

Victor Harbor Watercourses Floodplain Mapping Study

City of Victor Harbor



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Appendix A Flood Plain Mapping

1 Introduction

1.1 Background

Southfront have been engaged by the City of Victor Harbor to undertake floodplain and hazard mapping of the major rivers within greater Victor Harbor area, which will form part of a future Stormwater Management Plan. The study areas include the Inman and Hindmarsh Rivers, a catchment in the vicinity of Waterport Drive in western Port Elliot, and the two creek channels in southern Encounter Bay, as shown in Figure 1.1.



Figure 1.1 Floodplain Mapping Scope

This floodplain mapping study has investigated the existing flood risk within these four regions, and briefly explored stormwater management opportunities to improve the existing level of flood protection in key vulnerable areas.

Floodplain mapping of these watercourses was previously undertaken as part of the Victor Harbor Flood Management Plan (KBR, 2010). This study will provide an update to the previous mapping by moving from SOBEK and HEC-RAS models towards TUFLOW, combined with new AR&R rainfall runoff modelling procedures for generating model inflow hydrographs.

Flood inundation depth and flood hazard maps are attached to this report in Appendix A, and provide a visual representation of flood risk throughout the greater Victor Harbor region.

2 Catchment Features

2.1 Study Area

The study area consists of four major channels within the greater Victor Harbor Region; the Inman River, Hindmarsh River, Port Elliot West catchment (Waterport Road) and the Southern Encounter Bay channels.

The study area is largely contained within The City of Victor Harbor, with just a small part of the Port Elliot West model stretching into Alexandrina Council land.

Major catchment regions were delineated and are shown in Figure 2.1. Generally, the catchments upstream of the study area are rural in nature with little development. A larger percentage of residential and commercial land exists closer to the ocean. The rural land upstream is characterised by hilly, open terrain, with grades becoming relatively flat towards the ocean. The catchments for the Inman and Hindmarsh Rivers are very large, extending more than half way across the Fleurieu Peninsula. A summary of catchment sizes is presented in Table 2.1.

Table 2.1 Catchment Size Information

Catchment	Area (km ²)
Inman River	190
Hindmarsh River	112
Port Elliot West Model	8.20
Southern Encounter Bay Model	1.85

The Inman River and Hindmarsh River catchments are very large when compared to that of the Port Elliot West and southern Encounter Bay catchments, as can be seen in Figure 2.1 which shows the extent of each catchment.

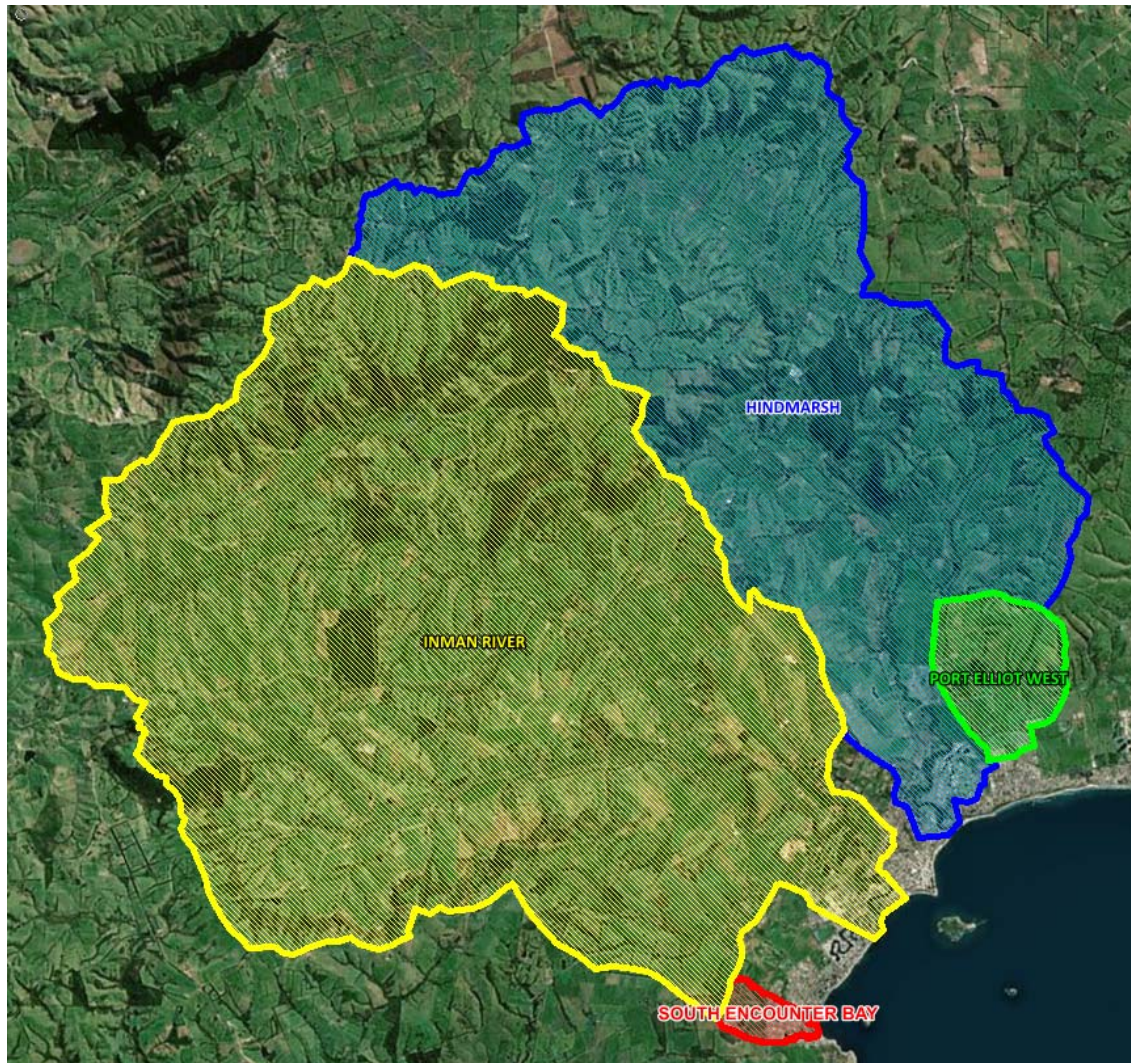


Figure 2.1 Catchment for Each Channel Region

2.2 Topography

A digital terrain model (DTM) was derived from the surface elevation model provided by AAM, and is shown in Figure 2.2. Red-coloured areas have relatively higher elevations than blue-coloured areas, as defined in the legend.

This region is characterised by steep hills to the west of the township, which grade towards the ocean via major rivers and smaller channels. The lower reaches of the catchments are much flatter, and contain both residential and commercial precincts. Water exits the system from the mouths of the Inman and Hindmarsh Rivers, and in southern Encounter Bay through the underground stormwater drainage network. For the region in Port Elliot West, flows cross the council boundary and drains into a series of basins within the Alexandrina Council area.

The surface elevation model shows a region of high elevation separating the Inman and Hindmarsh rivers, and a high-elevation ridge along the western side of Encounter Bay, forcing water out towards the ocean. The urban drainage network predominately flows into the major channels and then out to sea.

Land use within the study area varies with location, with high-density urban residential living occurring closer to the ocean, and a mixture of rural residential and agricultural use in hillier areas further inland. In the Port Elliot West region there is commercial land usage north of Waterport Road, and new urban housing developments around Ocean Road. There are many reserves along the shoreline and around the major rivers, including recreational parks and sports grounds.

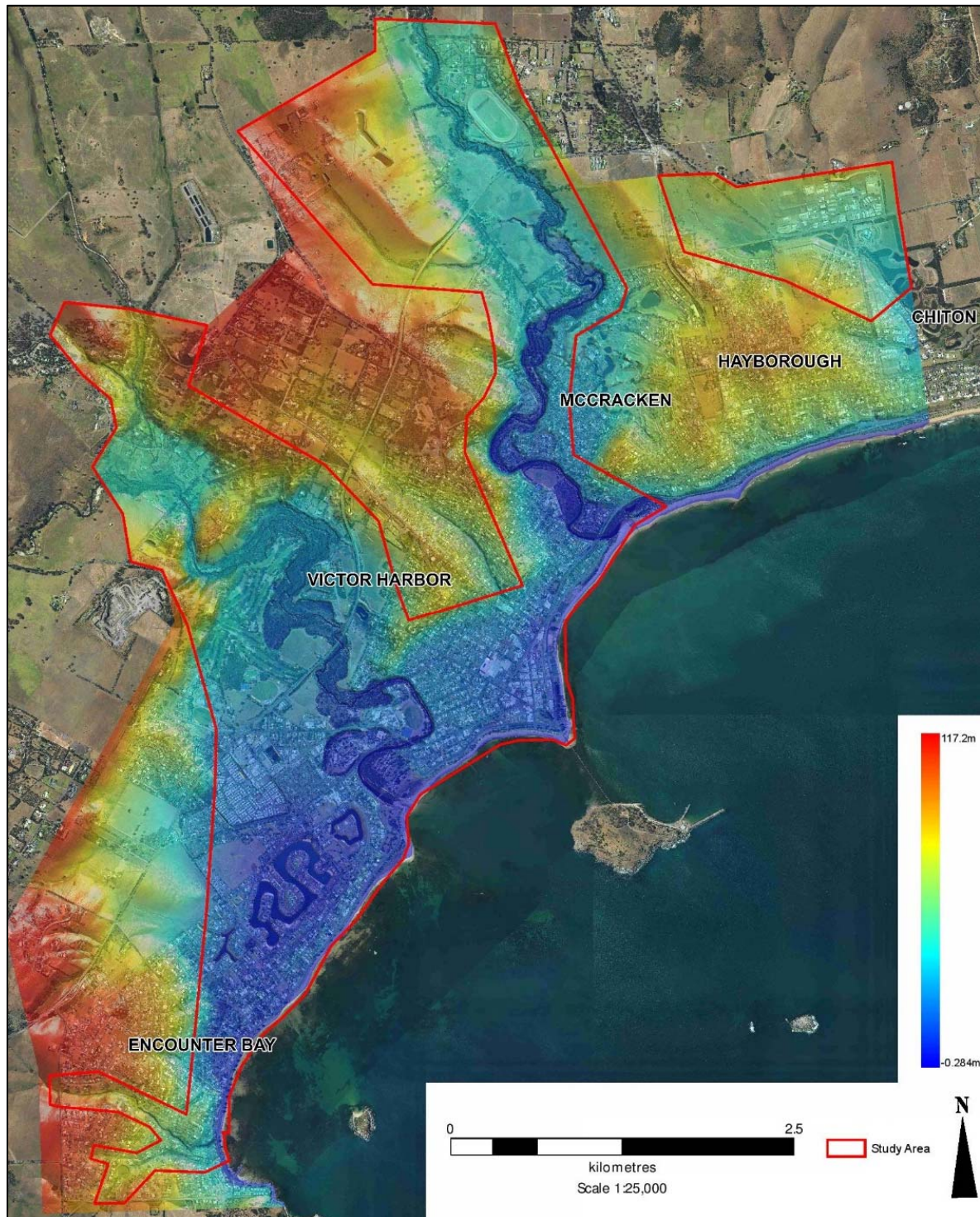


Figure 2.2 City of Victor Harbor Topography

2.3 Existing Structures and Drainage Infrastructure

There are a number of bridges over both the Inman and Hindmarsh rivers, varying in size from local roads over small culverts to multi-span arterial road crossings. Major bridges crossing the rivers are listed in Table 2.2 below.

Table 2.2 Major River Crossings

Bridge Location (road)	Study Area	Crossing Type
Victor Harbor Railway Line	Hindmarsh River	Rail Bridge
Hindmarsh Road	Hindmarsh River	Pedestrian Bridge
Hindmarsh Road	Hindmarsh River	Road Bridge
Wattle Drive	Hindmarsh River	Pedestrian Bridge
Bashams Road	Hindmarsh River	Pedestrian Bridge
Welch Road	Hindmarsh River	Road Bridge
Encounter Bikeway	Inman River	Pedestrian Bridge
George Main Road	Inman River	Road Bridge
Kullaroo Road	Inman River	Pedestrian Bridge
Armstrong Road	Inman River	Road Bridge
Swains Crossing Road	Inman River	Road Bridge
Cartwright Road	Inman River	Road Bridge
Stock Road	Inman River	Road Culvert
Ocean Road	Port Elliot West	Road Culvert

In the southern Encounter Bay region, there is a network of drainage infrastructure including detention basins, side entry pits and underground stormwater pipes, as shown in Figure 2.3. There is a similar arrangement in the Port Elliot West model, which is shown in Figure 2.4.

