Appendix C   Hydrology and Consequence Assessment

Eurobodalla Southern Storage

Hydrology and Consequence Assessment

Prepared for: Eurobodalla Shire Council
Reference No: 30012127_R06_V02
23/06/2017

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SMEC

ABBREVIATIONS AND ACRONYMS

AEP	Annual Exceedance Probability
ANCOLD	Australian National Committee on Large Dams
ARR	Australian Rainfall and Runoff
CL	Continuing Loss (mm/hr)
DCF	Dam Crest Flood
DEM	Digital Elevation Model (DEM)
DTM	Digital Terrain Model
DoC	NSW Department of Commerce
ESS	Eurobodalla Southern Storage
FSL	Full Supply Level
GSAM	Generalised Southeast Australia Method
GSDM	Generalised Short Duration Method
IL	Initial Loss (mm)
k_c	Catchment routing parameter used in rainfall-runoff model
PAR	Population At Risk
PLL	Potential Loss of Life
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
Q	Discharge (m³/s)
RFFE	Regional Flood Frequency Estimate
T_c	Catchment lag time used in rainfall-runoff model (hr)

1 INTRODUCTION

1.1 General

As part of the Eurobodalla Southern Storage Environmental Assessment and Detailed Design project, a hydrology and consequence assessment has been undertaken. The key outcomes required of the analysis are to:

- Assess the consequences associated with a dambreak.
- Establish the catchment flood frequency curve and define the required spillway capacity.

The scope of work for the hydrology and consequence assessment involves the following tasks:

Hydrology Study

- Review and update previous flood hydrology study in accordance with the most recent version of Australian Rainfall and Runoff (Ball et.al. 2016) and relevant ANCOLD and NSW DSC guidelines.
- Develop a catchment model and estimate the catchment runoff for flood events between the 1 year and PMF.
- Undertake reservoir routing to establish frequency-discharge and frequency-elevation graphs.

Dam Break Study

- Review and update previous dam break study in accordance with relevant ANCOLD and NSW DSC guidelines including modelling of the following scenarios:
  - Design Flood (PMF) with and without dam failure
  - Design Flood (PMF) prior to dam construction
  - 1 in 100 AEP flood with and without failure
  - Sunny day failure
  - Cofferdam failure.
- Development of flood inundation maps suitable for inclusion in the DSEP and Client GIS system.
- Undertake a Failure Impact Assessment.

1.2 Available Data

A variety of information was made available for the analysis as follows:

- NSW Department of Commerce (DoC) (2005) – Hydrology Study
- DoC (2006) – Dambreak Study
- River cross sections from DoC (2006)
- Lidar of the dam and surrounds
- Northrop (2016) – Tyrone Bridge Replacement Hydrology and Hydraulic Assessment Report

In addition, further information was supplied independently as follows:

- SRTM terrain data for regions outside the supplied lidar extents
- Flow Data from the NSW Pineena database
- Flow data from the NSW waterinfo website

2 HYDROLOGY

2.1 General

An analysis of the catchment hydrology was undertaken for the purposes of developing a flood frequency curve. The flood frequency curve has in turn been used to assist in developing the spillway design, embankment freeboard and the cofferdam design.

The hydrologic analysis involved undertaking the following sub tasks:

- Develop a rainfall runoff model for the study area
- Analyse regional rainfall data to develop an understanding of catchment runoff processes
- Calibration of rainfall runoff model utilising recorded data and existing studies
- Generate design hydrographs and a flood frequency curve for Eurobodalla storage
- Generate inflow hydrographs and a flood frequency curve for Eurobodalla storage cofferdam

2.2 Catchment Hydrology

The study area is located on an unnamed tributary of the Tuross River with a catchment area of 4.6 km^2. Of that catchment, the proposed Eurobodalla Southern Storage (ESS) will command a smaller portion of that tributary with a catchment area of 1.6 km^2. The overall catchment ranges in elevation from approximately RL 1.0 m (at the confluence with the Tuross River) to RL 153 m, and consists predominately of forested area, with a small portion of lowland farming. A thematic image showing the variation in elevation across the catchment is shown in Figure 2-1.
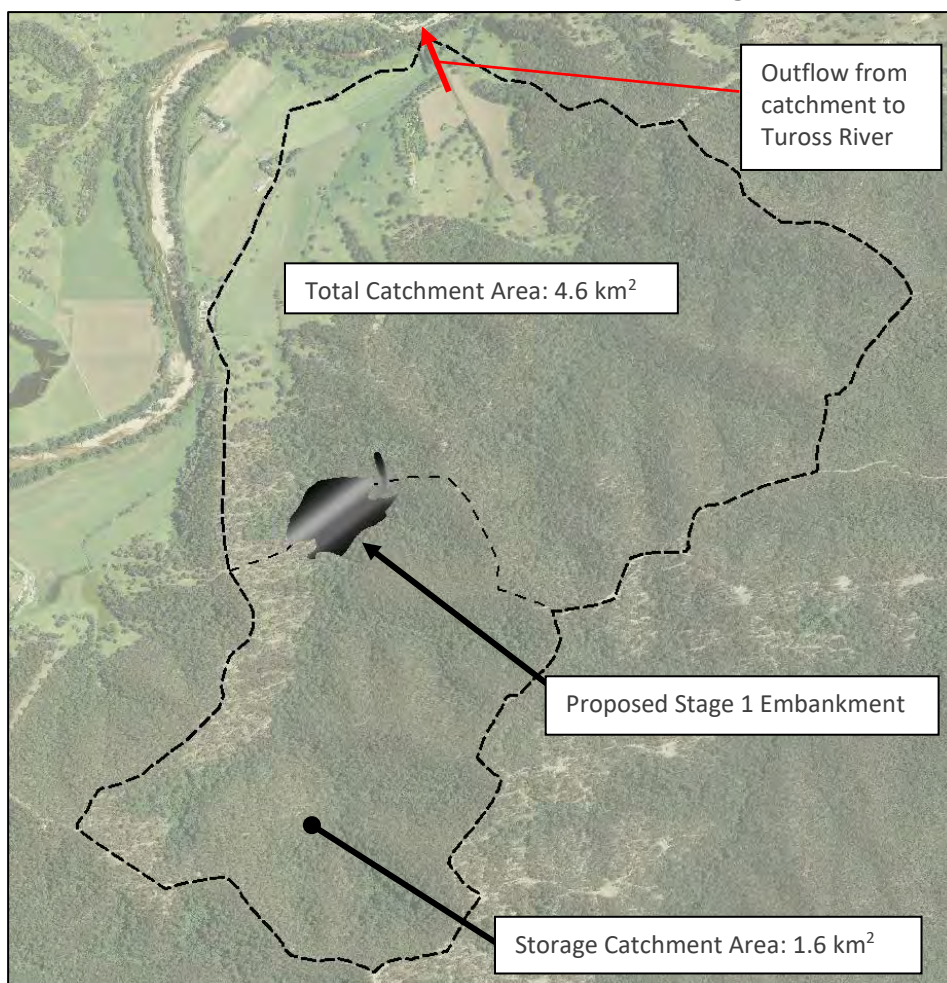
Figure 2-1 – Catchment Area

2.3   Flood Frequency Analysis

2.3.1   Regional Analysis

Data from adjacent catchments has been analysed to develop an understanding of rainfall runoff processes in the region and to further assist in developing rainfall runoff model parameters for the study catchment. A flood estimate for the 1 in 100 AEP storm event has been generated for the purposes of defining a target flow to use in the rainfall runoff model calibration. The flow has been estimated through:

- Flood frequency analyses derived from peak observed flows from nearby catchments.
- A regional flood frequency analysis based on data across the south east coast.
- Regional Rational Method

The outcomes have also been compared against the results from other studies.

2.3.1.1   Historic Flood Frequency Curve (FFC)

Flow data was obtained from a range of gauging stations in the Eurobodalla region and surrounds, with Flood frequency analyses undertaken on these data sets to obtain flow estimates for a range of probability events. The flood frequency analysis was undertaken using the Tuflow Flike software package, the curve fit adopted was Generalised Extreme Value (GEV) with optimised LH moments.

The catchment areas for the majority of regional catchments are substantially larger than the study catchment. The flows have been modified to allow a direct comparison with the modelled catchment area upstream of the ESS. The flows have been modified based upon catchment area applying the following equation as described in Grayson et.al. (1996):

$$Q_{(Unregulated\ Catchment)} = Q_{(Gauged\ Catchment)} \times (A_{(Unregulated\ Catchment)} / A_{(Gauged\ Catchment)})^{0.7}$$

Where:

Q = Discharge (m^3/s)

A = Catchment Area (km^2)

Note: Exponent can vary between 0.5 and 0.85. If data is available, the exponent may be calibrated, otherwise, 0.7 is typically applied and has been adopted for use in this analysis. The outcomes are listed in Table 2-1.

Table 2-1 – Regional Flood Frequency Analysis Results

Gauge Location	Catchment Area (km²)	Record Length (years)	Peak Flow (m³/s)		Selected Eurobodalla Peak Flow[1] (m³/s)	
			1 in 50 AEP	1 in 100 AEP	1 in 50 AEP	1 in 100 AEP
Araluen @ Neringla	111	20	141	159	4.5	5.1
Tuross Rvr at Eurobodalla	1,586	40	2,600	3,056	13.0	15.3
D/S Wadbilliga Jn	920	53	2,254	2,605	16.5	19.0
Brogo @ North Brogo	460	43	1,516	2,002	18.0	23.8
Wadbilliga	122	43	533	629	16.0	18.9
Tuross at Tuross Vale	93	61	183	208	6.7	7.6
Narira Rv @ Cobargo	92	44	521	602	19.1	22.1

Gauge Location	Catchment Area (km²)	Record Length (years)	Peak Flow (m³/s)		Selected Eurobodalla Peak Flow[1] (m³/s)	
			1 in 50 AEP	1 in 100 AEP	1 in 50 AEP	1 in 100 AEP
Murrah Rv @ Quaama	38	38	327	377	22.2	25.7
Rutherford Brown Mtn	15	77	132	157	17.2	20.5
				Range	5 - 26	
				Smallest catchment area range	21 - 26	

1 Flows calculated for catchment upstream of proposed ESS (prior to storage construction)

Of the results presented the most appropriate for extrapolation to the study catchment are those with the longest period of record and the smallest catchment areas. Utilising these criteria, the Rutherford Brown Mountain and Murrah River @ Quaama provide the outcome in which greatest confidence should be placed.

2.3.1.2   RFFE

In addition to the outcomes described above, a regional analysis has been undertaken using a newly developed regional procedure called Regional Flood Frequency Estimate (RFFE) described in Ball et.al. (2016). RFFE has been computed utilising the relevant website (accessed 9/12/16) as follows:

http://rffe.arr-software.org/

The regional analysis draws upon nearby gauging stations from a database of catchments across the South East Coast of NSW. The estimated flow is 55 m³/s for the catchment upstream of the proposed ESS (prior to storage construction) 5% and 95% confidence intervals being 19 m³/s and 148 m³/s. It is noted that in the lower end of the range, the results are strongly influenced by a single gauge with a catchment area of 0.5 km² located in an urban environment.

2.3.1.3   Rational Method

The rational method outcomes for the analysis have been developed using the approach described in IEAust (1987). They have been computed using a regression analysis approach as described in McDermott & Pilgrim (1982). The outcomes for this analysis are based upon a substantially shorter data set in comparison with the RFFE outcomes and it is, therefore, expected to be less accurate than that approach. The flow has been computed using IEAust (1987) rainfalls and a C_{10} value of 0.4.

2.3.1.4   Other Studies

DoC (2005) estimated the 1 in 100 AEP to range between 18.05 m³/s and 37.65 m³/s. The 1 in 100 AEP flow estimates have been derived using flows generated from a RORB rainfall runoff model. The model has been calibrated using regional delay parameter equations. The outcomes for the PMF were compared against a variety of PMF estimation references. The 1 in 100 AEP peak flow was not specifically reviewed as part of the works.

2.3.1.5   Summary

The outcomes from the various analyses for a catchment area of 1.6 km² (equivalent to size of ESS catchment) are presented in Table 2-2.

Table 2-2 - Flow estimation comparison

Method	1 in 50 AEP (m³/s)	1 in 100 AEP (m³/s)
RFFE	42.3	54.6
Regional Rational Method	17.7	20.9
Historical FFC (small catchments)	17 - 22	21 - 26
DoC (2005)	14 - 29	18 - 38

The above outcomes present a wide range of flows between 18 m³/s and 55 m³/s for the 1 in 100 AEP flood. The RFFE approach presents a flow which is substantially larger than the other estimates. It is noted from the RFFE analysis, that there is a wide potential range of flows and it is not considered that this estimate should be considered to be a more accurate representation than other procedures.

In developing routing parameters for the catchment model, it is considered that the higher end of the flow range (greater than 25 m³/s) should be targeted. The outcomes for this study are primarily focussed upon spillway sizing and, therefore, the more extreme events. Given that the approach targeted in DoC (2005), has focussed predominately on appropriate PMF flows, however, it is considered that the upper end of that range (~ 35 m³/s) should be used as an upper calibration target flow. Flows in the range of 25 m³/s to 35 m³/s are to be targeted for calibration purposes.

2.4   RORB Catchment Model

2.4.1   General

A RORB runoff routing model has been used to simulate the hydrologic performance of the ESS catchment.

The RORB (Laurensen et al 2010) model simulates the catchment routing characteristics utilising a representation of the stream network and the parameters k_c and m. The effect of the catchment in delaying the runoff response from rainfall is represented by k_c and the non-linearity in the storage discharge relationship for the catchment is represented by m. The model also incorporates a loss model to account for rainfall lost to groundwater stores, evaporation and various other sinks.

The RORB model was initially developed for existing conditions for calibration purposes. The proposed embankment and spillway was then incorporated into the model for design runs for determining spillway operation probabilities. The characteristics of the model which require specific consideration for the purposes of model calibration are as follows:

- Sub-catchment area and stream network delineation
- Parameter selection
- Design temporal and spatial patterns

These aspects are discussed further below.

2.4.2   Sub-Catchment Area and Stream Network

The RORB model consists of an area of 4.6 km², which was delineated into 25 sub-catchments ranging in size from 0.10 km² to 0.26 km².

The number of sub areas for the model has been influenced by the locations at which the model outputs are required (embankment location). The competing influences for catchment delineation are:

SMEC

- A preference for between 3 and 5 sub areas upstream of any point where flow measurements are required.
- A preference to keep sub areas across the catchment to as similar a size as possible.
- A preference to reduce the impact of large point source inflows to the downstream inundation area when modelling to estimate inundation consequences.

The fraction impervious value for each sub area was selected based upon the degree of development and land use in each sub area. The adopted impervious values are outlined in Table 2-3.

Table 2-3 – Adopted Zoning Imperviousness

Land Type/Planning Zone	Fraction Impervious
Agricultural Land	0.10
Forested Area	0.05
Road	0.70
Water Storage	1.00

The stream network was established based upon the overland flow paths as indicated by surface contour information. RORB allows a choice of five different reach types to be selected (1=natural, 2=excavated and unlined, 3=lined channel or pipe, 4=drowned reach, 5=dummy reach). All reaches were set to natural reaches during the calibration of the existing conditions model. Design model runs incorporating the proposed embankment and spillway, consisted of applying drowned reach types for reaches situated in the FSL storage area upstream of the embankment. The remaining reaches were set as natural reach types.

Details of the catchment model are shown in Figure 2-2 and Table 2-4.
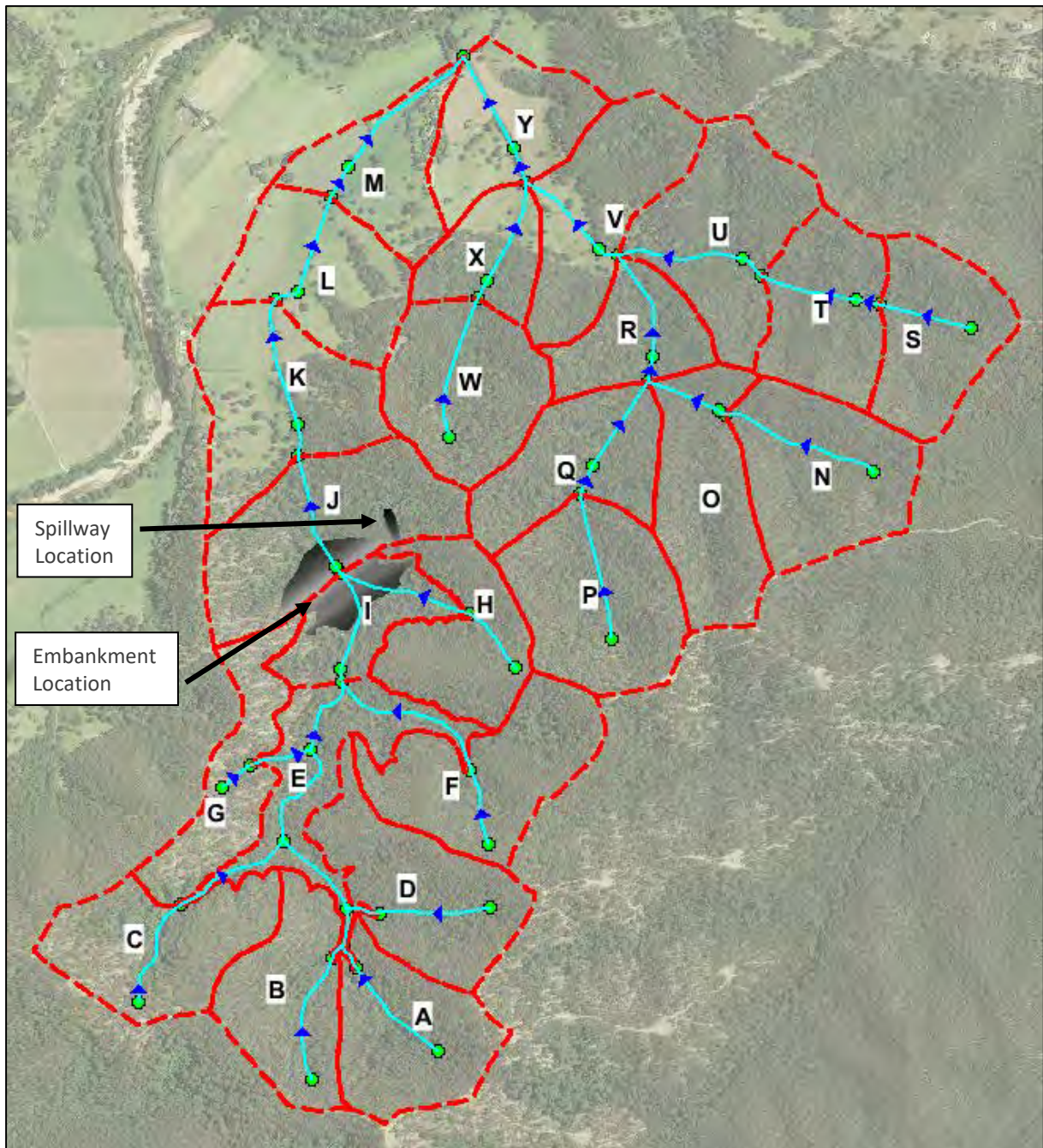
Figure 2-2 - Sub-catchment delineation and storage embankment location

Table 2-4 - Sub-catchment details

Sub-Area Number	Sub-Area	Area (km^2)	Fraction Impervious
1	A	0.186	0.05
2	B	0.176	0.05
3	C	0.202	0.05
4	D	0.213	0.05
5	E	0.179	0.05[1]
6	F	0.218	0.05
7	G	0.145	0.05
8	H	0.165	0.05
9	I	0.117	0.05[1]
10	J	0.256	0.05
11	K	0.234	0.078
12	L	0.186	0.091
13	M	0.143	0.119
14	N	0.22	0.05
15	O	0.197	0.05
16	P	0.236	0.05
17	Q	0.169	0.05
18	R	0.119	0.051
19	S	0.171	0.05
20	T	0.169	0.05
21	U	0.224	0.05
22	V	0.203	0.061
23	W	0.204	0.05
24	X	0.102	0.065
25	Y	0.166	0.09

1 Fraction impervious adopted to be 1.0 for scenarios with the Eurobodalla Southern Storage present

2.4.3   RORB Modelling Parameters

2.4.3.1   *General*

In the absence of rainfall and flow data across the catchment for calibration, parameters have been determined giving consideration to past studies and data from adjacent catchments. Initial estimates for various parameters have been identified and then varied as required to ensure consistency against the target flow and expected catchment delay.

2.4.3.2   *K_c Value*

The (Laurensen et al 2010) recommended approach for selecting RORB runoff routing model parameters k_c and m is to calibrate the catchment file utilising rainfall and runoff data for selected historical storm events. Where there is insufficient data on or near the catchment under investigation, then the approach can be to apply regional equations, to review available data from similar catchments and also to review outcomes from other studies.

Regional Equations

There are a range of regional equations to determine the catchment delay which typically takes the form of:

kc = b × Ad

Where:

A = area in (km²)

b = Coefficient

d = Coefficient

The regional equation developed for eastern NSW, and the value adopted in DoC (2005) are shown in Table 2-5, indicates the expected kc value to represent the catchment delay would be 1.49.

Table 2-5 - Regional equation kc value

Estimate	Equation	Kc
Eastern NSW (Kleemola)	$k_c = 1.22 * A^{0.46}$	1.49
DoC (2005)	NA	1.52

2.4.3.3   m Value

Although, the m parameter can vary in the range of 0.6 to 1.0, it is recommended in IEAust (1997) that a value of 0.8 be used in absence of evidence supporting an alternative value. Therefore, the value of 0.8 has been adopted for the purposes of developing design storm hydrographs.

2.4.3.4   Losses

The RORB rainfall runoff model can be run with one of two loss models. An initial loss is incorporated into both models. In addition, either a constant continuing loss or a proportional loss is included in the model.

Ball et.al. (2016) provide guidance on models to apply and values to adopt when undertaking rainfall runoff modelling. When undertaking extreme flood analyses, it is preferred that the continuing loss model be used, since there is explicit guidance on how to evaluate continuing losses for extreme flood events. There is no similar guidance available for the proportional loss model. In more frequent events, it is open to the practitioner as to which loss model to apply.

The continuing loss model has been applied for this study.

Laurensen et al (2010) recommend that the loss values should be determined through a calibration utilising rainfall and runoff data from selected historical storm events. Where there is insufficient data on or near the catchment under investigation, then the approach can be to apply regional values, to review available data from similar catchments or other studies and to undertake reconciliation against independent flood frequency estimates.

Potential ranges of loss values suitable for the catchment area were analysed using the ARR data hub recommended loss values derived using prediction equations and from the DoC (2005).

The potential range of loss values and the adopted values are outlined in Table 2-6.

Table 2-6 - Details of potential range of loss values

Source	Initial Loss (mm)	Continuing Loss (mm/hr)
ARR DataHub Regional losses	15	5.4
DoC (2005)	40	2.5

In reviewing the catchment delay, consideration was given to the regional time of concentration formula for NSW as presented in McDermott & Pilgrim (1982). The formula is also presented in IEAust (1987) and takes the form $t_c = 0.76 \times A^{0.38}$.

Where:

t_c = minimum time of hydrograph rise (hr)

A = catchment area (km^2)

The formula is most typically referred to as a time of concentration formula, but has been derived from minimum times of hydrograph rise which are highly correlated with catchment lag, noting that the lag should be greater than the minimum time of hydrograph rise.

The minimum time of hydrograph rise per the above formula is 0.9 hr at the storage site and it is therefore to be expected that the critical storm duration would be between 1 and 2 hrs.

2.4.3.5 Validation procedure

The RORB model was run with an ensemble of the 1 in 100 AEP temporal patterns as may be sourced from the data hub website (accessed 9/12/16) at the following location:

http://data.arr-software.org/

The model was run with the following aims:

- Peak flow target value of 25 - 35 m^3/s for the 1 in 100 AEP event.

- Critical duration flow in the range of 1 – 2 hrs

- Catchment delay parameter around 1.5

- Losses are broadly consistent with the datahub and previous studies estimates

- Similar parameter sets should be achievable for both the ensemble temporal patterns and the Generalised Short Duration Method (GSDM) temporal pattern at the 1 in 100 AEP event. The 1 in 100 AEP event represents the point where the flood frequency curve transitions between temporal patterns.

It may be noted that Ball et.al.(2016) propose that the approach to undertaking design event analyses should use either ensemble of storm temporal patterns for each duration with fixed initial loss values or alternatively a monte carlo analysis of initial loss and burst temporal patterns. For extreme flood analyses, it is recommended in Ball et.al. (2016) that the former process be adopted.

The critical duration event for the study catchment is expected to be a short duration event and notwithstanding the recommendations of Ball et.al. (2016), there is no ensemble of short duration extreme rainfall patterns available for analyses.

For the purposes of undertaking the validation, both the short duration temporal pattern described in BoM (2003) and the ensemble approach have been applied.

The validation procedure resulted in the outcomes presented in Table 2-7 and the parameter set listed in Table 2-8. The results of the validation runs are representative of the calculated flow at the location of the proposed ESS, prior to construction of the storage.

Table 2-7 - Results for Validation Run at Eurobodalla Southern Storage Location

AEP (1 IN X)	Temporal Pattern	Peak Flow (m³/s)	Inflow Critical Duration (hr)	Time of rise (hr)	K_c
50	Ensemble	23.2	0.75	0.7	1.2
50	GSDM	23.5	1.0	0.5	0.42
100	Ensemble	26.7	0.75	0.7	1.2
100	GSDM	26.7	1.0	0.7	0.42
PMF	GSDM	313	0.25	0.2	0.42

With respect to the various modelling aims, the following outcomes were achieved.

- A peak flow at the lower end of the target range has been achieved for both sets of temporal patterns. The target flow has been achieved for both sets of temporal patterns.

- The critical duration is at the lower end of the target range for both sets of temporal patterns with the ensemble temporal pattern flows dropping below the lower end of the range. This is suggestive of a very responsive catchment.

- The catchment delay parameter was found to vary between the two approaches. A consistent delay parameter could not be achieved for both sets of temporal patterns while meeting other criteria. The Catchment delay parameter for the GSDM study is substantially below the suggested figure of 1.5.

- Continuing losses have been applied which are consistent with DoC (2005). Lesser initial losses have been applied, however. The values adopted are less than suggested by the regional analysis from the datahub.

- It was not possible to achieve similar delay parameter sets and meet the other design criteria for the calibration.

The PMF estimate is of a similar magnitude to the upper estimate presented in DoC (2005). This figure is in turn high when compared against the world's largest recorded floods as reported in WMO (2009). An envelope curve presented in that document reports that for a catchment area of 1.6 km², flows of up to 220 m³/s have been observed.

There is no reason, therefore, to consider that the 1 in 100 AEP target flow criterion is too low. The difficulty in achieving the target flow of 25 m³/s without using very low losses and a very short catchment delay is suggestive that the flow may be an upper estimate of the 1 in 100 AEP peak flow for the catchment.

It has been found that the various modelling criteria can best be met by adopting a target flow of around 25 m³/s. No solution was identified which could meet the various criteria using a larger target flow of 35 m³/s or higher. The values provided in Table 2-8 are the recommended RORB parameters to be adopted for rainfall runoff modelling for the catchment upstream of the proposed ESS.

Table 2-8 - Adopted RORB parameters

Parameter	Value
m	0.8
K_c	1.2 - Ensemble
	0.42 - GSDM
Initial Loss (mm)	1.0
Continuing Loss (mm/hr)	2.5

2.5   Design Flood Modelling

2.5.1   Design Rainfall

2.5.1.1   General

Design rainfalls were developed for the catchment and applied to the RORB model to derive a runoff frequency relationship for flood frequency modelling.

The Eurobodalla storage catchment area was estimated to consist of 100% rough terrain to determine the PMP depths.

2.5.1.2   Design Rainfall Estimation

Design rainfall depths used in the development of the RORB storm files were obtained as follows:

- 1 in 50 and 1 in 100 AEP design rainfalls and others more frequent were estimated using the on line Bureau of Meteorology (BoM) website tool located at http://www.bom.gov.au/water/designRainfalls/ifd/index.shtml.   It may be noted that currently there are two IFD relationships available on this website, being 1987 and 2016 data sets. The 2016 data set has been applied in this analysis.

- Areal reduction factors were applied to rainfalls using the procedure described in Ball et. al. (2016).

