

## 15 DESIGN EVENTS MODELLING

### 15.1 Design Rainfall

The temporal patterns and design rainfalls for design events up to the 1%AEP were derived from procedures in AR&R giving a relationship between rainfall intensity, duration, and frequency (known as IFD). The following key parameters were obtained from the Bureau of Meteorology website based on procedures in AR&R87.

**Table 15.1: Geographic Rainfall Factors for Bega and Brogo Rivers Catchment**

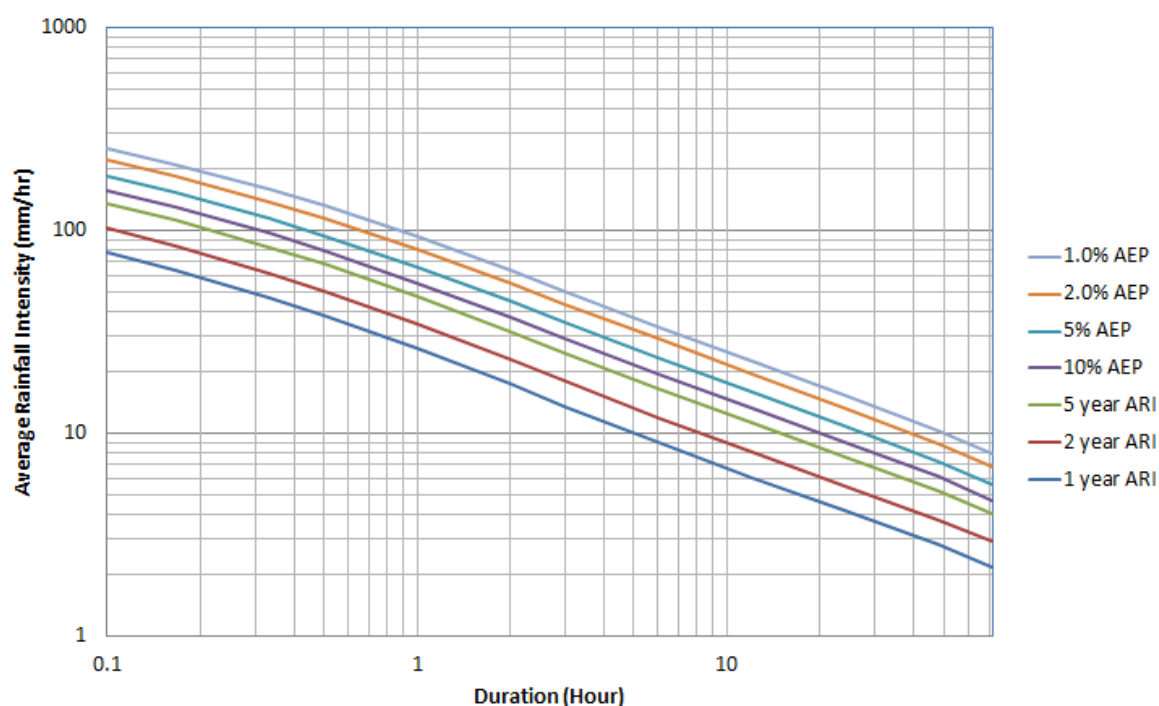
Factor	Value
Skewness G	0.17
F2	4.23
F50	15.69

**Table 15.2: Log Normal Intensities for Bega and Brogo Rivers Catchment**

Duration	2 Year ARI	50 Year ARI
1 hr	34.93	78.02
12 hr	8.08	18.8
72 hr	2.95	6.6

The basic IFD data above was applied in the XP-RAFTS model for design events of 10%, 5%, 2% and 1% AEP events. A graph of the IFD data showing the relationship between rainfall intensity, duration, and frequency is shown below.

**Figure 15.1: Design Rainfall Relationship – Intensity Duration Frequency**



As often found in practice, shorter duration rainfall events have higher rainfall intensities for a given frequency, with longer duration storms having lower average intensities but larger volumes. The critical duration flood event, with respect to flows, may not necessarily be of the largest average intensity or largest total volume. Therefore, a number of durations were run as part of XP-RAFTS modelling for design storms to find the critical duration of the storms that produce the flow peaks at Bega and Candelo. The established critical durations for various design storms are presented in Table 15.3.

Jellat Jellat Flats significantly impacted estimation of critical duration due to provision of offline storage absorbing backwater from downstream reaches. It was noted that for all events, apart from the 0.2%AEP and PMF, the long durations storms (i.e. 36 and 48 hours) were governing both upstream and downstream of the Bega/Brogo River junction. For the 0.2% event a short duration of 9 hours was critical at the Bega and Brogo River junction, while for the PMF event the 24 hour storm was critical.

To allow for the effects of the Jellat Jellat Flats storage, long duration storms were further assessed for these two events as it was found that the Jellat Jellat Flats storage has significant impact on flood levels. Based on an inspection of hydrographs from XP-RAFTS, a 72 hour storm was used for comparison against the 9 hour short duration event for the 0.2%AEP event, as it had a relatively large volume and peak flow that was expected to fill up the Jellat Jellat Flats storage and provide higher flood levels in the lower reaches of the Bega River. Based on a comparison of 0.2%AEP flood levels the 9 hour storm was critical in the upper reaches, particularly upstream of the Princes Hwy, and the longer duration 72 hour storm was critical in the lower reaches extending from Mogareeka to Bega. Similarly the 72 hour storm was used for the PMF event to take into account these additional storage effects, but it was found that the 24 hour storm was also critical in the lower reaches of the study area.

**Table 15.3: Critical Durations for Various Design Flood Events Based on Peak Flows at Bega/ Brogo Rivers Junction**

Description	CRITICAL DURATION					
	10% AEP	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF
Bega River at Bega U/S Junction	36 hour	36 hour	36 hour	36 hour	9 hour	18 hour
Brogo River at Bega U/S Junction	48 hour	48 hour	48 hour	48 hour	36 hour	24 hour
Bega-Brogo River Junction	36 hour	36 hour	36 hour	36 hour	9 hour*	24 hour
Bega River downstream of Jellat Jellat Flats	36 hour	36 hour	36 hour	36 hour	72 hour*	24 hour
Candelo Creek at Candelo	48 hour	48 hour	48 hour	48 hour	9 hour**	3 hour

\* Unsmoothed Temporal Pattern – 0.2%AEP 9hr WV1MAR10 used as per ARR99 unsmoothed GSAM patterns

\*\* Unsmoothed Temporal Pattern – 0.2%AEP 9hr DE4FEB55 used as per ARR99 unsmoothed GSAM patterns

## 15.2 Rainfall For Probable Maximum Flood (PMF)

Rainfall for the PMF, referred to as Probable Maximum Precipitation (PMP) is derived for short and long duration storm events in line with the Generalised Short Duration Method (GSDM), Revised Generalised Tropical Storm Method (GTSMR) and Generalised South-East Method (GSAM) as outlined in AR&R and the Bureau of Meteorology guidelines. The GSDM method outlines PMP estimates for storm durations up to 6 hours in duration, and the GSAM and GTSMR methods are for durations up to 120 hours. For the Bega River catchment the applicable methods were GSDM and GSAM.

