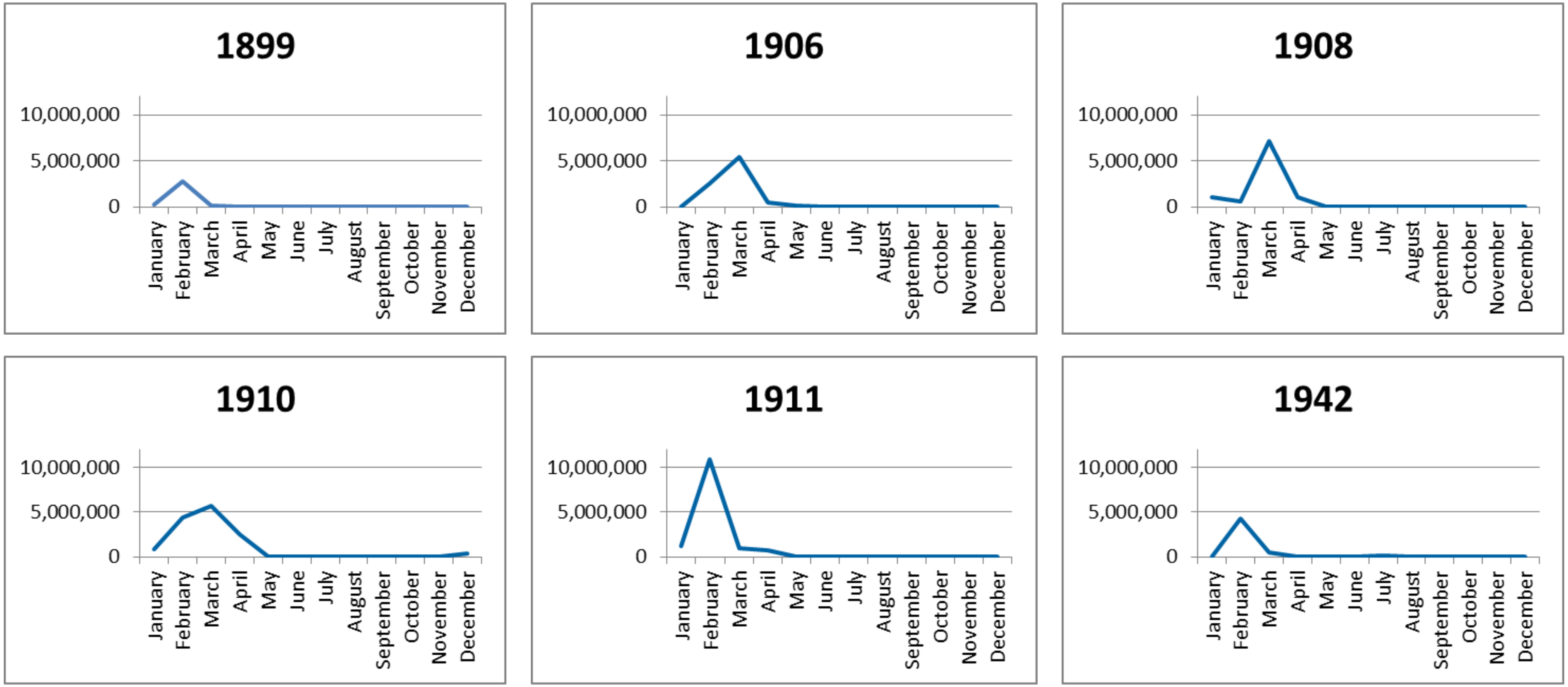


Figure 5-7 Hydrographs of selected years 1947, 1953, 1972, 1988, 1997 and 2003

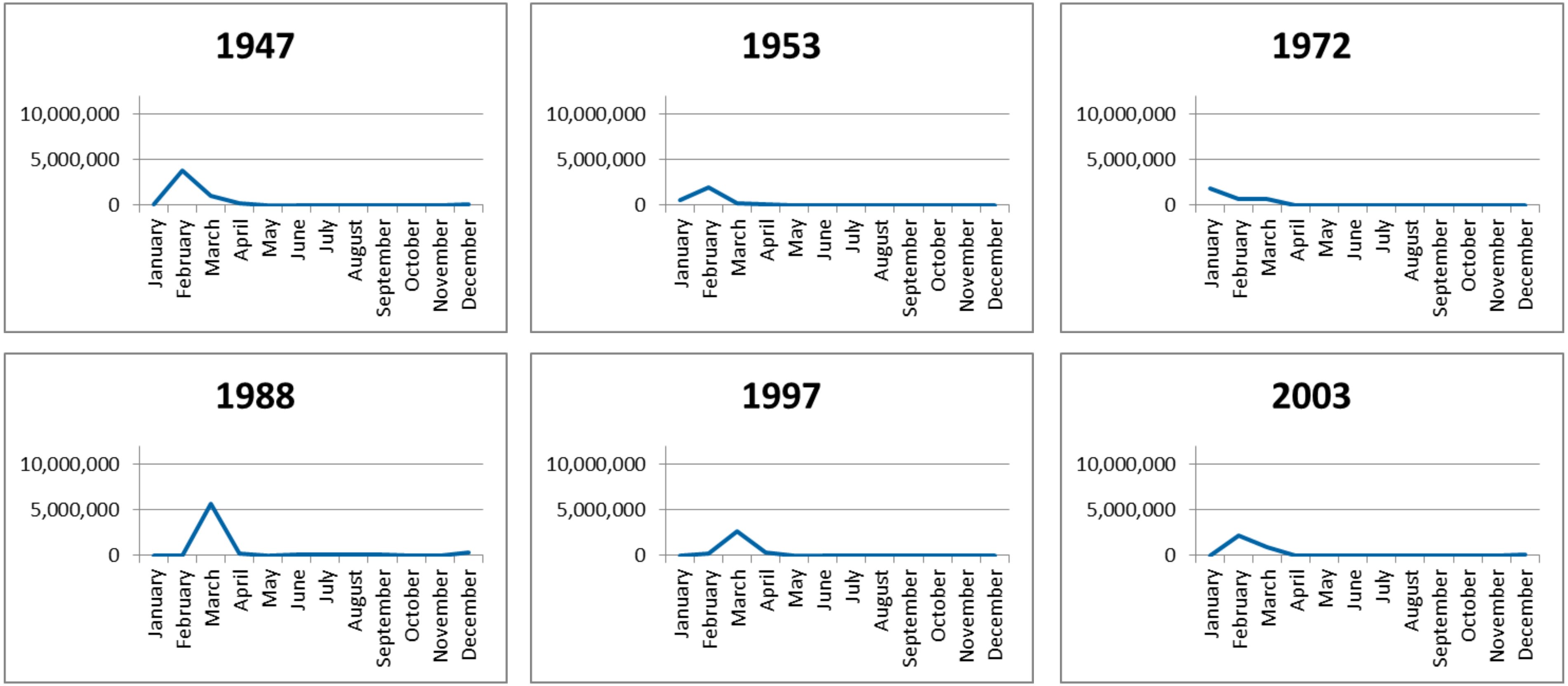
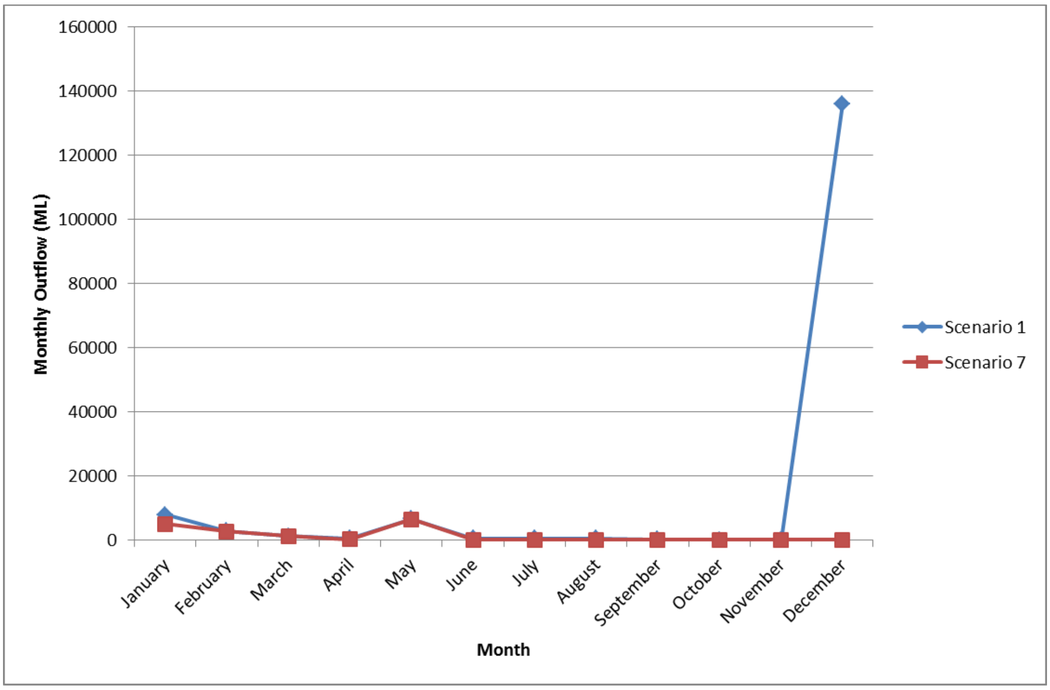