- Probable Maximum Precipitation (PMP) rainfall depths for durations up to 6 hours were developed using Bureau of Meteorology (2003).

- Probable Maximum Precipitation (PMP) rainfall depths for durations greater than 6 hours were developed using the Generalised Southeast Australia Method (GSAM) procedure described in BoM (2006).

- Rainfall depths between 1 in 100 and 1 in 2,000 AEP events for durations of 6 hours and less were determined using the growth curves described in Jordan et.al. (2005).

- Rainfall Depths between 1 in 2,000 AEP and PMP were determined using the interpolation technique described in IEAust (1999), with the AEP of the PMP assigned as 1 in 10,000,000.

Design rainfalls used in the analysis are included as Appendix A.

2.5.1.3   Design Temporal Patterns

As part of the revision to ARR, the recommended approach for applying temporal patterns has been modified to incorporate an ensemble of patterns to simulate the variability in resultant catchment flows. The Bureau of Meteorology has developed ensembles of temporal patterns for 12 different regional areas, to assess the regional characteristics. Regional areas consist of samples of 10 temporal patterns for each duration within the AEP groups of rare, intermediate and frequent.

The ensemble of temporal patterns developed for the Southern Slopes (Mainland) was adopted for this study.

Model calibration runs consisted of determining the average flow from the range of 10 patterns trialled for each duration, ultimately determining the critical duration.

The design hydrographs for storm events up to the 1 in 100 AEP were developed by selecting the temporal pattern which best fits the average peak flow. Selection was based on the temporal pattern that produced the closest match to the target peak flow, where the target flow is the average of all modelled temporal patterns. A hydrograph with peak flow which was within 10% of the average flow

SMEC

was targeted. An additional criterion applied to select the relevant hydrograph where several are available to select from is to adopt a pattern which is more uniform.

Plots of the critical duration ensemble patterns being 9 hr and 12 hr are shown below in Figure 2-3 and Figure 2-4, respectively.
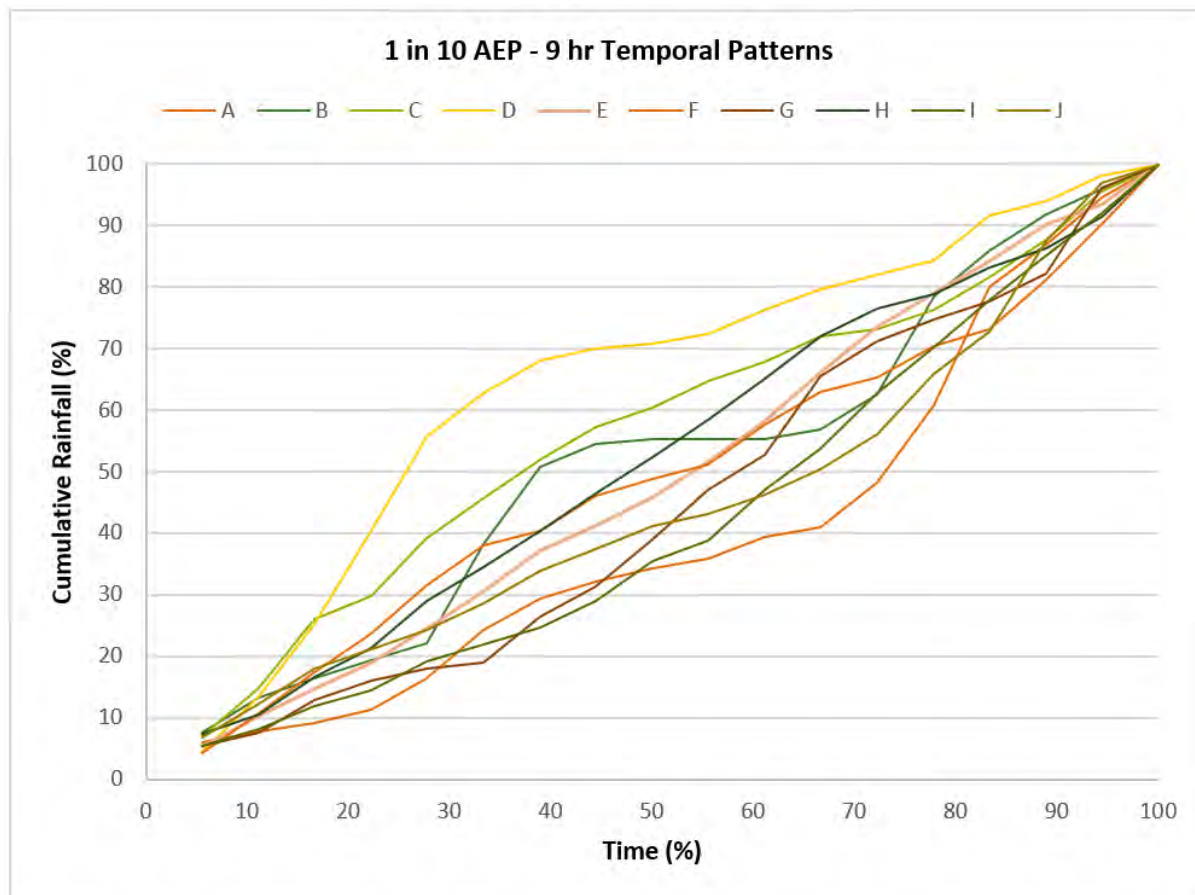
Figure 2-3 - 1 in 10 AEP - 9hr ensemble of temporal patterns
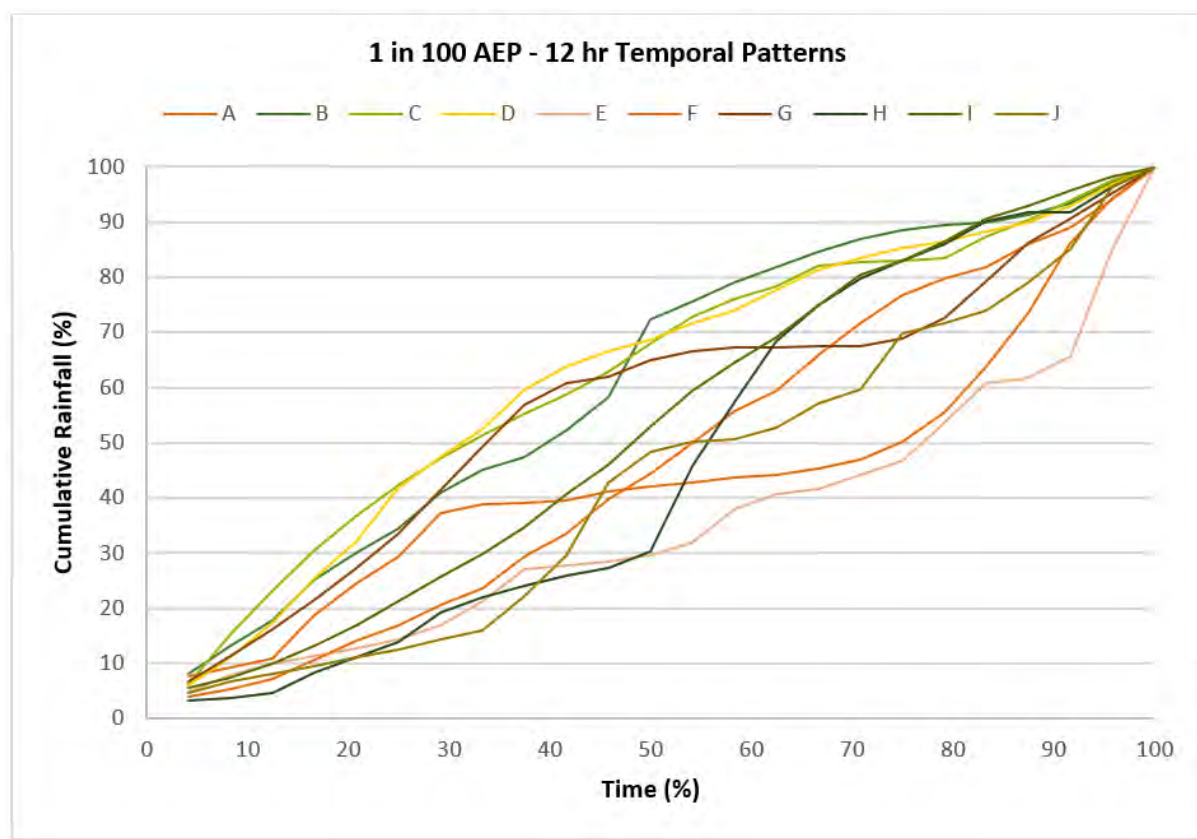
Figure 2-4 - 1 in 100 AEP - 12hr ensemble of temporal patterns

For rainfalls between an AEP of 1 in 100 and PMP:

- GSAM temporal patterns were adopted (Bureau of Meteorology 2006) for storm durations greater than 24 hours

- GSDM temporal patterns (Bureau of Meteorology 2003) were adopted for storm durations of 6 hours or less.

For storm durations between 6 hours and 24 hours, the results using both GSAM and GSDM temporal patterns were computed.

Pre-burst patterns described in Jordan et al (2005) were utilised for short duration GSDM storms.

2.5.1.4   Design Spatial Patterns

The selection of the design spatial patterns was based on the recommendations presented in Ball et.al. (2016). For annual storm return periods less than a 1 in 100 AEP a uniform spatial pattern was adopted, due to the small catchment size. For annual storm return periods greater than 1 in 100 AEP, the GSDM spatial pattern was applied over the catchment, with the storm centred over the catchment area of the storage.

2.5.1.5   Design Losses

The recommendations of Ball et.al. (2016) were applied for the purposes of defining design event losses as follows:

- Initial loss was held constant for all storm events up to the 1 in 10,000,000 AEP. The initial loss for the PMF was set to zero.

- Continuing Loss was held constant at 2.5 mm/hr for floods up to a 1 in 100 AEP magnitude. Losses were then reduced over the range of extreme events to 1 mm/hr for the PMF event.

Losses were varied as detailed in Table 2-9. These represent the recommended continuing loss values to be adopted within the catchment upstream of the proposed ESS for flood events less frequent than 1 in 100 AEP.

Table 2-9 - Adopted Continuing Loss

Storm AEP	Continuing Loss (mm/hr)
1 in 100	2.50
1 in 1,000	2.08
1 in 10,000	1.73
1 in 100,000	1.44
1 in 1,000,000	1.20
1 in 10,000,000	1.00
PMF	1.00

2.6 Eurobodalla Southern Storage Modelling

2.6.1 Model Development

2.6.1.1 General

As part of the RORB modelling for Eurobodalla Southern storage, it was necessary to include the relevant reservoir characteristics including the elevation storage and storage discharge relationships.

The key characteristics for the storage have been set as part of the concept design development and review process. The characteristics of the storage are outlined in Table 2-10.

Table 2-10 Storage characteristics

Characteristic	Value
Embankment Crest Elevation (m AHD)	50.0
Embankment Length (m)	360
Spillway Crest Elevation (m AHD)	47.7
Storage Volume at Spillway Crest (ML)	3,120
Storage Volume at Embankment Crest (ML)	3,836

Details are included below.

2.6.1.2 Elevation Discharge Relationship

The proposed spillway takes the form of a concrete crest constructed in a trapezoidal shaped concrete chute. The spillway dimensions from the original concept design have been reviewed and considered to be reasonable. The overall dimensions from the concept design have been adopted for use in this study and key dimensions are listed in Table 2-11.

Table 2-11 Storage characteristics

Characteristic	Value
Spillway chute base Width (m)	15
Spillway side slopes	1 in 1
Crest height above base (m)	0.8

The chute has been designed to ensure that the hydraulic control remains at the spillway crest. The discharge coefficient has been computed from first principles using two separate publications. Tracy (1957) has been used to determine the discharge coefficient for the main weir, while Ghodsian (2004) has been used to estimate the flow in the triangular portions at the end of the weir.

Tracy (1957) suggests that the discharge coefficient will vary between 1.6 and 2.3 depending upon the hydraulic head.

The formula in Ghodsian (2004) is presented below.

$Q = 16/(25 \times \sqrt{5}) \times C_{d} \times \sqrt{2g} \times h^{5/2}.x.\tan(\Theta)$

$C_{d} = 0.984 (h/H)^{0.025}$.

Where

Θ = Side slope angle

h = hydraulic head (m)

H = Weir height (m)

Losses in the approach channel, although these are minor, is calculated using Manning's Formula with a friction coefficient of "n" = 0.035.

The computed elevation discharge relationship has been checked using a 1D representation of the chute and crest using a HECRAS model. The HECRAS model indicates that a similar peak discharge is obtained, with a slightly different curve shape. The first principles relationship has been adopted for use in the study as shown in Figure 2-5.
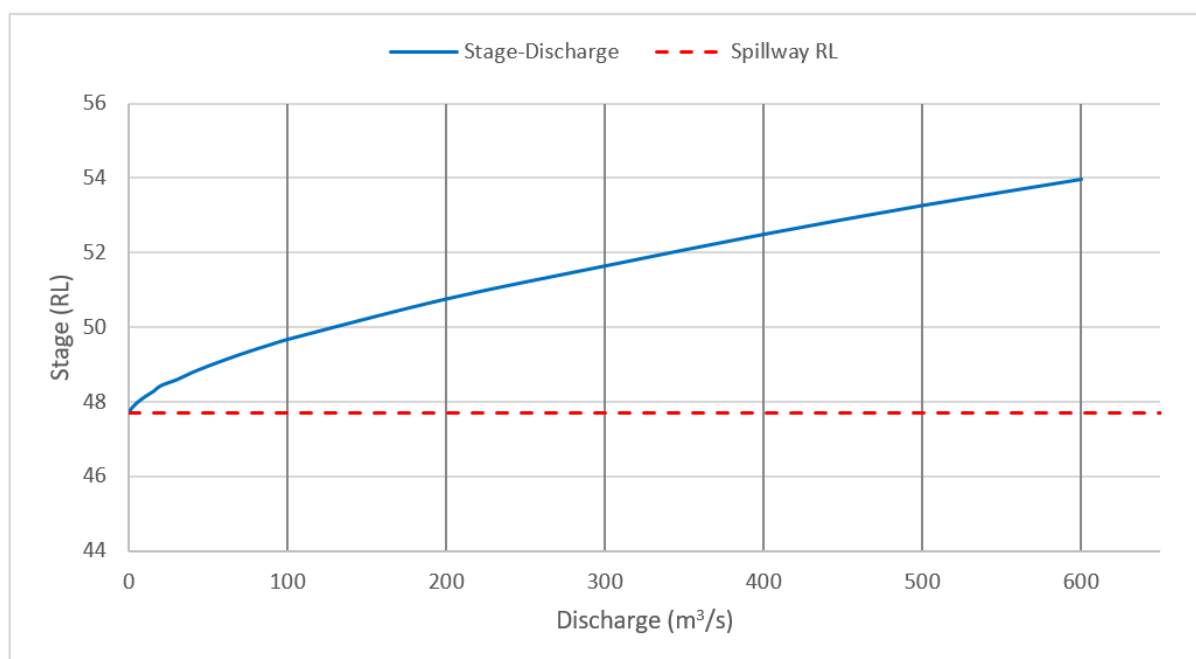
Figure 2-5 Spillway stage - discharge relationship

2.6.1.3 Elevation Storage Relationship

SMEC has derived a stage storage relationship for the storage area utilising the 12D software package in conjunction with lidar data supplied by Eurobodalla Shire Council. The relationship is detailed as Figure 2-6.
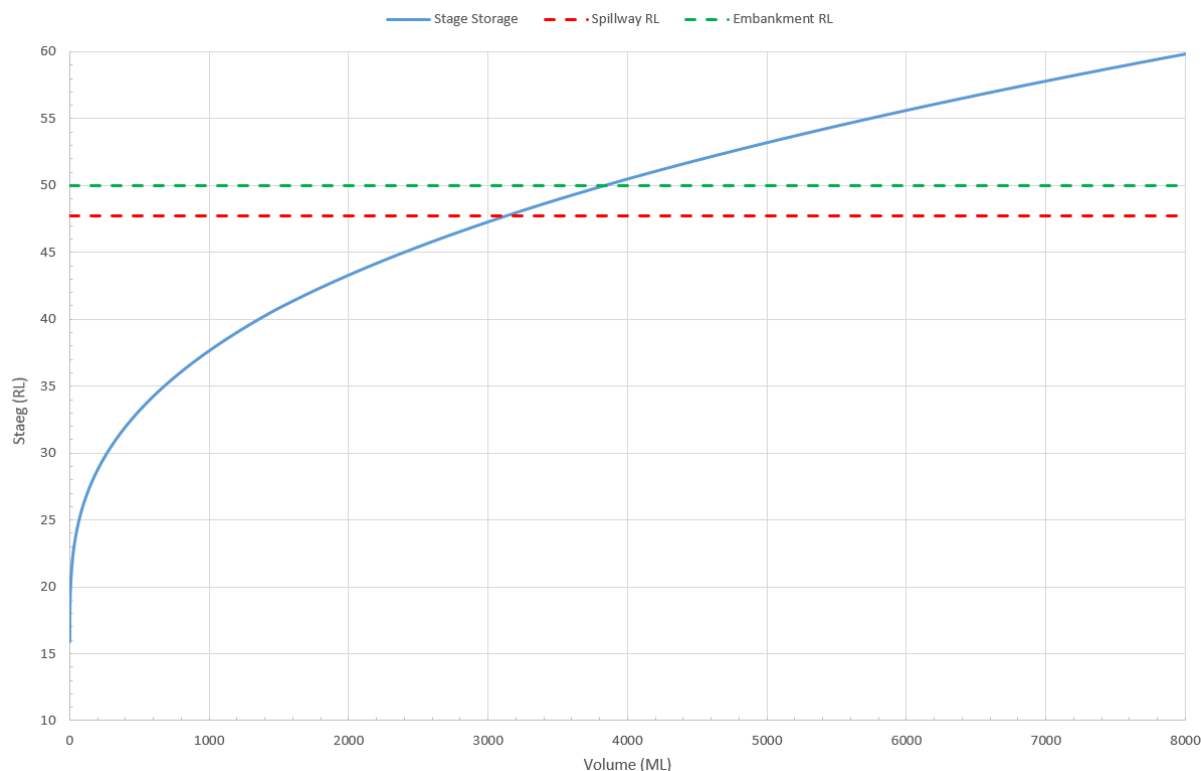
Figure 2-6 - Stage - storage relationship

2.6.2 Modelled Outcomes

2.6.2.1 General

The RORB modelling for the ESS was undertaken to establish the following:

- Establish the flood frequency curve for flood events
- Determine PMF

2.6.2.2 Flood Frequency Curve

The rainfall runoff model with the storage being present was run to establish the flood frequency curve and the results are presented as Table 2-12, Figure 2-7 and Figure 2-8. It is noted that the peak inflow values shown in Figure 2-8 are for very short duration flood events, 15 or 20 minute storms. The peak outflow events are longer duration events with the critical duration storms events typically being 9 hour storms and 3 hour storms for the very extreme events.

The peak outflow and peak reservoir of the storage for the 1 in 10,000,000 AEP and PMF flood events are reported to be equivalent values. The PMF levels are marginally higher than for the 1 in 10,000,000 AEP flood in this case for the following reasons:

- Starting reservoir level for both scenarios has been adopted to be at FSL. This is considered appropriate for the 1 in 10,000,000 AEP event due to the expectation that the reservoir will typically be operated at close to FSL.

- Consistent continuing loss values adopted as outlined in Table 2-9.

- Similar initial loss values adopted with zero for the PMF scenario compared to 1mm for the 1 in 10,000,000 AEP flood event. The 1mm initial loss is based on the calibration results as provided in Table 2-8. The negligible difference in these adopted loss values is further influenced by the large water surface relative to the catchment size.

Table 2-12 - Results for Eurobodalla Storage

AEP (1 IN X)	Peak Storage Elevation (m)	Inflow (m³/s)	Inflow Critical Duration (hrs)	Peak Outflow (m³/s)	Outflow Critical Duration (hrs)
2	47.8	15.4	0.33	2.4	9
5	47.9	23.0	0.33	3.7	9
10	48.0	28.5	0.25	4.7	9
20	48.0	34.2	0.25	5.7	9
50	48.1	40.8	0.33	8.3	12
100	48.1	48.5	0.25	9.1	9
1,000	48.3	126.5	0.25	16.4	9
10,000	48.5	188.1	0.25	24.9	9
100,000	48.7	261.4	0.25	37.1	9
1,000,000	49.0	333.2	0.25	53.4	3
10,000,000	49.4	390.0	0.25	76.1	3
PMF	49.4	390.0	0.25	76.1	3
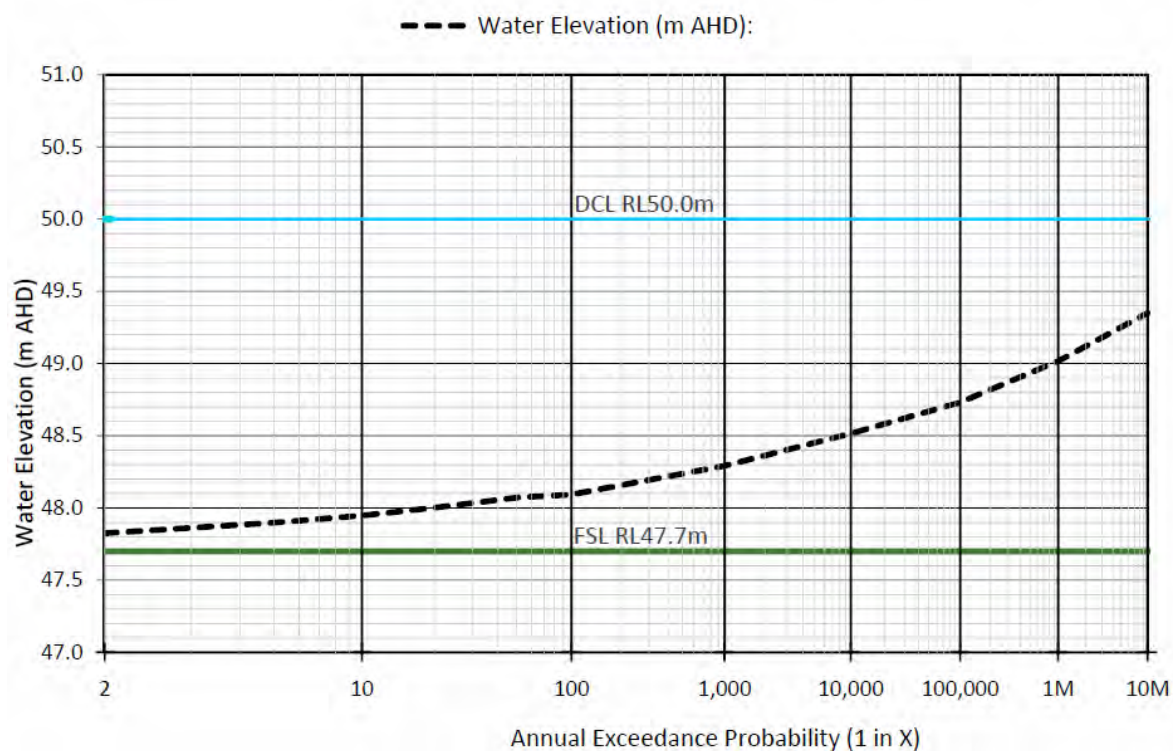
Figure 2-7 – Elevation Frequency Curve
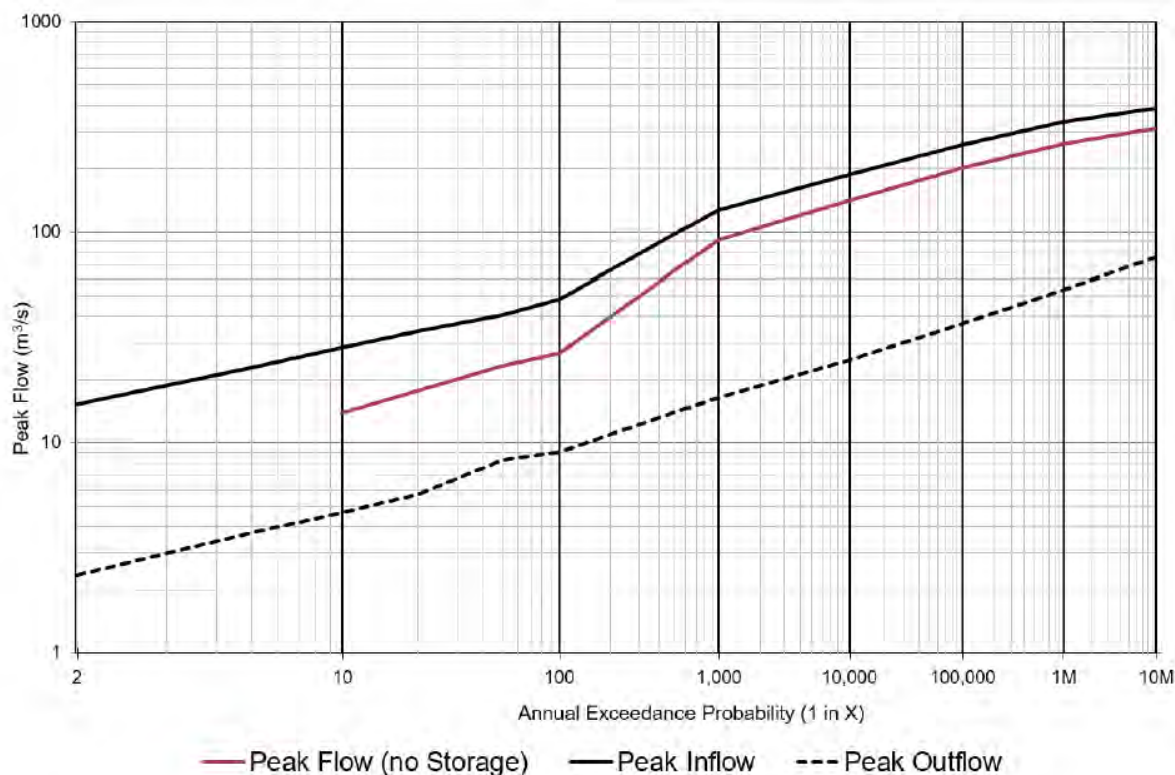
Figure 2-8 – Flood Frequency Curve with storage present

2.7 Cofferdam

2.7.1 General

For the construction of the storage embankment to occur, a waterway diversion is required. The waterway diversion is proposed to consist of a cofferdam which will also act as a storage that will supply water for the construction works. The cofferdam is to be designed with the capacity to pass a 1 in 10 AEP flood without overtopping.

The RORB model developed for the ESS flood frequency curve has been used to determine the embankment height required to pass a 1 in 10 AEP flood without overtopping.

2.7.2 Cofferdam Investigation

The cofferdam spillway is to be designed to be integral with the ESS low level outlet, which in turn comprises a 1200 mm diameter pipe. The spillway will act as uncontrolled crest (culvert) structure. The capacity of the culvert has been computed as an open channel flow structure up to the height of the culvert and from there as an inlet control culvert.