Due to limitations of the current XP-RAFTS software used for modelling, the spatial distribution for the GSAM method could not be incorporated and a uniform distribution was used across the catchment which is expected to provide more conservative estimates for the PMF.

As per current AR&R99 guidelines, the temporal patterns applied to 0.2% AEP used GSDM smoothed and GSAM unsmoothed methods that provide a gradual flood frequency curve from large to extreme flood events.

### 15.3 Ocean Water Levels from Site Specific Analysis

For the current study a detailed site-specific analysis of ocean water levels for design events was undertaken opposite of adopting the conservative default 1% AEP water level of 2.6 mAHD, in line with recommendations of Flood Risk Management Guide (OEH, 2010). This approach provided a downstream boundary that is more applicable to the particular entrance, a more accurate estimate of ocean levels and a less conservative modelling results, in particular flood levels within the floodplain.

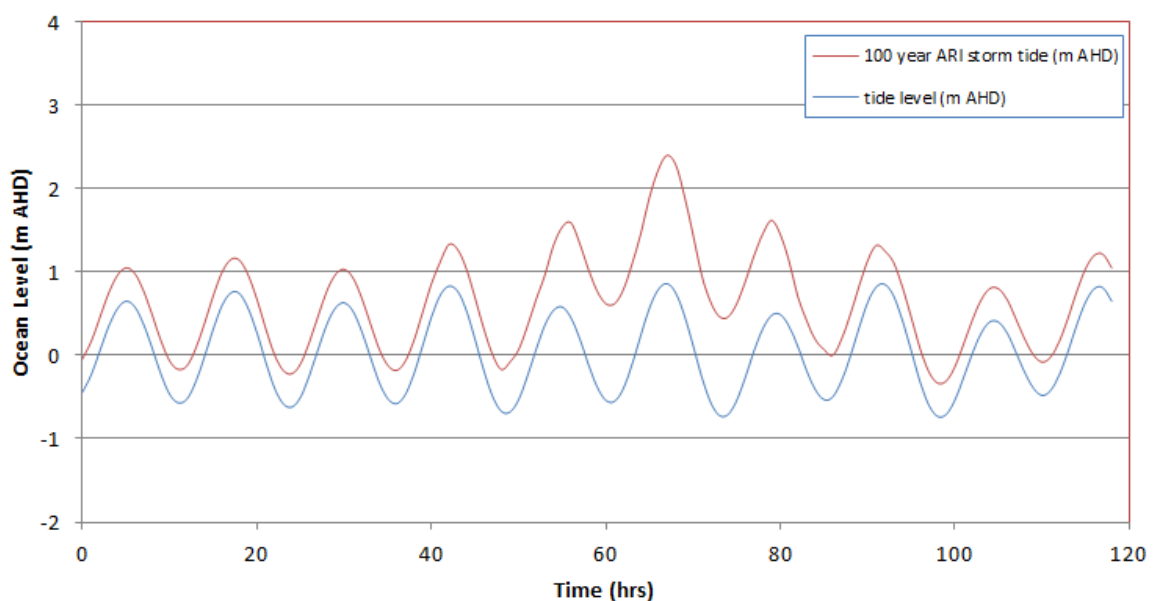
The site specific analysis included modelling in specific coastal modelling software programs (SWAN, ACES, and SBEACH) to determine design flood levels for application as downstream boundary conditions.

The procedure steps allowed for determination of the wave setup for storm events ranging from 1 to 100yr ARI including nearshore wave transformation, refraction coefficient and shoaling. A typical high tide was selected after observation of tidal data. The storm surge levels were estimated using the Fort Denison water level analysis (Figure C2 of the Appendix C of the attached "09698FortDenSeaLevRiseStudy.pdf"). Figure 7.1 from the FRM guide was then used for the timing of storm and tide.

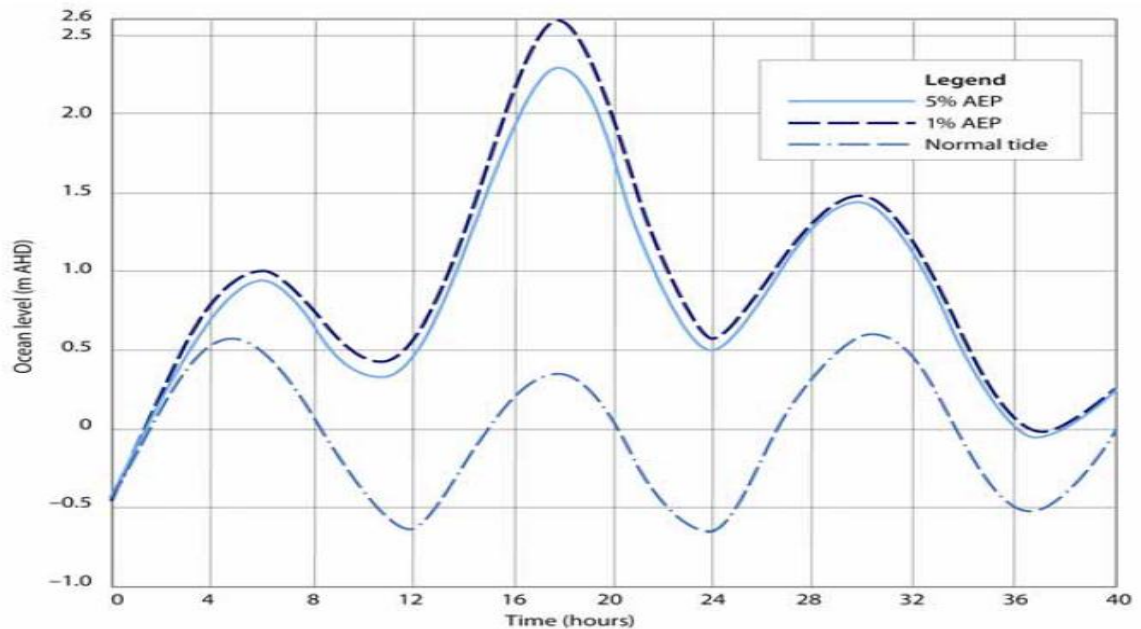
While the default boundary condition is targeted towards the 1% AEP event, the site specific analysis can apply to a range of different sized flood events. The site specific analysis resulted in peak ocean levels ranging from about 2.02 mAHD in the 20% AEP, to 2.40 mAHD for the 1% AEP. The different components used in developing the site specific analysis are presented in Appendix B.

The analysis provides the mean storm water level that varies with time (unsteady). A plot of the 1% AEP design tailwater levels developed by the site-specific analysis is shown in Figure 15.2 and the default 1% AEP tailwater in Figure 15.3.

**Figure 15.2: Site Specific Analysis – 1% AEP event**



**Figure 15.3: Default Ocean Level – 1%AEP event (as per Figure 7.1 OEH, 2010)**



In general site specific ocean levels were used in base case modelling for the current study, while the peak 1%AEP default ocean level of 2.6m AHD from Figure 7.1 of OEH guideline (refer Figure 15.3) was considered in an additional sensitivity run (refer Section 16). The results of sensitivity analysis confirmed that model is not sensitive to change in approach and that the differences in flood levels in case of using the default 2.6m AHD sea level are negligible. Results of this assessment can be found in Section 16.2.1.