Table 5-6 ANOSIM results summary for EB1 versus RW2+EB3 for all analysed years

Year[1; 2]	ANOSIM		Significance level		
	Global-R	P-value	0.1	0.05	0.001
1969	0.058	0.148	ns	ns	ns
1965	0.142	0.066	*	ns	ns
1982	0.098	0.088	*	ns	ns
1952	-0.056	0.857	ns	ns	ns
2007	-0.048	0.876	ns	ns	ns
1909	-0.045	0.844	ns	ns	ns
1994	0.149	0.025	*	*	ns
1913	-0.075	0.987	ns	ns	ns
1998	-0.049	0.808	ns	ns	ns
1988	-0.072	0.986	ns	ns	ns
1928	-0.081	0.999	ns	ns	ns
1976	-0.082	1.0	ns	ns	ns
1918	-0.004	0.414	ns	ns	ns

[1]The table is presented in ascending order of total annual flows; [2]Theoretical yield (110,000 ML/a); ns = not (statistically) significant; * (statistically) significant

Figure 5-8 1965 hydrograph (EB1 vs RW2+EB3; theoretical yield 110,000 ML/a)

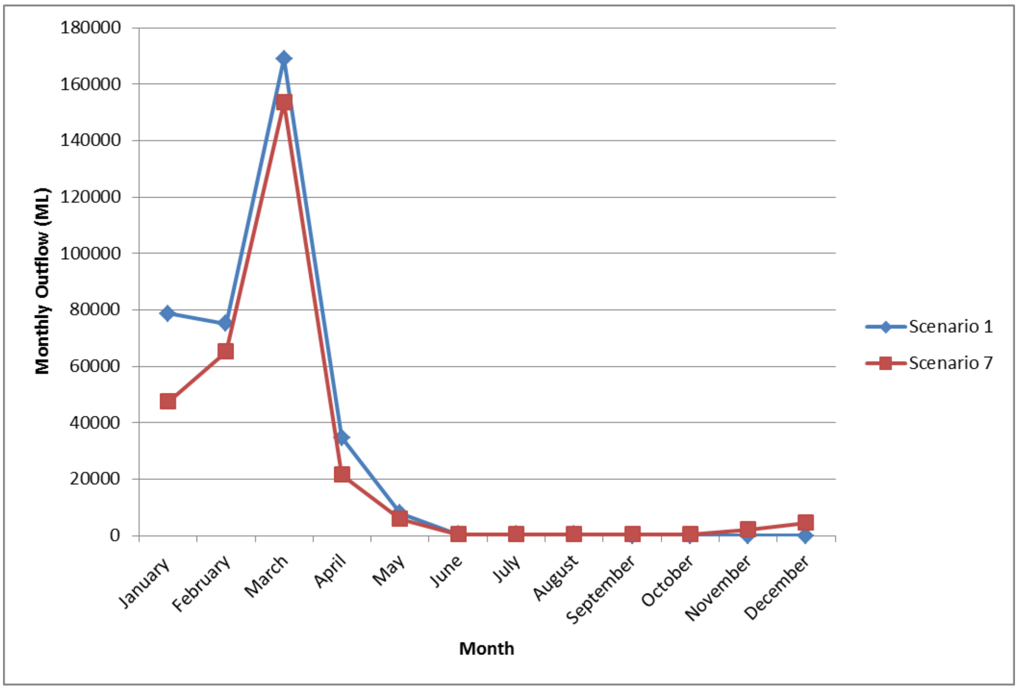


* Scenario 1 = EB1 (base case); Scenario 7 = RW2+EB3 (with yield predicted at 110,000 ML/a).