The storage characteristics have been incorporated into the RORB model and it has been run with 1 in 10 AEP design events to establish the required height of the cofferdam embankment. As part of the modelling, drowned reach types were adopted for reaches within the FSL of the cofferdam. It was also necessary to include the relevant storage - discharge characteristics of the cofferdam. Stage storage and stage discharge relationships for the cofferdam were developed, and are shown in Figure 2-9 and Figure 2-10.
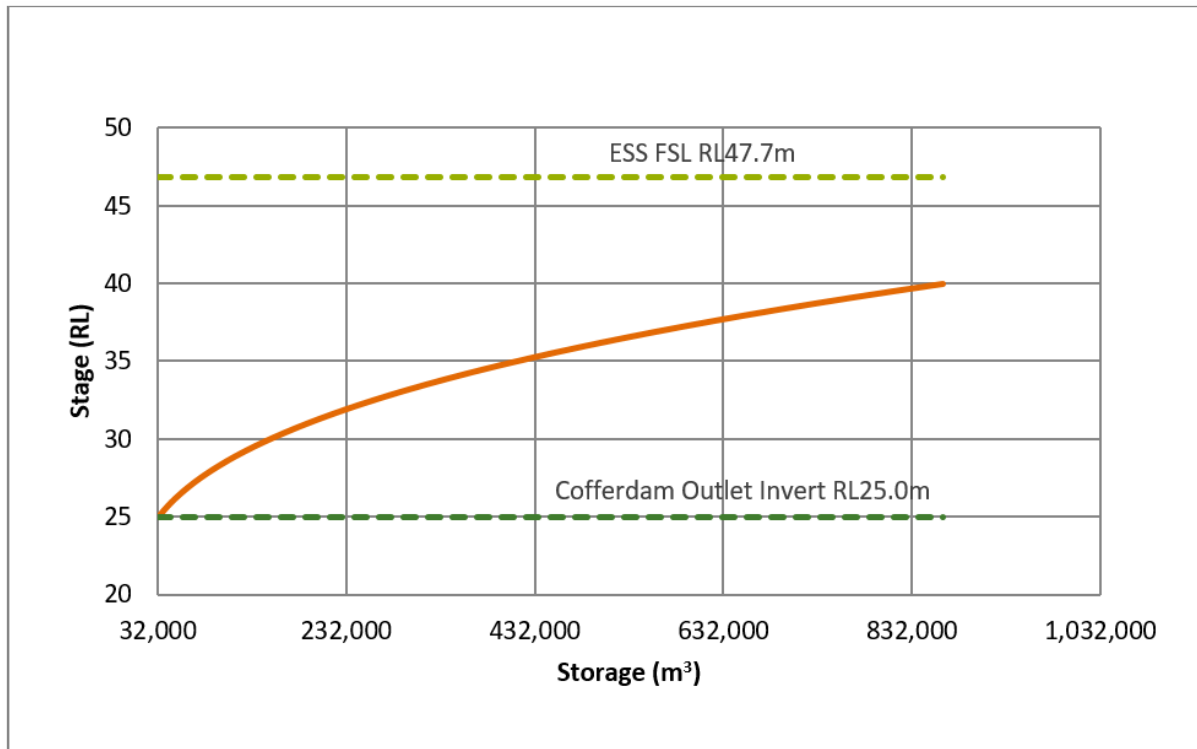
Figure 2-9 - Stage-storage for Cofferdam
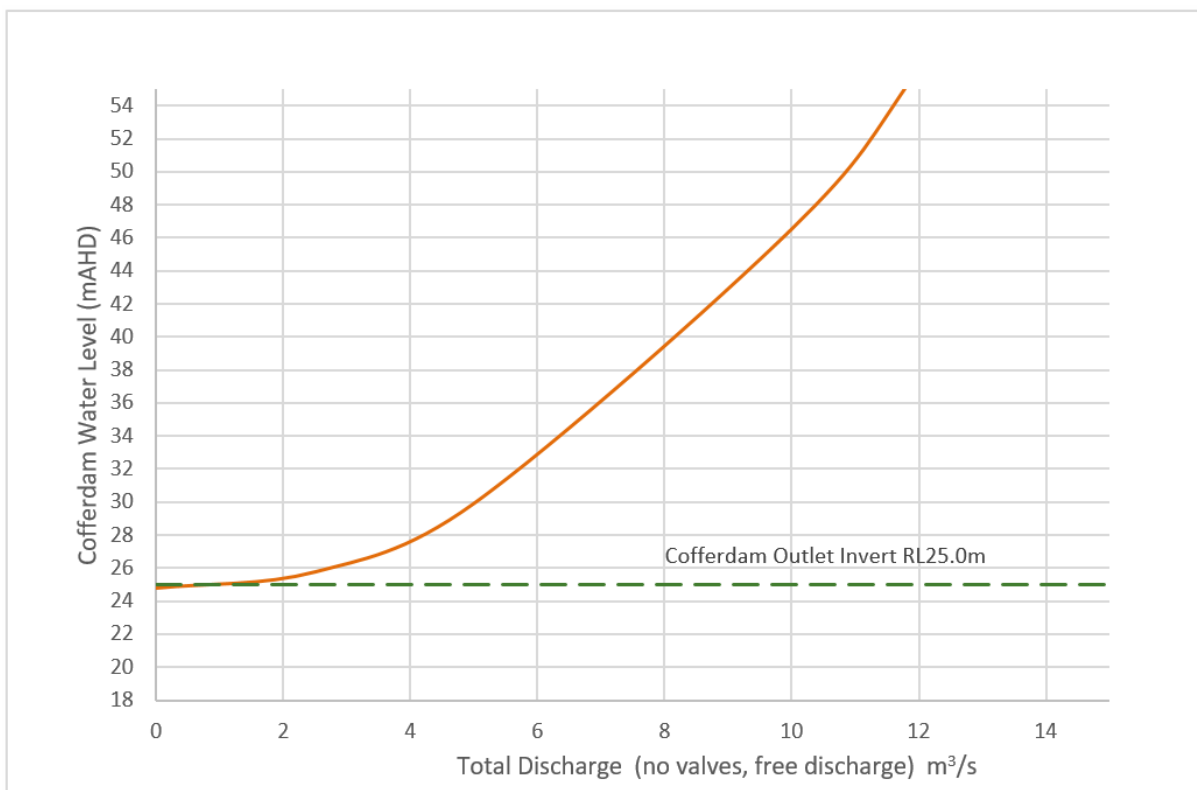
Figure 2-10 - Stage-discharge for Cofferdam

The modelling indicated that the characteristics detailed in Table 2-13 would be required for the cofferdam.

Table 2-13 – Cofferdam Design Characteristics

Characteristic	Value
Outlet pipe invert (m AHD)	25.0
1 in 10 AEP peak discharge (m³/s)	3.9
Peak 1 in 10 AEP Water Level (m AHD)	28.6
Embankment Crest Level (m AHD)	30 (nominal)

2.8 Coincident Flooding

2.8.1 General

Coincident flooding needs to be considered as part of the dambreak modelling to ensure that the impact of flows in the Tuross River can be modelled appropriately.

The catchment area of Tuross River is estimated to be 1,586 km^2, while the ESS catchment area is 1.6 km^2, or 0.1 % of the larger catchment. Furthermore, the critical duration of the PMF event is 3 hours which represents a convective event centred over the ESS catchment. A convective event of the size modelled for the PMF would be limited in extent. While a large flow may occur in the Tuross River, it is not likely to be an event with a similar probability. In addition, the timing of the Tuross River flood peak is unlikely to coincide with that emanating from the ESS. The minimum time of hydrograph rise for the ESS is estimated to be around 0.9 hr while the minimum time of rise of the Tuross River is estimated to be around 12 hr.

For the purposes of assessing the impact of Tuross River flooding on the dambreak event, a range of flood events have been modelled to test the sensitivity of modelled outcomes to coincident flow magnitude:

1. PMF dam breach with a 1 in 100 AEP storm event on Tuross River

2. PMF dam no-breach with a 1 in 100 AEP storm event on Tuross River

3. PMF dam breach with a 1 in 1 AEP storm event on Tuross River

4. PMF dam no-breach with a 1 in 1 AEP storm event on Tuross River

5. PMF existing conditions (no storage) with a 1 in 100 AEP storm event on Tuross River

6. 1 in 100 AEP dam breach with a 1 in 10 AEP storm event on Tuross River

7. 1 in 100 AEP dam no-breach with a 1 in 10 AEP storm event on Tuross River

The results of the modelling indicate that a coincident flood in the Tuross River with an AEP of 1 in 100 is the critical scenario for flood failure scenarios of the ESS and this has therefore been adopted for assessing the consequence category of the storage as outlined in Section 4.2.1.

For the purposes of modelling the impacts, a water level frequency analysis has been undertaken for the Tuross River using the Tuross River at Eurobodalla streamflow gauge. The peak water level from the gauge has been adopted for use in the hydraulic dambreak model.

The outcomes from Flood frequency and water level frequency analyses are presented in Figure 2-11 and Figure 2-12.
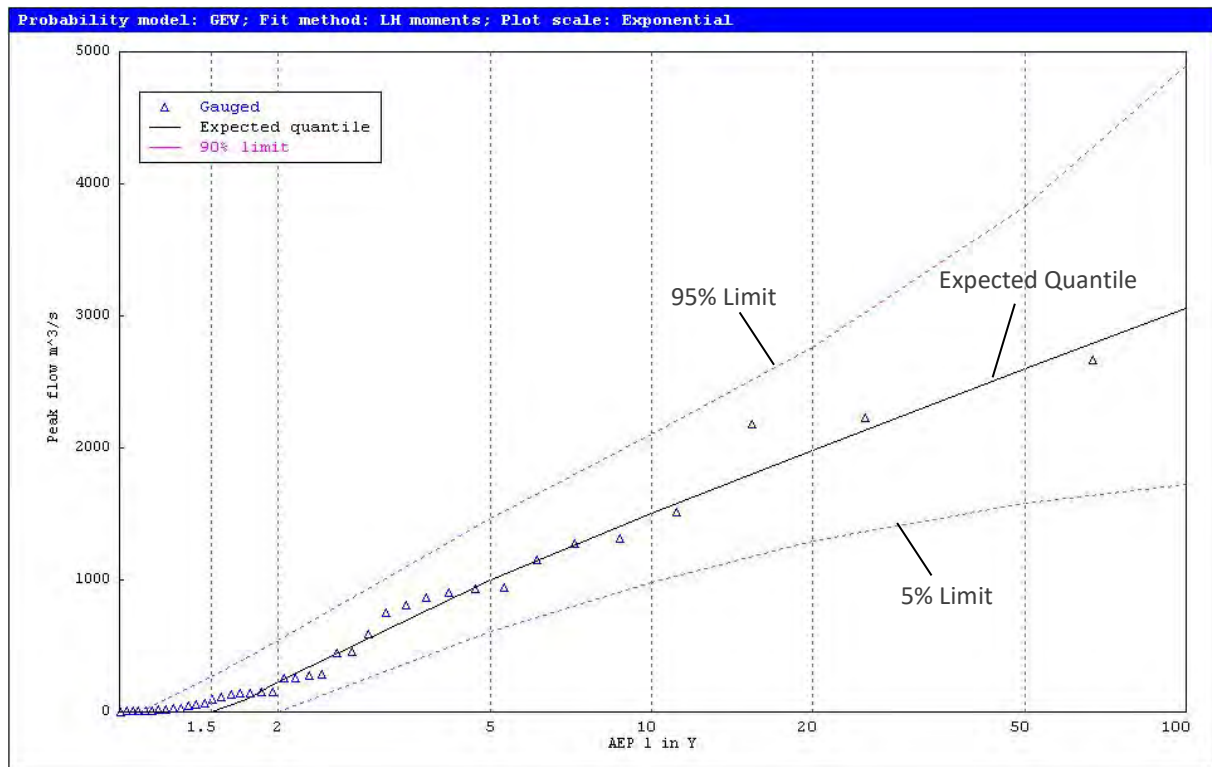
Figure 2-11 - Tuross River Flood Frequency Curve
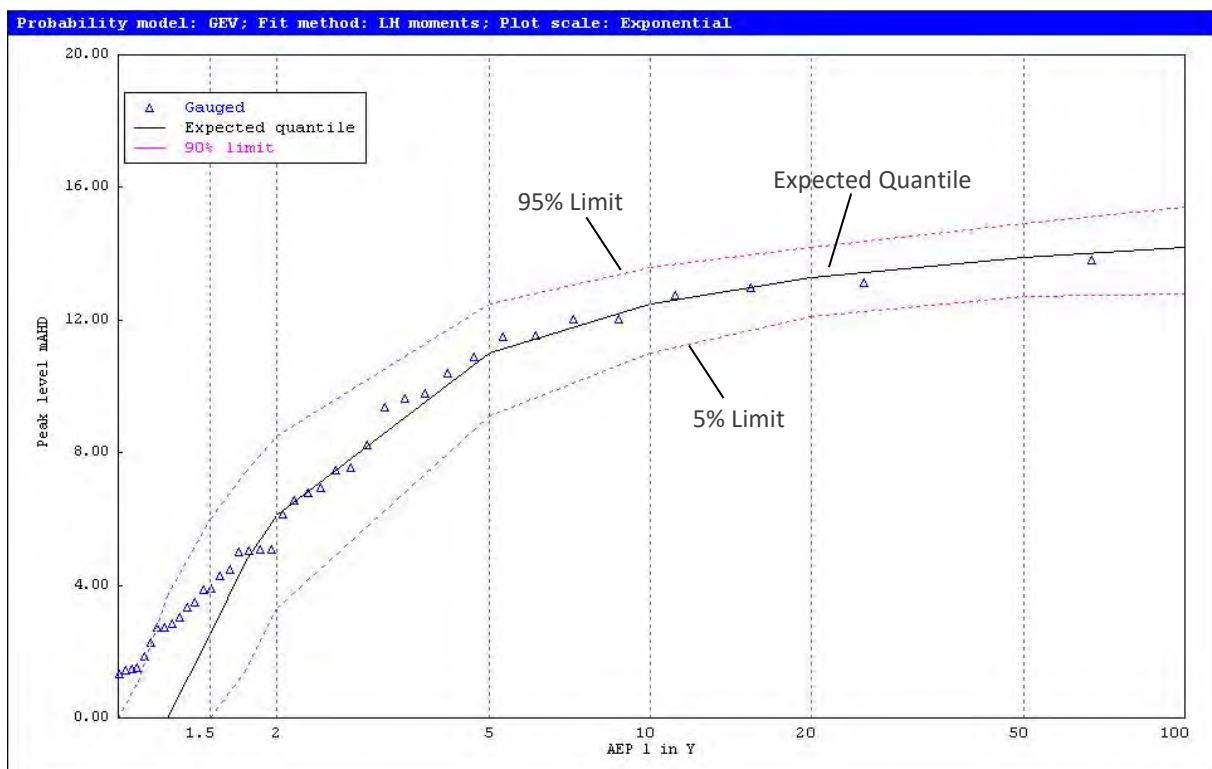
Figure 2-12 - Tuross River Water Level Frequency Curve

2.9   Transparent Storage

As part of the project requirements, the works required to ensure that the storage acts in a transparent manner up to the 1 in 10 AEP is to be considered. The requirement for the storage to act in a transparent manner will be confirmed during the environmental assessment.

The storage is on a very small catchment with an ephemeral watercourse. The size of the catchment upstream of the ESS is approximately 1.6 km2 and forms approximately 0.1% of the Tuross River catchment with an approximate area of 1,586 km2. The catchment downstream of the ESS is predominantly agricultural land with flow directed through a constructed channel.

In order to faithfully produce a transparent system, it would be necessary to monitor inflows from natural watercourses to the reservoir. This is considered to be impractical for the Eurobodalla Southern Storage due to the number of small tributaries which contribute small inflows. Further, it is anticipated that a substantial portion of flow could travel as subsurface flow in this catchment, further complicating inflow measurement.

Should the storage be required to be transparent, it is suggested that water levels be monitored and releases made from the storage through the outlet works accounting for inflows pumped from the Tuross River. The results of the flood

Based on the hydrological assessment, the estimated peak inflow rates and total inflow volumes for flood events up to the 1 in 10 AEP flood are presented in Table 2-14. The total inflow volume represents the maximum volume of water required to be released from the storage following a storm event to meet the transparent storage criteria i.e. should this be a requirement of the storage.

Table 2-14 – Estimated Storage Inflow Rates and Volume

AEP (1 in X)	Peak Inflow Rates m³/s (Storm Duration)	Total Inflow Volume ML (72hr Storm Duration)
2	15.4 (20 mins)	192
5	23.0 (20 mins)	321
10	28.5 (15 mins)	410

3   DAM BREAK ANALYSIS

3.1   General

A dam breach hydraulic model is required to establish the extent and other characteristics of a dam breach flood. This is in turn used to quantify the consequences.

A Tuflow 2D hydraulic model has been used to model the dam break for the storage. Dam break parameters adopted for the analysis have been estimated taking into account the specific nature of the ESS.

The dam break model utilises input hydrographs developed from the RORB upstream of the storage. The model routes those hydrographs through the storage and downstream zone to a logical end point being the bridge over the Tuross River at Princes Hwy where the valley widens out and the incremental flood depths as a result of dam breach are expected to be negligible. A variety of scenarios were modelled using different inflow hydrographs and different assumptions regarding an embankment breach. As follows:

- Sunny Day failure of the dam commencing at FSL

- 1 in 100 AEP flood without failure

- 1 in 100 AEP flood with failure

- PMF without failure

- PMF with failure

- 1 in 10 AEP flood with cofferdam failure

The major tasks undertaken as part of dam break modelling are as follows:

- Identification of dam breach mechanisms and selection of model.

- Computation of outflow hydrograph through the breach as affected by the breach.

- Routing of the outflow hydrograph through the downstream valley considering the storage, frictional resistance, and important downstream structures such as road crossings.

- Mapping of the flood levels determined from the modelling.

Storage characteristics as discussed above have been applied in the analysis. For the failure mode, discharge from breaching was determined using the breach formation specifications discussed below.

3.2   Tuflow Modelling

3.2.1   Model Setup

Tuflow models require either a Digital Elevation Model (DEM) or a Digital Terrain Model (DTM) as input. A DEM is an unmodified elevation surface at a defined grid spacing derived from some input data set, while a DTM is typically considered to be a refined DEM with additional finer scale elements such as break lines, infrastructure or drains incorporated into the surface. For the purposes of dam break modelling, it is generally considered that a DEM provides a sufficient level of detail. For this study, however, a DTM has been developed by incorporating a limited number of additional elements.

For the purposes of modelling undertaken as part of this study, the DTM describes the terrain elevations and in conjunction with roughness parameters defines the surface over which flows will pass. Lidar data covering part of the region was provided by Eurobodalla Shire Council and the remainder of the catchment was defined using a combination of SRTM data and cross sectional

information from a previous dambreak study. A DTM was developed for modelling using the supplied data.

Roughness elements have been defined consistently throughout the model as a Manning's n value for each cell, based on land use. All adopted Manning's roughness values, by land-use type are shown in Table 3-1, with Material ID as used in *Tuflow*. The same information is displayed as areas of different land use in Figure 3-1.

Table 3-1 – Tuflow Modelling Roughness Parameters

Material ID	Manning's n	Description/Use
3	0.040	Waterways, grasslands with Scattered Trees/Open Space
9	0.070	Pasture, lands beyond floodway
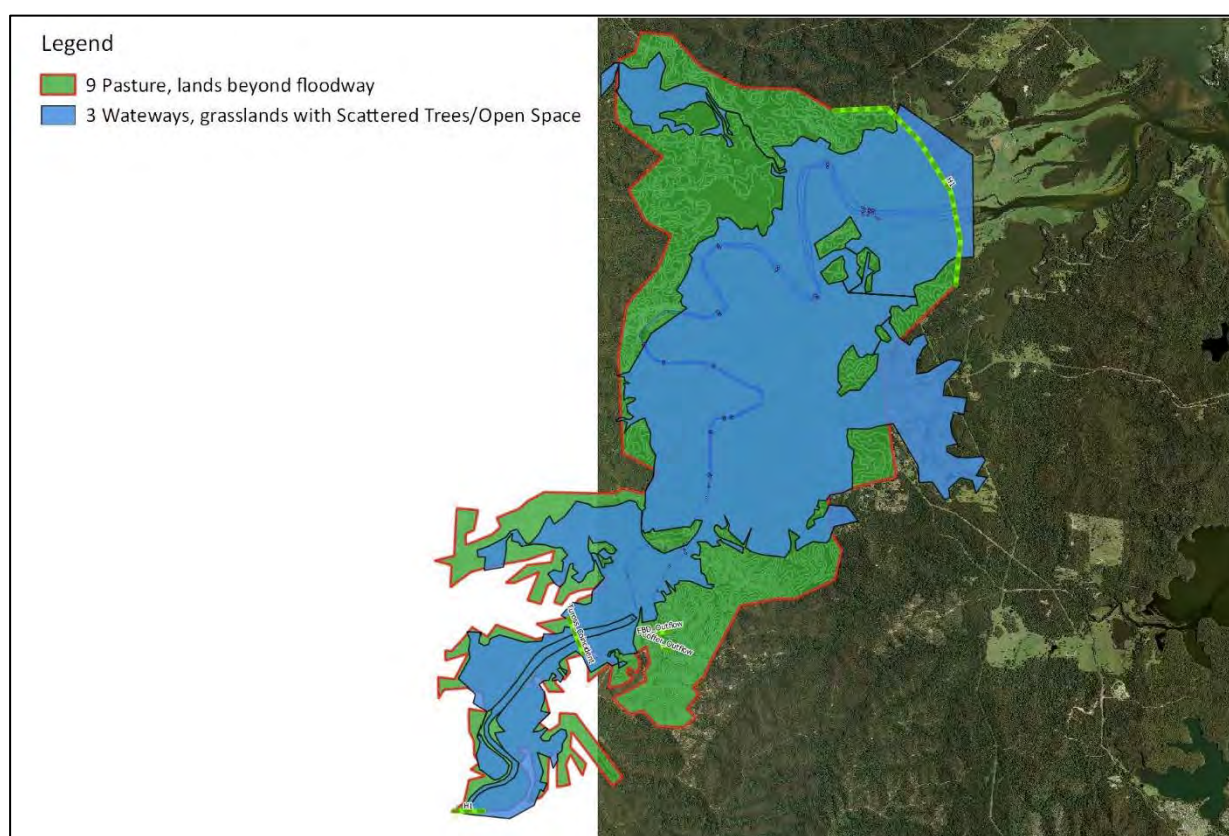
Figure 3-1 - Coverage of land use type / roughness categories adopted

Further turbulence losses are modelled through an eddy viscosity, calculated using the Smagorinsky model with a coefficient of 0.5. The model domain covers an area of 57 km², shown in Figure 3-2.

Tuflow uses a regular (square) grid for computation and outputs. The selection of cell size to adopt for modelling is a trade-off between practicable computational runtimes to allow multiple scenarios to be modelled and sensitivity testing to be carried out, versus accuracy of results. A 10-metre grid size was adopted, providing a good balance between runtime and representation of topographical features.
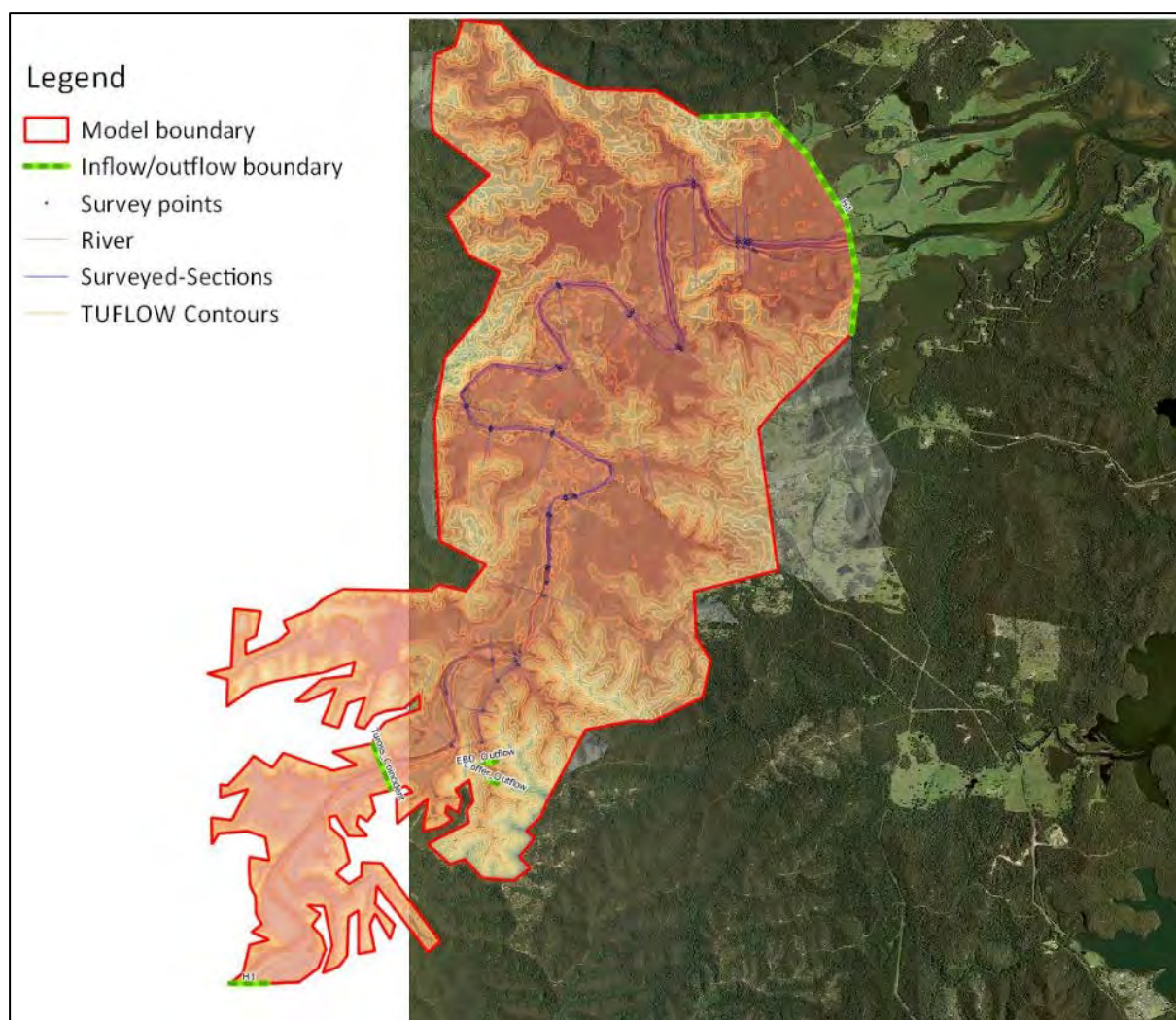
Figure 3-2: Model Domain and Boundary Conditions

Computational time steps was 4 seconds for model runs. The maximum simulation times was 9 hr, allowing full passage of the breach floods through the model domain.

3.2.2   Boundary Conditions

For both Sunny Day and flood overtopping breach models, the downstream boundary around 1.7 km downstream of Princes Hwy bridge is a constant slope of 0.001 (1 in 1,000).

For all models, a flood inflow was allowed for in the Tuross River. The boundary was represented as a water level. The water level was selected based upon the required event probability as determined in the elevation frequency relationship described above. The minimum water level adopted for use in the modelling for any scenario was 1.0 m AHD representing the mean oceanic water level. For the various modelling scenarios, the coincident flow in the Tuross River was established by running the model for a duration of 9 hours to establish stable initial conditions prior to the start of the breach hydrograph.

For all modelled breach scenarios, breach hydrograph was input as an inflow boundary at the toe of the storage. The breach hydrograph was generated separately in HECRAS. Consideration was given to including a variable geometry file in the Tuflow model, but run times were found to be excessively long.

3.2.2.1   *Model Convergence*

Model convergence of TUFLOW models have been examined through interrogation of the Mass Balance Error. TUFLOW literature typically suggests good modelling practice is to target a convergence with less than 1% mass error. This target was achieved across the various model runs except one model with mass error of 4%. It is important to note that all the modelled scenarios are effectively transient models which can induce mass balance error fluctuations.

3.2.3   Model Results

Model results are presented as inundation maps (refer to Appendix E), show depth of flooding and the product of depth and velocity ('DV') for selected areas of interest. The extent of the inundation for each of the scenarios are shown with the maximum depth and peak DV of properties within the inundated area shown as points.

As shown in Appendix E, the incremental flood depths at the downstream model limit are less than 0.3m for all scenarios indicating that the consequences of dam failure beyond the model boundary are expected to be negligible.

3.3   Breach Parameters

3.3.1   General

The dambreak analysis has comprised a range of sub tasks as follows:

- Analyse the embankment design detail and model no breach case
- Determine appropriate Flood and Sunny Day breach parameters.
- Establish flood breach parameter variability
- Develop a dambreak model and run for design scenarios

3.3.2   Dam Breach Parameters

Guidance has been sought from the literature in developing the breach parameters. USBR (2014a) evaluates a variety of equations and suggests that there are two appropriate equations for use in estimating breach time and width. These recommended equations are those developed in Froehlich (2008) and Xu and Zhang (2009). Typically, there is scope to vary breach parameters with failure scenario where the differing cases may be either piping or overtopping. Given that the concept dam design can pass the PMF with additional freeboard, it is unduly conservative to model an overtopping failure. Breach parameters have, therefore, been based upon a piping failure mechanism. The same parameters have been applied for both Sunny Day and flood failure scenarios. The dam breach regression equations typically require use of reservoir storage volume as an input. The total storage at the peak of a PMF has been applied in this case.