## 15.4 Climate Change Impacts

Climate change effects can significantly affect flooding with increases of flood levels resulting from sea level rise and/or potential increases in rainfall intensities. The current Floodplain Risk Management Guideline and other related documents provide a number of options for the management of climate change conditions:

### **Floodplain Risk Management Guideline: Practical Consideration of Climate Change (OEH/ formerly DECC, 2007)**

This guideline provides floodplain managers and planners with sea level rise and rainfall intensity considerations applicable to NSW. Estimates for low level, mid-level and high sea level change are presented in this document. The guideline recommends an assessment with increased rainfall intensity and storm volume as part of modelling sensitivity analyses. Various management options are presented for different approaches to tackle climate change effects.

### **Anticipated Response of Coastal Lagoons to Sea Level Rise (Haines 2008)**

This paper describes some of the hydrodynamic behaviour of ICOLLS's and their anticipated response to sea level rise. It is quoted that an increase in mean sea level would result in an upward and landward translation of ocean beach profiles. With respect to coastal lagoons, a sea level rise may cause the entrance sand berm to move inland and to build up to a higher level relative to local topography. "The increase in berm height can be expected to match the increase in sea level rise, given that the berm is built primarily by wave run-up processes" (Haines 2008, page 51).

## **NSW Flood Risk Management Guide: Incorporating sea level rise benchmarks in flood risk assessments (OEH 2010)**

This guide updates the sea level rise section of the Floodplain Risk Management Guideline: Practical Consideration of Climate Change (DECC 2007). The benchmarks for NSW sea level rise are drawn from the NSW Sea Level Rise Policy Statement (2009). This guide identifies that:

“flood studies need to account for the coincidence of catchment and coastal flooding and their varying influence with location in the tidal waterway in deriving flood levels and associated flood planning levels. This requires appropriate assumptions on downstream boundary conditions and initial water levels and examination of a number of combined catchment and coastal flooding scenarios” (OEH 2010).

The appendix of OEH (2010) provides recommendations on modelling of the interaction of catchment and coastal flooding.

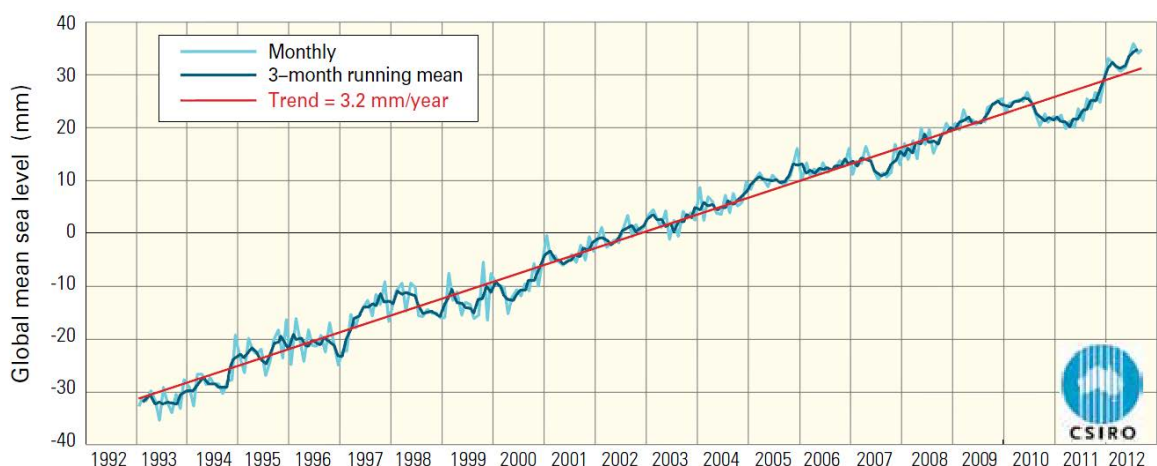
Bega River would be classified as Class 3 – catchments that normally drain directly to the ocean via shoaled entrances, or Class 4 – catchments with normally closed or partially blocked entrances. Modelling approaches are recommended including the use of steady/unsteady flow models, fixed/dynamic entrance conditions, initial water levels, envelope approaches to flood planning levels, default/site specific analyses of downstream boundary conditions, and sea level rise predictions.

### **15.4.1 Sea Level Rise**

Former DECCW guidelines indicated that the 2007 Intergovernmental Panel for Climate Change (IPCC) Fourth Assessment Report provided a range of future sea level rises depending on the level of future carbon emissions. The upper range scenario allows for 59 cm sea level rise by 2095, plus an additional allowance for ice-melt uncertainty of up to 20 cm. Further to this, CSIRO advocate up to a further 12 cm of sea level rise due to local effects caused by the East Australia Current, totalling 91cm. Thus, the upper-range scenario is for 91 cm of sea level rise for the NSW coast by the end of the century, and similarly 40cm by 2050. The former NSW Sea Level Rise Policy Statement (2009) similarly recommends sea level rise estimates of 0.4m and 0.9m for the 2050 and 2100 scenario respectively, relative to the 1990 mean sea level. The same benchmarks are recommended in the OEH Floodplain Risk Management Guide (OEH, 2010), however current NSW Government policy, introduced following the commissioning of this study, requires councils to adopt their own sea level rise projections.

As indicated above the global trend in mean sea-level rise can vary with location around the world. However a plot of the global trend sourced from the BoM Annual Climate Summary 2012 is shown in Figure 15.4 below showing that sea levels since 1993 has been generally rising at an average rate of 3.2mm/year. This plot shows a definite upward trend in sea level rise,

**Figure 15.4: Measurements of Global Mean Sea Level from Satellites from Jan 1993 to Dec 2012, source: CSIRO (BoM, 2012)**



Sea level rise estimates used in the preliminary FPL's developed in the current study source 0.4m SLR for the Scenario 1 and 0.9m for Scenario 2 based on the abovementioned estimates. This is consistent with the project brief developed prior to the change in NSW Government policy and is consistent with best currently available information including the NSW Chief Scientist and Engineer Report (April 2012). In addition to sea level rise impacting on the ocean levels, the effects of sea level rise can potentially increase the accumulation of sand along the river bed and at the berm and associated heights at the entrance opening. There have also been recent changes to the NSW Government policy on sea level rise associated with the stage 2 Coastal Reforms. It is therefore recommended that council undertake a more detailed assessment of the potential impacts of sea level rise on flood risk as part of developing its Floodplain Risk Management Plan.

#### **15.4.2 Increase of Rainfall Intensities**

Climate change can also include increases in rainfall intensities in addition to sea level rise, although the changes to rainfall patterns and distributions are less known or predictable. Climate change associated with increases in rainfall can impact on hydrologic and hydraulics processes, where flows and flood levels can be affected. The current OEH guidelines advocate assessment of this impact for an increase range of 10-30% for consideration in floodplain management process. For the current study a 10 and 30% increase in rainfall intensities were assessed during the sensitivity analysis, using the 1%AEP event as a base case.