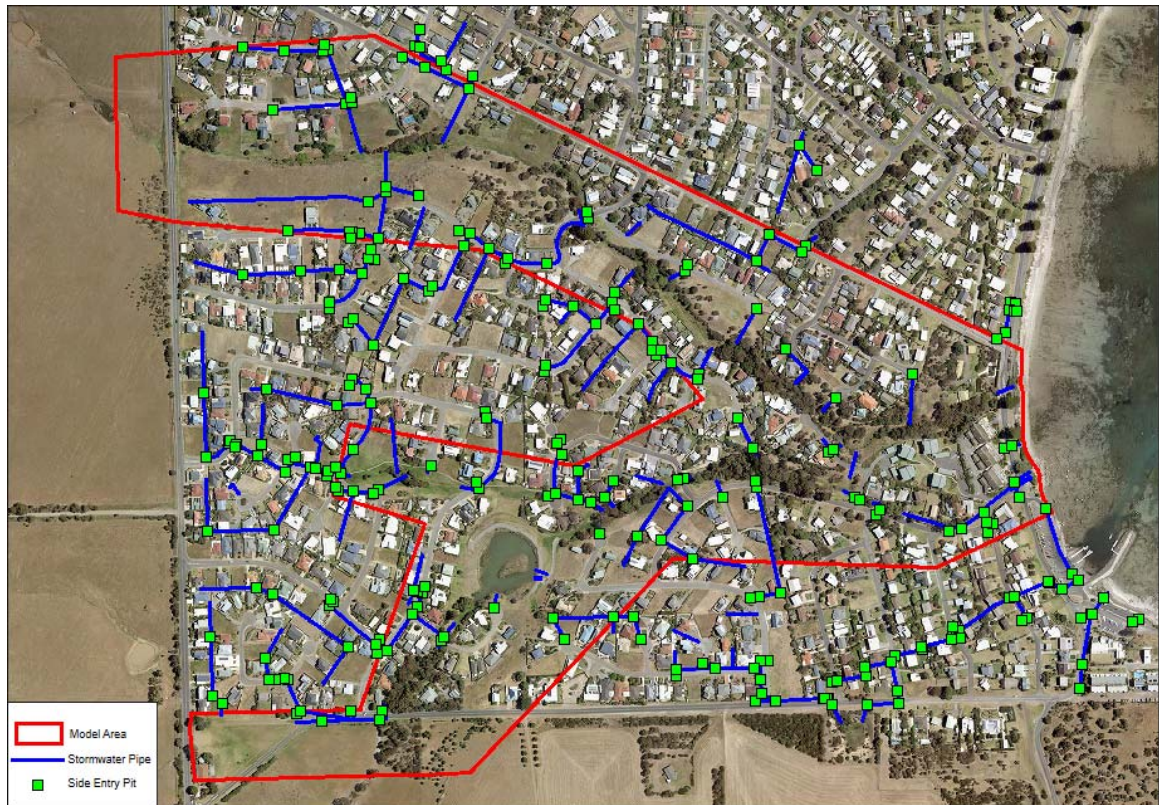


Figure 2.3 Southern Encounter Bay Stormwater Infrastructure

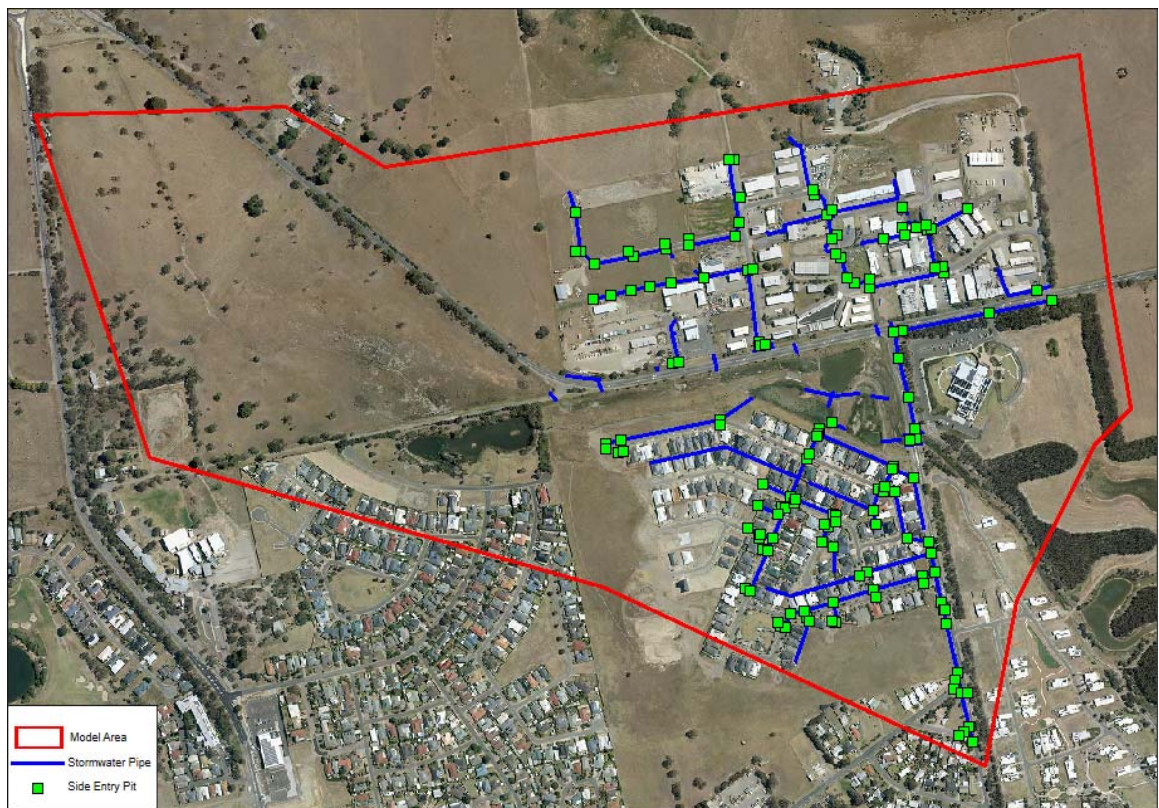


Figure 2.4 Port Elliot West Stormwater Infrastructure

2.4 Previous Studies

Flood modelling of this area has been undertaken previously, including:

- *Hindmarsh River Flood Study* by Connell Wagner (1998)
- *Flood Mapping of the Inman River* at Victor Harbor by Tonkin Consulting (2001)
- *Victor Harbor Flood Plain Mapping* by KBR (2010).

The results from these studies were used to compare and validate the findings of this study.

3 Hydrology

3.1 Modelling Approach

Hydrological modelling of the river systems was undertaken using two types of flood routing modelling software, including:

- RORB – to generate inflow hydrographs for the Inman River, Hindmarsh River and the Port Elliot West catchment; and
- DRAINS – to generate inflow hydrographs throughout the smaller, urban subcatchments of the southern Encounter Bay catchment;

Both the above hydrological models use design rainfall event-based approaches for the transformation of rainfall into a flood hydrograph using a simplified model of the physical processes involved. The flood hydrograph is then applied in the two-dimensional flood modelling software to represent the maximum floodplain extent for a given rainfall event.

Surface water gauges exist on the Inman River and Hindmarsh River. Streamflow data from these gauges enabled Flood Frequency Analysis (FFA) of the major river systems allowing for calibration and validation of input parameters into the RORB hydrological models (see Section 3.3).

Regional Flood Frequency Estimation (RFFE) of the major catchments enabled further validation of the hydrological models of the peak flows generated for each Annual Exceedance Probability (AEP) (see Section 3.4).

Hydrological modelling of the large rural Inman River, Hindmarsh River and Port Elliot West catchments are the primary focus of this section. Analysis of the Encounter Bay urban catchment was undertaken in DRAINS and is outlined in Section 3.9

The flood events modelled for this study include the 5% AEP (1:20 year), 2% AEP (1:50 year), 1% AEP (1:100 year) and 0.2% AEP (1:500 year) as well as the Probable Maximum Flood (PMF).

3.1.1 Data Sources

Development of design flood hydrographs for each river system was undertaken using a combination of the following resources:

- Bureau of Meteorology (BoM) for Intensity-Frequency-Duration (IFD) rainfall data;
- Australian Rainfall and Runoff (ARR) 'Data Hub' for temporal patterns, areal reduction factor (ARF) parameters and storm losses relevant to the catchment areas;
- WaterConnect (Enviro Data SA) for streamflow data for the Inman and Hindmarsh Rivers;
- The Regional Flood Frequency Estimation (RFFE) online tool developed for Australian Rainfall and Runoff 2016; and
- Review of previous hydrological studies of the river systems.

3.1.2 IFD Rainfall Data

In 2016 the Australian Rainfall and Runoff (ARR) guideline update was released. The update (the first major revision since 1987) was the result of an improved understanding of the Australian

rainfall landscape, gained through collection and analysis of 30 years of additional rainfall data from over 8,000 rainfall gauges across the nation. By combining contemporary statistical analyses and techniques with an expanded database, the new 2016 Intensity-Frequency-Durations (IFDs) were developed which are able to provide more accurate design rainfall estimates for Australia (Rainfall IFD Data System, Bureau of Meteorology, 2017).

The new design rainfall estimation procedures have been applied to the hydrological modelling of the major rivers of the greater Victor Harbor Region. Key variations from the previous study include the new 2016 Intensity-Frequency-Duration (IFD) dataset, updates to applied temporal patterns (a suite of region specific temporal patterns, rather than a single pattern) and changes to the rainfall probability terminology. The main term used to describe 2016 design rainfalls is Annual Exceedance Probability (AEP); the probability or likelihood of an event occurring or being exceeded within any given year, expressed as a percentage.

Table 3.3 lists the probability terminology used for the 2016 design rainfalls. The rainfall frequency descriptor defines the temporal pattern 'bin' (frequent, infrequent and rare) which are applied to the design rainfalls. Table 3.1 explains the new terminology with reference to the rainfall durations that have been used for this study.

Table 3.1 Rainfall and Runoff Terminology Comparison

Australian Rainfall and Runoff Design Rainfall Terminology			
ARR 1987 ARI (years)	ARR 2016 AEP (%)	ARR 2016 Exceedances per Year (EY)	Frequency Descriptor
20 year	5	0.05	Infrequent
50 year	2	0.02	
100 year	1	0.01	
500 year	0.2	0.002	Rare
PMF	N/A	N/A	Extreme

Design IFD data has been obtained from the Bureau of Meteorology (BoM) for use within RORB and DRAINS to generate inflow hydrographs. Rainfall intensity data obtained for the Hindmarsh River catchment is presented in Table 3.2.

Table 3.2 IFD Design Rainfall Intensities (mm/hr) for Hindmarsh River Catchment

Duration	Annual Exceedance Probability (AEP)			
	5%	2%	1%	0.2%
30 min	21.80	29.40	35.20	41.20
45 min	17.20	23.07	27.47	32.13
1 hour	14.40	19.40	23.10	27.00
1.5 hour	11.33	15.20	18.13	21.20
2 hour	9.55	12.85	15.30	17.90
3 hour	7.50	10.10	12.10	14.17
4.5 hour	5.89	7.98	9.56	11.24
6 hour	4.97	6.75	8.10	9.53
9 hour	3.89	5.31	6.38	7.52

Duration	Annual Exceedance Probability (AEP)			
	5%	2%	1%	0.2%
12 hour	3.26	4.46	5.36	6.32
18 hour	2.53	3.46	4.15	4.88

A comparison of the updated 2016 rainfall dataset to the previous 1987 data indicates variations of over $\pm 20\%$ for certain rainfall intensities. Table 3.3 below demonstrates relatively significant reductions in rainfall intensity for the shorter duration storms, while the longer duration (greater than 6 hours) tend to show a relatively significant increase (particularly for the 12 and 24 hour storms).