The results and the adopted parameters are detailed below:

3.3.2.1   *Breach Width*

USBR (2014a) presents a range of breach width parameter equations and suggest that the Xu and Zhang (2009) and Froehlich (2008) equations are the best of those available for medium and low erodibility dams. USBR (2014a) discusses the application of Xu and Zhang (2009) to low erodibility dams in particular and supports the approach of using erodibility as a basis for altering breach parameters. It queries the validity of the particular equations described in Xu and Zhang (2009) for low erodibility, by observing that the population of cases upon which the relationship is based is small and of questionable quality. There are no suitable alternative dam breach equations available, however, which incorporate erodibility. The approach applied in this report has been to adopt the low

erodibility approach equations nominated in Xu and Zhang (2009) and to check the sensitivity of outcomes to alternative measures as indicated in Froehlich (2008).

The data used to assist in assessing the average breach width is presented below:

- Xu and Zhang (2009) Low erodibility dam – 31 m (~1 times dam height)
  - Input parameters: Height=35 m; Volume = 3,836ML; ZD (Zoned dam); P (piping); LE (low erodibility)

- Froehlich (2008) – 43 m (1.2 times dam height)
  - Input parameter: Volume = 3,836ML

- USACE (2007) Range – 17 m - 175 m (0.5 to 5 times dam height)
  - Input parameter: Height=35m

Summary

- Most likely estimated range using equations: 30 m - 45 m

- Adopted: 31.5 m (Base width 13 m)

- Sensitivity tested over range: to 45 m (Base width 25 m)

3.3.2.2   Breach Development Time

Breach development time may be characterised as breach initiation and breach development phases. USBR (2014a) indicates that Froehlich (2008) provides the best estimate of breach development time as the database upon which the Xu and Zhang (2009) equations are based may include a mixture of breach initiation and development phases. Notwithstanding this observation, the application of breach development times to the analysis is considered to be quite conservative in the case of the ESS for the following reasons:

- It is noted the dam break model simulates the breach development phase and there is, therefore, a case for applying breach development time estimates. It should be recognised, however, that breach initiation time, which can be multiples of the development time is not simulated as part of the dam break modelling process. A piping failure initiating through the base of the dam would be expected to be slow given the substantial width of the embankment base relative to the stored volume. This particular feature is considered to support the adoption of a longer breach development time.

- The ESS is not likely to fail as a result of overtopping. Any piping failure which commences during a flood has a high probability of initiating in the upper zone near the embankment crest and progressing down. This style of failure progression is considered to be consistent with the failure times that form the basis of Xu and Zhang (2009) equations. There will be no overtopping/initiation phase to allow downstream supporting fill to be removed prior to the breach development phase.

- The Stage 1 ESS incorporates a substantial upstream berm (~30 m wide) at a height of 10 m below the embankment crest. This feature is expected to substantially impact upon breach progression. It is considered that the most likely failure progression would incorporate an initial piping failure of the upper 10m of embankment followed by a slower progression through the lower 25 m.

The breach development time approach for Eurobodalla Southern Storage is as follows:

- The upper portion of the embankment will fail in typical fashion. Froehlich (2008) is to be applied

- The lower portion of the bank will fail more slowly and Xu and Zhang (2009) is to be applied to that portion of the embankment.

The data used to assist in assessing the breach development time is presented below:

- Xu and Zhang (2009) Low erodibility dam – 3 hr
  - Input parameters: Height=35 m; Volume = 3,836ML; ZD (Zoned dam); P (piping); LE (low erodibility)
- Froehlich (2008) – 0.3 hr (around 20 min)
  - Input parameters: Height=35 m; Volume = 4,073,000 m3
- USACE (2007) Range – 0.1 hr to 4 hr (6 to 240 min)

Summary

- Estimated breach development time range using equations: 0.3 hr-3 hr

- Adopted: Total 3 hr (0.3 hr top 10m, 2.7 hr bottom 25 m)

- Sensitivity tested over range: to 1 to 3 hrs (total)

3.3.2.3   Breach Side Slopes

USACE (2007) suggests that side slopes can vary across the range of 0-1 H, (X H:1 V).

It is assumed that the breach will progress through to the dam foundation and that the side slopes would be 0.5H:1V. The side slopes are considered to be representative of the short-term stability characteristics of the earth and rockfill embankment.

3.3.2.4   Cofferdam

As part of the assessment, a dam breach of the cofferdam has also been modelled. The cofferdam is proposed to be around 12 m high, with a crest width of approximately 30 m. Given the relatively low stored volume, it is not considered realistic that a Sunny Day failure would be capable of producing a rapid breach and sudden release of stored volume. No Sunny Day failure has, therefore been modelled.

The cofferdam comprises a very wide crest width with compacted earthfill. For the purposes of modelling a flood failure, reasonably conservative parameters have been adopted. No sensitivity assessment has been completed since the overall consequences are estimated to be small.

Of the equations trialled, the most conservative outcomes have been adopted

- Average breach width - 20 m (Froehlich, 2008 for piping with 428,000 m^3). Conservatively adopt base width of 20 m.

- Breach time – 0.3 hr (Froehlich, 2008 for height of 12 m)

- Base Elevation – 18 m.

- Side Slopes - 0.5H:1V

3.3.3 Summary

The breach parameters to be applied for the various analyses are summarised in Table 3-2.

Table 3-2 – Breach Parameters

Breach Characteristic	ESS		Cofferdam
	Adopted Value	Sensitivity	Adopted Value
Time for breach development (hour)	3	1	
Breach Side Slope (Y horizontal in 1 vertical)	0.5	NA	0.5
Breach Base Elevation (m AHD)	16	NA	18
Bottom width (m)	13	25 - 69	20

3.4 Sensitivity Analysis

A sensitivity analysis was undertaken to assess the impact of varying base width and development time parameters. In addition, peak outflow was compared against regression equations for peak outflow. The outcomes are detailed in Table 3-3.

Table 3-3 – Breach Parameter Sensitivity (PMF Breach)

Breach Development Time (hr)	Breach Base Width (m)	Peak Discharge (m³/s)		
		Modelled	**Froehlich (2016)**	**Xu & Zang (2009)**
Total 3 hr (top portion 0.3 hr)	13.4	770	2,560	1,360
Total 1 hr (top portion 0.3 hr)	13.4	1,745		
Total 3 hr (top portion 0.3 hr)	25	827		
Total 3 hr (top portion 0.3 hr)	69	1,745		

The data set indicates that the peak outflow is sensitive to both time of failure and breach width. The outcomes are, however, far more sensitive to breach time than width.

The regression equations estimating peak outflows suggest that the peak discharge is a lower bound estimate. However, in reviewing the case studies presented in Froehlich (2016) the modelled flows are consistent with a range of observed failures.

The sensitivity of the consequence outcomes to variability in parameters is considered further below.

3.5 Other Studies

A single previous study has been undertaken investigating the dambreak consequences for ESS and is reported on in DoC (2006). The breach parameters adopted for the two studies are reported upon in Table 3-4.

Table 3-4 – Comparison Breach Parameters

Breach Characteristic	This Study	NSW Dept. Commerce (2006)
Time for breach development (hour)	3	0.45
Breach Side Slope (Y horizontal in 1 vertical)	0.5	1
Breach Height (m)	35	35
Bottom width (m)	13	13
Depth of overtopping required for breach erosion (m)	0	0

The only parameter which varies between the two studies is breach development time.

There is some variability in breach development time between the two studies and it is observed from the sensitivity analysis that peak breach outflows can vary substantially with variability in breach development time. Both DoC (2005) and this study have relied on empirical equations as the basis for selecting parameters, with the approach to utilising the outcomes varying between the studies. DoC (2005) has developed breach parameters applying recommendations described in ANCOLD (1994) and MacDonald and Langridge-Monopolis (1984), while this study has drawn upon the recommendations presented in USBR (2014a). The approach described in ANCOLD (1994) is considered to be dated and is superseded by later approaches. MacDonald and Langridge-Monopolis (1984) includes a database of case studies which is considered to be a useful reference.

The concept of breach development time has been discussed in some detail in USBR (2014a). It is noted in that document that breach time is comprised of an initiation phase and a development phase. In the initiation phase, the downstream face of the embankment is eroded, without activating release of any stored water volume. This initiation phase can vary in duration, and could be minutes or hours depending upon the duration and depth of overtopping. The breach development phase represents the time following the initiation phase when the erosion face progresses upstream of the crest and stored water contributes to the breach development. It is observed that breach time equations appear to represent only the development phase.

Based upon the above, utilising empirical equations to represent breach time is conservative if the breach is modelled as initiating as soon as the water level reaches the dam crest. The approach undertaken in this study is to apply a longer breach time to account for the fact that overtopping is not expected to occur and the embankment is quite substantial relative to the storage size.

4 CONSEQUENCE ASSESSMENTS

4.1 General

The consequences of failure have been assessed for the dams in accordance with ANCOLD (2012). The assessment procedure expresses the consequences of dam failure in terms of the damages and loss to Eurobodalla Shire Council and third parties, the Population At Risk (PAR) and Potential Loss of Life (PLL). The failure consequences depend upon the number and type of properties within the inundated area for the reservoir.

Aerial photographs were used to assess the number of properties and type within the inundated area. The number of properties affected were identified and counted manually using the available aerial imagery.

Residential properties are identified with a point located on each residence as shown on the inundation maps provided in Appendix E. No populated building types other than residential have been identified, although substantial number of farm sheds have been included for cost estimation purposes.

Flood affected properties were identified using a GIS software package (QGIS). Maximum water depth and Maximum DV outputs from the TUFLOW simulation are used as inputs where residential building point locations are overlain on the Tuflow outputs to determine the relevant DV value.

PAR was identified based upon the information available from the most recent census undertaken by the Australian Bureau of Statistics as available at www.abs.gov.au.

PLL was computed using the method described in Graham (1999), but also giving consideration to the approach described in USBR (2014b).

Further detail on the computational procedure is provided below.

4.1.1 Population At Risk (PAR)

According to the ANCOLD Guidelines the definition of PAR includes all people who would be directly exposed to flood waters assuming they took no action to evacuate. The PAR within the floodplain is a function of the number and type of infrastructure located within the floodplain. An exposure factor has been computed for PAR in each infrastructure category to account for the fact that most building types are not populated at the maximum rate at all times. Day, Night, and Weighted (daily average weighted by time – 10 hours of day, 14 hours of night) exposure factors were applied and the worst case is reported on. The basis for estimating PAR in this study is detailed in Table 4-1.

Table 4-1: PAR Rates

Infrastructure	Occupancy Rate	Source	Day Exposure	Night Exposure	Weighted Exposure
Residential dwelling (persons/dwelling)	2.4	ABS Data	50%	100%	79%
Commercial zoned building (persons per m²)	0.1	BCA A06 D1.13*	66%	0%	28%
Industrial zoned building (persons per m²)	0.02	BCA A06 D1.13	66%	0%	6%

* - Building Code of Australia, No. of persons accommodated

4.1.2 Potential Loss of Life (PLL)

PLL is most typically computed using the method described in Graham (1999) which is in accordance with the recommendations presented in ANCOLD (2012). It is recognised, however that the industry is currently giving serious consideration to updating the PLL estimation methodology by applying techniques described in USBR (2014b). The proposed update proposes an approach where the fatality rate is a function of flood severity as estimated from the product of depth and velocity.

The approach may be considered to be consistent with Graham (1999), which proposes that fatality rates vary with flood severity, but in a stepped fashion, rather than as a continuum.

The approach adopted for use in this study utilises a combination of the two approaches. It is proposed that the fatality rate should be a function of the depth and velocity product as defined in USBR (2014b), but also incorporating an extrapolation of the relationship described in that document.

A typical application of Graham (1999) would result in a fatality rate of 1%. Given the low severity flooding which occurs in the inundated area, it is considered that a fatality rate of 1% may overstate the likely consequences.

4.1.2.1 Graham (1999) Methodology

The approach to assigning fatality rates as described in Graham (1999) utilises the following inputs:

- Warning time for people exposed to the life-threatening flood
- Severity of flood event and types of failure scenario used in the evaluation
- Understanding of floods

Fatality rates are presented in Table 4-2.

Table 4-2: Graham (1999) Fatality Rates

Severity	Warning Time	Understanding	Fatality Rate	
			Average	**Range**
HIGH	No warning	N/A	0.75	0.3 to 1.0
	15 – 60	Vague	Use the value above and apply the number of people who remain in the flood plain after the warning is issued	
		Precise		
	> 60	Vague		
		Precise		
MEDIUM	No warning	N/A	0.15	0.03 to 0.35
	15 – 60	Vague	0.04	0.01 to 0.08
		Precise	0.02	0.005 to 0.04
	> 60	Vague	0.03	0.005 to 0.06
		Precise	0.01	0.002 to 0.02
LOW	No warning	N/A	0.01	0 to 0.02
	15 – 60	Vague	0.007	0 to 0.015
		Precise	0.002	0 to 0.004
	> 60	Vague	0.0003	0 to 0.0006
		Precise	0.0002	0 to 0.0004

Graham (1999) indicates that the onset of medium severity conditions may be judged to occur at a Depth and Velocity product (DV) of 4.6 m^2/s. In most cases, community understanding of flooding is

judged to be vague and warning time is most typically set to be zero. Further, most commonly, the average fatality rate is applied to assessments.

4.1.2.2   Alternate PLL Estimation Methodology

ANCOLD (2012) supports both the use of Graham (1999) and the use of alternative methodologies provided they have been derived from a similar or more extensive database than that used to develop the approach described in Graham (1999). The approach described in USBR (2014b) meets these conditions and it proposes varying the fatality rate with DV as a continuous function, rather than the stepped approach described in Graham (1999). The proposed approach does not, however detail the fatality rates associated with the lowest end of the DV range. The process of assigning a fatality rate to each individual property requires that the relationship be extended. In order to undertake the extrapolation, it is proposed that the following approach be applied:

- PAR should not be counted at properties where flood inundation depths are less than 50 mm.

- At DV of 0.4 m^2/s and lower, a fatality rate of 0.0002 should be applied.

- Between a DV of 0.4 m^2/s and 2.8 m^2/s, the fatality rate should be linearly increased up to a value of 0.01.

- Above a DV of 2.8 m^2/s, the fatality rate should be as described in USBR (2014b).

The 50mm flood inundation depth cut-off limit is proposed as a practical minimum, below which any flood impacts are considered to be negligible.

The resulting fatality rates for various methods are presented in Figure 4-1.
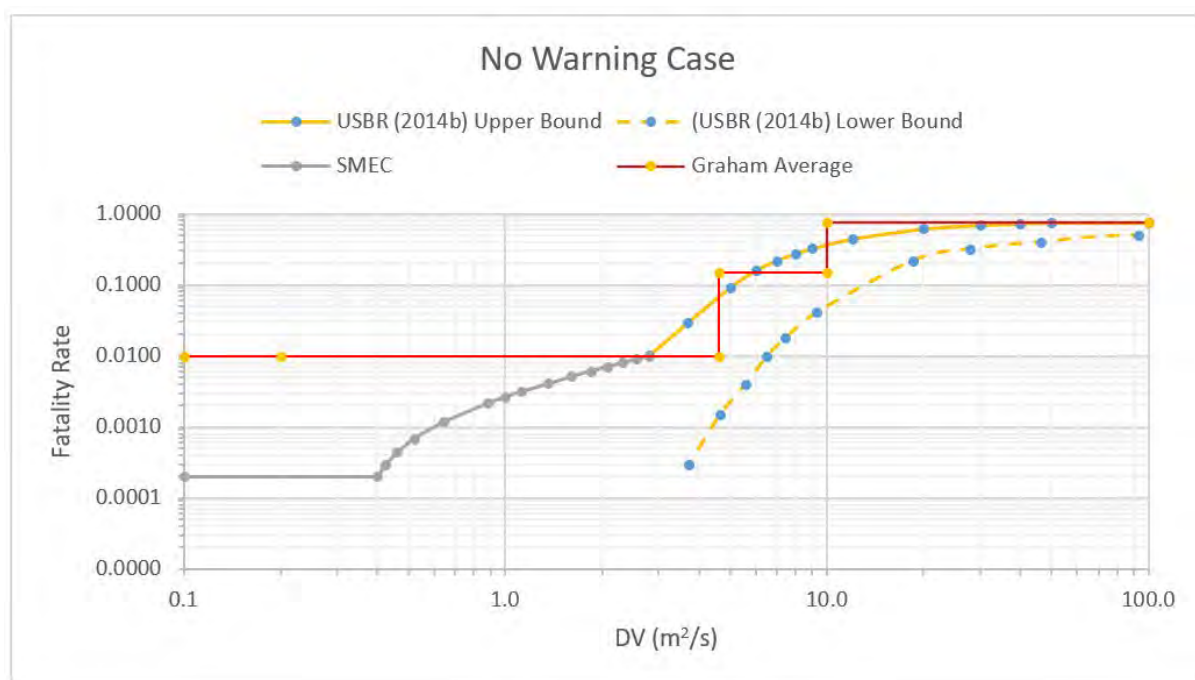

Figure 4-1: Comparison Fatality Rates.

The above figure presents the average Graham (1999) relationship, the upper and lower 'no warning' fatality rate lines as presented in USBR (2014b) and an extrapolation of the upper bound warning line. An extrapolation of the lower bound line would result in an effective fatality rate of zero at a DV of 2 m^2/s.

There is no reference in Graham (1999) which stipulates the lower DV bound for the lower fatality rate. A DV of 0.4 m^2/s has been selected to be consistent with the low hazard flood range described in EA (2010).

For the purposes of estimating the PLL, the DV has been computed from the TUFLOW model. The maximum DV has been recorded and transcribed onto affected properties/buildings via GIS for each of the modelled scenarios. The fatality rate for the property has been assigned based on the DV value for each case.

It is relevant to note that utilising the DV product alone does not account for circumstances where large depths can create hazardous circumstances. This factor has been considered, but it is not considered necessary to adjust the approach for the following reasons:

- In cases with very low DV values and large depths, the rate of rise is likely to be slow enough that substantial portions of the population would have sufficient time to evacuate once they have been alerted to the rising flood waters, even if the alert coincides with the flood waters rising to the property level.

- The base fatality rate is set at 1 in 5,000 which provides allowance for some number of fatalities even at low DV values.

The same procedure has been applied for assessing breach and no breach PLL. ANCOLD (2003) cautions against utilising dam breach fatality rates when assessing loss of life for no breach scenarios. The reasoning behind this limitation is that dam breach flood waves are frequently associated with far more violent conditions than a typical naturally occurring flood. It is considered that incorporating DV into the analysis as a variable adequately allows for the differences in flood severity and, therefore, it is appropriate to utilise the same procedure for both scenarios. It is considered to be of relevance to note that the fatality rate curves described in USBR (2014b) have been derived using a database of observed floods which incorporates both dam break and natural flood events.

In developing and applying a fatality rate line, it may be observed that the upper bound line has been used for all modelled scenarios, where USBR (2014b) provides the flexibility to select from within a range using the case studies as a guide. In reviewing the case studies, specifically the lowest reported DV cases, it is observed that in many cases fatalities are associated with people in cars, frequently when they attempt waterway crossings. This suggests that fatality rates have the potential to be higher where there is a higher concentration of waterway crossings. Given the prevalence of crossings within the inundated area, it is considered appropriate to adopt the higher fatality rates for this study.

4.1.2.3   PLL of Itinerants

The PLL of itinerant road users has been explicitly estimated for the ESS due to the particular circumstances for that storage. Typically, road users are captured in the dambreak fatality rates associated with residential and commercial PAR. In the case of the ESS, however, the residential PAR is very low and it is judged that the PLL associated with road users is not adequately quantified. In particular, the PLL has been estimated at the location where flood flows cross Eurobodalla Road upstream of the Tuross River.

The procedure used to estimate itinerant road user PLL is described in Campbell et.al. (2013). The key factors affecting potential fatalities are flood severity, no. of passengers and road conditions. The PLL is determined as the product of a number of variables which are listed in Table 4-3.

Table 4-3: PLL of Itinerant Road Users Parameters

Variable	Description	Adopted Value
$P_{T:S}$	Probability of a vehicle being exposed to the dambreak hazard Given duration of dambreak event there is small chance (20%) that it occurs in early morning without damage; or all road users see the damaged road afterwards and stop safely.	0.8
$P_{NE:T}$	Probability that a vehicle driver attempts to cross the hazard of any of those that arrive during the hazard The sight distance of the road suggests that all users will see the hazard, however there may be those whom deem it to be low risk and attempt to cross.	0.5
$P_{A:NE}$	Probability of an accident of the hazard is not avoided During the peak of the flooding the depths may be 2-3m and velocities 1-5m/s. Even for cautious speeds (<50km/h) the accident probability is high.	0.8
$V_{D:A}$	Fatality rate of vehicle occupants if an accident occurs The culvert across Eurobodalla Road may be eroded and swept away causing a vehicle ignorant of its loss to fall into the watercourse and swept into Tuross River; or the vehicle may be swept directly into the Tuross River. The vehicle ends up in deep, fast flowing water and the probability of drowning is very high.	0.9
PAR_V	Population at risk within vehicle or vehicle occupancy rate Regional data is used from: https://chartingtransport.com/tag/car-occupancy/	1.55 persons/vehicle
PLL_I	Potential Loss of Life of Itinerant road users Calculated using multiplicative product of other variables in this table.	0.45

The outcome presented above is considered to be an appropriate estimate to apply to both the Sunny day and the PMF dambreak scenarios.

The above PLL has not been applied to scenarios where the Tuross River has flooded and caused Eurobodalla Road to be cut. Likewise, it has not been applied to the PMF scenario without a breach. The PMF without breach is likely to result in flooding of Eurobodalla Road, albeit for a shorter duration lesser severity than the breach case, so adopting this approach is considered to result in a conservative estimate of the incremental consequences for flood failure.

4.1.3   Damages and Loss

The consequence category is computed in part utilising the severity of damage and loss. Guidance on computing this factor is provided in ANCOLD (2012) and standard tables are applied as part of the computation process. The severity is broadly classified in terms of damage costs, business, health and social and environmental impacts.

4.1.3.1   Costs

Infrastructure costs are primarily based upon the number and type of properties which are affected by flood waters. Damages per building type have been judged to vary with DV as documented in Table 4-4. Unit rates for damages (complete destruction of asset) have been assumed for a variety of property types identified within the floodplain as detailed in Table 4-5.

Table 4-4: Damage Cost Unit Rates

DV (m²/s)	Damage Cost as Percentage of Property Damage Cost	Comment
0-0.40	20%	Exterior and water damage
0.41-0.60	40%	Minor structural and water damage
0.61-1.00	60%	Moderate structural damage
1.01-2.00	100%	Major structural damage and floatation – destruction of property
2.01-4.60	100%	Major structural damage and floatation – destruction of property
4.61-15.00	100%	Major structural damage and floatation – destruction of property

Table 4-5: Damage Cost Property Type (for complete destruction of property)

Property Type	Damage Cost per Property
Residential	400,000 AUD / property
Commercial (Rural)	100,000 AUD / property
Community	500,000 AUD / property

NB: Damage cost is only associated with building inundation of 50mm or above

From the above table, Commercial (rural) refers to major outbuildings associated with a farm.

It was observed that there is a small timber bridge across the Tuross downstream of the storage site, which is expected to be impacted in a dambreak event. It is understood that the truss bridge across the Tuross River at the Princes Hwy at the downstream end of the model is at approximately RL7.3m and that it is therefore not expected to be inundated during a sunny day failure event. While it is possible that the smaller timber bridge may be damaged in a natural flood, for the purposes of this study, it has been assumed that it would be destroyed. It is estimated that the wooden bridge may cost in the order of $300,000 to replace, while the Princes Hwy Bridge is a more substantial structure and may cost anywhere up to $5,000/m² or around $10M to replace.

The damage cost of other utilities infrastructure such as roads, power, water or communications were not quantified. The inundation extents were reviewed and no centralised utility infrastructure was observed to be counted separately. Distributed infrastructure would likely be damaged by flood waters, however in aggregate is unlikely to push any scenario up an order of magnitude damage cost.

In addition to property damage, an assumption has been made that repair works would be required for the embankments in the case of failure. It is estimated that the repair costs would be in the order of $30M and the value of the lost water from the reservoir an additional $3M.

4.1.3.2 Business, Health and social and environmental impacts

Impacts on the dam owner's business are estimated to be either medium or major. In particular, the major aspects are related to the effect on the owner's credibility and political implications.

The overall impact of potential dam breach on health, social and environmental aspects is estimated to be minor due to the modest inundation extent associated with the dambreak.

Severity of damage tables summarising the assessed outcomes are included as Appendix C.

4.1.4   Consequence Assignment

Consequence categories can be assigned to the various failure scenarios of the basin. The Consequence Category is determined in accordance with the advice provided in DSC (2010). The Consequence Category associated with a scenario depends upon:

- the PAR or PLL; and
- the severity of damage and loss caused by dam failure.

The criteria for the consequence categories are given in Table 4-6 and Table 4-7. Populations at risk and severity of damage and loss are detailed in the sections above. Based on these estimates the Consequence Categories were assigned to the failure scenarios considered.

Table 4-6 PAR Based Consequence Categories (ANCOLD, 2012)

Incremental PAR	Severity of Damage and Loss			
	Minor	**Medium**	**Major**	**Catastrophic**
<1	Very Low	Low	Significant	High C
>1 to <10	Significant Notes 2	Significant Notes 2	High C	High B
>10 to <100	High C	High C	High B	High A
>100 to <1,000	Note 1	High B	High C	Extreme
> 1000		Note 1	Extreme	Extreme

Note 1:	With a PAR in access of 100, it is unlikely damage will be minor. Similarly with a PAR in excess of 1,000 it is unlikely damage will be classified as medium.
Note 2:	Change to High C where there is the potential of one or more lives being lost.

Table 4-7 PLL Based Consequence Categories (ANCOLD, 2012)

Incremental PLL	Severity of Damage and Loss			
	Minor	**Medium**	**Major**	**Catastrophic**
<0.1	Very Low	Low	Significant	High C
>0.1to <1	Significant	Significant	High C	High B
>1 to <5	Note 1	High C	High B	High A
>5 to <50		High A (Note 2)	High A	Extreme
> 50		Note 1	Extreme	Extreme

Note 1:	With an incremental PLL equal to or greater than (1) , it is unlikely damage will be minor. Similarly with an incremental PLL in excess of 50 it is unlikely damage will be classified as medium.
Note 2:	Where PLL is in the range of ≥5 to <10, the Category level can be reduced to High B.

4.2 Modelled Outcomes

4.2.1 Modelled Results

The outcomes of the consequences assessment undertaken in accordance with the above listed procedures is detailed in Table 4-8. Results using both Graham (1999) and the alternative approach are detailed.

Table 4-8 Incremental PAR, PLL and Cost Consequences.

Consequence	Modelled Scenario		
	PMF	**Sunny Day**	**Cofferdam (DCF)**
Breach PAR	55	0.0	55
No Breach PAR	55	0.0	55
PAR Incremental	**0.0**	**0.0**	**0.0**
Breach PLL*	1.00 (0.45)	0.45 (0.45)	1.00 (0.65)
No Breach PLL*	0.46 (0.00)	0.00 (0.00)	0.46 (0.56)
PLL Incremental*	**0.54 (0.45)**	**0.45 (0.45)**	**0.54 (0.08)**
Breach Damage Cost	$49.9 M	$33.3M	$17.8 M
No Breach Damage Cost	$16.9 M	$0.0 M	$16.6 M
Damage Cost Incremental	**$33.0 M**	**$33.3 M**	**$1.2 M**

* - Number is brackets represent PLL computed using the alternative methodology

The outcomes presented above represent the most severe case, where day, night, and weighted PAR and PLL have been modelled separately. For the PMF the most severe case is a different Tuross River coincident flooding scenario for the two PLL estimation techniques (see Section 4.2.2 for more discussion). More complete details of the assessment are included as Appendix C.

The use of a consistent relationship between DV and PLL in the alternative methodology represents a departure from the approach recommended in ANCOLD (2003) where different fatality rates are suggested for breach and no breach cases. It is suggested that in the case where the rate of rise of floodwater is similar in breach and no breach cases, there is no strong justification for varying the fatality rate. The rate of rise in the inundation zone was reviewed at a number of cross section locations. The absolute times of rise are small in all cases and as a result, the rate of rise is not expected to impact upon the fatality rate between different scenarios.