Table 5-7 1965 monthly outflows (EB1 vs RW2+EB3; theoretical yield 110,000 ML/a)

Month	Monthly flow (ML)	
	Scenario EB1 (base case)	Scenario RW2+EB3 (theoretical yield 110,000 ML/a)
January	7873.3	5030.1
February	2767.1	2713.1
March	1212.5	1176.5
April	540	180
May	6622.7	6424.7
June	540	0
July	558	0
August	558	0
September	216	0
October	0	0
November	0	0
December	135928.7	0

Figure 5-9 1982 hydrograph (EB1 vs RW2+EB3; theoretical yield 110,000 ML/a)

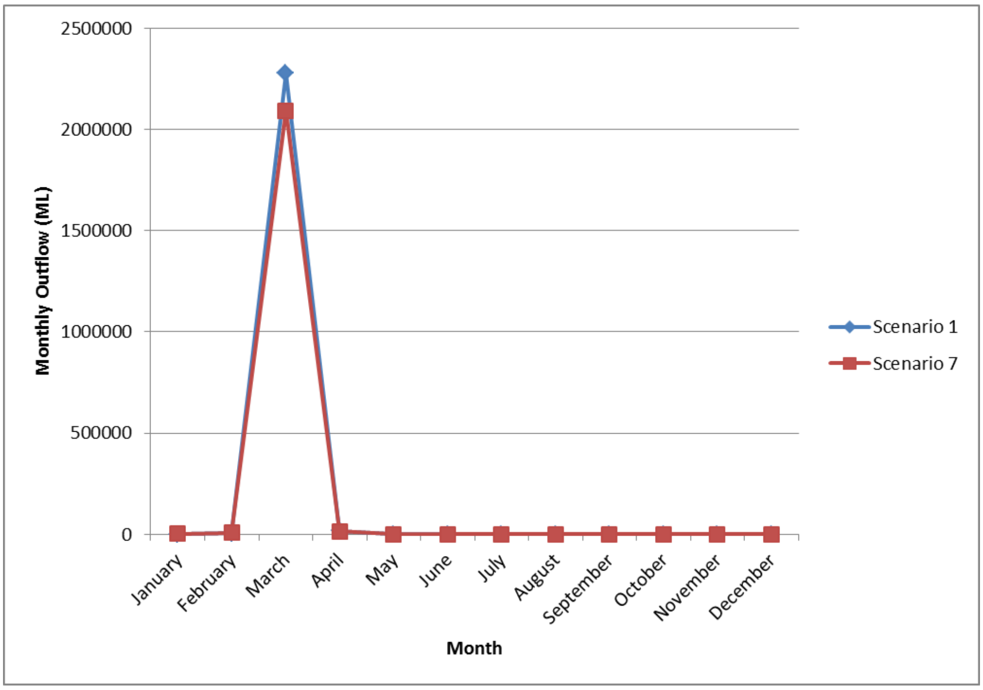


* Scenario 1 = EB1 (base case); Scenario 7 = RW2+EB3 (with yield predicted at 110,000 ML/a).

Table 5-8 1982 monthly outflows (EB1 vs RW2+EB3; theoretical yield 110,000 ML/a)

Month	Monthly flow (ML)	
	Scenario EB1 (base case)	Scenario RW2+EB3 (theoretical yield 110,000 ML/a)
January	78671.3	47487.1
February	75224.3	65211.5
March	169063.5	153578.2
April	34786.7	21587.2
May	8052.6	5951.8
June	540	540
July	558	558
August	486	558
September	0	540
October	0	504
November	0	2180.1
December	0	4618.9

Figure 5-10 1994 hydrograph (EB1 vs RW2+EB3; theoretical yield 110,000 ML/a)



* Scenario 1 = EB1 (base case); Scenario 7 = RW2+EB3 (theoretical yield (110,000 ML/a)).

Table 5-9 1994 monthly outflows (EB1 vs RW2+EB3; theoretical yield 110,000 ML/a)

Month	Monthly flow (ML)	
	Scenario EB1 (base case)	Scenario RW2+EB3 (theoretical yield 110,000 ML/a)
January	0	3058.3
February	3500	6480
March	2275157.1	2092742.9
April	17427.7	13328.9
May	558	558
June	540	540
July	558	558
August	144	558
September	0	540
October	0	540
November	0	468
December	0	486

5.3 Discussion

The results of the flow and statistical analysis have indicated that minimal change in hydrological regime of the Fitzroy River downstream of the Fitzroy Barrage will occur to the estuarine environment as a result of the proposed Project. In all years, flows downstream of the Fitzroy Barrage were not significantly different between the base case (EB1) and development scenarios of EB2 and RW1+EB1. The only potential change in flow occurs in extreme dry years or years with a large outflow event in an otherwise extreme dry year, under the upper limit development scenario (RW2+EB3).

At yields of 76,000 ML/a and 110,000 ML/a, flows downstream of the Fitzroy Barrage under development scenario RW2+EB3 are likely to be maintained for a longer period of time during extreme dry years compared to the base case. When a large outflow event occurs in an otherwise extreme dry year, flows downstream of the Fitzroy Barrage under development scenario RW2+EB3 are likely to initially be reduced during the outflow event followed by an increase in the duration of base flows. This flow regime occurs as initial flows are stored within the impoundments prior to spilling, which subsequently allows for the maintenance of base flows following the event.

The maintenance of base flows (in the order of 500 ML/month) for a longer period of time during extreme dry years is not expected to impact the ecological values of the Fitzroy River estuary or the downstream marine environment. Extreme dry years (as distinct from seasonal dry periods) are known to have a detrimental effect on riverine environments and the maintenance of flows during otherwise low to no flow conditions may actually improve habitat conditions by increasing habitat and resource availability, maintaining/restoring habitat connectivity and improving water quality (Caruso, 2001; Bond et al. 2008). Due to the small volume of the flows downstream of the Fitzroy Barrage, the maintenance of base flows are unlikely to have significant impact on habitat value within the estuary or the marine environment, located approximately 60 km downstream. The maintenance of base flows will, however, serve to prolong the operation of the Fitzroy Barrage fishlock and therefore provide habitat connectivity and fauna movement during these periods.

The reduction in flow downstream of the Fitzroy Barrage during a large outflow event in an otherwise extreme dry year is also considered unlikely to impact the ecological value of the estuarine and marine environments located downstream of the Fitzroy Barrage. Analysis of years with a similar hydrograph (i.e. large outflow event in an otherwise dry year) indicates that the significant reduction in flows downstream of the Fitzroy Barrage will occur on rare occasion only. No significant differences in flows were observed between the base case and development scenario RW2+EB3 for all other years with a large outflow event in an otherwise dry year, that were investigated. The flow reduction that is predicted to occur during these rare events is also small in relation to the volume of the outflow event and the volume of water in the estuary and downstream marine environment. Any changes in the freshwater-tidal interface that may have occurred as a result of the reduction in flow are minimised as a result of the existing Fitzroy Barrage. The rarity that a significant reduction in flow will occur combined with the existing impacts on the freshwater- tidal interface that occurs as a result of the Fitzroy Barrage indicate that impacts to the ecological value of the estuarine and marine environments downstream of the Fitzroy Barrage are unlikely.