Table 3.3 Percentage Change between 1987 and 2016 Rainfall Intensity Values

Duration	Annual Exceedance Probability (AEP)		
	5%	2%	1%
30 min	-15.4%	-20.0%	-23.3%
1 hour	-13.7%	-17.8%	-20.9%
2 hour	-8.2%	-11.6%	-13.6%
3 hour	-3.6%	-5.7%	-7.2%
6 hour	+5.6%	+5.6%	+6.0%
12 hour	+13.2%	+15.5%	+17.6%
24 hour	+16.3%	+18.9%	+20.5%
48 hour	+15.1%	+15.1%	+16.1%
72 hour	+15.1%	+13.6%	+13.2%

3.1.3 Australian Rainfall and Runoff – Data Hub

The Australian Rainfall and Runoff Data Hub is an online tool that allows for convenient access to the design inputs required to undertake flood estimation in Australia. By entering the catchment centroid location (latitude and longitude), the tool will output catchment specific parameters, such as storm losses (initial loss and continuing loss), Areal Reduction Factors and Temporal Patterns. The output parameters are able to be read directly into the RORB model and were used as a starting point before calibration/verification of the model was undertaken.

Data Hub suggests the following losses for the Victor Harbor rural catchments:

- Initial Loss: 23 mm
- Continuing Loss: 4.7 mm/hr

3.2 RORB Hydrological Modelling

Hydrological modelling of the Inman River, Hindmarsh River and Port Elliott West Catchments was undertaken using RORB. RORB is a computer based general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce runoff hydrographs at any location (RORB manual, V2, 2010).

Subcatchments and reach lengths were delineated for each of the river systems based on 5 metre contours provided by Council. All catchments were assumed to be rural with 0%

impervious fraction (urban areas downstream were found to have a negligible impact on hydrological results for the events modelled). Reach slopes ranged from to 7% in the upper reaches of the catchments down to 0.2% at the downstream end.

3.2.1 *kc* and *m*

There are two important modelling parameters used in RORB; '*kc*' and '*m*'. *m* is a dimensionless exponent used to measure of the catchment's non-linearity and *kc* is a dimensionless coefficient related to the storage within the catchment. The values used for *kc* and *m* influence the size, shape and profile of an outlet hydrograph and are dependent on catchment features and stream lengths. The value of *kc* is very dependent of the value of *m* and is the principal parameter used in RORB in order to calibrate the model. Typically an *m* value of 0.8 is used (ARR 2016).

Comprehensive hydrological modelling of the Hindmarsh and Inman River catchments was undertaken in 2005 by KBR as part of the *Victor Harbor Flood Plain Mapping and Urban Stormwater Management Plan*. That study undertook RORB model calibration against historical streamflow and rainfall event data within the Inman and Hindmarsh Rivers, as well as the neighbouring Currency Creek and Myponga River catchments. A trial and error method using 'FIT' runs of the RORB model was undertaken in order to calibrate model exponents and determine appropriate values for parameters *kc* and *m* for each of the river systems.

That study adopted the parameter values shown in Table 3.4.

Table 3.4 **Adopted *kc* and *m* Parameters (KBR, 2005)**

Catchment	<i>kc</i>	<i>m</i>
Inman River	15	0.8
Hindmarsh River	20	0.9

A review of the previous modelling was undertaken using the RORB models developed for this study as a basis for comparison. This review found an appropriate approach towards model calibration of parameters *kc* and *m* for both the Hindmarsh River and Inman River catchments. Model calibration assumptions were consistent with those recommended by the updated ARR 2016 guideline. It should be noted that an *m* value of 0.9 was selected for the Hindmarsh River, which provided a better fit for output hydrographs based on observed events. These values were therefore adopted for the remaining model runs for this study.

The Port Elliot West catchment was not modelled with RORB under the previous Study. Australian Rainfall and Runoff suggests the following regional relationship developed for ungauged systems in the south-east region of South Australia (for catchments of less than 100km²):

$$K_C = 0.6A^{0.67} \text{ (when } m = 0.8 \text{)}$$

A *kc* value of 2.46 with an *m* value of 0.8 was therefore adopted on the basis of the above formula. It should be noted that this formula was also used to develop the Inman River *Kc* value, which proved difficult to calibrate with the available streamflow/rainfall data.

3.2.2 Loss Model

ARR 2016 currently recommends the use of the IL/CL model as the most suitable for design flood estimation for both rural and urban catchments (ARR, Book 5, 2016). This loss model is based on two parameters; the Initial Loss (IL) (the water required to wet up the catchment) and Continuing Loss (CL). The output of these models is the rainfall excess that is then used to generate a direct flow hydrograph.

This parameter configuration was selected as the loss model to be used within RORB. The IL/CL model uses constant values for initial loss and continuing loss for a flood event. The exact parameters used were obtained from ARR Data Hub and were updated through calibration, validation and Monte Carlo scenario modelling of the hydrological model.

3.2.3 Monte Carlo Simulation

Design flood estimation within RORB was undertaken using the Monte Carlo simulation method for each catchment. The Monte Carlo approach involves undertaking thousands of simulations where the stochastic factors (such as rainfall, temporal patterns and initial loss) are sampled to represent the joint probability of such factors to provide a more realistic representation of the flood peak.

The Monte Carlo method was selected as it is a recognition that design floods (e.g. peaks flows) can result from a variety of combinations/factors, rather than from a single combination as is assumed with the typical 'design event' approach. For example, the same peak flood could result from a large, front-loaded storm on a dry catchment, or a moderate, more uniformly distributed storm on a saturated catchment.

The simulation used a range of rainfall depths, durations, temporal patterns and initial losses to produce a flood frequency distribution curve. Initial loss values are taken by sampling within an expected variability range of the original value (i.e. 23 mm), while rainfall depths/durations, temporal patterns and areal reduction factors are drawn from the information sourced by Data Hub. The flood frequency distribution curve is used to estimate the peak flow for each AEP event and identify critical storm durations and temporal patterns within the Monte Carlo result output.

Flood frequency curves were developed based on the kc , m and IL/CL parameters outlined above (based on initial input values), with the output locations located at the surface flow gauging stations on the Inman and Hindmarsh Rivers. Peak flows recorded from the flood frequency distribution curves are outlined below with the critical storm duration.

Table 3.5 Initial Verification Monte Carlo Simulation Peak Flow Results (at gauge site)

Catchment	Peak Flow from Flood Frequency Curve Peak Flows (m ³ /s)			
	5%	2%	1%	0.2%
Hindmarsh River	25 (12 hour)	41 (18 hour)	57 (18 hour)	110 (18 hour)
Inman River	92 (18 hour)	170 (18 hour)	250 (18 hour)	450 (36 hour)

RORB output at the gauge sites was selected such that direct comparison of the results to the Flood Frequency Analysis (as outlined below) could be undertaken. These models are to be referred to as the verification models. Final results will consider outputs from the full catchment.

3.3 Flood Frequency Analysis

Regional Flood Frequency Analysis (FFA) involves the fitting of a probability model to recorded streamflow annual flood peaks to determine the relationship between event magnitude and exceedance probability.

Flood frequency methods avoid the need to consider the complex processes and joint probabilities involved in the transformation of rainfall into flood. However, this method is heavily dependent on both the length of available record and its representativeness to the catchment and climatic conditions of interest, as they are based on the assumption of stationary data series (ARR 2016). FFA was undertaken in order to validate results of the Monte Carlo simulation and refine input parameters to ensure accurate representation of the flood peak.

3.3.1 Streamflow Data

Flood Frequency Analysis has been undertaken for the Inman River and Hindmarsh River catchments. Daily streamflow data (in ML/day) was obtained from the WaterConnect website at the sites described in Table 3.6.

Table 3.6 Surface Flow Gauge Sites

Site ID	Location	Record Period
A5011027	Hindmarsh River, upstream of estuary (Welch Road)	1969 – 2016 (47 years)
A5010503	Inman River, upstream of Victor Harbor Sewerage Treatment Works	1995 – 2019 (24 years)

The data obtained at the gauge sites was simplified into annual maximum flows (in m³/s) and ranked in accordance to magnitude. It should be noted that the largest annual flow within the Inman River was in 2016 (with 24 years of data), while the largest recorded flow within the Hindmarsh River was recorded in 1971 (with 47 years of data – the 2016 event was ranked as the eighth largest).

It should also be noted that the WaterConnect website logs Historic Site Status for each of the gauge sites. In September 2016 the Inman River site reported the following:

“Levels currently exceed gauged levels and flow at site. As a result, flow data will be missing during this time.”

The date of that occurrence coincides with the largest flow recorded at the Inman River site (to date) and is likely an indication that flow recording ceased prior to the peak of the flood (and is therefore under-recorded).

3.3.2 FFA Modelling Approach

Flood Frequency Analysis for the two sites was undertaken using TUFLOW-FLIKE.

The Log Pearson Type III (LP3) probability model was selected with the Bayesian fit method.

The probability model fits are shown in Figure 3.1 and Figure 3.2 below. The plots are presented in the Gumbel scale such that results of the FFA can be compared directly to those of the Monte Carlo flood frequency distribution curves.