The location of inundated infrastructure is detailed on the inundation maps included as Appendix E. The consequence category is presented in Table 4-9.

Table 4-9 Dam Consequence Category

Event	Severity	PLL*	Consequence Category
PMF (Graham, 1999)	Major	0.54	High C
PMF (alternative)	Major	0.45	High C
Sunny Day (Graham, 1999)	Major	0.45	High C
Sunny Day (alternative)	Major	0.45	High C
Cofferdam (DCF) (Graham, 1999)	Minor	0.54	Significant
Cofferdam (DCF) (alternative)	Minor	0.08	Very Low

* - Incremental

The consequence category for the main dam is not sensitive to the PLL estimation methodology. The cofferdam consequence category is sensitive to the PLL computation methodology.

The outcomes from the alternative analysis allows for a more detailed consideration of the PLL, taking into account the severity of flood waters in the vicinity of individual residences. The results using the alternative PLL estimation methodology return a lower PLL than the Graham (1999) methodology. The reason for the difference is considered to be predominately due to the difference in fatality rate for the breach and no breach scenarios. When applying Graham (1999), there is a substantial difference in fatality rates, while the alternative method suggests that there is very little difference in fatality rates. The number of properties in different DV zones is documented in Appendix F.

In the case of Eurobodalla Southern Storage Cofferdam, the modelled PLL outcomes provide justification for applying either a Significant or a Very Low Consequence Category. It is recommended that the PLL computed using the alternative methodology be adopted for the following reasons:

- The DV histograms documented in Appendix F indicate that there is little difference in flooding conditions in a dambreak and no dambreak event.
- The modelling of outcomes with different coincident flows indicates that the majority of consequences where they occur are due to natural or pre-breach flooded conditions
- Adopting the more conservative Graham (1999) methodology results in a more conservative consequence category. Given the similarity in pre-and post dambreak flooded conditions, the more conservative category does not appear to be justified.

The adopted consequence categories for the various scenarios are presented in Table 4-10.

Table 4-10 Adopted Dam Consequence Category

Event	Consequence Category
PMF	High C
Sunny Day	High C
Cofferdam (DCF)	Very Low

4.2.2   Sensitivity

A sensitivity analysis has been undertaken which considers the variability in outcomes as breach parameters are varied. The impact of parameters on peak outflows has been considered above. It was found that the peak outflows are relatively sensitive to breach width, but are far more sensitive to the time of breach development. Now, the analysis considers the impact of consequences as detailed in Table 4-11.

Table 4-11 PMF Dam Breach Parameter Sensitivity Analysis

Event	Breach Width (m)	Breach Time (hr)	Severity	PLL[1]
PMF - adopted parameters[2]	13.4	3 (top portion 0.3)	Major	0.001
PMF - Breach Time Check	13.4	1 (top portion 0.3)	Major	0.009
PMF - Breach Width Check	25	3 (top portion 0.3)	Major	0.005
PMF - Breach Width Check	69	3 (top portion 0.3)	Major	0.012

1- Incremental

2 - For comparison of dam breach parameter sensitivity the 1 in 100 AEP Tuross River level is used (critical case when ignoring itinerants).

The figures presented in the above table are for the alternative PLL computation methodology, since the PLL computed using the Graham (1999) method is invariant. The results in Table 4-11 are all for the condition where there is a 1 in 100 AEP flood event occurring in the Tuross River (as the more critical case when ignoring the impact of itinerants).

The outcomes indicate that the consequences are not sensitive to breach width or to the selected breach time.

In addition to the above the impact of coincident flooding in the Tuross has been reviewed as discussed in Section 2.8.1 and the impact of embankment failure during a 1 in 100 AEP flood event has been reviewed.

It is determined that embankment failure during a 1 in 100 AEP flood event the PAR is 2 and the PLL is 0.01 (excluding itinerants).

It is determined that the PMF consequences are sensitive to the coincident flood event in the Tuross River.

Table 4-10 lists the case where the Tuross River level is at the 1 in 100 AEP flood event.

In the case where the Tuross River level is at mean spring tide level the PAR is 0 and the PLL is 0.45 for both PLL estimation methods. The PLL of 0.45 comes from the itinerant roads user of Eurobodalla Rd whom are not at risk in the PMF without dambreak event when the Tuross River level is at mean spring tide level. For this case, the consequence category would be High C in a flood breach case.

Both water levels in the Tuross River are detailed in Appendix C.

4.2.3   Other studies

A single other study has assessed dambreak consequences, being NSW Dept. Commerce (2006). The key variable controlling the risk of the dam is either the PAR or the PLL. The estimated values for the two studies are presented as Table 4-12.

Table 4-12 Comparative PLL and Consequence Categories

Event	Analysis	Incremental PAR	Incremental PLL	Consequence Category
DCF/PMF	This Study	0	0.45	High C
	DoC (2006)	0	0	High C
Sunny Day	This Study	0	0.45	High C
	DoC (2006)	0	0	High C

The figures presented above from DoC (2006) nominate a zero figure for PAR and PLL and a severity of medium. DoC (2006) adopts High C which matches with the severity and PAR only if it is considered that there is reasonable potential for a loss of life. DoC (2006) does not discuss reasons why High C has been adopted and it is judged likely it has been selected on the basis that the travel time of the floodwave is relatively short and presumably has the potential for loss of life from itinerants.

The results from the above table indicate that the assessed PAR from this study is the same as from the previous study. The incremental PLL is assessed to come from itinerants along Eurobodalla Road. The consequence category from this study is stated to be the same as that presented in DoC (2006) i.e. High C for both flood and sunny day scenarios.

5   REFERENCES

Allen, P.H. (1994), *Dam Breach Mechanisms*, ANCOLD Bulletin No. 97, August 1994.

Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M and Testoni I, (Editors), 2016, *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Commonwealth of Australia

Bureau of Meteorology (2003), *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method*, June 2003.

Bureau of Meteorology (2006), *Guide to the Estimation of Probable Maximum Precipitation: Generalised Southeast Australia Method*, October

Campbell J, Barker G, Southcott P, and Wallis M, (2013) *Flooded cars: estimating the consequences to itinerants exposed to dambreak floods on roads.* Proceedings of the joint NZSLD and ANCOLD Conference, Rotorua, New Zealand, October.

DSC (2010) *Consequence Categories for Dams*, DSC3A, Updated May 2014.

Engineers Australia (2010) *Australian Rainfall and Runoff Revision project 10: Appropriate Safety Criteria for People*, Stage 1 Report, April 2010.

Froehlich, D.C. (2008) *Embankment Dam Breach Parameters and Their Uncertainties*. ASCE, Journal of Hydraulic Engineering, 134(12):1708-1721, Dec 2008.

Froehlich, D.C. (2016) *Predicting Peak Discharge from Gradually Breached Embankment Dam*. ASCE, Journal of Hydrologic Engineering, vol.21, Issue 11, Nov 2016.

Ghodsian, M. (2004), *Stage Discharge Relationship for Triangular Weir*, International Journal of Civil Engineering, Vol. 2, No. 1, March 2004

Graham, W.J. (1999), *A Procedure for Estimating Loss of Life Caused by Dam Failure*, DSO-99-06, USBR, September 1999.

Institute of Engineers Australia (1987) *Australian rainfall and Runoff, A Guide to Flood Estimation*.

IEAust (1999). *Australian Rainfall and Runoff. Book 6: Estimation of Large and Extreme Floods*. Institution of Engineers, Australia

Jordan, P., Nathan, R., Mittiga, L. and Taylor, B. (2005) *Growth Curves and Temporal Patterns of Short Duration Design Storms for Extreme Events*, Aust. J. Wat. Res. Vol 9, N0.1, 2005.

MacDonald, T.C. and Langridge-Monopolis, J. (1984) *Breaching Characteristics of Dam Failures*, ASCE Journal, Hydraulics Division, vol. 110, no.5, May 1984.

Northrop (2016), *Tyrone Bridge Replacement Hydrology & Hydraulic Assessment Report*, Project Ref 166016, Report to NSW Public Works Department, September 2016.

McDermott, G.E. and Pilgrim, D.H. (1982) *Design Flood Estimation for Small Catchments in New South Wales*, Australian Water Resources Council, Technical Paper no. 73.

NSW Department of Commerce (2005), *Stony Creek Site No.2 Dam, Flood Hydrology Study - Draft*, Report No. 05095, Report to Eurobodalla Shire Council, December 2005.

NSW Department of Commerce (2006), *Stony Creek Dam Dambreak Study - Draft, Report No. DC06022*, Report to Eurobodalla Shire Council, March 2006.

Tracy, H.J. (1957) *Discharge Characteristics of Broad Crested Weirs*, Geological Survey Circular 397, US Department of the Interior.

USACE (2007) *Risk Assessment for Dam Safety*, Dam Failure Analysis Toolbox, September 2007.

USBR (2014a) *Evaluation of Erodibility-Based Embankment Dam Breach Equations*, Report No. HL-2014-02, June 2014.

U.S. Bureau of Reclamation (USBR) (2014b) *RCEM – Reclamation Consequence Estimating Methodology*, Dam Failure and Flood Event Case History Compilation, Interim DRAFT, US Department of the Interior, Bureau of Reclamation, Jun 2014.

WMO (2009) *Manual on Estimation of Probable Maximum Precipitation Precipitation (PMP)*, Publication No. WMO No. 1045.

Xu, Y. and L.M. Zhang (2009) *Breaching Parameters for Earth and Rockfill Dams*. Journal of Geotechnical and Geoenvironmental Engineering, 135(12):1957-1970.
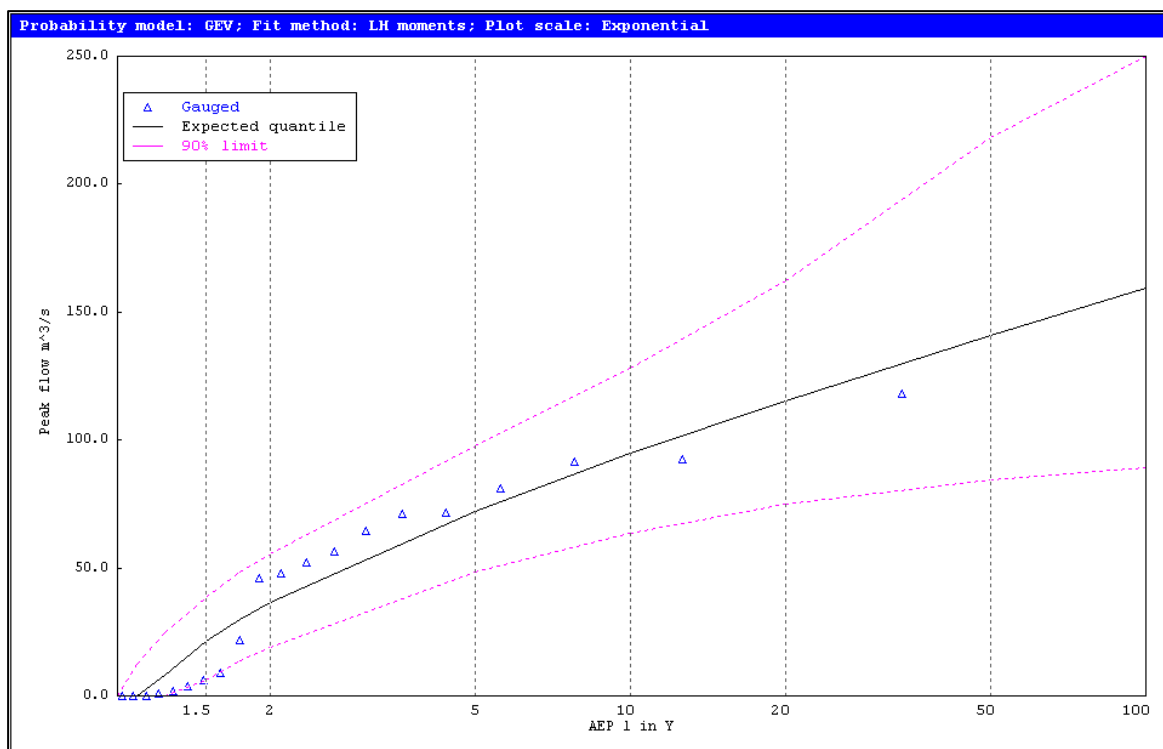
Appendix A HISTORICAL FLOOD FREQUENCY CURVES

Figure A-1: #217006 - Araluen @ Neringla Flood Frequency Curve

Title: 217006 - Araluen at Neringla
Nominated L moment shift = 0

L moment	Value
1	41.817
2	21.937
3	2.938
4	-1.560

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	24.363	24.596	8.518	1.000		
a	33.212	32.898	6.507	0.313	1.000	
k	0.055	0.072	0.183	0.384	0.403	1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-28.6	-59.8	-3.4	1.53
1.10	-5.4	-21.9	12.4	0.87
1.25	8.3	- 6.2	24.7	0.48
1.50	21.2	6.1	38.6	0.09
1.75	29.8	13.7	48.2	-0.17
2.00	36.4	19.2	55.7	-0.37
5.00	72.2	48.4	97.5	-1.50
10.00	94.7	63.6	127.9	-2.25
20.00	115.4	74.8	162.3	-2.97
50.00	141.0	84.5	218.3	-3.90
100.00	159.4	89.3	270.8	-4.60

```
Probability model: GEV; Fit method: LH moments; Plot scale: Exponential
```
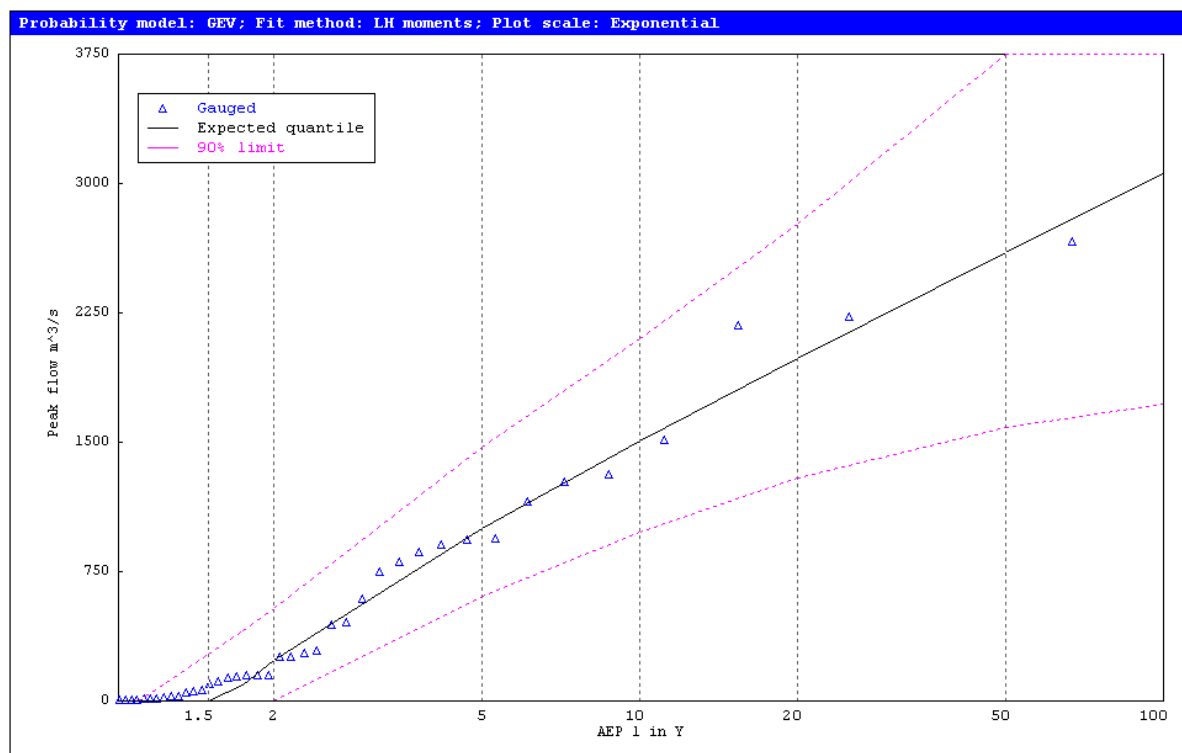
Figure A-2: #218008 - Tuross Rvr at Eurobodalla Flood Frequency Curve

Title: 218008 - Tuross Rvr at Eurobodalla
Optimized L moment shift = 4

L moment	Value
1	0.000
2	0.000
3	0.000
4	0.000

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	-17.714	-38.611	184.044	1.000		
a	686.135	747.018	281.382	-0.558	1.000	
k	0.012	0.050	0.220	-0.382	0.817	1.000

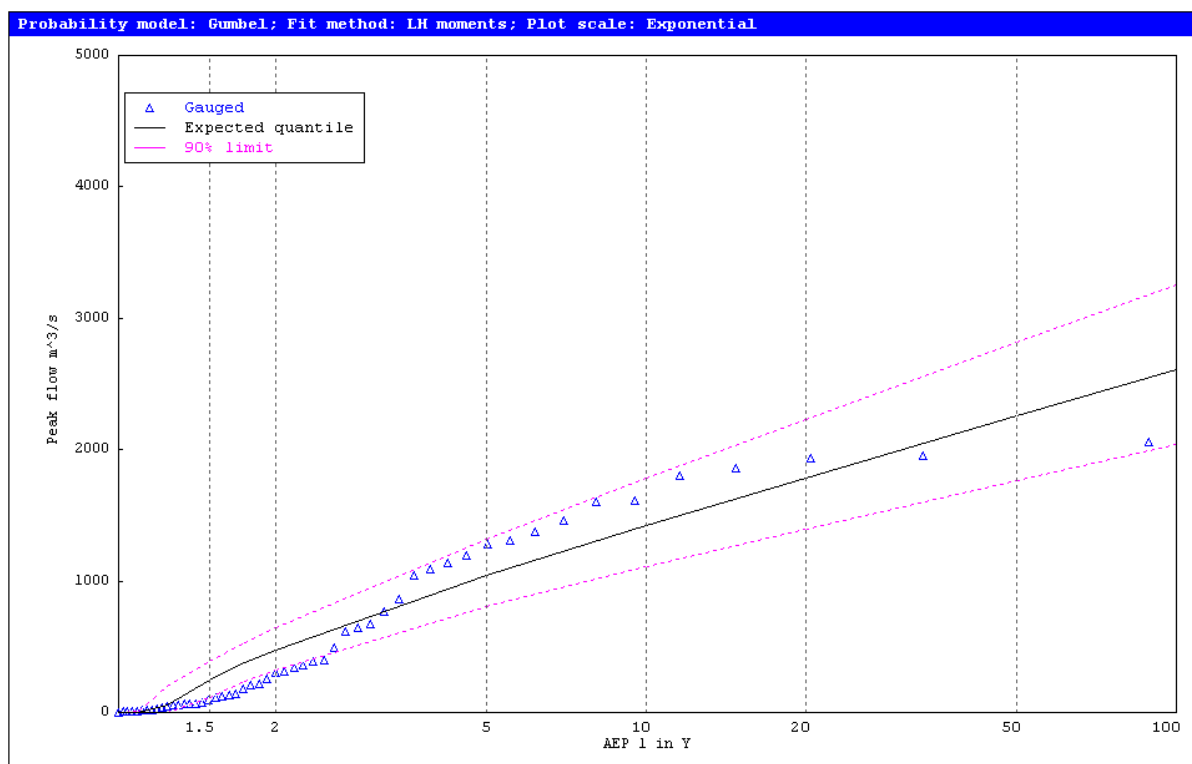| AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-1076.4	-3341.7	-196.0	1.53
1.10	-620.9	-1776.4	-24.1	0.87
1.25	-345.1	-1059.7	107.7	0.48
1.50	-82.3	-515.6	271.0	0.09
1.75	95.9	-231.1	410.3	-0.17
2.00	233.2	-59.5	539.1	-0.37
5.00	1002.5	605.3	1468.0	-1.50
10.00	1506.3	980.8	2102.4	-2.25
20.00	1985.5	1294.7	2762.4	-2.97
50.00	2599.8	1584.0	3827.5	-3.90
100.00	3055.7	1723.8	4897.4	-4.60

Figure A-3: #218005 - D/S Wadbilliga Jn Flood Frequency Curve

Title: 218005 D/S Wadbilliga Jn
Nominated L moment shift = 0

L moment	Value
1	584.064
2	348.224
3	116.097
4	8.114

Gumbel Fit Results

Parameter	LH	Mean	Std dev	Correlation	
tau	294.090	293.224	72.944	1.000	
a	502.382	502.364	62.409	0.231	1.000

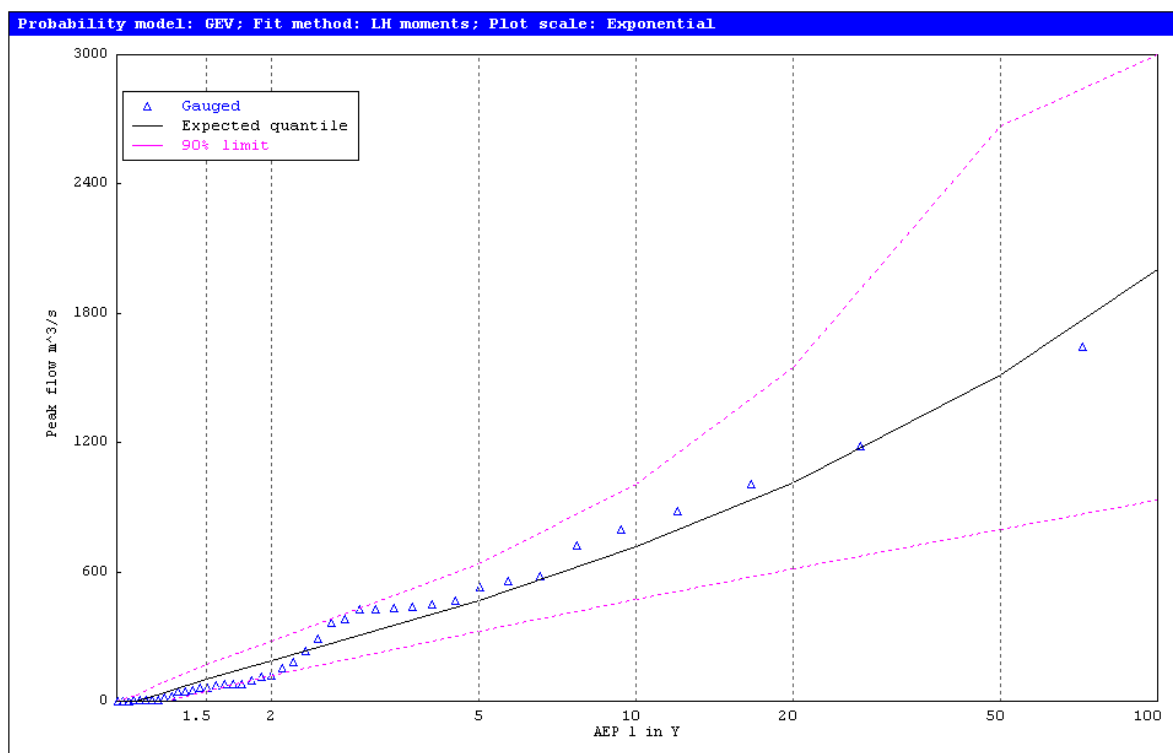| AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-474.2	-685.9	-272.0	1.53
1.10	-145.3	-300.5	13.7	0.87
1.25	55.0	-82.2	199.9	0.48
1.50	246.8	110.3	392.5	0.09
1.75	377.3	232.5	531.7	-0.17
2.00	478.2	323.7	643.4	-0.37
5.00	1047.6	802.7	1316.7	-1.50
10.00	1424.6	1107.4	1779.8	-2.25
20.00	1786.3	1395.3	2231.4	-2.97
50.00	2254.4	1764.2	2817.4	-3.90
100.00	2605.1	2040.7	3256.9	-4.60

Figure A-4: #219013 - Brogo @ North Brogo Flood Frequency Curve

Title: 219013 Brogo at North Brogo
Nominated L moment shift = 0

L moment	Value
1	305.365
2	186.499
3	72.218
4	26.082

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	119.021	121.016	32.681	1.000		
a	182.120	182.428	31.252	0.652	1.000	
k	-0.315	-0.281	0.147	0.342	0.246	1.000

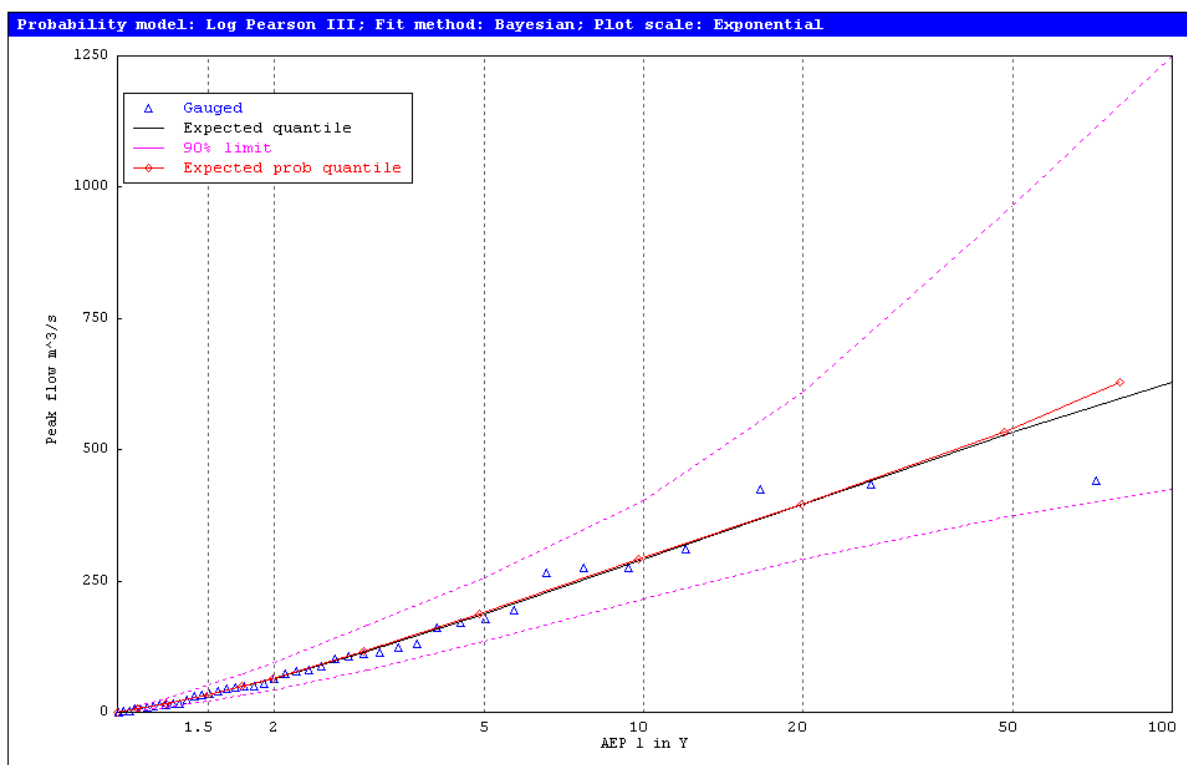| AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-102.0	-184.5	-37.3	1.53
1.10	-20.2	-66.3	29.4	0.87
1.25	38.5	-6.0	92.1	0.48
1.50	102.1	48.3	169.3	0.09
1.75	150.0	87.0	229.0	-0.17
2.00	189.8	118.0	278.3	-0.37
5.00	468.1	322.9	638.6	-1.50
10.00	715.3	471.7	1006.7	-2.25
20.00	1014.1	614.5	1546.3	-2.97
50.00	1516.4	797.2	2671.3	-3.90
100.00	2002.0	933.0	4031.3	-4.60

Figure A-5: #218007 - Wadbilliga Flood Frequency Curve

Title: 218007 Wadbilliga
Flood model: Log Pearson III

Zero flow threshold: 0.000
 Number of gauged flows at or below flow threshold =   0