6 Hydrologic investigations and modelling

6.1 Introduction

Flood hydrological investigations have been undertaken by GHD for the Eden Bann Weir and Rookwood Weir designs and to inform the EIS. The investigations built on previous work undertaken by SunWater in 2008 to estimate peak flow rates at various locations along the Fitzroy River, including at Eden Bann Weir and the proposed Rookwood Weir site and included an assessment of the large flood that occurred in late December 2010/early January 2011.

6.2 Scope of works

The scope of works in relation to flood hydrological investigations and assessment included:

- Review and update SunWater's runoff-routing model of the catchment
- Calibrate the run-off routing model using historical flood events
- Carry out a flood frequency analysis at gauging stations within the catchment in accordance with Book IV, Australian Rainfall and Runoff (IEAust 1999)
- Estimate the design rainfall depths for the catchment and provide design flood inflow hydrographs for the 1 in 1 year, 1 in 2 year, 1 in 5 year, 1 in 10 year, 1 in 20 year, 1 in 50 year and 1 in 100 year AEP flood events at required locations for inclusion into the hydraulic model
- Estimate climate change impacts within the catchment.

6.3 Approach and methodology

6.3.1 Flood frequency analysis

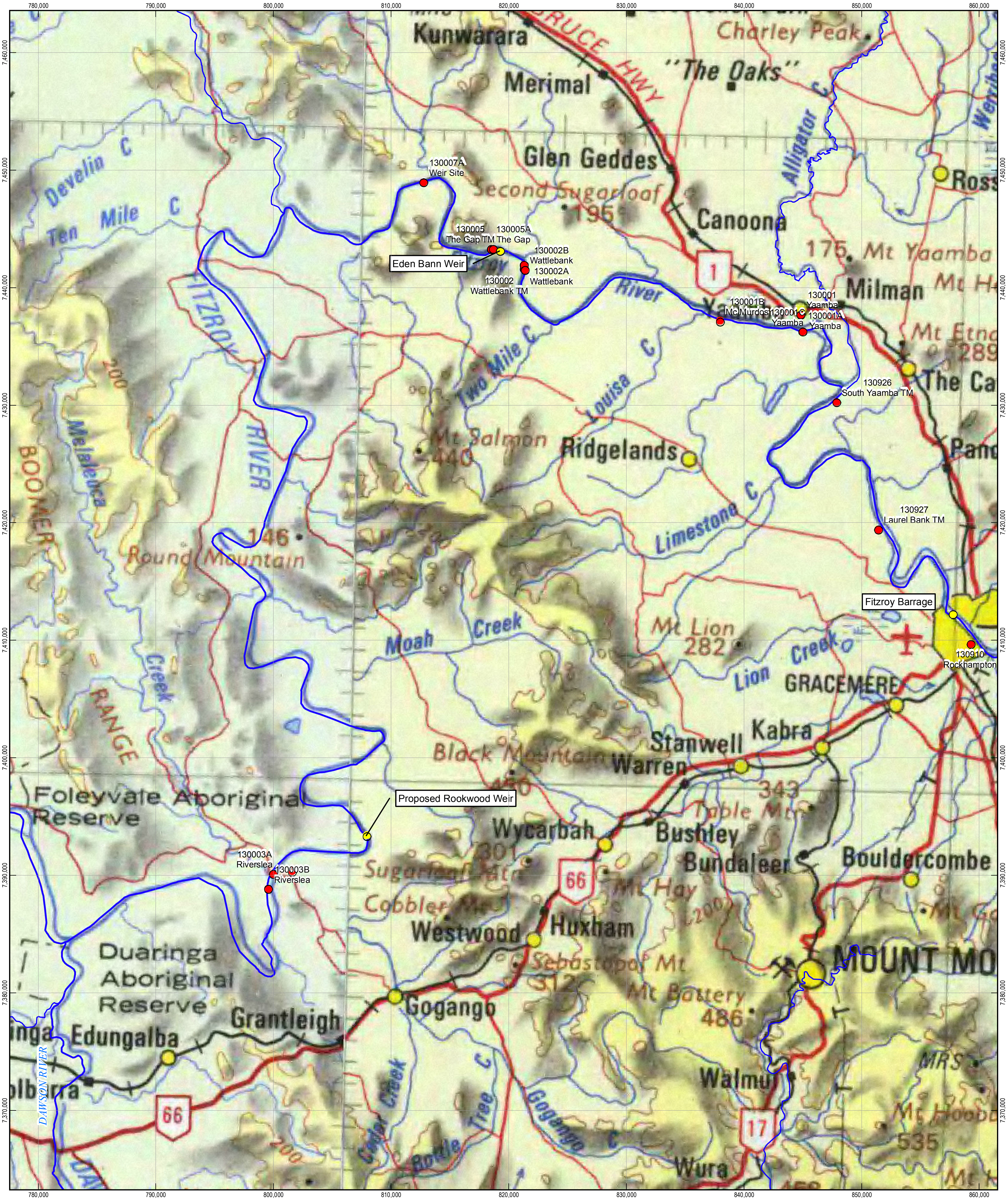
A flood frequency analysis was undertaken using data from the following stream flow gauging stations located on the Fitzroy River in the vicinity of the Project:

- Yaamba - the Yaamba site was treated as a conglomeration of data from gauges 130 001A, 130 001B and 130 001C.
- Wattlebank - the Wattlebank site record was derived by combining data from gauges 130 002A and 130 002B.
- Riverslea - the Riverslea gauge site record was obtained by merging data from gauges 130 003A and 130 003B.
- The Gap – the entire record for the Gap used data from gauge 130 005A.

The gauging stations are described in Table 6-1. Figure 6-1 shows the locations of the stations in relation to the Project.