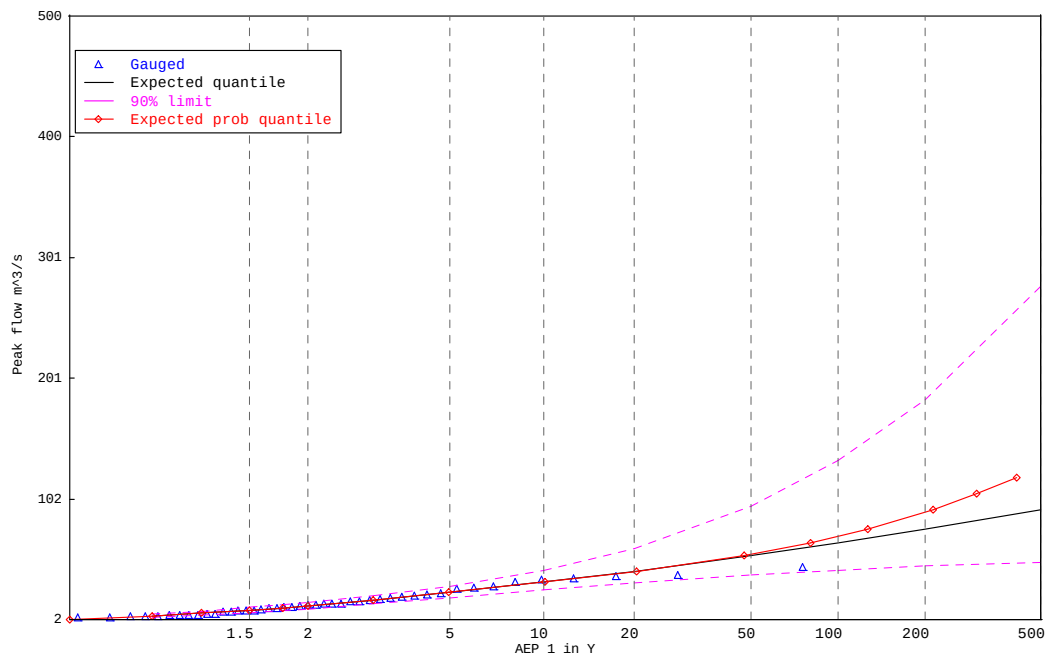


Figure 3.1 Hindmarsh River FFA Probability Curve TUFLOW-FLIKE Output

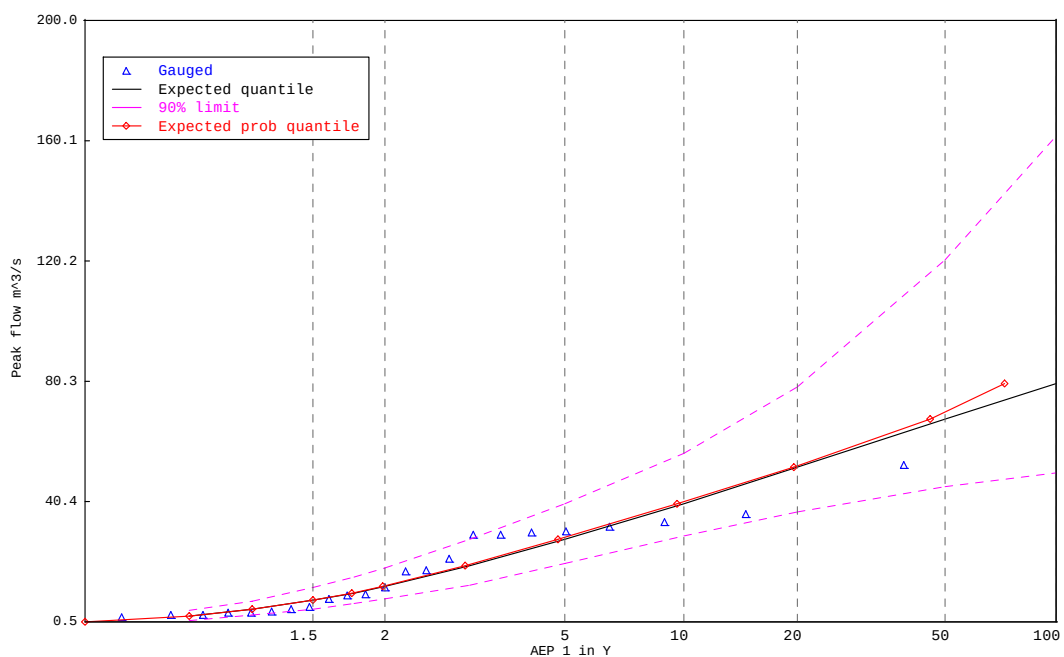


Figure 3.2 Inman River FFA Probability Curve TUFLOW-FLIKE Output

Review of the Hindmarsh FFA curve indicates relatively tight upper and lower confidence limits up to the 1% AEP when compared to the Inman River results. The FFA results for key AEP events are summarised in Table 3.7 below and compared to that of the initial Monte Carlo simulation flood curve distribution results.

Table 3.7 FFA Results Summary and Comparison

Catchment	AEP	FFA Expected Probability Quantile Peak Flow (m ³ /s)	Monte Carlo (initial run)
Hindmarsh River	5%	42	25
	2%	54	41
	1%	66	57
Inman River	5%	51	92
	2%	70	170
	1%	89	250

Comparison of the FFA results and the initial Monte Carlo simulation runs indicate a close correlation within the Hindmarsh River model. Inman River however is shown to differ by a relatively large magnitude. This is likely due to limitations of the Inman River input streamflow data, potentially missing records during large flow events and the relatively short record period.

It should be noted that the Monte Carlo results for the Hindmarsh River are within the upper and lower confidence limits of the FFA results, as shown in Figure 3.1. The Inman River results however far exceed the FFA upper limit – although the FFA results are questionable.

3.4 Regional Flood Frequency Estimation (RFFE)

Regional Flood Frequency Estimation (RFFE) is an online tool developed for Australian Rainfall and Runoff. The RFFE technique is a data-driven approach, which attempts to transfer flood characteristics from a group of gauged catchments to ungauged locations of interest where design floods need to be estimated. A range of different methods are available to extract regional flood information from the pooled data and to transfer the relevant information to an individual ungauged catchment in the region. The RFFE techniques use the results of at-site Flood Frequency Analysis as basic data.

RFFE analysis was undertaken for the Hindmarsh and Inman River Catchments. Those results (and the upper and lower confidence limits) are shown in Table 3.8 below.

Table 3.8 RFFE results

Catchment	AEP	Peak Flow (m ³ /s) (Expected Quantile)	5% Confidence Limit (m ³ /s)	95% Confidence Limit (m ³ /s)
Hindmarsh River	5%	37	13	140
	2%	55	16	206
	1%	74	18	273
Inman River	5%	77	23	255
	2%	108	29	385
	1%	135	34	517

Expected peak flow quantiles and confidence limit curves are shown in Figure 3.3 and Figure 3.4. It should be noted that the Hindmarsh River results show very close correlation between RFFE and FFA results and minor difference between the RORB Monte Carlo simulations. The Inman River results however show a relatively large spread between the three set of results, with wide confidence limits in both the FFA and RFFE results.

3.5 Evaluation of Design Flows

An evaluation of the initial verification RORB model was undertaken such that the fit between expected flood peaks could be aligned between the different estimation methods. The verification RORB model was updated by modifying the input losses (initial and continuing) before rerunning the Monte Carlo simulation. It should be noted that the model performs variable sampling of initial loss values (amongst other stochastic factors), while continuing loss is modelled as a constant.

The Hindmarsh River model was shown to provide the best fit to FFA and RFFE flows, with the peak flows of the RORB Monte Carlo simulations slightly lower than those shown for RFFE and FFA results. Loss values within the model were reduced to enable a better fit by trial and error.

Revised Monte Carlo simulation modelling indicated that the best fit to RFFE and FFA results occurred with the following loss values:

- Initial loss: 23 mm (starting value)
- Continuing loss: 3 mm/hr (reduced from 4.7 mm/hr)

The revised flood frequency curves are shown in the figure below with all three models showing very close correlation. A summary of the values for comparison is also shown in Table 3.9.

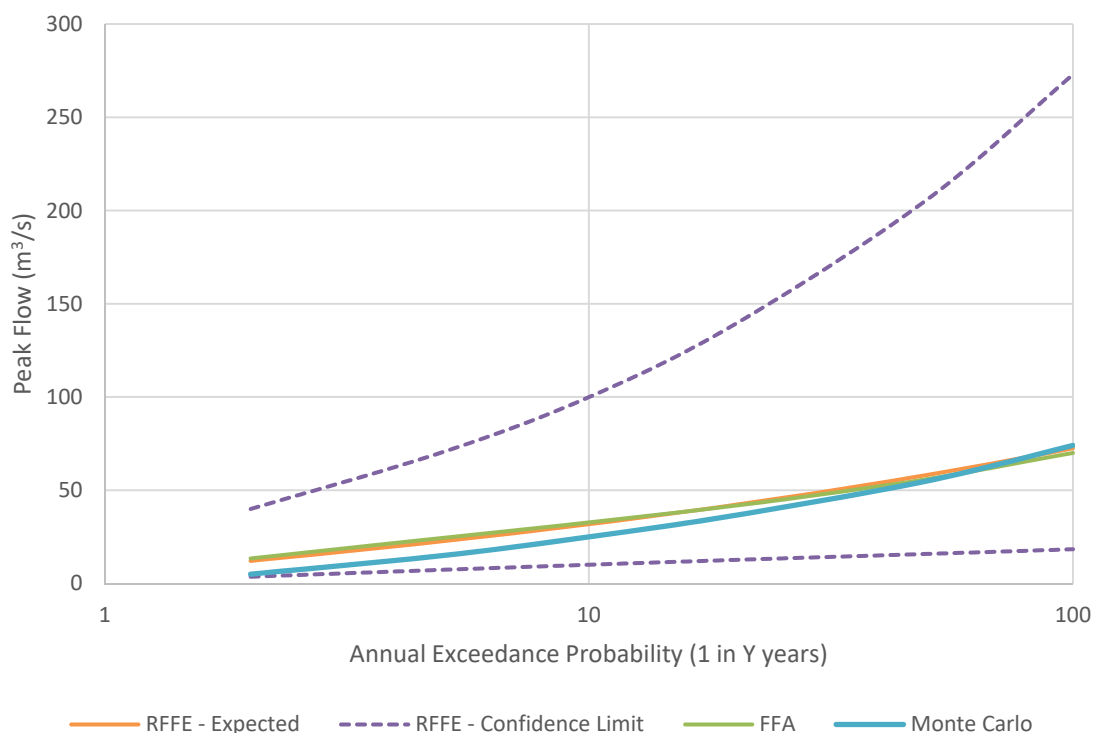


Figure 3.3 Hindmarsh River Flood Frequency Curve Comparison

The Inman River proved to be more difficult to validate based on FFA and RFFE results. Loss values within the RORB model were increased significantly (even unrealistically) to match the results of the FFA and RFFE models. In addition, confidence in the FFA results was questionable given the shorter record period and errors in reporting large flow events.

Loss values for the Monte Carlo simulation of the neighbouring Hindmarsh River RORB model were therefore adopted for the Inman River model (i.e. 23 mm initial loss, 3mm/hr continuing loss). It should be noted that a 3mm/hr initial loss was adopted for the Inman River in previous KBR, Tonkin and Connell Wagner hydrological models.

The Inman River flood frequency curves are shown in Figure 3.4. As noted, the results are moderately higher than those of the FFA and RFFE results. However it should be noted that they fall in the mid-range of the confidence limits of the RFFE assessment.

A summary of the values for comparison is shown in Table 3.9.

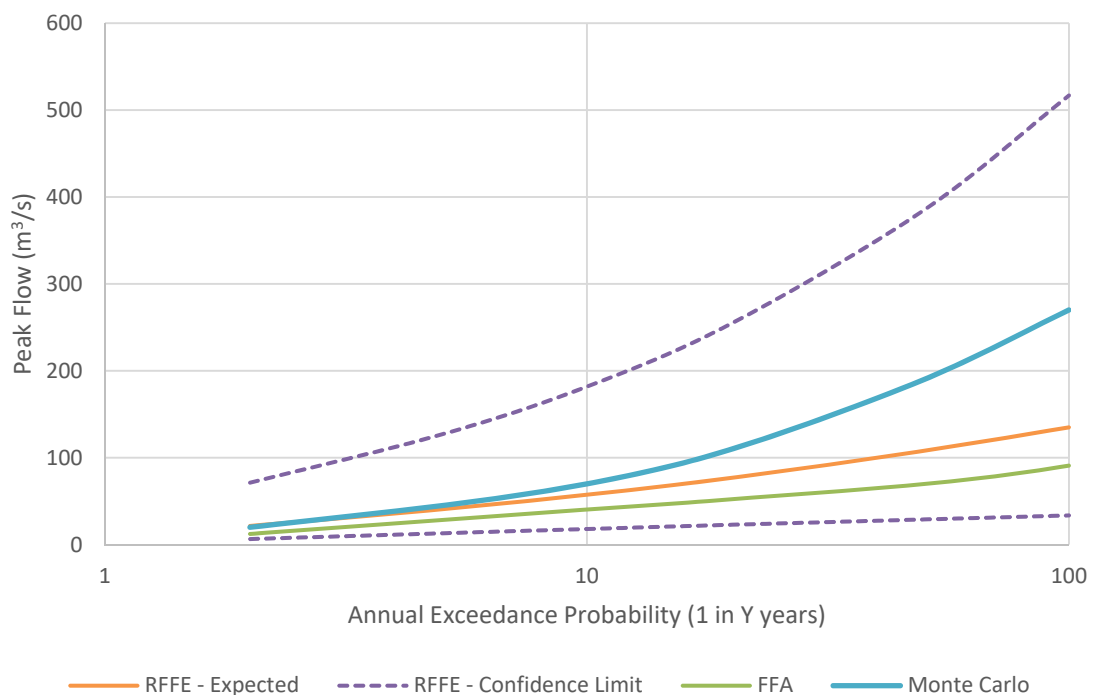


Figure 3.4 Inman River Flood Frequency Curve Comparison

Comparison values between the RORB models (initial and revised verification models) and the FFA and RFFE results are shown in Table 3.9.

Table 3.9 Revised RORB Verification Model Results and Comparison

Catchment	AEP	Peak Flow (m ³ /s)				
		Monte Carlo (initial verification model)	Flood Frequency Analysis	Regional Flood Frequency Estimation (RFFE)	Monte Carlo (revised verification model)	2005 KBR Study ARR 1987 (at gauge station)
Hindmarsh River	5%	25	42	36.5	35	43
	2%	41	54	55	52	52
	1%	57	66	74	70	59
	0.2%	110	130	-	132	-
Inman River	5%	92	51	77	110	112.2
	2%	170	70	108	190	172.1
	1%	250	89	135	270	220.4
	0.2%	450	156	-	495	-

It should be noted that these values are slightly higher than the predicted peak flows used in the 2005 KBR study. This is likely due to a number of factors, including; revised modelling techniques, updated 2016 rainfall data (up to 20% higher intensities in the longer duration storms) and new rainfall temporal patterns.