#### **15.4.3 Management Options for Climate Change Impacts**

Apart from the recommended assessment of the impact of these changes on flooding affectations in existing conditions, considering the need to provide solid planning tools for a longer development period, the predicted sea level rise and increases in rainfall intensities can be considered for inclusion in modelling as a basic condition for design events. This issue requires a thorough communication with and consideration of Council to assure that the most appropriate and economically justifiable approach is adopted.

For the purpose of this study, the sea level rise component and rainfall intensity increase effects were not included in basic design modelling, however they were considered separately as part of the Sensitivity Analysis. In addition, a preliminary assessment of flood planning levels was carried out including sea level rise (SLR) and results presented for the existing conditions and year 2050 and 2100 projections. Refer Appendix H.

Based on these scenarios a number of approaches can be adopted in the Floodplain Risk Management of the study area. It is understood that Council will further consider the impacts of climate change in the next stage of the floodplain risk management process during preparation of the Floodplain Risk Management Study.

### **15.5 Coincidental Flooding from the River and Ocean**

In order to establish the appropriate downstream conditions, an estimate was made for the joint probability of coincidence of occurrence of the peak flows from the Bega River and water level conditions from the ocean. The peak levels from the ocean were adjusted to coincide with the peak flow in the main river, using a more frequent design water level in line with current floodplain management practice. For example, peak flows in the river under the 0.2% AEP flood event corresponded with peak water levels from the ocean producing the 2% AEP flood event.

This approach is considered to be more appropriate than if the same AEP was used for both the catchment flows and ocean levels, which is less likely to occur and was considered to be too conservative as it would significantly exceed the given AEP. The combinations of catchment and ocean flooding with different sized flood combinations are shown in Table 15.4 based on the estimated joint probabilities.

**Table 15.4: Coincidental Flooding of Bega River from Catchment and Ocean Flooding**

River Flood Frequency	Ocean Levels Frequency	Basis for Recommendation
PMF	1% AEP	Estimated ( based on current FRM practice)
0.2% AEP	2% AEP	Estimated ( based on current FRM practice)
1% AEP*	5% AEP	Joint probability assessment and envelope as per current OEH guidelines with managed and unmanaged cases
1% AEP*	tide	Joint probability assessment and envelope as per current OEH guidelines
5% AEP*	1%AEP	Joint probability assessment and envelope as per current OEH guidelines
2% AEP	5% AEP	Estimated ( based on current FRM practice)
5% AEP	10% AEP	Estimated ( based on current FRM practice)
10% AEP	20% AEP	Estimated ( based on current FRM practice)

\*envelope approach required for establishment of preliminary flood planning levels

In line with recommendations in OEH guidelines the highlighted scenarios were considered in preparation of Preliminary Flood Planning Levels.

For the base case modelling, managed entrance conditions were applied with the exception of 5%AEP catchment with 1% AEP Ocean flooding, where the entrance was initially open to provide a conservative estimate where floodwaters from the ocean can fill the lagoon and tidal waterway early in the storm. In addition to these conditions an unmanaged entrance was assessed to consider no maintenance of the entrance and the effects on preliminary flood planning levels.

Peak flowrates and corresponding peak ocean levels are presented in Table 15.5.

**Table 15.5: Catchment and Ocean Flooding Combinations**

DESCRIPTION	UNIT	JOINT PROBABILITY PARAMETERS							
AEP catchment flooding	-	10%	5%	2%	5%	1%	1%	0.2%	PMF
Peak Flowrate catchment flooding at Bega-Brogo Rivers junction*	m <sup>3</sup> /s	5056	6373	7810	6373	9231	9231	10424/8791	23395
Peak Flowrate catchment flooding at Mogareeka	m <sup>3</sup> /s	3319	4165	5194	4224	6350	6110	5635/7196	18734
AEP ocean flooding	-	20%	10%	5%	1%	tide	5%	2%	1%
Peak Level Ocean Boundary Condition	m <sup>3</sup> /s	2.02	2.09	2.18	2.40	0.86	2.18	2.29	2.40

\*Quickflow component only



## 15.6 Assessment of Blockage

### 15.6.1 Basis for Assessment

As part of the recent revision of publication Australian Rainfall and Runoff currently underway, the Stage 2 Report has been recently released (February 2013) and includes blockage issues, blockage analysis and assessment, design and analysis of drainage systems, pit and pipe blockages, the management of blockage, generic blockage factors, and further recommendations. Alternative methodologies for the analysis and assessment of blockages outlined in the report include:

- Scheme A (based on papers by Barthelmess, Rigby, Silveri, et al);
- Scheme B (by Witheridge); or
- Generic Blockage Factors.

**Scheme A Methodology** uses a method based on the likelihood and consequences of blockage. The likelihood uses a qualitative base value for debris reaching a structure based on the consideration of debris availability, mobility, and transportability, to determine a “Debris Potential” for different sized events.

Using Scheme A, events are categorised as:

- Design floods smaller than 20 year ARI;
- Design floods between 20 year and 200 year ARI;
- Design Floods greater than the 200 year ARI.

Following an estimate of the length of the longest 10% of debris that could arrive at the site (termed  $L_{10}$ ), quantitative estimates for blockages at structures are provided in terms of a percentage blockage where 100% blockage represents a fully blocked structure. The consequences of blockage are then rated as either Very Low, Low, Moderate, High or Very High and the previously determined percentage blockages modified. In this way the likelihood and consequences are used in a risk based assessment.

**Scheme B Methodology** incorporates likelihood and consequences into the assessment with 5 levels of consequence based on the cost of damages, health, environment, social impacts, impacts to the community, and legal implications. Once categories for likelihood and consequences are determined, a risk matrix is used to determine a “risk level”. Following the determination of risk level, approaches to blockages at individual hydraulic structures are recommended in addition to the degree of blockage for cross drainage structures or not-cross drainage structures. Blockage conditions are quoted as “design” or “severe” blockage under different flood events with suggested values available for adoption.

**Generic Blockage Factors** are provided in the Stage 2 report of AR&R – Blockage

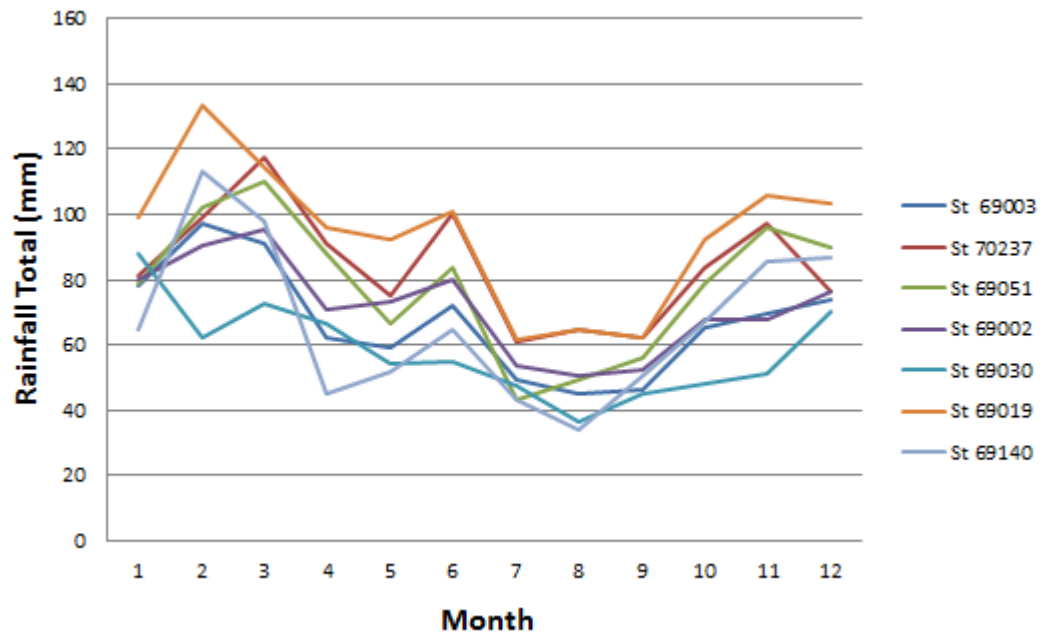
A table of generic blockage factors are provided depending on type of structure, design or severe blockage. However, some factors stated under this method are difficult to estimate without specific knowledge of debris accumulation potential for different sized floods, and would rely on significant assumptions on blocking conditions.