Table 6-1 Stream gauging stations summary details

Gauge	Station name (Fitzroy River at)	Site		AMTD (km)	Control	Maximum gauged flow (m^3/s)	Date	Catchment area (km^2)	Number of gaugings	Gauge type
		Opened	Closed							
130 001A	Yaamba	01/10/1914	09/11/1927	108.8	Rock weir	17,997	31/01/1918	136,398	64	Staff
130 001B	McMurdos	01/10/1927	31/03/1951	111.4	Sand gravel	10,535	21/01/1951	136,356	132	Staff
130 001C	Yaamba	01/10/1950	31/12/1973	108.8	Rock weir	15,085	18/02/1954	136,398	75	Staff
130 002A	Wattlebank	30/11/1918	05/10/1958	137.9	Sand gravel	17,962	17/02/1954	135,933	232	Staff
130 002B	Wattlebank	19/08/1994	01/07/2002	139.0	Control weir	4,353	08/09/1998	135,932	25	Pressure sensor level recorder
130 003A	Riverslea	19/03/1922	01/10/1974	274.4	Rock outcrop	19,553	15/02/1954	131,385	130	Staff
130 003B	Riverslea	01/10/1974	Current	276.0	Gravel-rock	15,211	08/01/1991	131,385	130	Pressure sensor level recorder
130 05A	The Gap	30/04/1964	Current	142.1	Weir	14,550	10/01/1991	135,757	222	Pressure sensor level recorder



LEGEND

Weir Sites	Principal Rivers
Stream Gauges	River Tributary

Paper Size A3

0 2.5 5 7.5 10

Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 55

N

Gladstone Area Water Board | SunWater

Gladstone Area Water Board, SunWater
Lower Fitzroy River Infrastructure Project

Job Number	41-20736
Revision	0
Date	31 Oct 2014

Gauging Station Locations

Figure 6-1

G:\41\20736\GIS\Projects\MXD\260_Hydrological_Analysis\4120736_276_Rev_A.mxd

Suite 4, Level 2, 1-3 Annand Street Toowoomba QLD 4350 Australia T 61 7 4633 8000 F 61 7 4613 1687 E tbamail@ghd.com W www.ghd.com

Data source: GA - Raster Premium 1:1 Million Scale Mosaic (2005). GHD - Weir Sites, Stream Guages (2011). Created by: CM

An annual series approach was adopted for the flood frequency analysis using a Log Pearson III distribution (LPIII) based on the peak flow rate in each water year. The water year was assumed to start in October and concluded in September of the following year. Each streamflow record was examined to identify those years where data was missing. Where streamflow records were missing for part of a year, daily rainfall records were scrutinised to determine whether that period coincided with very high rainfall totals. In certain instances, the water year peak flow rate was retained where it was evident the rainfall coinciding with missing streamflow data period was significantly lower than the rainfall that generated the peak flow recorded in that particular water year. Where rainfall was higher for a period of missing gauging data than the largest recorded flow within a water year, that water year was discarded from the analysis.

All gauging data was organised into water years with the analysis determining the percentage of data that was coded into the categories 'good', 'fair', 'suspect', 'poor' and 'missing' for each water year. In general, the Riverslea and Yaamba gauges appear to have better quality gauging data compared to Wattlebank and The Gap. The Riverslea and Yaamba gauges have relatively fewer water year records where more than half the record is missing, and have a greater proportion of water years where more than 75% and 90% of the record is coded as being good quality or fair quality data.

The outcomes of the flood frequency analysis concluded that the Riverslea and Yaamba sites are considered reliable estimates for the lower Fitzroy River. Given that the Riverslea site has the longer record, it was determined that the Riverslea gauge be adopted.

6.3.2 Flood hydrology model

6.3.2.1 Model description

Consistent with SunWater (2008), the URBS[2] split mode of operation (Carroll, 2004) was adopted for the Project. In this mode the distinction is made between catchment routing and river channel routing.

For catchment routing, rainfall is initially routed through a time-area diagram which takes into account the effect of catchment slope, urbanisation and fraction forested on concentration time for sub-catchment runoff. Following this, runoff is routed through a non-linear reservoir according to a defined storage-discharge relationship. Channel routing is based on the non-linear Muskingum model.

6.3.2.2 Catchment description

As per SunWater (2008), the Fitzroy River catchment was delineated into 113 sub-catchments.

SunWater (2008) identified that during floods, the area immediately upstream of Riverslea Crossing, at the confluence of the Mackenzie and Dawson Rivers, acts as a large detention basin and controls flows into the Fitzroy River. Using ALS data the height-volume relationship could be derived. The rating curve identifying the height-discharge relationship was determined from the hydraulic model developed for the Project (GHD 2012).

[2] Unified River Basin Simulator. URBS refers to a rainfall runoff routing model for flood forecasting and design. That is a model that unites a rainfall runoff model together with a runoff routing model.

6.3.2.3 Model calibration

To calibrate the model to the major historical floods that have occurred in the Fitzroy River Basin, seven historical storm events with satisfactory data were considered as described in Table 6-2. Table 6-2 also provides the total rainfall volume for the event which is seen to last for several weeks and incorporates storm bursts that yield flood peaks. The calibration aimed to match the peak outflow rate, volume of runoff, and the time to peak of each event.

Table 6-2 Estimated rainfall totals and AEP for historical storm events

Event	Average catchment rainfall (mm)	Estimated storm duration (days)	AEP event* (peak storm burst) (tyears)
January 1918	673	42	1 in 50
February 1954	500	28	1 in 50 to 1 in 100
May 1983	405	28	1 in 5 to 1 in 10
March 1988	144	28	1 in 5
December 1990	360	28	1 in 10
February 1991	142	28	1 in 5
December 2010/January 2011	642	42	1 in 20 to 1 in 50

* Based on the estimated design rainfall totals for the entire Fitzroy River catchment.

All seven events were major flood events however the January 1918 and December 2010 events were the largest events in terms of rainfall across the catchment (673 mm and 642 mm, respectively). The February 1954, May 1983 and the January 1991 events were the next biggest events generating rainfall totals between 400 mm and 500 mm. The March 1988 and February 1991 events generated the least rainfall with totals of approximately 140 mm across the catchment.

6.3.2.4 Adopted preliminary model parameters

A reasonably calibrated event obtains results within 10% between the simulated result and the recorded data. An average of the model calibration parameters was adopted for the design event hydrology. Two averages were computed:

- A straight average of the parameters; and
- As a function of recorded peak flow rate at the Riverslea gauge.

In deriving the averages, only those events with consistent results were considered. As such it was determined that the January 1918 and the February 1991 events cannot be calibrated satisfactorily. The 1918 event has been identified as having no pluviograph stations available and the 1991 event has been identified as lacking accurate pluviograph data for various stations.