3.5.1 RORB Model Revision

The RORB model was modified to output flow data taking into account all catchments (including those downstream of the gauge sites) to be used for hydraulic 2D modelling. Those results are shown in Table 3.10.

Table 3.10 RORB Model Peak Flow Results (full catchment)

Catchment	AEP	Peak Flow (m ³ /s)	
		Monte Carlo (revised model, output catchment outlet)	2005 KBR Study ARR 1987
Hindmarsh River	5%	50	50
	2%	73	67
	1%	96	87
	0.2%	220	225
Inman River	5%	134	140
	2%	221	203
	1%	302	225
	0.2%	538	580

3.6 Probable Maximum Flood

The Probable Maximum Flood (PMF) was modelled for all four catchments. The PMF is calculated by first determining the Probable Maximum Precipitation (PMP). For the Victor Harbor region the Generalised Short Duration Method (GSDM) for storms up to 3 hours and the Generalised Southeast Australia Method for storms of between 24 – 72 hours was used.

The PMP rainfall envelope is shown in Figure 3.5 below.

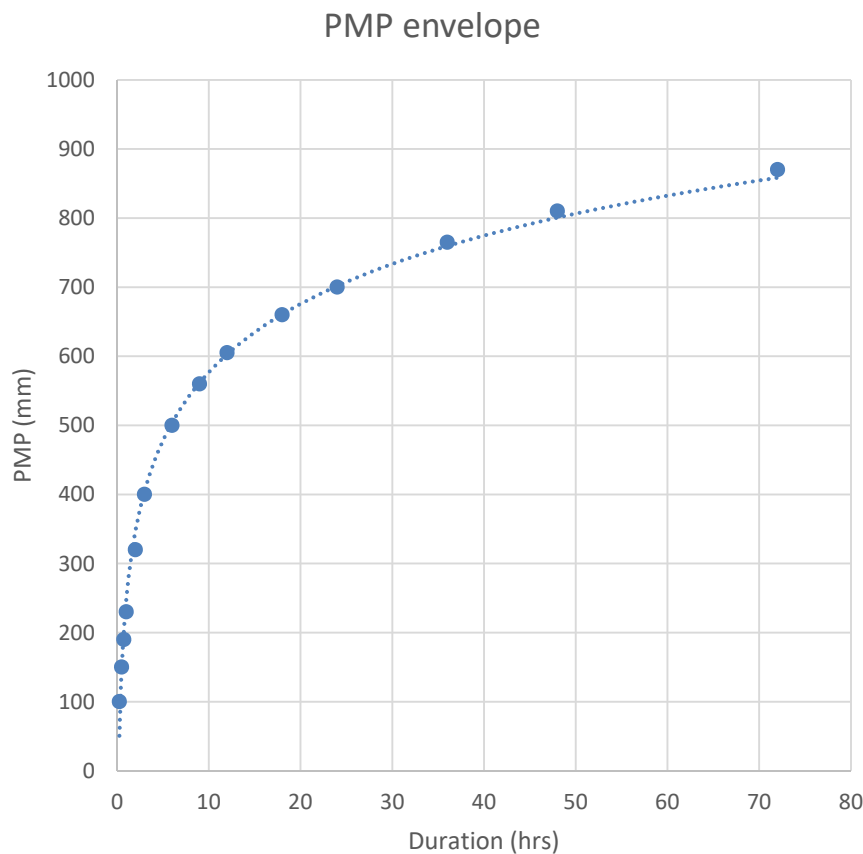


Figure 3.5 PMP Envelope

Flood depths were inserted into the RORB model (as rainfall intensifies). Losses were assumed to be low (i.e. 15mm initial loss and 0.1 mm/hr continuing loss) for all catchments.

PMF peak flows are outlined in Table 3.11.

Table 3.11 PMF flows

Catchment	PMF Peak Flow (m ³ /s)	Duration
Hindmarsh River	1140	12 hours
Inman River	4800	3 hours
Port Elliot West	400	1 hour

The PMF peak flows for the southern Encounter Bay catchment were calculated as described in Section 3.9.4.

3.7 Results Summary

The Monte Carlo model results were analysed, and initial losses and temporal patterns were selected based on the peak flows listed in Table 3.12. Individual storms were extracted from the RORB model with the parameters outlined in Table 3.12.

Table 3.12 RORB Results Peak Flow Summary

Catchment	AEP	Initial Loss (mm)	Continuing Loss (mm/hr)	Peak Duration	Selected Temporal Pattern (1 – 10)	Peak Flow (m ³ /s)
Inman River	5%	12	3	36 hours	1	134
	2%	17	3	36 hours	1	221
	1%	20	3	18 hours	10	302
	0.2%	35	3	18 hours	1	538
	PMF	15	0.1	3 hours	-	4800
Hindmarsh River	5%	14	3	18 hours	3	50
	2%	16	3	18 hours	8	73
	1%	21	3	18 hours	3	96
	0.2%	35	3	18 hours	2	220
	PMF	15	0.1	12 hours	-	1140
Port Elliot West	5%	12	3	2 hours	4	16
	2%	17	3	2 hours	4	23
	1%	19	3	2 hours	4	28
	0.2%	25	3	2 hours	4	46
	PMF	15	0.1	1 hour	-	400

As shown, critical durations range from 18 to 36 hours for Inman River (and 3 hours for the PMF). The 18 hour storm was critical for all Hindmarsh River AEP events (besides the PMF which had a critical duration of 12 hours), while the 2 hour storm was critical for the Port Elliot West catchment. Initial losses for the selected storms varied from 12 mm up to 35mm for corresponding temporal patterns for most catchment models.

3.8 DRAINS Hydrological Modelling

The southern catchment hydrology was modelled using DRAINS. As described in the model documentation (Watercom, 2018), DRAINS is a multi-purpose Windows program for designing and analysing urban stormwater drainage systems and catchments. DRAINS can model drainage systems of all sizes, from small to very large. Working through a number of time steps during the course of a storm event, it converts rainfall patterns to stormwater runoff hydrographs and routes these through networks of pipes, detention basins, channels and stream.

The total area flowing out into the ocean through the two major channels in this model was delineated into smaller subcatchments for migration with DRAINS. The model was run for various durations from 30 minutes to 12 hours, under 10 distinct temporal patterns, for 5%, 2% 1% and 0.2% AEP events as well as the PMF. This allowed the critical storm duration and temporal pattern for each AEP event to be determined, and then inflow hydrographs for each subcatchment could be produced. These hydrographs were used as inputs for the hydrodynamic model (discussed in greater detail in Section 4.2.7), and added directly into the channels at the location of stormwater pipe outlets or where the subcatchment would naturally drain to.

The model is largely comprised of urban residential land, however there are two larger, rural subcatchments draining into the top of the channel through culverts under Three Gullies Road. The boundaries of subcatchments were determined by analysing the DTM and Council provided aerial photography and contours of the area.

3.9 DRAINS Parameters

3.9.1 Impervious Areas and Runoff Coefficients

Sample sub-areas were selected to assess impervious site coverage within the southern model region. The selected sub-areas are summarised in Table 3.13, with one displayed in Figure 3.6 as an example of the process used. These sample sub-areas were selected as they were considered representative of the impervious site coverage of the surrounding area. Where there were empty plots of land, the polygons from existing properties were duplicated, to simulate future development within the subcatchment.



Figure 3.6 Impervious Fraction, Davies St Sub-Area

Table 3.13 Impervious Fraction, Sample Sub-Areas

Sub-area	Impervious (%)	Pervious (%)
Davies Street Subcatchment	51	49
Horizon Way Subcatchment	51	49

This split was further broken down into directly connected and indirectly connected impervious area, and these values used within DRAINS to determine the volume of runoff.

The catchment coverage characteristics for a ‘typical’ residential subcatchment for these same sub-areas are summarised in Table 3.14. These values were then tweaked on a subcatchment by subcatchment basis, depending on the land cover identified in aerial photography.

Table 3.14 Impervious Fraction Splits, Sample Sub-Areas

Sub-area	Directly Connected Impervious (%)	Indirectly Connected Impervious (%)	Pervious (%)
Davies Street Subcatchment	46	5	49
Horizon Way Subcatchment	45	6	49

For the non-residential catchments upstream of Three Gullies Road, runoff coefficients were determined based on land use and through visual inspection of aerial photography. These larger, rural subcatchments were given pervious percentages greater than 95%, comprising mostly empty land with sporadic houses and sheds.

3.9.2 Hydrological Model

The ILSAX model has been selected as the hydrological model within DRAINS, with depression storages of:

- Paved = 1 mm;
- Supplementary paved = 1 mm; and
- Grassed = varied depending on ARI.

3.9.3 Times of Concentration

Times of concentration for the urban subcatchments were estimated by determining the longest flow path to the outlet (generally from the furthest corner of the subcatchment). Given the small size of the subcatchments, the time of concentration was typically between 5 and 10 minutes for impervious surfaces, with an additional 15 minutes applied for pervious surfaces.

3.9.4 Probable Maximum Flood

In determining the PMF flows in this catchment, the DRAINS model required a number of changes. The hydrological model was modified such that the pervious area storage was reduced to 15 mm, and the soil parameters including antecedent rainfall depths and Horton's parameters were all the minimum allowable value of 0.1. Rainfall time series for a range of durations were manually entered into DRAINS, after having been calculated externally.

4 Hydraulic Modelling

4.1 Overview

Hydraulic modelling has been undertaken to define flood depths and extent for the 5%, 2%, 1% and 0.2% AEP flood events, as well as the probable maximum flood (PMF) event. Flood hazard maps have also been produced for each model for the 2%, 1% and 0.2% AEP flood events.

In the previous flood modelling undertaken, integrated 1D/2D hydrodynamic modelling software was used for only the largest catchments, and not for the channels in southern Encounter Bay. These channels have been modelled in this study using a combination of DRAINS and TUFLOW, bringing this model up to industry standard. Further, in this catchment in the previous study, localised rainfall on the catchment itself was ignored. In this study, the use of DRAINS provides further accuracy in these channels by modelling the rainfall caused by the residential land surrounding them.

4.2 Software Selection

Hydraulic floodplain modelling was carried out using the TUFLOW (and ESTRY) computer program jointly funded and developed by BMT WBM and The University of Queensland in 1990. TUFLOW (Two-dimensional Unsteady FLOW) is specifically orientated towards establishing flow and inundation patterns in coastal waters, estuaries, rivers, floodplain and urban areas where the flow behaviour is essentially 2 dimensional (2D) in nature and cannot or would be awkward to represent using a 1 dimensional (1D) model (BMT WBM, 2010).

The TUFLOW and ESTRY computational engines use third party software as their interface. These software are typically a text editor (eg. Notepad), a GIS platform (eg. MapInfo), 3 dimensional (3D) surface modelling software (eg. Global Mapper) and result viewing (eg. SMS).

The model area is divided into fixed rectangular cells that can be either wet or dry during a simulation. The model has the ability to simulate the variation in water level and flow inside each cell once information regarding the ground resistance, topography and boundary conditions are entered.

4.3 TUFLOW Modelling

4.3.1 Modelling Scope

The scope of TUFLOW modelling includes the major river channels of the greater Victor Harbor Region and their floodplains. In the Inman River model, there has been an initial concept level investigation of the potential effectiveness of levees along the banks of the Inman River in the vicinity of Stock Road and Swains Crossing Road.

Flood models for the Inman River and Hindmarsh River cover areas of 832 and 527 hectares in size respectively, not including the large upstream catchments draining into these rivers. The Port Elliot West model area is 126 hectares in size, and the southern Encounter Bay model is 55 hectares in size.

4.3.2 1D/2D Hydraulic Model Domains

The 1D network, including side entry pits, culverts and stormwater drains, were modelled based on GIS stormwater databases provided by Council. In instances where there was missing data (i.e. invert levels, pipe dimensions), engineering judgement was used to fill in the gaps.

The models were developed such that culverts and the underground drainage network were modelled in 1D, while overland flow paths on the ground surface were modelled in 2D using TUFLOW. The 1D and 2D domains within each model were hydro-dynamically linked, allowing flows in both domains to interact.