 Summary of Posterior Moments from Importance Sampling
 Parameter Name   Mean   Std dev   Correlation
    1 Mean (loge flow)   3.94059   0.23440   1.000
    2 loge [Std dev (loge flow)   0.41472   0.13873   -0.547 1.000
    3 Skew (loge flow)   -0.90909   0.38857   0.021 -0.478 1.000
 Note: Posterior expected parameters are the most accurate in the mean-squared-error sense.
    They should be used in preference to the most probable parameters
 Upper bound =   1438.38

| AEP 1 in Y	Exp parameter quantile	Monte Carlo 90% quantile probability limits		Mean(log10(q))	Stdev(log10(q))
1.010	0.57	0.08	2.2	-0.2668	0.4577
1.100	6.06	2.48	11.9	0.7775	0.2125
1.250	16.08	8.91	26.9	1.2069	0.1482
1.500	33.06	20.79	51.4	1.5219	0.1198
1.750	49.31	32.44	74.4	1.6959	0.1098
2.000	64.52	43.34	95.5	1.8127	0.1044
3.000	115.70	80.99	165.3	2.0654	0.0937
5.000	187.46	135.27	257.0	2.2737	0.0848
10.000	291.38	216.04	403.3	2.4640	0.0836
20.000	397.03	291.87	608.5	2.5982	0.0968
50.000	532.52	375.91	966.4	2.7273	0.1287
100.000	628.80	423.56	1311.0	2.8019	0.1574
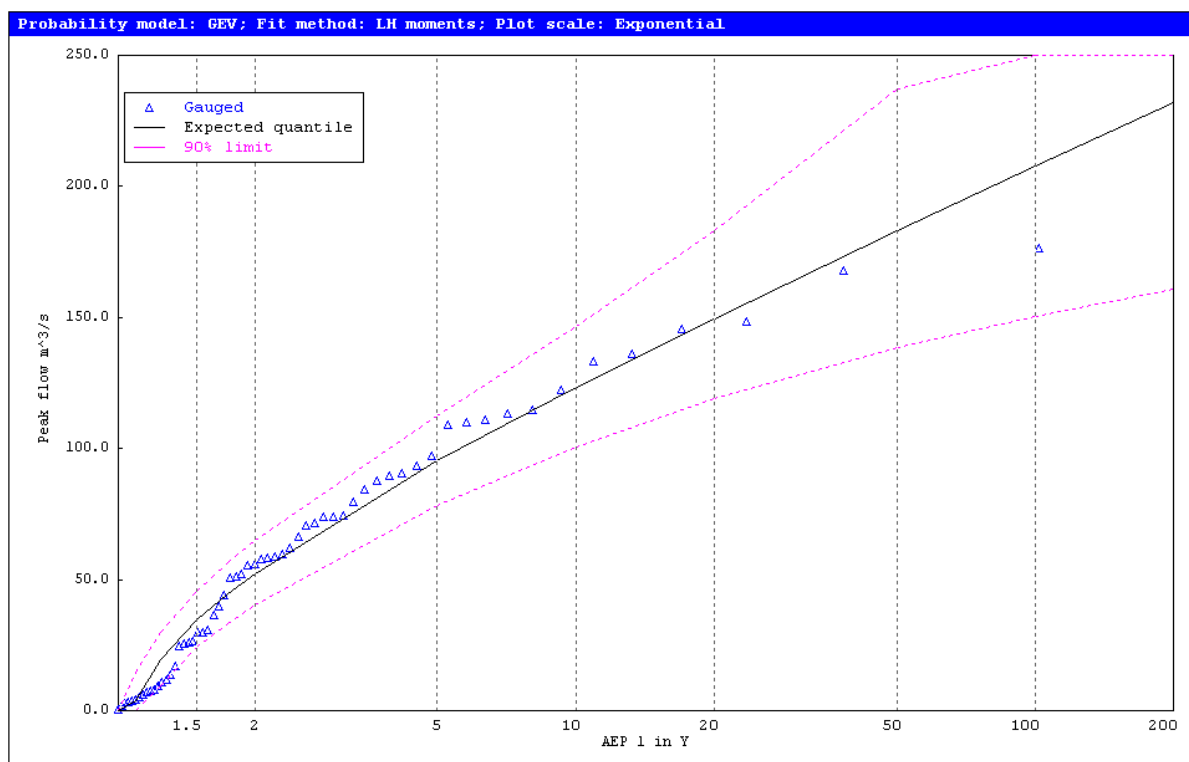
Figure A-6: #218001 - Tuross at Tuross Vale Flood Frequency Curve

Title: 218001 Tuross at Tuross Vale
Nominated L moment shift = 0

L moment	Value
1	59.575
2	26.357
3	4.073
4	1.280

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	38.009	38.016	5.616	1.000		
a	38.784	38.593	4.269	0.351	1.000	
k	0.022	0.025	0.098	0.365	0.379	1.000

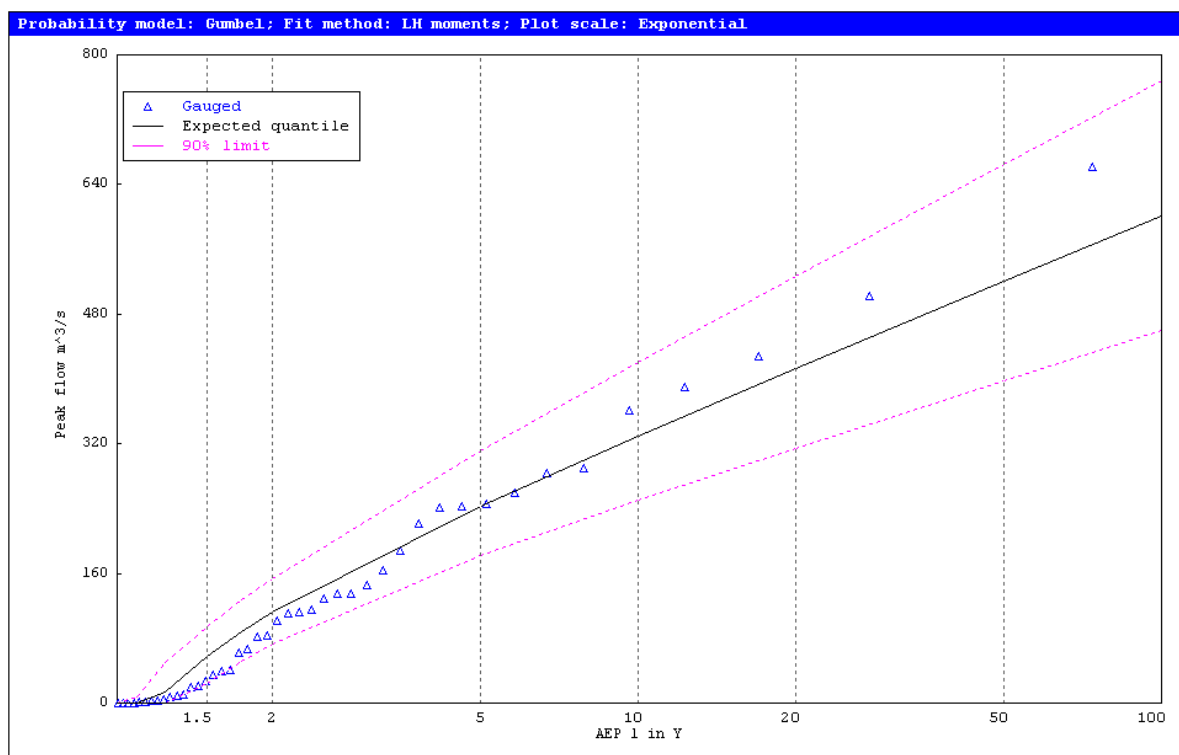| AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-22.3	-40.1	-6.0	1.53
1.10	3.8	-6.9	14.9	0.87
1.25	19.5	9.7	29.9	0.48
1.50	34.4	24.0	45.5	0.09
1.75	44.4	33.3	56.3	-0.17
2.00	52.2	40.3	64.8	-0.37
5.00	95.2	78.4	112.6	-1.50
10.00	123.2	100.8	146.4	-2.25
20.00	149.6	119.2	182.9	-2.97
50.00	183.1	138.5	237.4	-3.90
100.00	207.8	150.6	284.7	-4.60

Figure A-7: #219016 - Narira Rv @ Cobargo Flood Frequency Curve

Title: 219016 Narira Rv at Cobargo
Nominated L moment shift = 0

L moment	Value
1	135.974
2	80.253
3	27.784
4	9.585

Gumbel Fit Results

Parameter	LH	Mean	Std dev	Correlation	
tau	69.146	68.893	18.473	1.000	
a	115.780	115.827	15.781	0.231	1.000

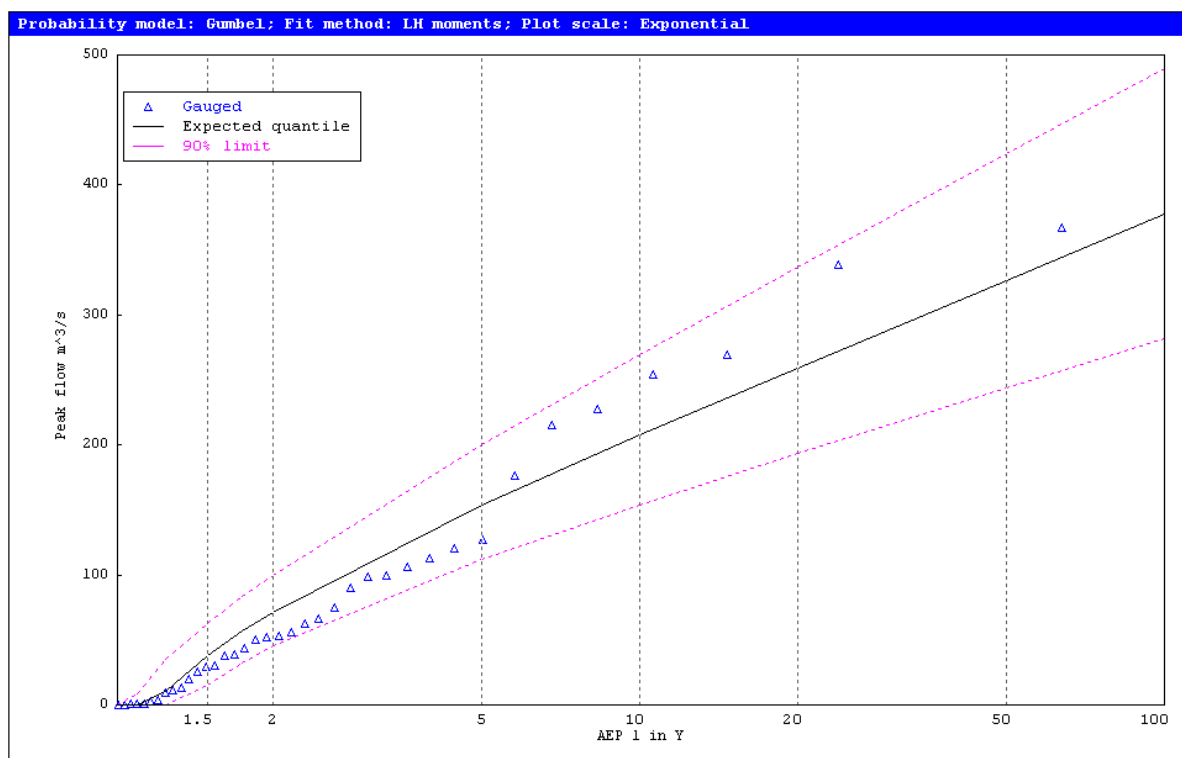| AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-107.9	-161.8	-56.4	1.53
1.10	-32.1	-71.7	8.1	0.87
1.25	14.0	-20.8	50.7	0.48
1.50	58.3	23.7	94.8	0.09
1.75	88.3	51.8	127.0	-0.17
2.00	111.6	72.6	153.1	-0.37
5.00	242.8	181.5	311.5	-1.50
10.00	329.7	250.2	421.1	-2.25
20.00	413.0	314.7	527.0	-2.97
50.00	520.9	397.9	664.8	-3.90
100.00	601.8	459.4	768.2	-4.60

Figure A-8: #219018 - Murrah Rv @ Quaama Flood Frequency Curve

Title: 219018 Murrah Rv at Quaama
Nominated L moment shift = 0

L moment	Value
1	86.607
2	50.062
3	18.785
4	8.280

Gumbel Fit Results

Parameter	LH	Mean	Std dev	Correlation	
tau	44.919	44.786	12.404	1.000	
a	72.225	72.292	10.631	0.228	1.000

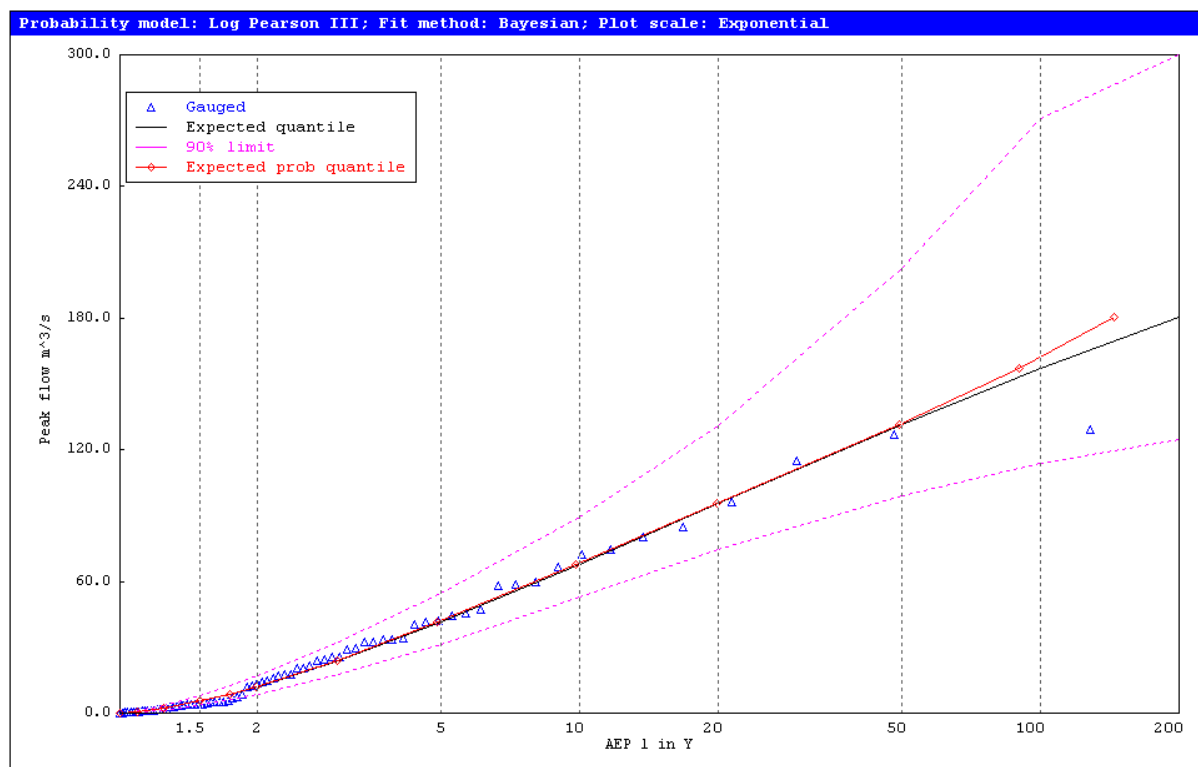| AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-65.5	-101.7	-31.2	1.53
1.10	-18.2	-44.9	8.9	0.87
1.25	10.5	-12.8	35.3	0.48
1.50	38.1	15.0	62.9	0.09
1.75	56.9	32.5	83.2	-0.17
2.00	71.4	45.4	99.7	-0.37
5.00	153.3	112.3	199.9	-1.50
10.00	207.5	154.1	269.0	-2.25
20.00	259.4	193.5	336.5	-2.97
50.00	326.7	244.1	423.8	-3.90
100.00	377.2	281.6	489.3	-4.60

Figure A-9: #219001 - Rutherford Brown Mtn Flood Frequency Curve

Title: 219001 Rutherford Brown Mtn
Flood model: Log Pearson III

Zero flow threshold: 0.000
 Number of gauged flows at or below flow threshold =   0

 Summary of Posterior Moments from Importance Sampling

Parameter Name	Mean	Std dev	Correlation
1 Mean (loge flow)	2.20354	0.20129	1.000
2 loge [Std dev (loge flow)	0.57757	0.10589	-0.531 1.000
3 Skew (loge flow)	-0.98112	0.28771	0.016 -0.561 1.000

Note: Posterior expected parameters are the most accurate in the mean-squared-error sense.
    They should be used in preference to the most probable parameters
Upper bound = 342.243

AEP 1 in Y	Exp parameter quantile	Monte Carlo 90% quantile probability limits		Mean(log10(q))	Stdev(log10(q))
1.010	0.04	0.01	0.2	-1.3959	0.3995
1.100	0.73	0.34	1.4	-0.1419	0.1831
1.250	2.34	1.40	3.7	0.3695	0.1267
1.500	5.50	3.71	8.0	0.7422	0.1026
1.750	8.81	6.19	12.5	0.9470	0.0946
2.000	12.07	8.65	16.9	1.0838	0.0905
3.000	23.78	17.54	32.5	1.3778	0.0823
5.000	41.34	31.33	54.6	1.6170	0.0745
10.000	67.99	52.77	89.5	1.8319	0.0706
20.000	95.77	74.32	131.2	1.9804	0.0779
50.000	131.73	98.98	202.5	2.1196	0.1008
100.000	157.19	113.67	271.1	2.1978	0.1227

Appendix B   DESIGN RAINFALL DEPTHS

Table B.1: Rainfall Depths for Eurobodalla Storage (1 in 2 AEP (50%) to 1 in 2,000 AEP (0.05%))

Areal rainfall depth (mm)

Duration		Annual Exceedance Probability (1 in X)									
	(mins)	2	5	10	20	50	100	200	500	1,000	2,000
0.25	15	12.51	17.57	21.22	24.84	30.0	34.1	39.0	45.8	51.4	57.5
0.33	20	14.46	20.33	24.48	28.70	34.5	39.2	44.9	52.7	59.2	66.2
0.42	25	16.01	22.49	27.14	31.86	38.3	43.5	49.8	58.4	65.6	73.3
0.5	30	17.36	24.44	29.49	34.51	41.5	47.0	54.0	63.3	71.1	79.5
0.75	45	20.67	29.15	35.19	41.19	49.3	55.5	64.0	75.1	84.2	94.1
1.0	60	23.35	33.03	39.85	46.52	55.4	62.4	72.0	84.4	94.6	105.7
1.5	90	27.96	39.61	47.58	55.47	65.7	73.6	85.2	99.7	111.7	124.7
2.0	120	31.94	45.24	54.36	63.17	74.6	83.3	96.6	113.0	126.6	141.2
3.0	180	39.05	55.29	66.21	76.69	90.4	100.9	117.0	136.7	152.9	170.5
6.0	360	56.48	80.33	96.35	112.0	133.1	149.3	173.2	203.3	228.2	255.2
9.0	540	70.50	100.8	121.3	141.5	169.5	191.3	221.2	260.2	321.8	360.4
12	720	82.40	117.9	142.5	166.7	200.8	227.9	262.2	308.3	381.3	427.1
24	1,440	115.4	167.1	204.3	242.5	298.2	344.0	390.6	460.4	570.4	639.9
36	2,160	136.3	199.8	246.8	295.8	367.9	427.9	482.0	568.0	703.7	789.5
48	2,880	150.2	222.5	276.9	334.5	418.9	489.8	548.8	646.8	801.3	898.9
72	4,320	167.0	250.8	315.4	384.9	485.3	570.0	635.7	749.2	928.2	1041.3

Table B.2: Rainfall Depths for Eurobodalla Storage (1 in 5,000 AEP to 1 in 10,000,000 AEP (PMP))

Areal rainfall depth (mm)

Duration		Annual Exceedance Probability (1 in X)									
(hrs)	(mins)	5,000	10,000	20,000	50,000	100,000	200,000	500,000	1,000,000	2,000,000	5,000,000	10,000,000
0.25	15	67.7	75.9	84.4	96.1	105.0	113.9	125.3	133.5	141.2	150.2	156.0
0.33	20	77.9	87.3	97.1	110.5	120.7	131.0	144.2	153.7	162.5	173.0	179.8
0.42	25	86.2	96.7	107.6	122.7	134.3	145.9	161.0	172.1	182.6	195.2	203.7
0.5	30	93.5	104.9	116.8	133.4	146.3	159.4	176.6	189.4	201.7	217.0	227.5
0.75	45	110.7	124.3	138.7	159.0	175.1	191.7	214.1	231.2	248.1	270.1	286.0
1.0	60	124.2	139.6	155.9	179.1	197.6	216.8	243.2	263.6	284.2	311.3	331.5
1.5	90	146.5	164.8	184.5	212.9	236.1	260.8	295.5	323.2	352.1	391.7	422.5
2.0	120	165.8	186.5	209.0	241.6	268.5	297.3	338.3	371.5	406.5	455.3	494.0
3.0	180	200.0	224.8	251.8	291.1	323.5	358.3	407.9	448.2	490.8	550.5	598.0
6.0	360	300.1	337.2	376.7	432.6	477.3	523.8	587.4	636.5	686.1	751.2	799.5
9.0	540	424.5	475.7	528.4	599.1	652.1	703.6	767.6	811.6	850.6	893.0	917.1
12	720	502.7	562.6	623.7	704.6	764.3	821.4	890.6	936.5	975.6	1015.1	1034.8
24	1,440	752.8	839.2	924.1	1030.2	1102.8	1166.0	1231.9	1266.1	1285.4	1287.0	1270.0
36	2,160	927.4	1031.3	1131.4	1252.5	1331.7	1396.8	1457.4	1481.5	1485.8	1461.0	1420.0
48	2,880	1054.9	1170.8	1280.8	1410.4	1491.9	1555.4	1607.3	1620.3	1610.0	1561.3	1500.0
72	4,320	1220.3	1351.0	1472.1	1609.7	1690.9	1748.3	1783.5	1777.7	1744.1	1660.0	1570.0

Appendix C   PAR AND PLL COMPUTATION DETAILS

Table C.1: No Breach PAR, PLL, and Damage Cost Computation Details using the Graham 1999 Methodology

		Eurobodalla Southern Storage – Graham 1999 Methodology				
		Flood Events (Without Failure)				
		Probable Maximum Flood			**1 in 100 AEP**	
	Case	Existing No Embankment Tuross = 1in100 AEP	No Breach Tuross = 1in100 AEP	No Breach Tuross = 1in1 AEP	No Breach Tuross = 1in10 AEP	Cofferdam No Breach Tuross = 1in100 AEP
		Life Safety				
Population At Risk - Day	persons	27.6	27.6	0	18.0	27.6
Population At Risk - Night	persons	55.2	55.2	0	36.0	55.2
Population At Risk - Weighted	persons	43.7	43.7	0	28.5	43.7
Potential Loss of Life (PLL) - Day	persons	0.46	0.46	0.00	0.45	0.46
Potential Loss of Life (PLL) - Night	persons	0.46	0.46	0.00	0.46	0.46
Potential Loss of Life (PLL) - Weighted	persons	0.46	0.46	0.00	0.46	0.46
		Third-party Losses				
Residential Buildings	no.	23	23	0	15	23
Rural Commercial Buildings	no.	15	15	0	10	15
Damages	$ M	17.0	16.9	0.0	3.5	16.6
Residential Damages	$ M	6.7	6.6	0.0	3.5	6.3
ESS Repair + Value of Water	$ M	0.0	0.0	0.0	0.0	0.0
Damage Severity		-	-	-	-	-
Hazard Potential						

Table C.2: Breach PAR, PLL, and Damage Cost Computation Details using the Graham 1999 Methodology

		Eurobodalla Southern Storage – Graham 1999 Methodology							
		Events with Embankment Failure							
		Sunny Day Failure	1 in 100 AEP	Cofferdam at DCF	**Probable Maximum Flood with Embankment Failure**				
	Case	**Breach Tuross = 1in1 AEP**	**Breach Tuross = 1in10 AEP**	**Cofferdam Breach Tuross = 1in100 AEP**	Breach Tuross = 1in100 AEP	Breach Tuross = 1in1 AEP	Breach Tuross = 1in100 AEP Breach Time Check	Breach Tuross = 1in100 AEP Breach Width=69m Check	Breach Tuross = 1in100 AEP Breach Width=25m Check
		Life Safety							
Population At Risk - Day	persons	0.0	19.2	27.6	27.6	0.0	27.6	27.6	27.6
Population At Risk - Night	persons	0.0	38.4	55.2	55.2	0.0	55.2	55.2	55.2
Population At Risk - Weighted	persons	0.0	30.4	43.7	43.7	0.0	43.7	43.7	43.7
Potential Loss of Life (PLL) - Day	persons	0.45	0.64	0.73	0.73	0.45	0.73	0.73	0.73
Potential Loss of Life (PLL) - Night	persons	0.45	0.83	1.00	1.00	0.45	1.00	1.00	1.00
Potential Loss of Life (PLL) - Weighted	persons	0.45	0.75	0.89	0.89	0.45	0.89	0.89	0.89
		Third-party Losses							
Residential Buildings	no.	0	16	23	23	0	23	23	23
Rural Commercial Buildings	no.	0	10	15	15	0	15	15	15
Damages	$ M	33.3	37.1	17.8	49.9	33.3	50.0	50.1	49.9
Residential Damages	$ M	0.0	3.8	7.5	6.6	0.0	6.7	6.8	6.6
ESS Repair + Value of Water	$ M	33.0	33.0	0.0	33.0	33.0	33.0	33.0	33.0
Damage Severity		**Major**	-	-	-	-	-	-	-
Hazard Potential		**High C**							

Table C.3: Incremental PAR, PLL, and Damage Cost Computation Details using the Graham 1999 Methodology

		Eurobodalla Southern Storage – Graham 1999 Methodology						
		Incremental Failure						
		1 in 100 AEP	Cofferdam at DCF	Probable Maximum Flood Incremental Failure				
	Case	Breach Tuross = 1in10 AEP	Cofferdam Breach Tuross = 1in100	Breach Tuross=1:100	Breach Tuross=1m	Breach Tuross=1:100 Breach Time Check	Breach Tuross=1:100 Breach Width=69m Check	Breach Tuross=1:100 Breach Width=25m Check
		Life Safety						
Population At Risk - Day	persons	1.2	0.0	0.0	0.0	0.0	0.0	0.0
Population At Risk - Night	persons	2.4	0.0	0.0	0.0	0.0	0.0	0.0
Population At Risk - Weighted	persons	1.9	0.0	0.0	0.0	0.0	0.0	0.0
Potential Loss of Life (PLL) - Day	persons	0.19	0.27	0.27	0.45	0.27	0.27	0.27
Potential Loss of Life (PLL) - Night	persons	0.38	0.54	0.54	0.45	0.54	0.54	0.54
Potential Loss of Life (PLL) - Weighted	persons	0.30	0.43	0.43	0.45	0.43	0.43	0.43
		Third-party Losses						
Residential Buildings	no.	1	0	0	0	0	0	0
Rural Commercial Buildings	no.	0	0	0	0	0	0	0
Damages	$ M	33.6	1.2	33.0	33.3	33.1	33.2	33.0
Residential Damages	$ M	0.3	0.9	0.0	0.0	0.1	0.2	0.0
ESS Repair + Value of Water	$ M	33.0	0.0	33.0	33.0	33.0	33.0	33.0
Damage Severity		**Major**	**Minor**	**Major**	**Major**	**Major**	**Major**	**Major**
Hazard Potential		**High C**	**Significant**	**High C**	**High C**	**High C**	**High C**	**High C**

Table C.4: No Breach PAR, PLL, and Damage Cost Computation Details using the Alternative PLL Methodology

	Case	Probable Maximum Flood			1 in 100 AEP	
		Existing No Embankment Tuross = 1in100 AEP	No Breach Tuross = 1in100 AEP	No Breach Tuross = 1in1 AEP	No Breach Tuross = 1in10 AEP	Cofferdam No Breach Tuross = 1in100 AEP
Eurobodalla Southern Storage - Alternative Methodology						
Flood Events (Without Failure)						
Life Safety						
Population At Risk - Day	persons	27.6	27.6	0	18.0	27.6
Population At Risk - Night	persons	55.2	55.2	0	36.0	55.2
Population At Risk - Weighted	persons	43.7	43.7	0	28.5	43.7
Potential Loss of Life (PLL) - Day	persons	0.53	0.51	0.000	0.46	0.51
Potential Loss of Life (PLL) - Night	persons	0.61	0.58	0.000	0.47	0.56
Potential Loss of Life (PLL) - Weighted	persons	0.58	0.55	0.000	0.47	0.54
Third-party Losses						
Residential Buildings	no.	23	23	0	15	23
Rural Commercial Buildings	no.	15	15	0	10	15
Damages	$ M	17.0	16.9	0.0	3.5	16.6
Residential Damages	$ M	6.7	6.6	0.0	3.5	6.3
ESS Repair + Value of Water	$ M	0.0	0.0	0.0	0.0	0.0
Damage Severity		-	-	-	-	-
Hazard Potential						