6.3.3 Design flood event hydrology

Design flood event hydrology included:

- Producing estimated design flood event hydrographs, using the calibrated URBS model with the design event rainfall data. The assessment of design floods was limited to 12 to 72 hour durations from the 1 in 1 year AEP event up to the 1 in 100 year AEP event
- Determining frequent to large event rainfall estimates using the DNRW Rainfall program (2005). Rainfall estimates were derived using the CRC-FORGE method (IEAust 1999) with the rainfall program (DNRW 205) allowing for areal reduction factors to convert point rainfall to areal estimates based on the methodology as outlined by Siriwadena and Weinmann. Adopted design rainfall depths are provided in Table 6-3 for various durations and AEP events. Table 6-3 also shows the adopted design rainfall losses
- Consideration of ARR temporal patterns (IEAust 1999) and GTSMR temporal patterns (BOM 2003). AAR temporal patters are recommended for frequent to large events for AEPs up to 1 in 100 year. GTSMR temporal patterns are meant to be applied to rare and extreme rainfalls. However these were considered for the Project as they represent storm durations up to and including five days. ARR temporal patterns only cover storm durations up to an including three days. Longer duration patterns were investigated as historical rainfall events were seen to persist for several days
- Basing the spatial distribution of rainfall on the topographic adjustment factors provided by the GTSMR technique
- Running the calibrated Fitzroy River Basin URBS model for a range of storm duration events with AEPs from the 1 in 1 to the 1 in 100 year AEP events. Table 6-4 summarises simulated peak discharges through the Riverselea gauging station. The critical storm duration for all design events was three days based on the ARR temporal patterns. Simulations were conducted using the GTSMR temporal patterns however it was fund that lower peak flow rates were estimated. Table 6-4 compares flood frequency results between historical (annual series) and estimated (simulated using the URBS model) data and indicates good agreement for all AEPs (differences of less than four per cent).

Table 6-3 Adopted design rainfalls and rainfall losses

AEP (1 in X year event)	Design rainfall (mm)					Rainfall losses (mm)	
	24 hs	48 hrs	72 hrs	96 hrs	120 hrs	Initial	Continuing
1	55	85	100	110	120	-	-
2	65	95	115	125	135	60	2.7
5	79	118	140	155	165	40	2.7
10	91	136	163	179	191	25	2.7
20	107	161	192	211	226	25	2.7
50	130	195	232	256	273	25	2.7
100	148	220	263	289	209	25	2.7

Table 6-4 Design flood event summary (Riverslea gauge)

AEP (1 in X year)	Critical duration (hrs)	Time to peak (hrs)	Peak flow (m^3/s)	Annual series (m^3/s)	Volume (GL)
2	72	210	2,200	2,300	2,100
5	72	203	5,600	5,500	5,100
10	72	206	8,200	8,230	7,300
20	72	209	11,400	11,200	10,000
50	72	208	15,500	15,400	13,400
100	72	208	19,700	18,800	16,900

Further the design flood hydrology considered the variability of rainfall across the Dawson, Isaac, Lower Fitzroy and Mackenzie River catchments. It is considered rare that a rainfall event will impact all the catchments within the same period of time with a consistent intensity. It is intended that by schematically applying rainfall events to selected catchments an understanding of the catchment response can be obtained (spatial distribution of rainfall sensitivity analysis).

Different scenarios were adopted applying rainfall to selected catchments to represent the most likely events as follows:

- Rainfall over all catchments
- Rainfall over the Isaac River catchment only
- Rainfall over the Mackenzie and Dawson Rivers catchments only
- Rainfall over the Mackenzie, Isaac and Lower Fitzroy Rivers catchments only
- Rainfall over the Mackenzie Dawson and Lower Fitzroy Rivers catchment.

The design rainfall totals were estimated for each scenario using the Rainfall application (DNRM 205). IRBS model files were created for each scenario for events ranging from the 1 in 1 year AEP to the 1 in 100 year AEP. The peak outflow rates at the Riverslea gauge site are summarised in Table 6-5.

Table 6-5 Design flood event rainfall variability sensitivity analysis summary (Riverslea gauge)

AEP(1 in X year)	Peak flow (m^3/s) per catchment scenario				
	All catchments	Isaac River	Mackenzie and Dawson Rivers	Mackenzie, Isaac and Lower Fitzroy Rivers	Mackenzie, Dawson and Lower Fitzroy Rivers
2	2,200	2,500	1,300	2,000	1,400
5	5,600	4,700	3,100	5,000	3,600
10	8,200	6,300	4,700	6,800	5,400
20	11,400	8,400	7,000	9,600	7,700
50	15,500	10,900	9,200	12,800	10,300
100	19,700	13,400	11,800	16,100	13,300

Table 6-5 indicates that the scenario representing rainfall across the whole catchment is the 'worst case' scenario as the peak outflow rates exceed the other scenarios, except for the Isaac River scenario during the 1 in 1 year AEP event. During this event an extra 200 m^3/s (approximately) is passing through the Riverslea gauge site. The scenario that closely represents the performance of the total catchment is when the Mackenzie, Isaac and Lower Fitzroy Rivers are receiving rainfall events.

6.3.4 Climate change

Potential climate change impacts were assessed using the Guidelines for Preparing a Climate Change impact Statement (EPA 2008). For Queensland catchments, a five per cent increase in rainfall intensity per degree of global warming must be used for the 1 in 100 year, 1 in 200 year and 1 in 500 year AEP events. The projected temperature increases specific to the central Queensland region are determined to be:

- +1.0 ˚C by 2030
- +2.0 ˚C by 2050
- +3.2 ˚C by 2070.

All three climate scenarios have been considered for the Project and rainfall intensity increases have been incorporated as follows:

- Five per cent by 2030
- Ten per cent by 2050
- Fifteen per cent by 2070.

The peak flow rates at the Riverslea gauge obtained from the hydrology model for each climate scenario as compared to the current peak flow for all catchments are provided in Table 6-6.

Table 6-6 Estimated climate change peak flows (Riverslea gauge)

AEP (1 in year)	Peak flow (m^3/s) (% increase)			
	Current	2030	2050	2070
2	2,300	2,600 (18)	3,00 (36)	3,400 (55)
5	5,500	6,500 (16)	7,300 (30)	8,100 (45)
10	8,200	9,100 (11)	9,900 (21)	10,800 (32)
20	11,200	12,700 (11)	14,000 (23)	15,300 (34)
50	15,500	17,100 (10)	18,700 (21)	20,300 (31)
100	19,000	21,600 (10)	23,500 (19)	25,500 (29)

The results suggest that a five per cent increase in rainfall intensity (2030 scenario 0 will increase peak flows by a minimum of approximately ten per cent. Increases in precipitation by 10 per cent (2,050 m scenario) will increase peak flows by a minimum of approximately 20 per cent. In 2070, an increase in precipitation by 15 per cent will increase peak flows by approximately 30 per cent.