4.3.3 2D Cell Size

Determining an appropriate 2D cell size to be used by TUFLOW requires a compromise between the accuracy of modelling necessary to sufficiently reproduce the hydraulic behaviour of the floodplain and limitations in computing power and processing time. In this instance, there are 4 distinct and unique model areas. The southern Encounter Bay model and the Port Elliot West model were both relatively small in size, and therefore a cell size of 1 metre was deemed appropriate for these two models. The Inman River and the Hindmarsh River models were comparatively much larger. Due to the broad-scale nature of the floodplains of these rivers, a cell size of 2 metres was selected for these models.

This is a substantial improvement over the previous modelling work, which had a cell size of 5 metres for all models besides the southern catchments, which used a cell size of 3 metres.

4.3.4 Time Step

Selection of an appropriate time step in the 2D domain is an important aspect of TUFLOW modelling, as it is directly proportional to the run time of the model. A smaller time step will produce more accurate results and is less likely to cause instabilities, however the simulation time can become limiting for very large models such as those for the Inman and Hindmarsh Rivers. Conversely, a large time step will shorten run times, but may lead to inaccurate results.

A time step of 1 second was used for all models in this study.

4.3.5 Topography

A DTM of the model domain areas was used to define the existing topography of the study area. This involved assigning elevations to individual cells within the 2D model domains. These are assigned at the cell centres, corners and mid-sides to enable interaction with surrounding cells within TUFLOW, which governs where water will and will not flow in a simulation.

4.3.6 Resistance Parameters

The bed resistance is an essential element used to determine the flow, and hence the water depth at any location within the 2D model domain. In TUFLOW, bed resistance values for 2D domains are most commonly created using GIS layers. These layers contain polygons spanning the model area, with material values in the attribute table. The material values specified in this way correspond to a user defined Manning's n value, associated with a specific type of land use. This approach makes it quick and easy to define the bed roughness of the model domain, and to change values if necessary for model calibration.

The bed resistance values used in the models have been taken from the 2010 study by KBR where available, and are specified in Table 4.1.

Table 4.1 Bed Resistance Parameters

Type of Land Use	Manning's Roughness Coefficient
Residential/Commercial	0.2
Overbank/Floodplain – Thick Vegetation	0.1
Roads	0.03
Weedy/Vegetated Channel	0.045
Rural – Open Farm Land	0.03
Rural – Moderate Vegetation	0.06
Main River Channel – Unvegetated	0.025
Developed Area	0.06
Sand ¹	0.024

¹ Adopted from Brisbane City Council guidelines

It should be noted that relatively high values of Manning's n are used for residential and commercial land to compensate for the removed building envelopes in the DTM.

The Manning's n value used for modelling culverts was 0.013.

4.3.7 Boundary Conditions

As part of the modelling, careful consideration was given to boundary conditions. Within the 2D domain, the boundary condition is the edge of the model. At the downstream boundary of the models for the Hindmarsh and Inman rivers, and the two channels in Encounter Bay, water flows out into the ocean.

In the 2010 KBR report, Victor Harbor tide data is listed, as summarised in Table 4.2.

Table 4.2 Victor Harbor Tide Data (KBR, 2010)

Parameter	Elevation (mAHD)
Highest Recorded Tide	+1.58
Mean High Water Springs	+0.33
Mean Sea Level	+0.02
Lowest Recorded Tide	-1.02
Victor Harbor Tide Datum	-0.579

In the nine years since this report, there has been additional tide data recorded, which has been analysed for use in the model. The same statistics, modified to reflect the full length of available data are shown in Table 4.3.

Table 4.3 Victor Harbor Tide Data (Southfront, 2019)

Parameter	Elevation (mAHD)
Highest Recorded Tide	+1.64
Mean High Water Springs	+0.33
Mean Sea Level	+0.05
Lowest Recorded Tide	-1.02
Victor Harbor Tide Datum	-0.579

The statistics are mostly similar to what they were in the 2010 study, however the highest recorded tide and the mean sea level have both slightly risen.

The 1998 report by Connell Wagner lists estimated peak tide levels at Victor Harbor for a range of ARIs, as calculated by BoM in 1989. These can be seen in Table 4.4.

Table 4.4 Victor Harbor Tide Average Recurrence Intervals

Average Recurrence Interval (years)	Tide level (mAHD)
5	1.24
10	1.36
20	1.47
50	1.61
100	1.70 ± 0.12

This information suggests that the highest recorded tide of 1.64 mAHD is between a 50 and a 100 year ARI tide level. However, these values were calculated thirty years ago, so their validity is uncertain.

At the downstream boundaries of the Inman, Hindmarsh and southern Encounter Bay models, where the water reaches the ocean, a 'HT' type boundary was specified along the edge of the model domain. This was used to specify a water level vs time relationship, simulating the ocean tide, and enabling a more realistic interaction between river flows and the sea level. See Section 4.4 for a sensitivity analysis of the downstream boundary conditions.

In the Port Elliot West model, which terminated well before the ocean near a series of detention basins, a 'HT' type boundary was applied at the edge of the model, with a water surface slope of 2%. This allowed water to 'disappear' once flood flows reached the downstream edge of the model, and ensured that results were not adversely affected at the edge of the model.

4.3.8 Inflows

The inflow hydrographs were applied at either the upstream boundary of the model, or directly into the channel depending on the location within the model. For the southern Encounter Bay model, which is mostly urban with some upstream rural inflows, the hydrograph for each urban subcatchment was applied directly into the channel. The purpose of this approach was to simulate the flow of the urban catchments through the underground drainage network and into the main channels, but ignoring the capacity of the pits and pipes. The location of model inflows for these two channels can be seen in Figure 4.1.

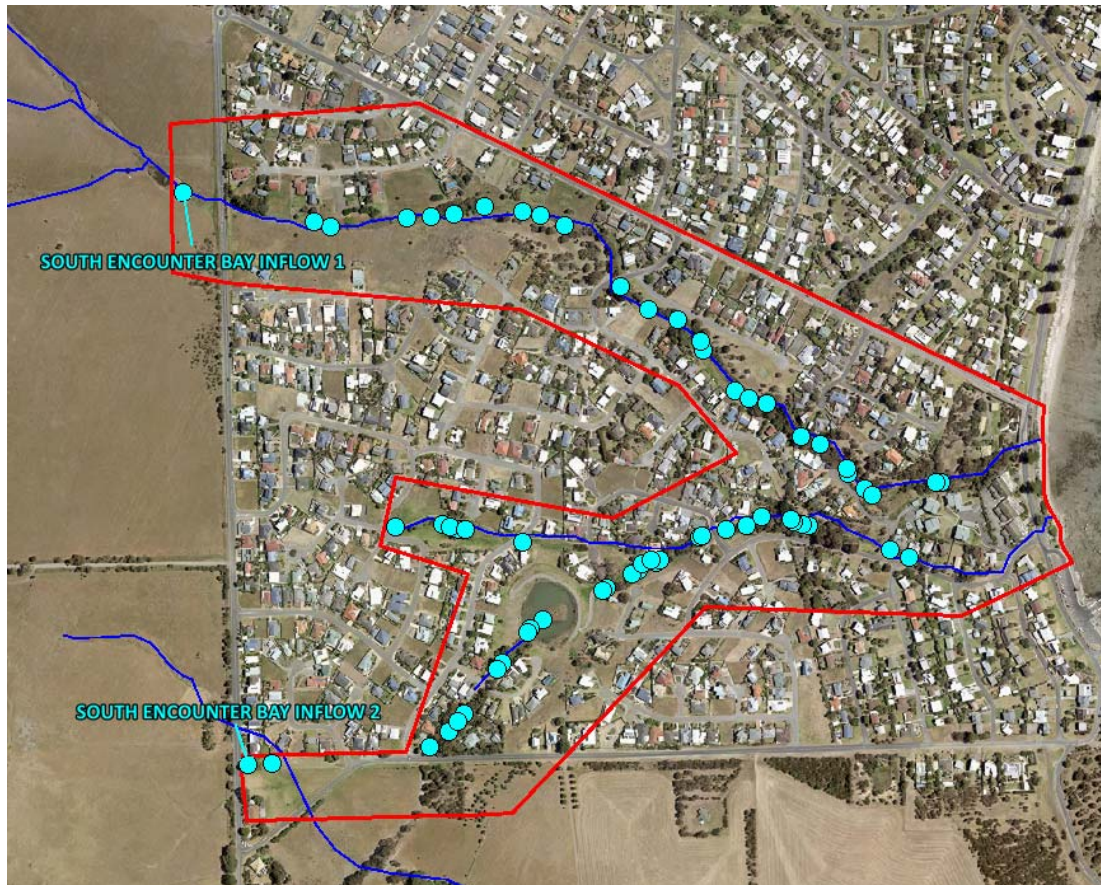


Figure 4.1 Southern Encounter Bay Model Inflow Locations

For the remaining three models, which have much larger catchments producing flows from upstream, the inflow hydrographs were applied directly into the 2D domain at the upstream boundaries. The location of model inflows for these systems can be seen in Figure 4.2, Figure 4.3 and Figure 4.4.