Table C.5: Breach PAR, PLL, and Damage Cost Computation Details using the Alternative PLL Methodology

		Eurobodalla Southern Storage - Alternative Methodology							
		Events with Embankment Failure							
		Sunny Day Failure	1 in 100 AEP	Cofferdam at DCF	Probable Maximum Flood with Embankment Failure				
	Case	Breach Tuross = 1in1 AEP	Breach Tuross = 1in10 AEP	Cofferdam Breach Tuross = 1in100 AEP	Breach Tuross = 1in100 AEP	Breach Tuross = 1in1 AEP	Breach Tuross = 1in100 AEP Breach Time Check	Breach Tuross = 1in100 AEP Breach Width=69m Check	Breach Tuross = 1in100 AEP Breach Width=25m Check
		Life Safety							
Population At Risk - Day	persons	0.0	19.2	27.6	27.6	0.0	27.6	27.6	27.6
Population At Risk - Night	persons	0.0	38.4	55.2	55.2	0.0	55.2	55.2	55.2
Population At Risk - Weighted	persons	0.0	30.4	43.7	43.7	0.0	43.7	43.7	43.7
Potential Loss of Life (PLL) - Day	persons	0.45	0.47	0.55	0.51	0.45	0.52	0.52	0.52
Potential Loss of Life (PLL) - Night	persons	0.45	0.49	0.65	0.58	0.45	0.59	0.59	0.58
Potential Loss of Life (PLL) - Weighted	persons	0.45	0.48	0.61	0.55	0.45	0.56	0.56	0.56
		Third-party Losses							
Residential Buildings	no.	0	16	23	23	0	23	23	23
Rural Commercial Buildings	no.	0	10	15	15	0	15	15	15
Damages	$ M	33.3	37.1	17.8	49.9	33.3	50.0	50.1	49.9
Residential Damages	$ M	0.0	3.8	7.5	6.6	0.0	6.7	6.8	6.6
ESS Repair + Value of Water	$ M	33.0	33.0	0.0	33.0	33.0	33.0	33.0	33.0
Damage Severity		Major	-	-	-	-	-	-	-
Hazard Potential		High C							

Table C.6: Incremental PAR, PLL, and Damage Cost Computation Details using the Alternative PLL Methodology

		1 in 100 AEP	Cofferdam at DCF	Probable Maximum Flood Incremental Failure				
	Case	Breach Tuross=1:10	Cofferdam Breach Tuross=1:100	Breach Tuross=1:100	Breach Tuross=1m	Breach Tuross=1:100 Breach Time Check	Breach Tuross=1:100 Breach Width=69m Check	Breach Tuross=1:100 Breach Width=25m Check
Eurobodalla Southern Storage - Alternative Methodology								
Incremental Failure								
Life Safety								
Population At Risk - Day	persons	1.2	0.0	0.0	0.0	0.0	0.0	0.0
Population At Risk - Night	persons	2.4	0.0	0.0	0.0	0.0	0.0	0.0
Population At Risk - Weighted	persons	1.9	0.0	0.0	0.0	0.0	0.0	0.0
Potential Loss of Life (PLL) - Day	persons	0.007	0.041	0.000	0.45	0.005	0.006	0.003
Potential Loss of Life (PLL) - Night	persons	0.012	0.083	0.001	0.45	0.009	0.012	0.005
Potential Loss of Life (PLL) - Weighted	persons	0.010	0.066	0.000	0.45	0.007	0.009	0.004
Third-party Losses								
Residential Buildings	no.	1	0	0	0	0	0	0
Rural Commercial Buildings	no.	0	0	0	0	0	0	0
Damages	$ M	33.6	1.2	33.0	33.3	33.1	33.2	33.0
Residential Damages	$ M	0.3	0.9	0.0	0.0	0.1	0.2	0.0
ESS Repair + Value of Water	$ M	33.0	0.0	33.0	33.0	33.0	33.0	33.0
Damage Severity		Major	Minor	Major	Major	Major	Major	Major
Hazard Potential		Significant	Very Low	Significant	High C	Significant	Significant	Significant

Appendix D   SEVERITY AND LOSS TABLES

EUROBODALLA SOUTHERN STORAGE PMF DAMAGE AND LOSS		SEVERITY LEVEL			
		Minor	Medium	Major	Catastrophic
TOTAL INFRASTRUCTURE COSTS	COST ESTIMATE				
Residential	$ M				
Commercial	$ M				
Community Infrastructure	$ M				
Dam replacement or repair cost	$30 M				
Total Estimated Cost	**$ 30 Million**		X		
IMPACT ON DAM OWNER'S BUSINESS					
Importance to the business			X		
Effect on services provided by the owner			X		
Effect on continuing credibility				X	
Community reaction and political implications				X	
Impact on financial viability				X	
Value of water in storage			X		
HEALTH AND SOCIAL IMPACTS					
Public health		X			
Loss of service to the community		X			
Cost of emergency management		X			
Dislocation of people		X			
Dislocation of businesses		X			
Employment affected		X			
Loss of heritage		X			
Loss of recreational facility		X			
ENVIRONMENTAL IMPACTS					
Stock and fauna		X			
Ecosystems		X			
Rare and endangered species		X			
OVERALL					
Highest level of severity of damage and loss				**Major**	

EUROBODALLA SOUTHERN STORAGE SUNNY DAY DAMAGE AND LOSS		SEVERITY LEVEL			
		Minor	Medium	Major	Catastrophic
TOTAL INFRASTRUCTURE COSTS	COST ESTIMATE				
Residential	$ M				
Commercial	$ M				
Community Infrastructure	$10 M				
Dam replacement or repair cost	$30 M				
Total Estimated Cost	**$ 40 Million**		X		
IMPACT ON DAM OWNER'S BUSINESS					
Importance to the business			X		
Effect on services provided by the owner			X		
Effect on continuing credibility				X	
Community reaction and political implications				X	
Impact on financial viability				X	
Value of water in storage			X		
HEALTH AND SOCIAL IMPACTS					
Public health		X			
Loss of service to the community		X			
Cost of emergency management		X			
Dislocation of people		X			
Dislocation of businesses		X			
Employment affected		X			
Loss of heritage		X			
Loss of recreational facility		X			
ENVIRONMENTAL IMPACTS					
Stock and fauna		X			
Ecosystems		X			
Rare and endangered species		X			
OVERALL					
Highest level of severity of damage and loss			**Major**		

Appendix E   FLOOD MAPS

Legend

Building Points
Depth x Velocity (DV) Ranges (m²/s)
- ● 0.0 to 0.4
- ○ 0.4 to 0.6
- ● 0.6 to 1.0
- ● 1.0 to 2.0
- ● 2.0 to 4.6
- ● 4.6 to 20
- ▪ Building Sheds

Inundation Depth* (m)
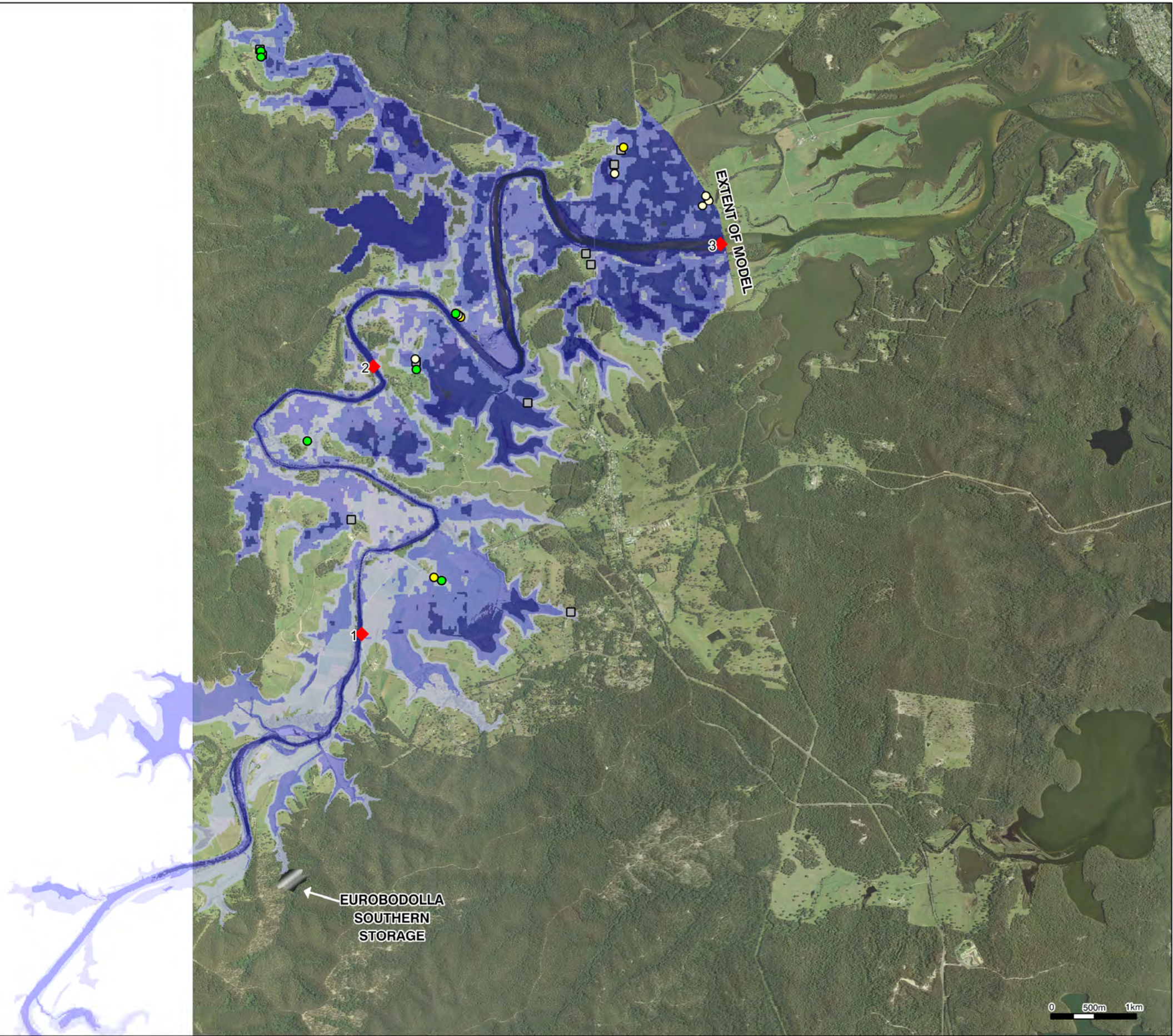- 0.05 to 5.00
- 5.00 to 10.0
- 10.0 to 15.0
- > 15.0

Time to Peak from Breach Peak
- ◆ Hour

*Depth is measured relative to ground level.
Within the Tuross River the depth is relative to the river bed.

EXTENT OF MODEL

EUROBODOLLA SOUTHERN STORAGE

0   500m   1km

EXTENT OF MODEL

COORDINATE SYSTEM Datum: GDA94 Projection: MGA Zone 56	NORTH	**DRAWING NO.** 30012127-E1	**REVISION** B	**CLIENT**
PAGE SIZE A3 **SCALE** 1 in 50,000		**CREATED BY** H. MALLEN	**DATE** 22-06-2017	eurobodalla shire council
PROJECT NO. 30012127 **PROJECT TITLE** Eurobodalla Southern Storage		**TITLE** **Sunny Day With Breach** Inundation Depth Map with Property DV		**CONSULTANT** SMEC Member of the Surbana Jurong Group Level 10, 71 Queens Road, Melbourne VIC 3004 www.smec.com

LEGEND

Building Points
Depth x Velocity (DV) Ranges (m²/s)
● 0.0 to 0.4
● 0.4 to 0.6
● 0.6 to 1.0
● 1.0 to 2.0
● 2.0 to 4.6
● 4.6 to 20
▪ Building Sheds

Inundation Depth* (m)
    0.05 to 5.00
    5.00 to 10.0
    10.0 to 15.0
    > 15.0

Time to Peak from Breach Peak
◆ hour

*Depth is measured relative to
ground level.
Within the Tuross River the
depth is relative to the river bed.

EXTENT OF MODEL

3 hr

2 hr

1 hr

EUROBODALLA
SOUTHERN
STORAGE

EXTENT OF MODEL

0   500m   1km

Legend

Building Points
Depth x Velocity (DV) Ranges (m²/s)

- 🟢 0.0 to 0.4
- ⚪ 0.4 to 0.6
- 🟡 0.6 to 1.0
- 🟠 1.0 to 2.0
- 🔴 2.0 to 4.6
- 🔴 4.6 to 20
- ⬜ Building Sheds

Inundation Depth* (m)

- 0.05 to 5.00
- 5.00 to 10.0
- 10.0 to 15.0
- > 15.0

*Depth is measured relative to ground level.
Within the Tuross River the depth is relative to the river bed.

EXTENT OF MODEL

EUROBODOLLA SOUTHERN STORAGE

0   500m   1km

EXTENT OF MODEL

COORDINATE SYSTEM	Datum: GDA94 Projection: MGA Zone 56	NORTH	DRAWING NO.	30012127-E3	REVISION	B	CLIENT	CONSULTANT

PAGE SIZE A3   SCALE 1 in 50,000

CREATED BY H. MALLEN   DATE 22-06-2017

PROJECT NO. 30012127   PROJECT TITLE Eurobodalla Southern Storage

TITLE **PMF Without Breach (Tuross River at 1 in 100 AEP Flood Level)**
Inundation Depth Map with Property DV

eurobodalla shire council

SMEC
Member of the Surbana Jurong Group
Level 10, 71 Queens Road, Melbourne VIC 3004
www.smec.com

Legend

Building Points
Depth x Velocity (DV) Ranges (m²/s)
- 0.0 to 0.4
- 0.4 to 0.6
- 0.6 to 1.0
- 1.0 to 2.0
- 2.0 to 4.6
- 4.6 to 20
- Building Sheds

Inundation Depth* (m)
- 0.05 to 5.00
- 5.00 to 10.0
- 10.0 to 15.0
- > 15.0

*Depth is measured relative to ground level.
Within the Tuross River the depth is relative to the river bed.

EXTENT OF MODEL

EUROBODOLLA SOUTHERN STORAGE

0   500m   1km

EXTENT OF MODEL

COORDINATE SYSTEM	Datum: GDA94 Projection: MGA Zone 56	NORTH
PAGE SIZE A3 SCALE 1 in 50,000		
PROJECT NO. 30012127 PROJECT TITLE Eurobodalla Southern Storage		

DRAWING NO.	30012127-E4	REVISION	B
CREATED BY	H. MALLEN	DATE	22-06-2017

TITLE PMF Existing (No Storage) (Tuross River at 1 in 100 AEP Level)
Inundation Depth Map with Property DV

CLIENT
eurobodalla shire council

CONSULTANT
SMEC
Member of the Surbana Jurong Group
Level 10, 71 Queens Road, Melbourne VIC 3004
www.smec.com

Copyright SMEC Australia Pty Ltd. All Rights Reserved. Aerial imagery © Eurobodalla Shire Council 2016

Legend

Building Points
Depth x Velocity (DV) Ranges (m²/s)
- 🟢 0.0 to 0.4
- ⚪ 0.4 to 0.6
- 🟡 0.6 to 1.0
- 🟤 1.0 to 2.0
- 🔴 2.0 to 4.6
- 🔴 4.6 to 20
- ⬜ Building Sheds

Inundation Depth* (m)
- 0.05 to 5.00
- 5.00 to 10.0
- 10.0 to 15.0
- > 15.0

Time to Peak from Breach Peak
- 🔴 Hour

*Depth is measured relative to ground level.
Within the Tuross River the depth is relative to the river bed.

EXTENT OF MODEL

EUROBODALLA
SOUTHERN
STORAGE

0   500m   1km

EXTENT OF MODEL

COORDINATE SYSTEM	Datum: GDA94 Projection: MGA Zone 56	NORTH	DRAWING NO. 30012127-E5	REVISION B	CLIENT	CONSULTANT
PAGE SIZE A3 SCALE 1 in 50,000			CREATED BY H. MALLEN	DATE 22-06-2017	eurobodalla shire council	SMEC Member of the Surbana Jurong Group
PROJECT NO. 30012127 PROJECT TITLE Eurobodalla Southern Storage			TITLE **1 in 100 AEP With Breach (Tuross River at 1 in 10 AEP Flood Level)** Inundation Depth Map with Property DV			Level 10, 71 Queens Road, Melbourne VIC 3004

Copyright SMEC Australia Pty Ltd. All Rights Reserved. Aerial imagery © Eurobodalla Shire Council 2016

Legend

Building Points
Depth x Velocity (DV) Ranges (m²/s)
- ● 0.0 to 0.4
- ○ 0.4 to 0.6
- ● 0.6 to 1.0
- ● 1.0 to 2.0
- ● 2.0 to 4.6
- ● 4.6 to 20
- ▢ Building Sheds

Inundation Depth* (m)
- 0.05 to 5.00
- 5.00 to 10.0
- 10.0 to 15.0
- > 15.0

*Depth is measured relative to ground level.
Within the Tuross River the depth is relative to the river bed.

EXTENT OF MODEL

EUROBODOLLA SOUTHERN STORAGE
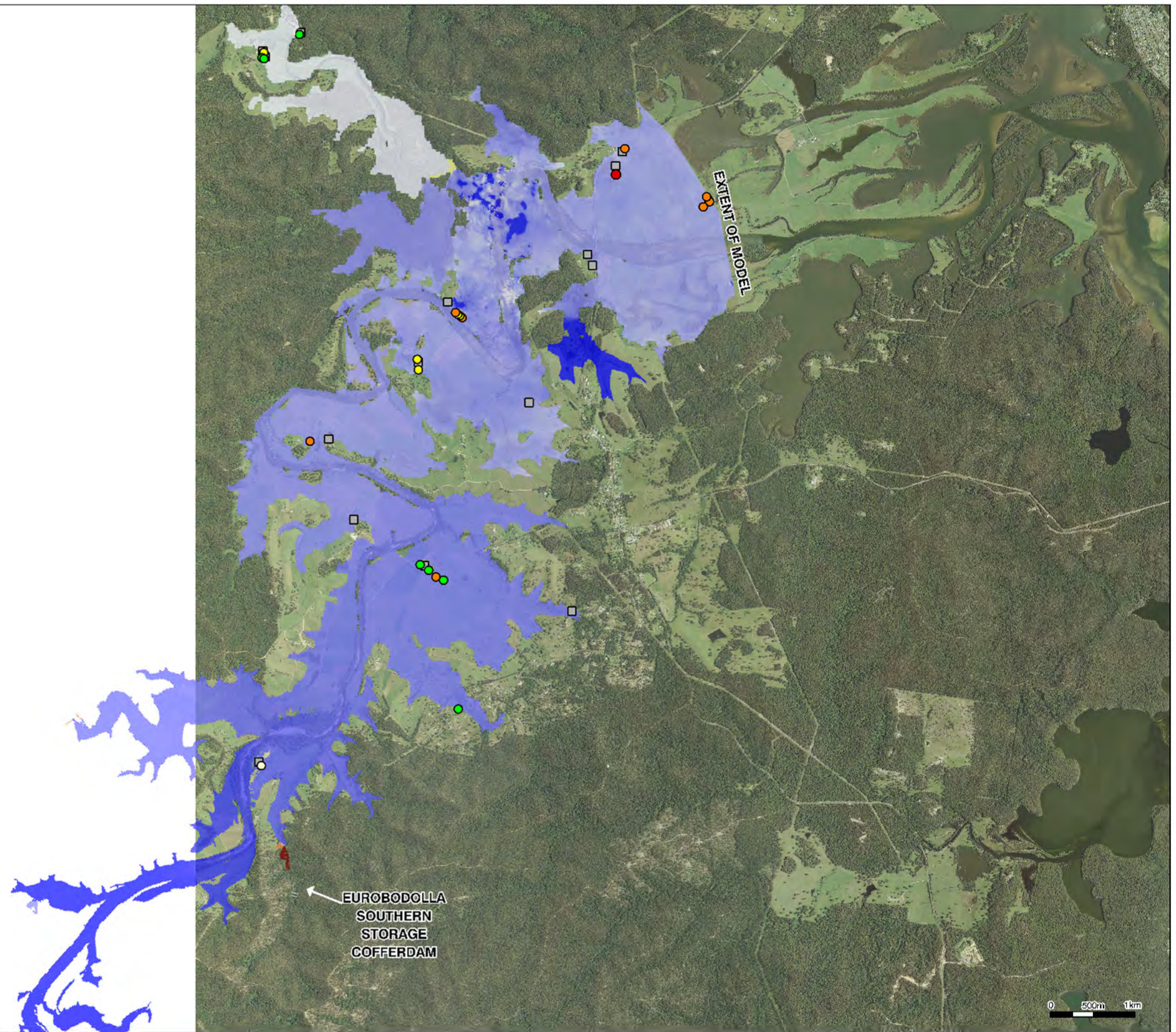
0   500m   1km

EXTENT OF MODEL

Legend

Building Points
Depth x Velocity (DV) Ranges (m²/s)

- 0.0 to 0.4
- 0.4 to 0.6
- 0.6 to 1.0
- 1.0 to 2.0
- 2.0 to 4.6
- 4.6 to 20
- Building Sheds

Inundation Depth* (m)

- 0.05 to 5.00
- 5.00 to 10.0
- 10.0 to 15.0
- > 15.0

Time to Peak from Breach Peak

- Hour

*Depth is measured relative to ground level.
Within the Tuross River the depth is relative to the river bed.

EXTENT OF MODEL

EXTENT OF MODEL

EUROBODOLLA
SOUTHERN
STORAGE
COFFERDAM

0   500m   1km

EXTENT OF MODEL

Legend

Building Points

Depth x Velocity (DV) Ranges (m²/s)

- ● 0.0 to 0.4
- ○ 0.4 to 0.6
- ● 0.6 to 1.0
- ● 1.0 to 2.0
- ● 2.0 to 4.6
- ● 4.6 to 20
- ▫ Building Sheds

Breach to No Breach Depth Difference (m)

- 0.000 to 0.001
- 0.001 to 0.010
- 0.010 to 0.020
- 0.020 to 0.050
- 0.100 to 1.000
- >1.0

EXTENT OF MODEL

EUROBODOLLA SOUTHERN STORAGE COFFERDAM

EXTENT OF MODEL

0   500m   1km

Legend

Building Points
Depth x Velocity (DV) Ranges (m²/s)
- ● 0.0 to 0.4
- ● 0.4 to 0.6
- ● 0.6 to 1.0
- ● 1.0 to 2.0
- ● 2.0 to 4.6
- ● 4.6 to 20
- ▫ Building Sheds

Breach to No Breach Depth Difference (m)
- 0.00 to 0.01
- 0.01 to 0.10
- 0.10 to 0.15
- 0.15 to 0.20
- 0.20 to 1.00
- >1.0

EXTENT OF MODEL

EUROBODOLLA SOUTHERN STORAGE COFFERDAM

EXTENT OF MODEL

0   500m   1km

www.smec.com

Legend

Building Points

Depth x Velocity (DV) Ranges (m²/s)

- ● 0.0 to 0.4
- ○ 0.4 to 0.6
- ● 0.6 to 1.0
- ● 1.0 to 2.0
- ● 2.0 to 4.6
- ● 4.6 to 20
- ▪ Building Sheds

Breach to No Breach Depth Difference (m)

- 0.00 to 0.01
- 0.01 to 0.10
- 0.10 to 0.15
- 0.15 to 0.20
- 0.20 to 1.00
- >1.0

EXTENT OF MODEL

EXTENT OF MODEL

EUROBODOLLA
SOUTHERN
STORAGE
COFFERDAM

0   500m   1km

EXTENT OF MODEL

COORDINATE SYSTEM	Datum: GDA94 Projection: MGA Zone 56	NORTH	DRAWING NO.	30012127-E10	REVISION	A
PAGE SIZE A3 SCALE 1 in 50,000			CREATED BY H. MALLEN		DATE 22-06-2017	

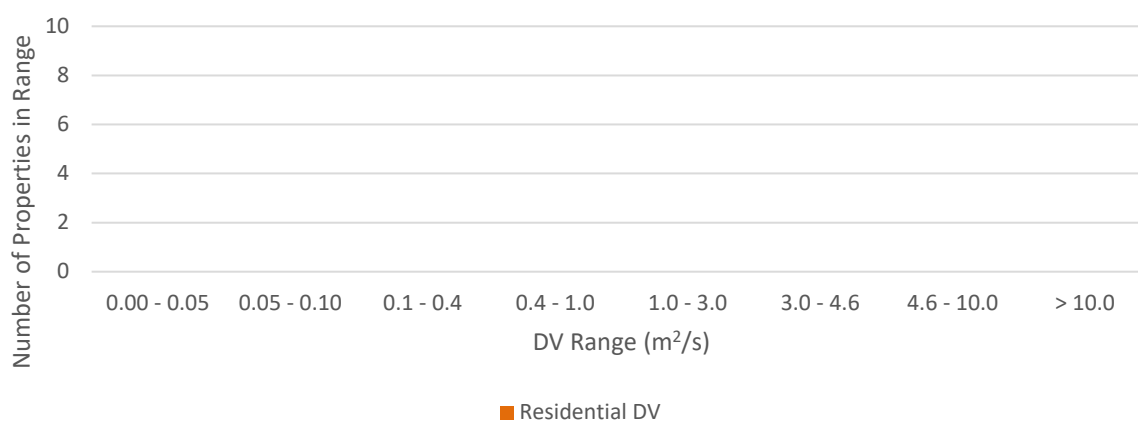| CLIENT	CONSULTANT
eurobodalla shire·council	SMEC Member of the Surbana Jurong Group Level 10, 71 Queens Road, Melbourne VIC 3004

PROJECT NO. 30012127 PROJECT TITLE Eurobodalla Southern Storage	TITLE	**Eurobodalla Dam 1 in 100 Breach to No Breach Depth Difference** Inundation Depth Difference Map (Tuross River at 1 in 10 AEP Level)

www.smec.com

Appendix F   PROPERTY FLOOD SEVERITY

Sunny Day With Breach

PMF With Breach (Tuross at 1 in 100 AEP Level)
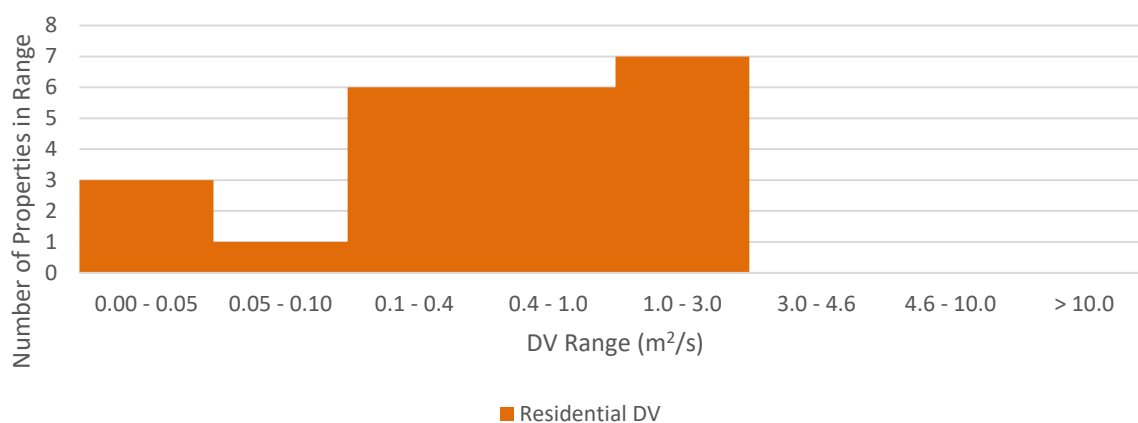