6.4 Flood flows

6.4.1 Eden Bann Weir

The existing (Eden Bann Stage 1) river geometry has been modelled for the 1 in 2 year, 1 in 5 year, 1 in 10 year, 1 in 20 year, 1 in 50 year and 1 in 100 year AEP events. The estimated peak water levels for the modelled AEPs pre- and post-development are compared and presented in Table 6-7 for locations along the Fitzroy River. The generated afflux for the design event (that is Eden Bann Weir raised to Stage 2, FSL 18.2 m) at each AEP in presented in Table 6-8 for locations along the Fitzroy River.

Appendix E presents the existing and post development flood extents for the 1 in 2 year, 1 in 5 year, 1 in 10 year, 1 in 20 year, 1 in 50 year and 1 in 100 year AEP scenarios.

6.4.2 Rookwood

The existing (pre-Rookwood Weir) Fitzroy River geometry has been modelled for the 1 in 2 year, 1 in 5 year, 1 in 10 year, 1 in 20 year, 1 in 50 year and 1 in 100 year AEP events. The estimated peak water levels for the modelled AEPs (pre- and post-development) are presented in Table 6-9 for locations along the Fitzroy River. The generated afflux for the design event (that is Rookwood Weir Stage 2 (FSL 45.5 m AHD) at each AEP is presented in Table 6-10 for locations along the Fitzroy River.

The peak water levels for all AEPs were assessed at the Capricorn Highway crossing and Foleyvale Crossing. The estimated peak water levels for the modelled AEPs are shown in Table 6-11. Only 1 in 2 year AEP event is modelled (refer to Section 7.4).

Appendix F presents the existing and post development flood extents for the 1 in 2 year, 1 in 5 year, 1 in 10 year, 1 in 20 year, 1 in 50 year and 1 in 100 year AEP scenarios.

6.4.3 Climate change

As the long-term development of the sites is being considered, the influence of climate change has been assessed using recommendations from the Final Report on the Inland Flooding Study (State of Queensland, 2010). Two climate change horizons were assessed, 2030 and 2070, which equate to an increase in rainfall of five per cent and 15 per cent respectively.

The resulting increase in existing peak water levels are displayed in Table 6-12 for the 2030 and 2070 horizons for Eden Bann Weir and the Rookwood site. The estimated climate change increase for the 2030 horizon is between 0.3 m and 0.9 m with the larger increases occurring with the 1 in 2 year and the 1 in 5 year AEP events. By 2070, the estimated increase in peak water levels is between 0.7 m and 2.7 m, with the larger increases again occurring with the 1 in 2 and the 1 in 5 year AEP events.

The increase in peak water levels under the 2030 and 2070 climate change horizons with the proposed raising of Eden Bann Weir and construction of Rookwood Weir are shown in Table 6-13 and Table 6-14. Table 6-13 and Table 6-14 represent the difference between estimated peak water levels for the development weir case with climate change and the estimated peak water levels for the development weir case.

Table 6-7 Estimated peak water levels for modelled AEPs pre- and post-development of Eden Bann Weir

Location	Peak water level (m AHD)											
	AEP 1 in 2 year		AEP 1 in 5 year		AEP 1 in 10 year		AEP 1 in 20 year		AEP 1 in 50 year		AEP 1 in 100 year	
	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post
Upstream	52.8	52.8	59.8	59.8	62.1	62.1	64	64	65.8	65.8	66.8	66.8
Riverslea gauge	45.9	45.9	53.3	53.3	56.1	56.1	59	59	61.2	61.2	62.1	62.1
Rookwood	44.2	44.2	51.4	51.4	54.1	54.1	56.7	56.7	58.8	58.8	59.5	59.5
Weir site gauge	19.9	22.5	26.2	26.6	28.7	28.9	30.6	30.7	32.4	32.6	33.9	34
The Gap Gauge	18.2	21.8	24.4	24.9	26.9	27.2	28.7	28.9	30.3	30.5	31.5	31.7
Eden Bann Weir	18.1	21.8	24.4	24.9	26.9	27.2	28.6	28.9	30.2	30.5	31.5	31.7
Wattlebank gauge	17.5	17.4	24	24	26.5	26.5	28.2	28.2	29.8	29.8	30.9	30.9

Table 6-8 Generated afflux for design event at Eden Bann Weir

Location	Afflux (m)					
	AEP 1 in 2 year	AEP 1 in 5 year	AEP 1 in 10 year	AEP 1 in 20 year	AEP 1 in 50 year	AEP 1 in 100 year
Upstream	0.0	0.0	0.0	0.0	0.0	0.0
Riverslea gauge	0.0	0.0	0.0	0.0	0.0	0.0
Rookwood	0.0	0.0	0.0	0.0	0.0	0.0
Weir site gauge	2.6	0.3	0.2	0.2	0.2	0.1
The Gap Gauge	3.6	0.5	0.3	0.3	0.3	0.2
Eden Bann Weir	3.6	0.5	0.3	0.3	0.3	0.2
Wattlebank gauge	0.0	0.0	0.0	0.0	0.0	0.0
Downstream	0.0	0.0	0.0	0.0	0.0	0.0

Table 6-9 Estimated peak water levels for modelled AEPs pre- and post-development of Rookwood on the Fitzroy River

Location	Peak water level (m AHD)											
	AEP 1 in 2 year		AEP 1 in 5 year		AEP 1 in 10 year		AEP 1 in 20 year		AEP 1 in 50 year		AEP 1 in 100 year	
	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post
Upstream	52.8	53.4	59.8	60	62.1	62.2	64	64	65.8	65.8	66.8	66.8
Riverslea gauge	45.9	49.7	53.3	54.1	56.1	56.5	59	59.2	61.2	61.3	62.1	62.7
Rookwood	44.2	49.2	51.4	52.5	54.1	54.7	56.7	57.1	58.8	58.9	59.5	59.6
Weir site gauge	19.9	19.9	26.2	26.2	28.7	28.7	30.6	30.6	32.4	32.4	33.9	33.9
The Gap Gauge	18.2	18.2	24.4	24.4	26.9	26.9	28.7	28.7	30.3	30.3	31.5	31.5
Eden Bann Weir	18.1	18.1	24.4	24.4	26.9	26.9	28.6	28.6	30.2	30.2	31.5	31.5
Wattlebank gauge	17.5	17.5	24	24	26.5	26.5	28.2	28.2	29.8	29.8	30.9	30.9
Downstream	16.3	16.3	23	23	25.6	25.6	27.1	27.1	28.5	28.5	29.7	29.7

Table 6-10 Generated afflux for design event at Rookwood Weir and locations on the Fitzroy River