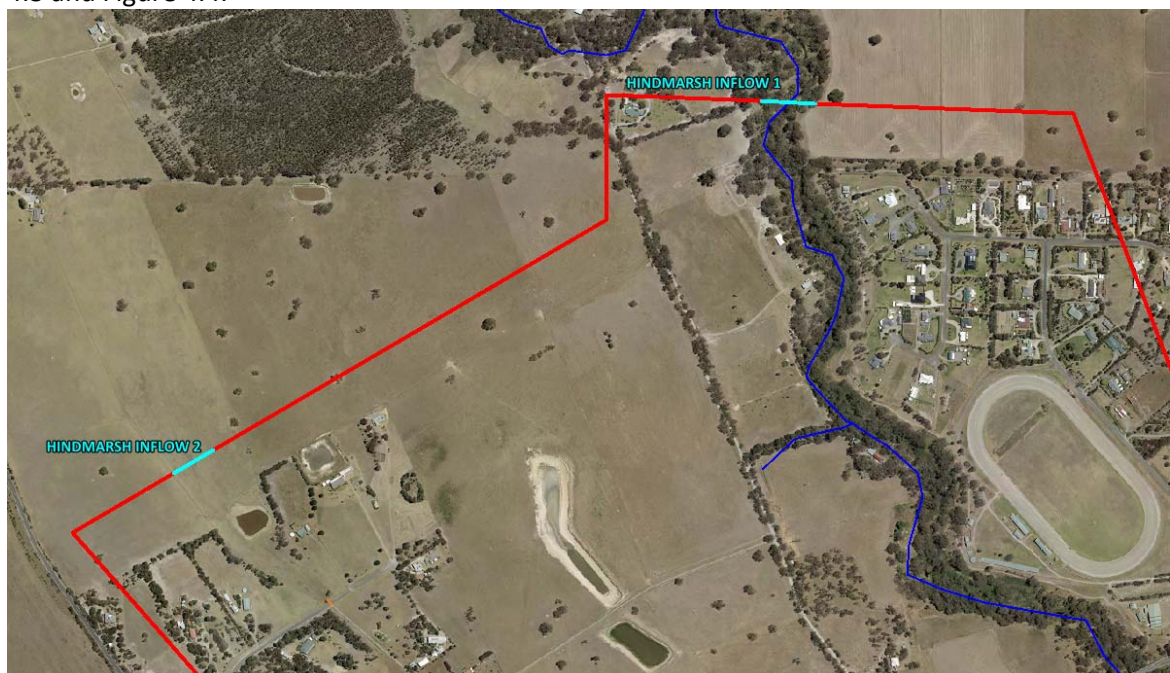


Figure 4.2 Hindmarsh River Model Inflow Locations



Figure 4.3 Inman River Model Inflow Locations

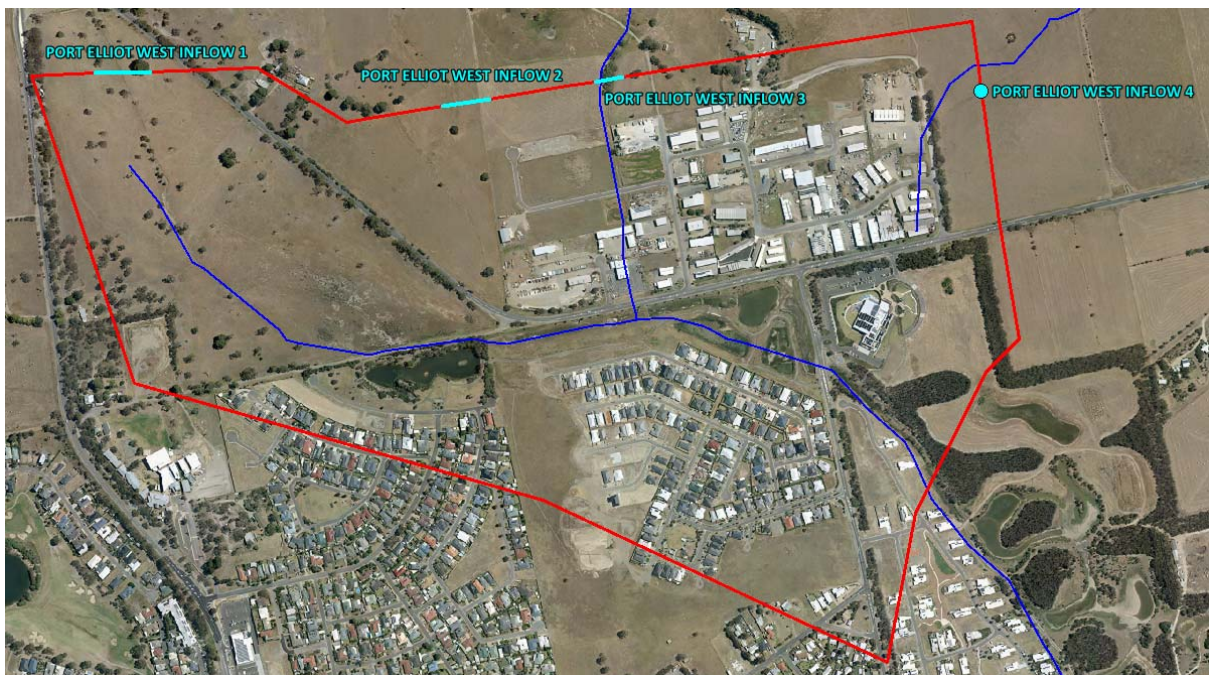


Figure 4.4 Port Elliot West Model Inflow Locations

Of particular note in the Port Elliot Model is that the easternmost inflow has been modelled as a single point rather than a line through a channel cross-section. As the channel in this location is poorly defined, the DTM was used to locate the low point and insert water there to better represent the flow from this catchment. The flow rates at each inflow location are given in Table 4.5 below.

Table 4.5 Peak Flow Rate at Each Inflow Location

Inflow Location	5% AEP Peak Flow (m ³ /s) (Storm Duration)	2% AEP Peak Flow (m ³ /s) (Storm Duration)	1% AEP Peak Flow (m ³ /s) (Storm Duration)
Hindmarsh Inflow 1	50 (18 hours)	73 (18 hours)	96 (18 hours)
Hindmarsh Inflow 2	4.9 (12 hours)	6.1 (12 hours)	7.1 (12 hours)
Inman Inflow 1	134 (36 hours)	221 (36 hours)	302 (18 hours)
Inman Inflow 2	14.5 (12 hours)	19.8 (12 hours)	24.2 (12 hours)
Inman Inflow 3	7.1 (12 hours)	8.9 (12 hours)	12.1 (12 hours)
PEW Inflow 1	10.1 (2 hours)	13.4 (2 hours)	15 (2 hours)
PEW Inflow 2	4.6 (2 hours)	6.1 (2 hours)	7 (2 hours)
PEW Inflow 3	2.8 (2 hours)	3.5 (2 hours)	4.1 (2 hours)
PEW Inflow 4	3.9 (2 hours)	5.1 (2 hours)	6.2 (2 hours)
SEB Inflow 1	0.96 (12 hours)	1.49 (9 hours)	3.3 (9 hours)
SEB Inflow 2	0.53 (12 hours)	0.86 (9 hours)	1.9 (9 hours)

4.3.9 Bridges

The surface elevation model acquired for this study had all major bridges clipped, instead displaying the river channel profile at crossings. To accommodate for this, bridges and major crossings have been modelled in TUFLOW using layered flow constrictions. Within these layers, the user can specify the geometry of a crossing, including the elevation of the obvert, bridge deck and top of rail, as well as the blocked percentage and form loss coefficients for each layer of bridge. These parameters allow TUFLOW to restrict flows through beneath a bridge appropriately to represent the effect they have on flow.

4.4 Assumptions

Throughout the flood modelling, several assumptions have been made.

- **Impact of climate change and a rising sea level:** A thorough analysis of the impacts of climate change and potential future sea level rise has not been undertaken. Instead, the sensitivity of the downstream boundary of all models has been considered, as discussed further in Section 4.4.
- **Pumping stations:** While a number of pumping stations were identified through Council supplied GIS databases, these have not been included in the TUFLOW modelling. Compared to the flows experienced during infrequent and rare flood events, the impact of these pumping stations was considered negligible.
- **1D network capacity:** Throughout the models, but most notably in the southern Encounter Bay and Port Elliot West models, side entry pits, stormwater pipes and culverts have been modelled as 1D network elements in TUFLOW. However, the inlet capacity of these pits and pipes has not been modelled. As this study is primarily concerned with broad-scale flood risk, the impact of any capacity limitations of the 1D network was assumed to be negligible.

4.5 Sensitivity Analysis

At the boundaries of models where the water flows out to the ocean, a sensitivity analysis was undertaken to determine how much of an impact sea level has on the depth of inundation in the lower reaches of the rivers. As a baseline for comparison, the mean high water springs of +0.33 mAHD as used in the previous report was first used in all models. Then to test for sensitivity, a high tide of +1.64 mAHD (the highest tide on record, as described in Section 4.2.7) was applied at the downstream boundary for the entire duration of the model run for all three systems which flow out to the ocean. This approach is conservative, in that a sinusoidal tide has not been used.

For the Inman River, Hindmarsh River and southern Encounter Bay models in a 5% AEP event, the floodplain was effectively unchanged despite this sea level increase, as the added water due to the downstream boundary condition was negligible compared to the water flowing down the river. As a 5% AEP flood event was the most frequent storm event modelled, and a high tide had a negligible impact, it was assumed that for less frequent events, where the river flow is much larger, a high tide would make a similarly negligible difference.

5 Existing Flood Behaviour

5.1 Floodplain Mapping

Floodplain mapping was performed for the critical duration for a range of storm events (as described in Section 3) for each of the models. Flood inundation and depth maps for the five storm events for each model are attached in Appendix A, while flood hazard maps for the 2%, 1% and 0.2% AEP events as specified by Council are also attached in Appendix A. The following sections provide a description of flood hazard as a concept, and commentary on the resulting floodplain and flood hazard maps.

5.2 Flood Hazard

Flood hazard is the potential for material damages, injuries and the loss of life as a result of a flood event. Flood hazard varies throughout a floodplain with changes to flow behaviour, such as in locations of low velocity flow, or areas where there is a minimal flood depth. In *Floodplain Management in Australia: Best Practice Principles and Guidelines*, SCARM Report 73 (SCARM, 2000), flood hazard is considered to be a combination of the following factors:

- Size of the flood: it is important to consider floods of varying severity, as different events may behave differently, altering flood hazard;
- Land use: existing and future land use can have a significant impact on flood hazard, through the creation or removal of evacuation routes, mismanagement of stormwater infrastructure, as well as a number of other factors;
- Population and awareness: the potential for future increases to population density, the level of flood awareness and readiness for evacuation can all impact flood hazard; and
- Emergency response capacity: the ability for emergency services and other related authorities to alert affected residents and businesses, and to assist with evacuations and rescues are key components of flood hazard.

SCARM (2000) defines four levels of flood hazard:

- **Low** – there are no significant evacuation problems. If necessary, children and elderly people could wade to safety with little difficulty; maximum flood depths and velocities along evacuation routes are low; evacuation distances are short. Evacuation is possible by a sedan-type motor vehicle, even a small vehicle. There is ample time for flood forecasting, flood warning and evacuation; evacuation routes remain trafficable for at least twice as long as the time required for evacuation.
- **Moderate** – fit adults can wade to safety, but children and the elderly may have difficulty; evacuation routes are longer; maximum flood depths and velocities are greater. Evacuation by sedan-type vehicles is possible in the early stages of flooding, after which 4WD vehicles or trucks are required. Evacuation routes remain trafficable for at least 1.5 times as long as the necessary evacuation time.
- **High** – fit adults have difficulty in wading to safety; wading evacuation routes are longer again; maximum flood depths and velocities are greater (up to 1.0 m and 1.5 m/s respectively). Motor vehicle evacuation is possible only by 4WD vehicles or trucks and only in the early stages of flooding. Boats or helicopters may be required. Evacuation routes remain trafficable only up to the minimum evacuation time.

- **Extreme** – boats or helicopters are required for evacuation; wading is not an option because of the rate of rise and depth and velocity of floodwaters. Maximum flood depths and velocities are over 1.0 m and over 1.5 m/s respectively.

The definition of these four flood hazard categories can be seen depicted below in Figure 5.1.

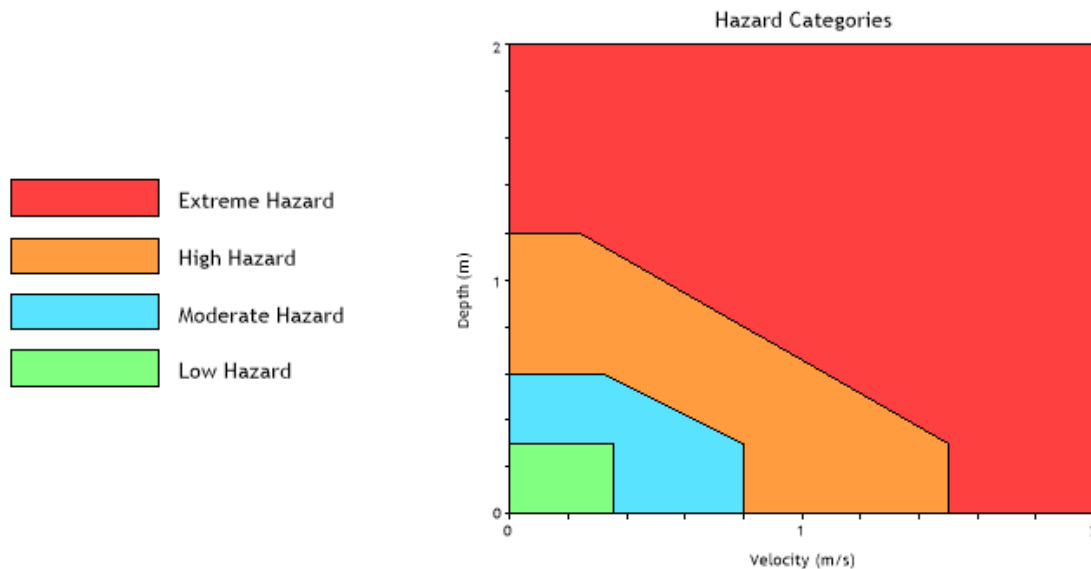


Figure 5.1 Flood Hazard as Defined by SCARM, 2000

As can be seen, regions of shallow water but with a high flow velocity, and conversely regions of deep water but with a low flow velocity are both considered hazardous, as well as a combination of the two factors. This classification approach should only be considered a rough estimate of flood hazard, as it does not consider all of the four factors previously discussed which contribute to flood hazard.

Within TUFLOW, the flood hazard as defined by SCARM was output from model runs, and this layer was classified to match the above colour scheme for use in producing flood hazard maps.

5.3 Inman River Floodplain Mapping Commentary

The Inman River mainly receives flow from the main channel south of Stock Road, however there are two smaller channels to the north that converge shortly upstream of the Stock Road crossing, and connect with the main channel after passing beneath it. Because of this, the Inman River was found to have two critical duration storm events, one for the major channel, and a separate one for the minor channels to the north.

Detailed flood depths at bridges and other key crossings have been provided, as this information is critical for determining suitable evacuation routes in flood events of different severity.