#### **Annual Average Rainfall**

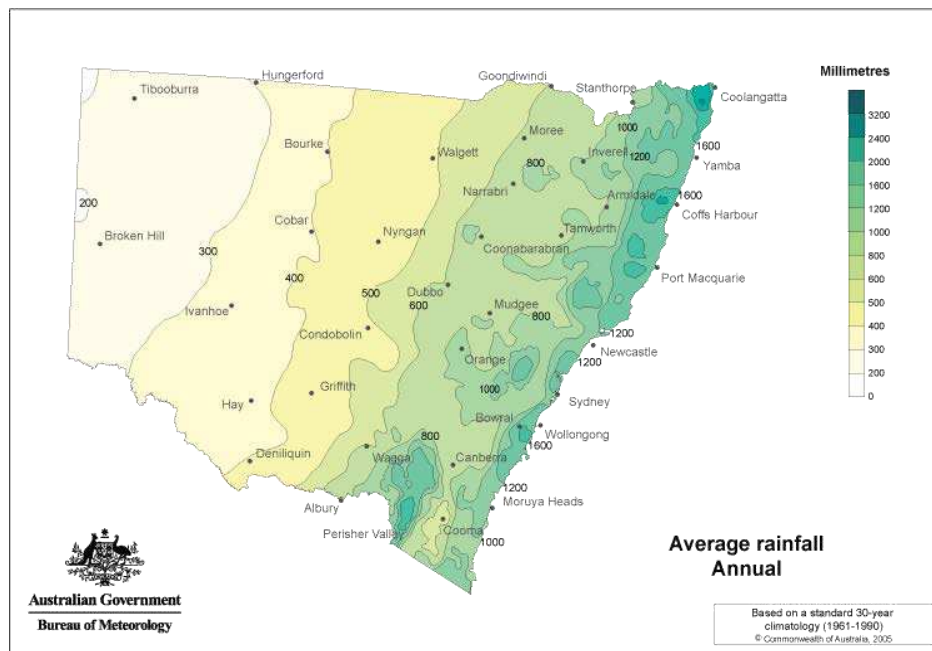
The annual average rainfall is used to help assess the debris potential for blockage calculations in the above Scheme A methodology. The temporal distribution of average annual rainfall was plotted for 7 sites within the catchment as shown in Fig. 15.5, indicating a medium level of variability. A spatial plot of average annual totals obtained from the Bureau of Meteorology is also presented (Fig 15.6) indicating a medium level of average annual rainfall near the Bega region.



**Figure 15.5: Temporal Distribution of Annual Rainfall**



**Figure 15.6: Average Annual Rainfall across NSW (source BoM)**



## 15.6.2 Adopted Approach

The adopted approach for this study applied the methodologies from Scheme A of the AR&R Stage 2 report on blockage, as it was considered to be relatively straight forward, while comprehensive analysis of blockage for the bridges modelled in the current study.

The debris availability describes the potential for the production of debris at source before mobilisation and transport of debris. The debris availability for the Bega/Brogo Rivers catchment has been categorised as “Medium” based on indicative characteristics in the AR&R Stage 2 Report. The **Medium** level of Availability is mainly based on:

- source areas that generally fall between High and Low categories, suggesting *Medium*.
- the existence of state forest areas, grazing land with stands of trees, suggesting *Medium*.
- Totals of Average Annual rainfall, and temporal distribution of Annual Rainfall, suggesting *Medium*.
- considerable fallen limbs, leaves and high levels of floor litter, suggests *High*.
- NOT Arid, suggest *Medium or Low*.
- NOT Urban with cars and stored loose material close to water courses, and NOT a considerable distance from watercourses, suggesting *Medium*.

Debris mobility refers to the ease with which the available debris is initially mobilised at the on-set of debris movement from the source and before the transportation phase. The debris mobility for the current study has been categorised as “Medium” based on indicative characteristics in the AR&R Stage 2 Report. The **Medium** level of Mobility is mainly based on:

- Areas generally falling between the High and Low Categories, suggesting *Medium*.
- Steep catchments in the upper reaches of the catchment and mild slopes in the mid to lower reaches of the catchment, suggesting *Medium*.
- Totals of Average Annual rainfall, suggesting *Medium*.
- With cover not considered to be Arid, suggesting *Medium to Low*.

The debris transportability refers to the “ease with which the mobilised debris is transported”. The debris transportability for the current study has been categorised as “High” based on indicative characteristics in the AR&R Stage 2 report. The **High** level of Transportability is mainly based on:

- Bed slopes less than 1% grade, suggested *Low* category.
- Wide streams relative to debris load dimensions, suggesting *High*.
- Banks are prone to scour during a design event, suggesting *High*.
- Totals of Average Annual rainfall, and temporal distribution of Annual Rainfall, suggesting *Medium*.

The three categories for the debris availability, mobility and transportability are combined to give (Medium-Medium-High). The Debris Potential is then read from the AR&R documentation as corresponding to a Medium level of debris potential. However given the large amount of damage and large degree of debris noted *during historic flood events*, the Debris Potential was re-categorised as High (refer to Council’s historic flood photos below).

Following the above assessment, further adjustments were made for the different sized design flood event. Blockage was estimated using the  $L_{10}$  parameter which represent the “length of the longest 10% of debris that could arrive at the site”. The  $L_{10}$  parameter has been assumed to be 8m based on available photographs.

**Figure 15.7: Debris and Blockages at Bridges (from Council's Flood Data Library)**







For the design events used in this study the base debris potential gives the following Blockage Levels.

**Table 15.6: Most Likely Blockage Levels Based on Debris Potential Alone**

Design Flood Event	Base Debris Potential	"Most Likely" Blockage Level ( $B_{DES}$ )		
		$W < L_{10}$	$L_{10} \leq W \leq 3 * L_{10}$	$W > 3 * L_{10}$
10% AEP	Medium	50%	10%	0%
5% AEP	Medium	50%	10%	0%
2% AEP	High	100%	20%	10%
1% AEP	High	100%	20%	10%
0.2% AEP	High	100%	20%	10%
PMF	High	100%	20%	10%

\*W refers to control dimension taken here as the diagonal measurement of the opening

Depending on the consequences associated with failure of a structure, the "most likely" blockage levels are adjusted to cater for the varying consequence levels. A possible ('severe') blockage level increases the likely blockages where the consequences of severe blockage levels are high.