Location	Afflux (m)					
	AEP 1 in 2 year	AEP 1 in 5 year	AEP 1 in 10 year	AEP 1 in 20 year	AEP 1 in 50 year	AEP 1 in 100 year
Upstream	0.6	0.2	0.1	0.0	0.0	0.0
Riverslea gauge	3.7	0.8	0.4	0.2	0.1	0.0
Rookwood	5.0	1.2	0.6	0.4	0.2	0.2
Weir site gauge	0.0	0.0	0.0	0.0	0.0	0.0
The Gap Gauge	0.0	0.0	0.0	0.0	0.0	0.0
Eden Bann Weir	0.0	0.0	0.0	0.0	0.0	0.0
Wattlebank gauge	0.0	0.0	0.0	0.0	0.0	0.0
Downstream	0.0	0.0	0.0	0.0	0.0	0.0

Table 6-11 Estimated peak water levels and generated afflux pre- and post-development of Rookwood on the Fitzroy River

AEP (1 in Y)	Peak water levels (m AHD)				Afflux (m)	
	Existing (pre-Rookwood)		Post-development		Foleyvale Crossing	Capricorn Highway crossing
	Foleyvale Crossing	Capricorn Highway crossing	Foleyvale Crossing	Capricorn Highway crossing		
2	57.01	60.26	57.23	60.35	0.22	0.09
5	62.59	64.11	-	-	-	-
10	64.15	65.07	-	-	-	-
20	65.48	66.02	-	-	-	-
50	66.85	67.11	-	-	-	-
100	67.77	67.79	-	-	-	-

Table 6-12 Estimated increase in existing peak water levels under climate change scenarios

	Increase in peak water level (m)											
	Climate change horizon 2030						Climate change horizon 2070					
AEP event	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100
Upstream end of model	0.9	0.8	0.5	0.6	0.4	0.4	2.4	2.2	1.6	1.5	1.1	1.1
Riverslea gauge	0.9	0.9	1.0	0.7	0.4	0.3	2.5	2.7	2.5	1.8	0.9	0.8
Rookwood	0.9	0.9	0.9	0.7	0.3	0.3	2.5	2.6	2.3	1.7	0.8	0.7
Weir site gauge	0.7	0.8	0.7	0.5	0.5	0.6	2.0	2.4	1.7	1.4	1.6	1.7
The Gap gauge	0.6	0.8	0.7	0.4	0.5	0.5	1.9	2.4	1.5	1.2	1.4	1.5
Eden Bann Weir	0.6	0.8	0.7	0.4	0.5	0.5	1.9	2.4	1.5	1.2	1.4	1.5

	Increase in peak water level (m)											
	Climate change horizon 2030						Climate change horizon 2070					
AEP event	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100
Wattlebank gauge	0.7	0.8	0.7	0.4	0.4	0.5	2.2	2.4	1.5	1.2	1.4	1.4
Downstream end of model	0.8	0.8	0.7	0.4	0.4	0.5	2.2	2.5	1.4	1.0	1.4	1.4

Table 6-13 Estimated increase in post-development (Eden Bann Weir) peak water levels under climate change scenarios

	Increase in peak water level (m)											
	Climate change horizon 2030						Climate change horizon 2070					
AEP event	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100
Upstream end of model	0.9	0.8	0.5	0.6	0.4	0.4	2.4	2.2	1.6	1.5	1.1	1.1
Riverslea gauge	0.9	0.9	1.0	0.7	0.4	0.3	2.5	2.7	2.5	1.8	0.9	0.8
Rookwood	0.9	0.9	0.9	0.7	0.3	0.3	2.5	2.6	2.3	1.7	0.8	0.7
Weir site gauge	0.7	0.8	0.7	0.5	0.5	0.6	2.0	2.4	1.7	1.4	1.6	1.7
The Gap gauge	0.6	0.8	0.7	0.4	0.5	0.5	1.9	2.4	1.5	1.2	1.4	1.5
Eden Bann Weir	0.6	0.8	0.7	0.4	0.5	0.5	1.9	2.4	1.5	1.2	1.4	1.5
Wattlebank gauge	0.7	0.8	0.7	0.4	0.4	0.5	2.2	2.4	1.5	1.2	1.4	1.4
Downstream end of model	0.8	0.8	0.7	0.4	0.4	0.5	2.2	2.5	1.4	1.0	1.4	1.4

Table 6-14 Estimated increase in post-development (Rookwood Weir) peak water levels under climate change scenarios

	Increase in peak water level (m)											
	Climate change horizon 2030						Climate change horizon 2070					
AEP event	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100
Upstream end of model	0.8	0.7	0.5	0.6	0.4	0.4	2.3	2.1	1.5	1.5	1.1	1.1
Riverslea gauge	0.5	0.7	0.9	0.7	0.4	0.3	1.4	2.4	2.3	1.7	0.9	0.9
Rookwood	0.4	0.7	0.8	0.6	0.3	0.2	1.0	2.1	2.0	1.6	0.8	0.9
Weir site gauge	0.7	0.8	0.7	0.5	0.5	0.6	2.1	2.4	1.7	1.4	1.6	1.7
The Gap gauge	0.6	0.8	0.7	0.4	0.5	0.5	1.9	2.4	1.5	1.2	1.4	1.5
Eden Bann Weir	0.6	0.8	0.7	0.4	0.5	0.5	2.0	2.4	1.5	1.2	1.4	1.5
Wattlebank gauge	0.8	0.8	0.7	0.4	0.4	0.5	2.2	2.4	1.5	1.2	1.3	1.4
Downstream end of model	0.8	0.8	0.7	0.4	0.4	0.5	2.2	2.5	1.4	1.0	1.4	1.4

For Eden Bann Weir, the estimated influence of climate change in 2030 is similar to the existing weir scenario with increases with the proposed weir raising being between 0.3 m and 0.9 m, with the higher increases associated with the 1 in 2 year and 1 in 5 year AEP events. The estimated influence of climate change in 2070 is also similar to the existing weir scenario with the proposed weir raising scenario yielding increases between 0.7 m and 2.7 m, with the higher increases associated with the 1 in 2 year and 1 in 5 year AEP events.

For the proposed Rookwood Weir the estimated influence of climate change in 2030 is similar to the existing weir scenario with increases with the proposed weir raising being between 0.2 m and 0.9 m, with the higher increases associated with the 1 in 2 year and 1 in 5 year AEP events. The estimated influence of climate change in 2070 is also similar to the existing weir scenario with the proposed weir raising scenario yielding increases between 0.9 m and 2.4 m, with the higher increases associated with the 1 in 2 year and 1 in 5 year AEP events.