PMF Without Breach (Tuross at 1 in 100 AEP Level)
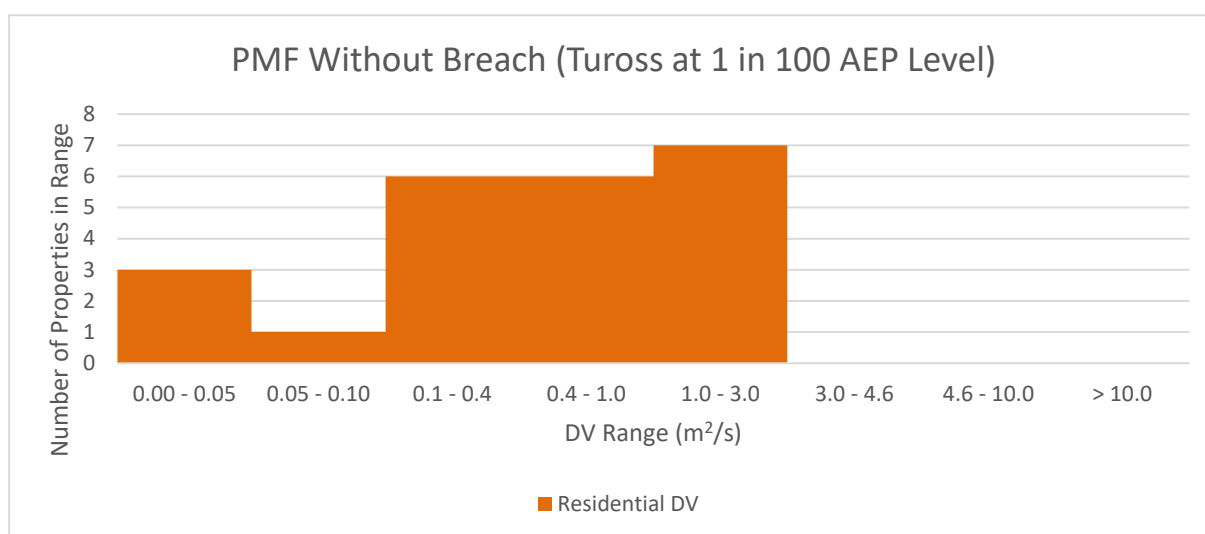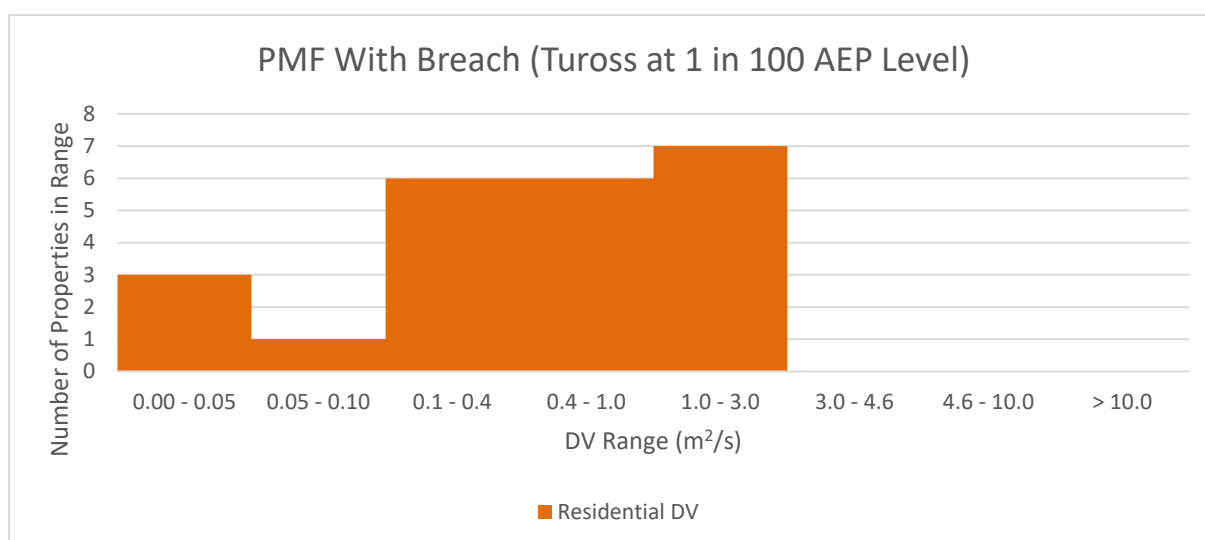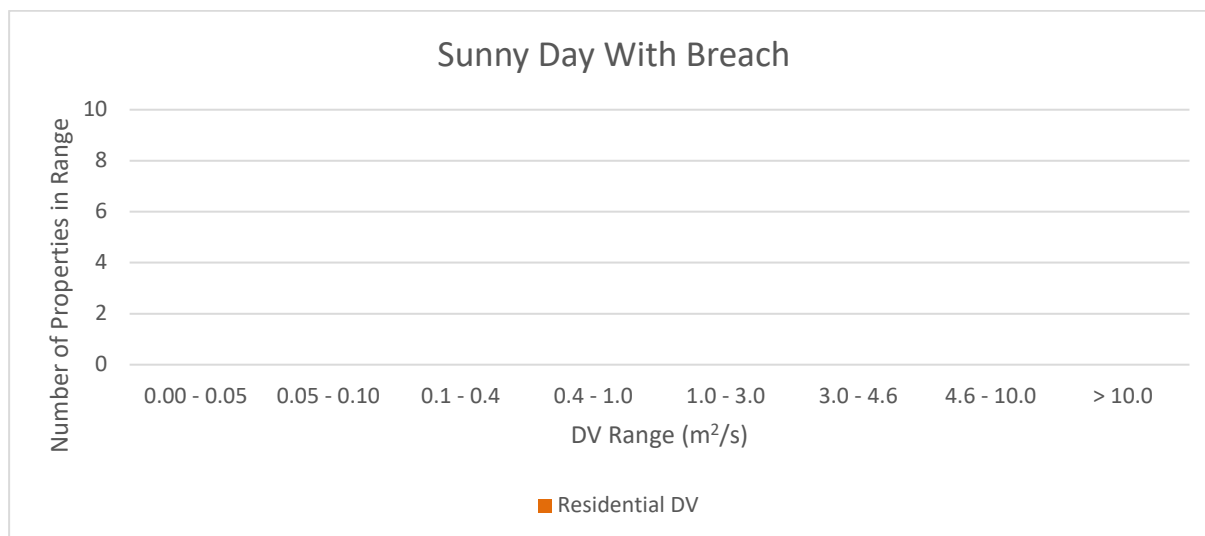
PMF With Breach (Tuross at 1 in 1 AEP Level)
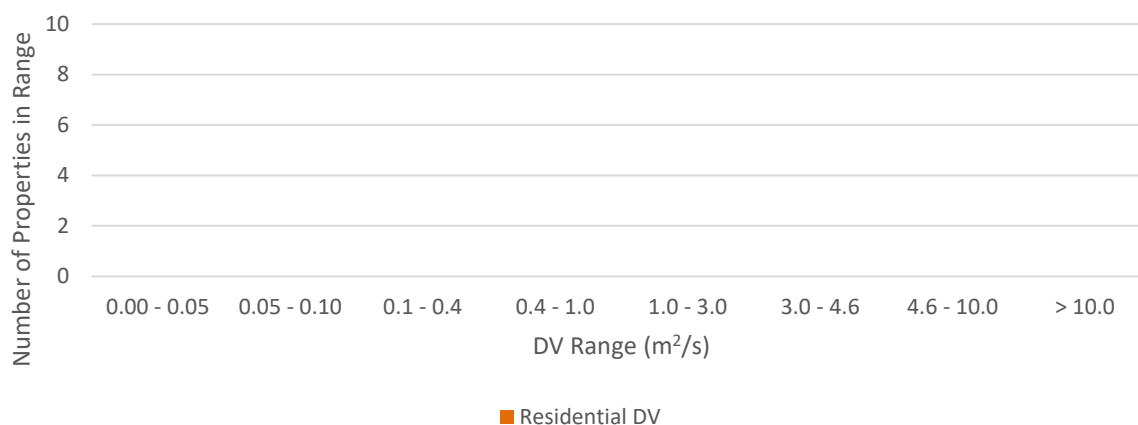
PMF Without Breach (Tuross at 1 in 1 AEP Level)
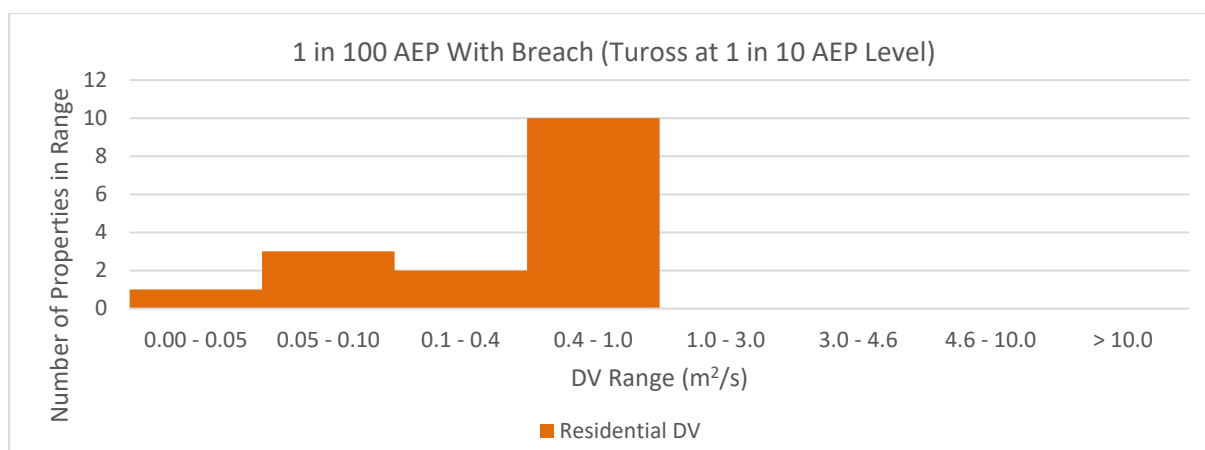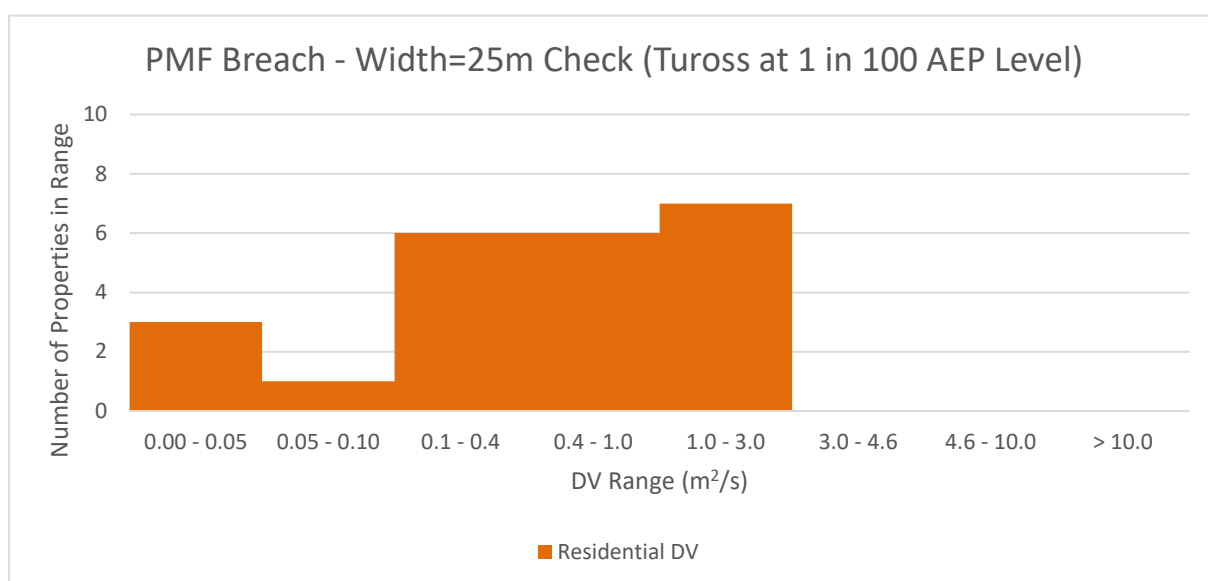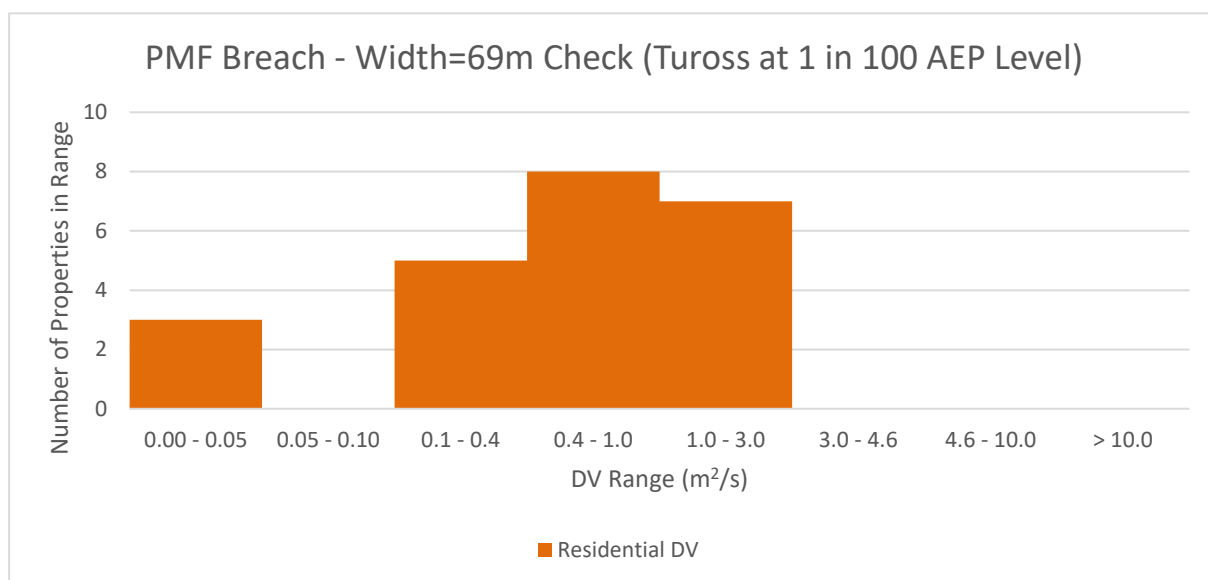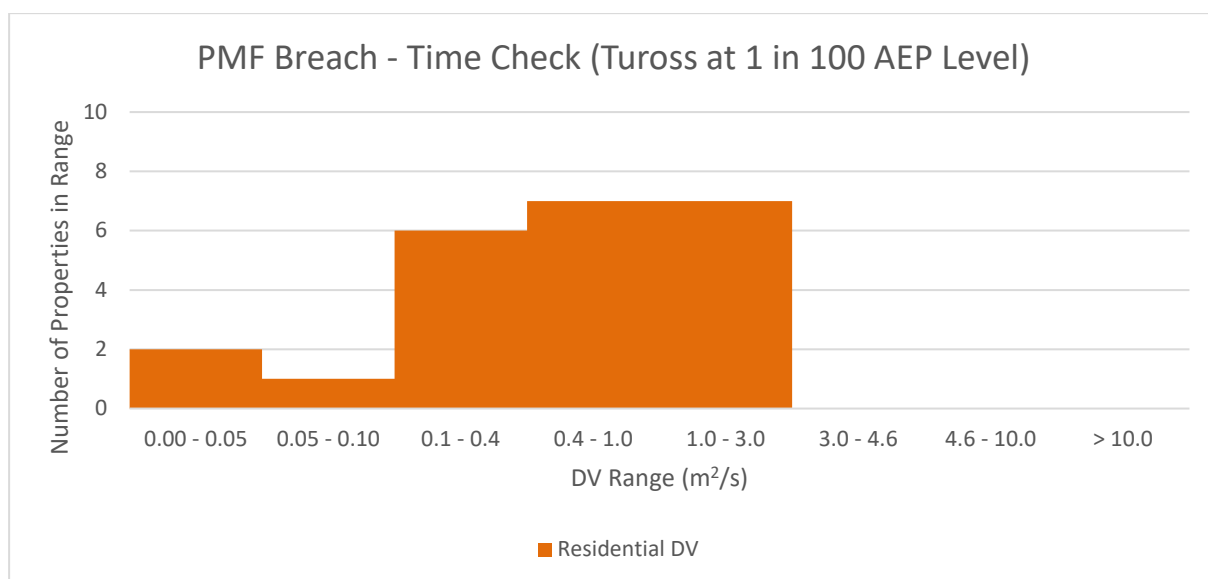

PMF Existing (No Embankment - Tuross at 1 in 100 AEP Level)

1 in 100 AEP With Breach (Tuross at 1 in 10 AEP Level)

Number of Properties in Range vs DV Range (m²/s)
Residential DV

1 in 100 AEP Without Breach (Tuross at 1 in 10 AEP Level)

Number of Properties in Range vs DV Range (m²/s)
Residential DV

Cofferdam DCF With Breach (Tuross at 1 in 100 AEP Level)

Number of Properties in Range vs DV Range (m²/s)
Residential DV

Cofferdam DCF Without Breach (Tuross at 1 in 100 AEP Level)

Number of Properties in Range vs DV Range (m²/s)
Residential DV

PMF Breach - Time Check (Tuross at 1 in 100 AEP Level)

Number of Properties in Range

DV Range (m²/s)

■ Residential DV

PMF Breach - Width=69m Check (Tuross at 1 in 100 AEP Level)

Number of Properties in Range

DV Range (m²/s)

■ Residential DV

PMF Breach - Width=25m Check (Tuross at 1 in 100 AEP Level)

Number of Properties in Range

DV Range (m²/s)

■ Residential DV

IMPORTANT NOTICE

This report is confidential and is provided solely for the purposes of assessing failure consequences for Eurobodalla Southern Storage. This report is provided pursuant to a Consultancy Agreement between SMEC Australia Pty Limited ("SMEC") and Eurobodalla Shire Council under which SMEC undertook to perform a specific and limited task for Eurobodalla Shire Council. This report is strictly limited to the matters stated in it and subject to the various assumptions, qualifications and limitations in it and does not apply by implication to other matters. SMEC makes no representation that the scope, assumptions, qualifications and exclusions set out in this report will be suitable or sufficient for other purposes nor that the content of the report covers all matters which you may regard as material for your purposes.

This report must be read as a whole. The executive summary is not a substitute for this. Any subsequent report must be read in conjunction with this report.

The report supersedes all previous draft or interim reports, whether written or presented orally, before the date of this report. This report has not and will not be updated for events or transactions occurring after the date of the report or any other matters which might have a material effect on its contents or which come to light after the date of the report. Unless expressly agreed otherwise in writing, SMEC does not accept a duty of care or any other legal responsibility whatsoever in relation to this report, or any related enquiries, advice or other work, nor does SMEC make any representation in connection with this report, to any person other than Eurobodalla Shire Council. Any other person who receives a draft or a copy of this report (or any part of it) or discusses it (or any part of it) or any related matter with SMEC, does so on the basis that he or she acknowledges and accepts that he or she may not rely on this report nor on any related information or advice given by SMEC for any purpose whatsoever.

DOCUMENT / REPORT CONTROL FORM

File Location Name:	I:\Projects\30012127 - Eurobodalla Storage\07 Working\08 Consequence Assessment\Report
Project Name:	Eurobodalla Southern Storage
Project Number:	30012127
Revision Number:	1

Revision History

Revision #	Date	Prepared by	Reviewed by	Approved for Issue by
Draft	13-03-17	J.Missen/T.Rhodes	C.Purss, R.Westmore	D.Evans
Rev1	23-06-17	H.Mallen	T.Rhodes	D.Evans

Appendix D   NSW Water quality objectives for the Tuross River

Protection Category	Indicator
Aquatic ecology	- total phosphorus: 25 µg/L - total nitrogen: 350 µg/L - chlorophyll-*a*: 5 µg/L - turbidity: 6–50 NTU (salinity) electrical conductivity: 125–2200 µS/cm - dissolved oxygen: 85–110 per cent - pH: 6.5–8.5 - chemical contaminants or toxicants: in accordance with Table 3.4.1 of the ANZECC 2000 Guidelines.
Visual amenity	- visual clarity and colour: - natural visual clarity should not be reduced by more than 20 per cent - natural hue of the water should not be changed by more than 10 points on the Munsell Scale - the natural reflectance of the water should not be changed by more than 50 per cent - surface films and debris: - oils and petrochemicals should not be noticeable as a visible film on the water, nor should they be detectable by odour - waters should be free from floating debris and litter - nuisance organisms: macrophytes, phytoplankton scums, filamentous algal mats, blue-green algae, sewage fungus and leeches should not be present in unsightly amounts.
Primary contact recreation	- turbidity: a 200 mm diameter black disc should be able to be sighted horizontally from a distance of more than 1.6 m (equates to approximately 6 NTU) - faecal coliforms: - BeachWatch considers waters are unsuitable for swimming if the median faecal coliform density exceeds 150 colony forming units per 100 millilitres (cfu/100 mL) across five samples taken at regular intervals not exceeding one month, or the second highest sample contains equal to or greater than 600 faecal coliform cfu/100 mL across five samples taken at regular intervals not exceeding one month - ANZECC 2000 Guidelines recommend a median over the bathing season of < 150 faecal coliforms per 100 mL, with four out of five samples <600/100 mL (minimum of five samples taken at regular intervals not exceeding one month) - Enterococci: - BeachWatch considers waters are unsuitable for swimming if the median enterococci density exceeds 35 cfu/100 mL across five samples taken at regular intervals not exceeding one month, or the second highest sample contains equal to or greater than 100 cfu/100 mL (enterococci) for five samples taken at regular intervals not exceeding one month - ANZECC 2000 Guidelines recommend median over bathing season of < 35 enterococci per 100 mL (maximum number in any one sample: 60-100 organisms/100 mL) - Protozoans: pathogenic free-living protozoans should not be present in bodies of fresh water

Protection Category	Indicator

	• algae, including blue-green algae: < 15 000 cells/mL
	• nuisance organisms:
	  – use visual amenity guidelines
	  – large numbers of midges and aquatic worms are undesirable
	• pH: 5.0-9.0
	• temperature: 15-35 °C for prolonged exposure
	• chemical contaminants:
	  – waters containing chemicals that are either toxic or irritating to the skin or mucus membranes are unsuitable for recreation
	  – toxic substances should not exceed the concentrations provided in Table 5.2.3 and Table 5.2.4 of the ANZECC 2000 Guidelines
	• visual clarity and colour: use visual amenity guidelines
	• surface films: use visual amenity guidelines.

Secondary contact recreation	• faecal coliforms: median bacterial content in fresh and marine waters of < 1000 faecal coliform cfu/100 mL, with four out of five samples < 4000 cfu/100 mL (minimum of five samples taken at regular intervals not exceeding one month)
	• *Enterococci*: median bacterial content in fresh and marine waters of < 230 enterococci cfu/100 mL (maximum number in any one sample: 450-700 cfu/100 mL)
	• algae, including blue-green algae: < 15 000 cells/mL
	• nuisance organisms:
	  – use visual amenity guidelines
	  – large numbers of midges and aquatic worms are undesirable
	• chemical contaminants:
	  – waters containing chemicals that are either toxic or irritating to the skin or mucous membranes are unsuitable for recreation
	  – toxic substances should not exceed values in tables 5.2.3 and 5.2.4 of the ANZECC 2000 Guidelines
	• visual clarity and colour: use visual amenity guidelines
	• surface films: use visual amenity guidelines.

Livestock water supply	• algae, including blue-green algae: increasing risk to livestock health is likely when cell counts of microcystins exceed 11,500 cells/mL and/or concentrations of microcystins exceed 2.3 µg/L expressed as microcystin-LR toxicity equivalents
	• salinity (electrical conductivity): recommended concentrations of total dissolved solids in drinking water for livestock are given in Table 4.3.1 of the ANZECC 2000 Guidelines
	• faecal coliforms: drinking water for livestock should contain less than 100 cfu/100 mL (median value)
	• chemical contaminants:
	  – refer to Table 4.3.2 of the ANZECC 2000 Guidelines for heavy metals and metalloids in livestock drinking water
	  – refer to Australian Drinking Water Guidelines (NHMRC and NRMMC 2004) for information regarding pesticides and other organic contaminants, using criteria for raw drinking water.

Irrigation water supply	• algae, including blue-green algae: should not be visible; no more than low algal levels are desired to protect irrigation equipment

Protection Category	Indicator
	• salinity (electrical conductivity): to assess the salinity and sodicity of water for irrigation use, several interactive factors must be considered including irrigation water quality, soil properties, plant salt tolerance, climate, landscape and water and soil management; for more information, refer to Chapter 4.2.4 of the ANZECC 2000 Guidelines • faecal coliforms: trigger values for faecal coliforms in irrigation water used for food and non-food crops are provided in Table 4.2.2 of the ANZECC 2000 Guidelines • heavy metals and metalloids: long term trigger values (LTV) and short-term trigger values (STV) for heavy metals and metalloids in irrigation water are presented in Table 4.2.10 of the ANZECC 2000 Guidelines.

Homestead water supply	• blue-green algae:
	  – recommend twice weekly inspections during danger period for storages with history of algal blooms
	  – no guideline values are set for cyanobacteria in drinking water
	  – in water storages
	  – counts of < 100 algal cells/mL are of no concern
	  – counts of >500 algal cells/mL - increase monitoring
	  – counts of >2000 algal cells/mL - immediate action indicated; seek expert advice
	  – counts of >6500 algal cells/mL - seek advice from health authority
	• turbidity:
	  – guideline value is 5 NTU
	  – <1 NTU desirable for effective disinfection
	  – >1 NTU may shield some micro-organisms from disinfection
	• total dissolved solids:
	  – < 500 mg/L is regarded as good quality drinking water based on taste
	  – 500 to 1000 mg/L is acceptable based on taste
	  – >1000 mg/L may be associated with excessive scaling, corrosion and unsatisfactory taste
	• faecal coliforms:
	  – zero faecal coliforms per 100 mL (0/100 mL)
	  – if micro-organisms are detected in water, advice should be sought from the relevant health authority
	• pH: 6.5-8.5
	• chemical contaminants: see Guidelines for Inorganic Chemicals in the Australian Drinking Water Guidelines (NHMRC & NRMMC 2004).

Drinking water	• blue-green algae:
	  – recommend twice weekly inspections during danger period for storages with history of algal blooms
	  – counts >500 algal cells/mL – increase monitoring
	  – counts < 2000 algal cells/mL – water may be used for potable supply
	  – counts >2000 algal cells/mL – immediate action indicated; seek expert advice
	  – counts >6500 algal cells/mL – seek advice from health authority
	  – counts >15 000 algal cells/mL – may not be used for potable supply except with full water treatment, which incorporates filtration and activated carbon
	• turbidity: site-specific determinant
	• salinity (electrical conductivity):

Protection Category	Indicator
	– <1500 µS/cm
	– > 800 µS/cm causes a deterioration in taste
	• faecal coliforms: zero faecal coliforms per 100 mL (0/100 mL)
	• total coliforms:
	– 95 per cent of samples should be zero coliforms/ 100 mL throughout the year
	– up to 10 coliform organisms may be accepted occasionally in 100 mL
	– coliform organisms should not be detected in 100 mL in any two consecutive samples
	• dissolved oxygen: > 6.5 mg/L (> 80 per cent saturation)
	• pH: 6.5-8.5
	• chemical contaminants: see ANZECC 2000 Guidelines, Section 6.2.2.
Aquatic foods (cooked)	• algae, including blue-green algae: no guideline is directly applicable, but toxins present in blue-green algae may accumulate in other aquatic organisms
	• faecal coliforms:
	– guideline in water for shellfish: the median faecal coliform concentration should not exceed 14 MPN/100 mL; with no more than 10 per cent of the samples exceeding 43 MPN/100 mL
	– standard in edible tissue: fish destined for human consumption should not exceed a limit of 2.3 MPN *E. coli* /g of flesh with a standard plate count of 100,000 organisms/g
	• toxicants (as applied to aquaculture activities):
	– metals: copper – less than 5 µgm/L; mercury – less than 1 µgm/L; zinc – less than 5 µgm/L
	– organochlorines: chlordane – less than 0.004 µgm/L (saltwater production); PCBs – less than 2 µgm/L
	• physicochemical indicators (as applied to aquaculture activities):
	– suspended solids: less than 40 micrograms per litre (freshwater)
	– temperature: less than 2 degrees Celsius change over one hour.