**Table 15.7: Blockage Levels Based on Likelihood and Consequences**

Design Flood Event	Consequence Level	Additional Risk	"Most Likely" Blockage Level (BDES)		
			W<L10	L10 ≤ W ≤ 3*L10	W>3*L10
10% AEP	High	Design	50%	10%	0%
5% AEP	High	Design	50%	10%	0%
2% AEP	High	Design	100%	20%	10%
1% AEP	Very High	Severe	100%	40%	20%
0.2% AEP	Very High	Severe	100%	40%	20%
PMF	Very High	Severe	100%	40%	20%

\*W refers to control dimension taken here as the diagonal measurement of the opening

The derived blockage factors were then applied to each individual bridge segment for the various sized design flood events (refer Appendix C). To simplify application in modelling the segmental blockage factors were then generalised for overall bridge span. The results are summarised in Table 15.8 below for each bridge in the modelled system.

**Table 15.8: Adopted Blockage Factors for Bridges within the Modelling Area**

Bridge	Blockage Factors (%)					
	10% AEP	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF
Princes Highway Large Span Bridge	0	0	10	20	20	20
Tarraganda Lane bridge – Bega River	10	10	20	40	40	40
Tarraganda Lane bridge – Bega River Anabranh	10	10	20	40	40	40
Tathra - Bermagui Bridge	10	10	20	40	40	40
Candelo	10	10	20	40	40	40

## 15.7 Design Event Runs

### 15.7.1 Scenarios for Design Modelling Runs

Following the hydrologic and hydraulic models calibration and adoption of design parameters, the hydrologic model was run for the design events, including 10%, 5%, 2%, 1%, 0.2% AEP and PMF events. This provided the quickflow component, with baseflows subsequently added to form total flow hydrographs required as input into the XP-SWMM2D hydraulic model. The effects of full water storages in Brogo and Cochrane Dam were considered by applying initial values set at full storage volumes/ levels for each design flood event.

XP-SWMM2D was then run for all design storms with a combination of catchment and ocean flooding subject to unsteady, dynamically changing berm conditions as discussed in Section 14.8.3. The combination of catchment and ocean flooding was run to include coincidental peaks as suggested in OEH guidelines. The timing of the peaks from the catchment flows and ocean levels was adjusted to occur simultaneously in order to produce the critical flood conditions.

Maps showing modelled design flood event parameters including flood extents, water levels, depths and velocities are presented in Appendix F and Appendix K.

### 15.7.2 Flood Behaviour

Both the Bega and Brogo River sub-catchments are noted to contribute significantly to flooding at Bega. In the 1%AEP flood the estimated peak flowrate along Bega River is 6160 m<sup>3</sup>/s and along Brogo River 3155 m<sup>3</sup>/s. At the junction the maximum combined peak flowrate was 9230 m<sup>3</sup>/s, well above that of any single river alone.

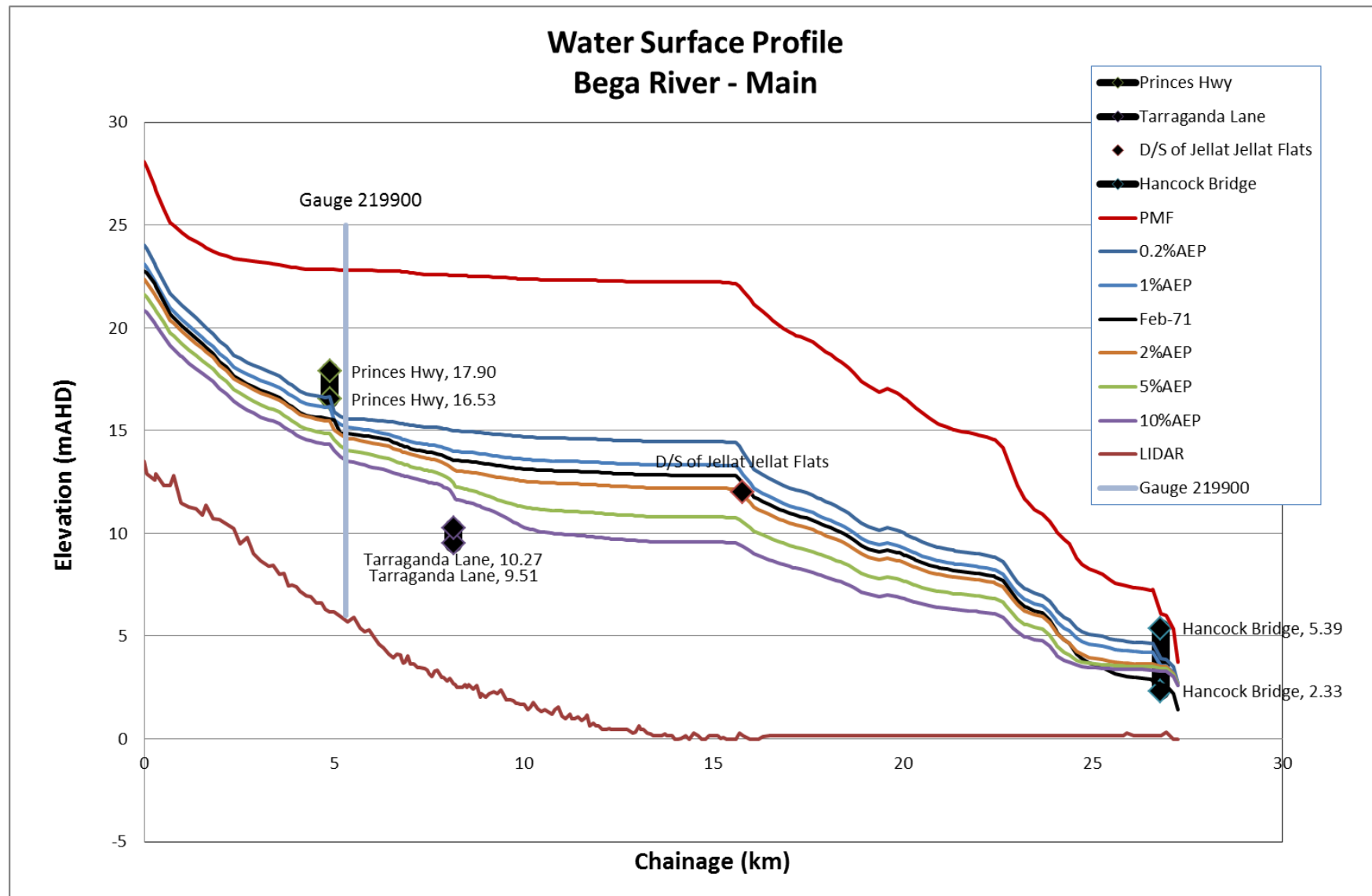
It was noted that flooding along Bega River is subject to a large reduction in peak flowrate from the Bega-Brogo Rivers junction to the outlet at Mogareeka. This is largely attributed to the influence that Jellat Jellat Flats has as a storage area plus the storage routing along Bega River floodplain.

Jellat Jellat Flats is an off-line storage between Bega and Mogareeka that gets engaged during significant flood events. Downstream of Jellat Jellat Flats flows are constrained into a relatively narrow river channel with limited conveyance capacity. The result is that flows get attenuated at Jellat Jellat Flats and the peak flowrates are significantly reduced in the lower reaches of Bega River, while a backwater effect occurs from the downstream channel impacting on upstream flood levels.

Overflows occurred from Jellat Jellat Flats into Wallagoot Lake in the PMF Event while for the other events flows between Jellat Jellat Flats and Wallagoot Lake were not linked.

A plot of the water surface profiles from the 10%, 5%, 2%, 1%, 0.2% AEP events and PMF events in conjunction with the joint probability approximations (i.e. 20%, 10%, 5%, 5%, 2%, 1%AEP respectively) are shown in Figure 15.6 below. The 1%AEP water surface profile indicates flood depths of about 10m, 11m, and 13m at the crossings along Bega River at Princes Hwy and Tarraganda Lane main bridge and at Jellat Jellat Flats respectively. PMF depths are about 17m, 20m and 22m respectively. Compared to previously developed (1979) water surface profiles supplied by Council, the profiles of the design events computed in the current study are higher.

Figure 15.8: Water Surface Profiles along Bega River





The computed design water levels at flood gauges within the modelling area and their relationship to the gauge RL is presented in the following table.

**Table 15.9: Design Flood Levels at Flood Gauges within the Modelling Area**

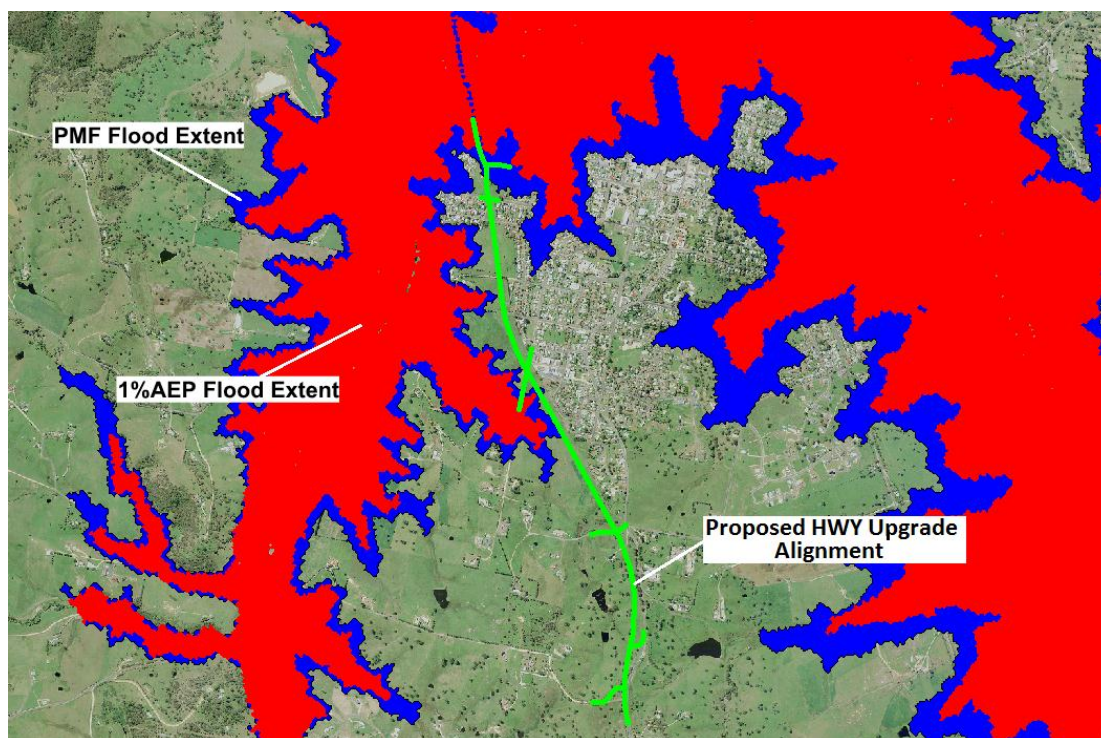
Station No	Station Name	Flood Gauge ORL (mAHD)	Coordinates		Design Flood Level mAHD / Local Gauge Level mRL					
			Latitude	Longitude	10%	5%	2%	1%	0.2%	PMF
219900	Bega River @ Bega	5.701	-36.669	149.830	13.96	14.48	14.99	15.49	15.86	22.83
					<b>8.26</b>	<b>8.78</b>	<b>9.29</b>	<b>9.79</b>	<b>10.16</b>	<b>17.13</b>
219026	Bega River @ Warraguburra	0.54	-36.702	149.872	9.85	11.02	12.38	13.48	14.6	22.32
					<b>9.31</b>	<b>10.48</b>	<b>11.84</b>	<b>12.94</b>	<b>14.06</b>	<b>21.78</b>
219410	Bega River @ Tathra (Bega Live)	0.00	-36.703	149.978	3.35	3.48	3.59	4.02	4.37	6.84
					<b>3.35</b>	<b>3.48</b>	<b>3.59</b>	<b>4.02</b>	<b>4.37</b>	<b>6.84</b>

NOTE: Highlighted values represent design flood levels relative to the local gauge. Local gauge levels at Station 219410 are to AHD.

Although some additional flood gauges operated by NSW State Water exist in the modelling area (Stations 219064- Brogo River @ Apps Crossing, 219063- Bega @ Darceys and 219065- Bega River @ Barrage), the flood level vs. gauge level relationship could not be established as the gauge zero level (ORL) was not available in mAHD.

An initial comparison of the 1%AEP and PMF flood extents in the vicinity of the proposed Princes Highway upgrade is shown below. The upgrade alignment is superimposed on the pre-upgrade flood extents computed in the current study. The upgrade appears to have minimal interaction with flooding based on the existing layout but the affectation might need to be confirmed once the construction is complete and WAE (Works-As-Executed) levels have been confirmed.

**Figure 15.9: Approximate Location of Proposed Upgrade**



The Candelo Creek model was run for the same set of design events with the modelled flood levels along the creek and flood maps shown in Appendix J and Appendix K respectively. The maps show a range of flood extents, with the 10%AEP event primarily confined to the creek in the area near Candelo Bridge but overflowing the banks and potentially cutting access to roads and affecting some properties in other areas. The affectation increases with less frequent events. The PMF is a significantly larger event with more overbank flow and properties affected.

Some areas of Candelo along the local flowpaths and creek gullies may be subject to additional flooding from backwaters (such as at Gordon Street, along Power Street, Eden and Pambula Streets, Tantawangalo-Mountain Road, the southern end of Myrtle-Mountain Road). Flooding along these areas could not be mapped as it would require significantly more survey data.

Modelling of Brogo Dam indicates that the dam is subject to overtopping above the dam crest level (118.10mAHD) in the PMF event based on Probable Maximum Precipitation Design Flood (PMPDF). This highlights that the dam might need to be raised to minimise potential for dam failure due to excessive overtopping. In terms of flood mitigation functionality of the dam, there is no available airspace which could be utilised for flood mitigation. Detailed dambreak studies for Brogo and Cochrane Dams are recommended to assess the likely consequences of dam failure on downstream properties.

### 15.7.3 Capacity of Hydraulics Structures

The modelling indicates that the existing large span Princes Highway Bridge has a capacity around the 0.2%AEP but no available freeboard in the 0.2%AEP (i.e. flood levels reach above the soffit level but are below the deck level). The Tarraganda Lane main bridge and Tarraganda Lane anabranch bridge are subject to overtopping in frequent events, such as the 10%AEP event and above. The Tathra-Bermagui bridge near Mogareeka is subject to overtopping in the 5%AEP event but with no available freeboard even in the smaller, 10%AEP event. Similarly, the bridge at Candelo has approximately 10%AEP capacity.

The overtopping of bridge sites for a range of different sized flood events, together with corresponding flood levels and associated freeboards, is listed in Tables 15.10 and 15.11.

**Table 15.10: Design Flood Levels at Hydraulic Structures**

CROSSING	Road Level (m AHD)	DESIGN FLOOD LEVEL (mAHD)					
		10% AEP	5% AEP	2% AEP	1% AEP	0.2%AEP	PMF
Princes Highway	17.90	14.32	14.86	15.48	16.15	16.63	22.86
Tarraganda Bridge – Bega River	10.27	12.09	12.62	13.23	14.08	15.04	22.57
Tarraganda Bridge – Bega River Anabranch	10.33	12.37	12.87	13.45	14.25	15.21	22.76
Tathra-Bermagui	3.45	3.36	3.50	3.64	4.43	4.65	7.24
Candelo Creek	101.48	100.42	101.60	102.25	102.92	103.45	107.67

\* highlighted values indicate overtopping of the structure

**Table 15.11: Freeboard at Hydraulic Structures**

CROSSING	Approx. Bridge Soffit Level (m AHD)	FREEBOARD TO BRIDGE SOFFIT (m)					
		10% AEP	5% AEP	2% AEP	1% AEP	0.2%AEP	PMF
Princes Highway	16.53	2.21	1.67	1.05	0.38	-0.10	-6.33
Tarraganda Bridge – Bega River	9.51	-2.58	-3.11	-3.72	-4.57	-5.53	-13.06
Tarraganda Bridge – Bega River Anabranh	9.71	-2.66	-3.16	-3.74	-4.54	-5.50	-13.05
Tathra-Bermagui	2.33	-1.03	-1.17	-1.31	-2.10	-2.32	-4.91
Candelo Creek	100.36	-0.06	-1.24	-1.89	-2.56	-3.09	-7.31

\* highlighted values indicate no freeboard to soffit of the bridge

\*negative value of freeboard indicates overtopping of the soffit with the depth of overtopping as per the value in the table

The results also indicate that the Princes Highway bridge at Bega River is subject to overtopping in events greater than the 0.2%AEP and overtopping of the deck by about 5.0m in the PMF event. In the 1%AEP design flood event the Princes Highway bridge had 0.38m of freeboard available to the soffit level. The results indicated that the PMF flows may overtop the Tarraganda Lane crossings by about 12.3m.

At the main Bega River crossing - Tarraganda Lane, the bridge is overtopped in all events modelled including the 10%AEP event. The freeboard in the 1%AEP event was about -4.6m (i.e. no available freeboard to the soffit) with the road level overtopping by about 3.8m in the 1%AEP event. In the 10%AEP event the road level is subject to overtopping by about 1.8m.

At the Tarraganda Lane anabranh crossing south of the main Bega River crossing, the bridge may be subject to overtopping by about 2.0m in the 10%AEP event and about 3.9m in the 1%AEP event. The results indicated that during the PMF, flows may overtop the Tarraganda Lane Anabranh crossing by 12.4m. The freeboard to the soffit level at this crossing was about -4.5m in the 1%AEP design flood event (i.e. no freeboard).

The Tathra bridge is subject to overtopping in the 5%AEP by about 0.1m, by about 1.0m in the 1%AEP, and by about 3.8m in the PMF. The available freeboard in the 1%AEP design event is about -2.1m to the soffit level, i.e. no freeboard.

The relatively low capacities at the bridge crossings may be indicative of the relatively large flows from the catchment, relatively restricted channel capacity particularly leading into Bega, and downstream of Jellat Jellat Flats, and high potential for blockages at bridge crossings due to debris potential, likelihood and consequences.

The estimated duration of overtopping of each bridge is presented in Table 15.12.

**Table 15.12: Duration of Bridge Overtopping in Design Events**

CROSSING	DURATION OF BRIDGE OVERTOPPING (Hours) **					
	10% AEP	5% AEP	2% AEP	1% AEP	0.2%AEP	PMF
Princes Highway	-	-	-	-	-	29.0
Tarraganda Bridge – Bega River	19.3	22.8	27.3	31.8	> 71.5	> 74.5
Tarraganda Bridge – Bega River Anabranh	21.8	27.8	29.8	32.8	> 71.5	> 74.5
Tathra-Bermagui	-	1.3	9.3	17.8	31.0	> 74.5
Candelo Creek	-	0.7	3.0	5.6	6.4	6.2

\* highlighted values indicate overtopping of the structure

\*\* May be subject to longer durations of bridge overtopping

The overtopping durations may need to be considered in the development of an emergency response to assist in preparing the community for flood events and evacuation plans. Although overtopping from real-time historic floods can vary from design floods, the PMF design event may be considered as an extreme case and could be used for this assessment to determine the duration of road closures. Further assessment of emergency management issues may be considered as part of the next stage of the flood planning process (Floodplain Risk Management Study).

The impact of Jellat Jellat Weir had an insignificant impact on major flooding as it overtopped by more than 8m in all modelled design events. The width of the floodplain controlling the ingress of water in the area of Jellat Jellat Weir is about 2km which represents a much larger waterway area compared to the weir.

## 15.7.4 Other Considerations

The effects of existing and future development were considered by:

- Using the LEP2012, where impervious percentages were applied to individual subareas in the hydrologic model. No changes to these percentages were applied for future development as the hydrology was not considered sensitive to imperviousness under the present condition of the catchment which is predominately rural landscapes including farmland and forested areas.
- The roughness for areas along the Bega/Brogo River floodplain represented used in the XP-SWMM2D model were based on aerial photographs, dated 2010 and assumed to apply to existing and future development as it is not envisaged that major changes to development will occur in this area due to existing level of flood affectation.
- The impact of the proposed RMS road upgrade included a preliminary assessment by superimposing the current 1%AEP and PMF flood extents using pre-upgrade conditions with the design layout, indicating that most of the proposed road is outside of the 1%AEP and PMF flood extents.
- Future consequences as a result of potential climate change conditions such as increased rainfall intensity and sea level rise has been assessed separately as part of the sensitivity analyses. Sea level rise was incorporated in the preparation of preliminary flood planning levels.