5.3.1 5% AEP

In the 5% AEP event, water largely remains within the channel of the Inman River. The maximum depth of water in the main channel is just over 5 metres between Kullaroo Road and River Road.

Water breaks out of the main river channel upstream of the crossing at Stock Road, spilling over both banks and then across this road and through properties. Nine properties are inundated in

this area, with flood depths ranging from less than 100 mm to 500 mm. Just downstream at the Swains Crossing bridge, a single house located very close to the main river channel is flooded to a depth of over 1 m. Excluding these problematic locations, the floodplain is generally contained to the main channel and overbank in this region upstream of the Victor Harbor Golf Club.

In the vicinity of the Victor Harbor Golf Club, again the flood is largely contained to the main channel, however a substantial volume of water spills across the middle of the golf course, with some holes and a number of small buildings at the southern edge of the course flooded to a depth of up to 1 metre. Water spills from the outside of the channel at the bend around the golf course and floods Canton Place and parts of the SA Water owned land around it to depths of 400 mm, limiting vehicular access back to Henderson Road.

The Victor Harbor Dog Park, which is situated right next to the main river channel, is heavily flooded to a depth of over 2 m in the south-western treed section. Some of the properties on Almond Avenue and Kullaroo Road experience flooding as water overtops the banks. The majority of affected houses are only inundated to depths of up to 250 mm, however three houses see flood depths in excess of 500 mm. Victor Harbor Oval is flooded to depths of up to 1.3 m, and shallower depths of 800 mm occur in the nearby tennis courts. Downstream of the George Main Road bridge, water spills over the bank and across Bay Road, flooding the Civic Centre with depths of up to 800 mm in the car park and around the main building.

The two holiday parks situated near the mouth of the Inman River, the Victor Harbor Holiday & Cabin Park to the north of Bay road and the NRMA Victor Harbor Beachfront Holiday Park closer to the ocean, both experience flooding in the 5% AEP event. The Victor Harbor Holiday & Cabin Park experiences flooding around the perimeter of the park, with those buildings on the outside affected by mostly water at a depth of 250 mm to 500 mm. The other holiday park is affected much more seriously, with depths ranging from 300 mm at some caravan sites to upwards of 1.8 m at the permanent cabins.

Flood Characteristics at Key Roadways and Bridges

Shallow water flows across Glenvale Road at the location of the culvert, with a maximum water depth of 150 mm. Bay Road also sees flooding in two locations, just west of the roundabout to George Main Road at a depth of 400 mm, and near the entrance to the Victor Harbor Holiday & Cabin Park, also to depths of 400 mm.

Cartwright Road Bridge: Water depth of 490 mm

Stock Road Crossing: Water depth of 1.00 m

Swains Crossing Road Bridge: Water depth of 1.52 m

Footbridge upstream of Armstrong Road: Water depth of 1.24 m

Armstrong Road Bridge: Freeboard of 2.87 m from bridge deck to water

Kullaroo Road Footbridge: Freeboard of 200 mm from bridge deck to water

Newland Bridge, George Main Road: Freeboard of 630 mm from bridge deck to water

Encounter Bikeway Footbridge: Freeboard of 400 mm from bridge deck to water

5.3.2 2% AEP

Flood behaviour is largely the same for the 2% AEP event as it is for the 5% AEP event, but with depths generally increasing throughout the model. However, there are some areas newly inundated in the increase to this design event.

In the area surrounding the Stock Road crossing, the flood behaviour is similar, but the increased volume of water results in more extensive flooding near the intersection of Stock Road and Swains Crossing Road. The water depth at this location is in excess of 1 metre.

As compared to the 5% AEP event, the extent of flooding throughout the Victor Harbor Golf Club course is much more significant. Many additional holes undergo shallow flooding to depths of up to 300 mm, while the deepest flooding on the course exceeds 1.3 m. Downstream, there is much more water accumulating on the upstream side of Bridge 24 on Armstrong Road, resulting in water spilling down into Canton Place from the southern end.

Unlike in the 5% AEP event scenario, Victor Harbor High School experiences shallow inundation, largely to a maximum depth of 250 mm but with some buildings seeing as much as half a metre. Downstream of the George Main Road bridge crossing, the Victor Harbor Holiday & Cabin Park experiences much more significant flooding than in the 5% AEP storm. Where water was just around the perimeter of the site, it is now spread over the majority of the cabins, with just a handful of buildings near the entrance safe from the floodplain. The depth of water within this site reaches over 1 m near the edge, and 500 mm within the main area housing the majority of cabins. Similarly, the other holiday park is flooded to much greater depths, however the extent of flooding is largely unchanged.

Flood Characteristics at Key Roadways and Bridges

Water flows across Glenvale Road, limited by the culvert capacity, and reaches a depth of 400 mm along the road. The intersection of Stock Road and Swains Crossing Road is flooded to a depth of 900 mm. Downstream, there is very shallow water of around 100 mm across George Main Road between the high school and the Civic Centre, and relatively deep water on Bay Road between the two holiday parks of depth 800 mm.

Cartwright Road Bridge: Water depth of 580 mm

Stock Road Crossing: Water depth of 1.05 m

Swains Crossing Road Bridge: Water depth of 2.26 m

Footbridge upstream of Armstrong Road: Water depth of 1.80 m

Armstrong Road Bridge: Freeboard of 2.36 m from bridge deck to water

Kullaroo Road Footbridge: Water depth of 290 mm

Newland Bridge, George Main Road: Freeboard of 35 mm from bridge deck to water

Encounter Bikeway Footbridge: Freeboard of 25 mm from bridge deck to water

5.3.3 1% AEP

Flood behaviour observed by the 1% AEP event is similar to that of the 2% AEP event, just with larger flow depths, however there are some new areas of inundation when compared to the 2% AEP flood.

The spread of water around the Stock Road crossing is slightly larger, inundating two additional properties at the fringe of the 1% AEP floodplain. The overall depth of the floodplain is significantly increased relative to the 2% AEP event, particularly at the Stock Road and Swains Crossing Road intersection.

The overall shape of the floodplain within the Victor Harbor Golf Club remains largely unchanged, however the depth of inundation consistently increases throughout. The water accumulating upstream of Bridge 24 along Armstrong Road significantly increases, but still does not flow across the road surface.

Just downstream of this bridge crossing, water overflows the banks in the vicinity of the dog park, spilling out into the police station car park, inundating it to depths of 600 mm and the station buildings to 100 mm. Shallow water flows around the TAFE car park, but does not breach it. Along George Main Road at the intersection with Broderick Terrace, water spills across the road up to a depth of 400 mm.

Victor Harbor High School faces worse flooding, despite the actual extent of inundation remaining largely the same, with flooding shown to increase by over 200 mm as compared to the 2% AEP flood, resulting in depths of up to 700 mm.

Similarly, the extent of flooding throughout the two holiday parks is close to that of the 2% AEP event, with only a handful of additional cabins inundated, but with depths throughout these locations increasing significantly.

Flood Characteristics at Key Roadways and Bridges

Cartwright Road Bridge: Water depth of 700 mm

Stock Road Crossing: Water depth of 1.10 m

Swains Crossing Road Bridge: Water depth of 2.87 m

Footbridge upstream of Armstrong Road: Water depth of 2.29 m

Armstrong Road Bridge: Freeboard of 1.92 m from bridge deck to water

Kullaroo Road Footbridge: Water depth of 660 mm

Newland Bridge, George Main Road: Water depth of 360 mm

Encounter Bikeway Footbridge: Water depth of 230 mm

5.3.4 0.2% AEP

As would be expected for such a rare storm event, in the 0.2% AEP flood there is widespread flooding throughout the catchment. Surrounding the Stock Road crossing, a number of dwellings which were outside of the 1% AEP floodplain are now inundated, and flood depths in this area have generally increased by around 1 to 1.5 m. The water in the main channel extends much further north towards Stock Road, increasing the water depth at houses which were inundated in the 1% AEP flood.

The majority of the golf course within the oxbow is submerged, with depths of up to 1.8 m at some holes. Substantial water breaks out of the floodplain and spills across Canton Place, creating a deep pond in the vacant land between this road and Armstrong Road.

To the southwest of the Armstrong Road bridge, between the Encounter Bay Football Club and the Encounter Centre, water spills onto the road and flows towards the broadly residential area to the west of the lower Inman River channel. This is brought on by water exceeding the overbank in the area between the golf course and the Armstrong Road bridge, where water gets very backed up over a number of hours before finally giving way and spilling.

This is stockpiled by water flowing along Broderick Terrace from George Main Road, vastly increasing the total volume of water flowing through the residential area. Rosetta Village and Bay Village Retirement Communities, Encounter Lakes and the properties generally surrounding these locations are all severely affected, with many houses and businesses inundated to depths in excess of 500 mm.

A number of properties on Inman Street and Newland Street are within the floodplain, with a flood depth in the range of 200 mm to 700 mm. Almost the entirety of both holiday parks is submerged in very deep water, with only a small portion of the NRMA Victor Harbor Beachfront

Holiday Park remaining outside the floodplain. Substantial water flows into Kent Park and forms reasonably sized ponds of up to 500 mm deep.

Flood Characteristics at Key Roadways and Bridges

Cartwright Road Bridge: Water depth of 850 mm

Stock Road Crossing: Water depth of 1.15 m

Swains Crossing Road Bridge: Water depth of 4.34 m

Footbridge upstream of Armstrong Road: Water depth of 3.29 m

Armstrong Road Bridge: Freeboard of 1.17 m from bridge deck to water

Kullaroo Road Footbridge: Water depth of 1.02 m

Newland Bridge, George Main Road: Water depth of 830 mm

Encounter Bikeway Footbridge: Water depth of 630 mm

5.3.5 PMF

Flood Characteristics at Key Roadways and Bridges

Cartwright Road Bridge: Water depth of 2.16 m

Stock Road Crossing: Water depth of 14.9 m

Swains Crossing Road Bridge: Water depth of 16.6 m

Footbridge upstream of Armstrong Road: Water depth of 5.80 m

Armstrong Road Bridge: Water depth of 1.66 m

Kullaroo Road Footbridge: Water depth of 3.10 m

Newland Bridge, George Main Road: Water depth of 2.99 m

Encounter Bikeway Footbridge: Water depth of 2.63 m

5.3.6 2% AEP Flood Hazard

In a 2% AEP storm event flood, the flood hazard is quite pronounced throughout the Inman River floodplain. The main channel and the surrounding floodplain both largely fall into the extreme flood hazard category, due to large flood depths. One property, located very close to the Swains Crossing Road bridge, faces extreme flood hazard due to its proximity to the river. Other locations with extreme flood hazard include a section of the Victor Harbor Golf Club in the oxbow of the river channel, some of the houses between Kullaroo Road and Almond Avenue, the Victor Harbor Oval and nearby tennis courts, the high school and the Civic Centre, and large parts of both holiday parks near the mouth of the river.

There is high flood hazard typically further back from the river channel in surrounding land and within some parts of the floodplain. Surrounding the intersection of Stock Road and Swains Crossing Road there is high flood hazard, including through a number of dwellings. As compared to the extreme hazard classification, the high flood hazard is much more prevalent through the Victor Harbor Golf Club and surrounding areas including across Canton Place. Many of the properties on Almond Avenue and Kullaroo Road see extreme or high flood hazard, as water overflows the riverbank and along the roadways. Similarly, there is widespread high flood hazard throughout the holiday parks as would be expected considering the extent of flooding observed.

5.3.7 1% AEP Flood Hazard

The 1% AEP flood hazard map is quite similar to that of the 2% AEP flood. Generally speaking, more of the floodplain is considered extremely hazardous than in the 2% AEP flood hazard map.

Specifically, the intersection of Stock Road and Swains Crossing Road has moved from high flood hazard to extreme flood hazard. This means that properties along Stock Road between this intersection and the river crossing are trapped, with extreme flood hazard on either side.

Further downstream along the river, there is a high to extreme flood risk across George Main Road near the intersection with Broderick Terrace, and a moderate to high flood risk across the same road around the roundabout leading up to the George Main Road bridge. This isolates the Civic Centre, the high school and the recreation centre, and requires vehicles to drive through very dangerous water if an evacuation has not finished by the time that the flooding occurs. The section of Bay Road from the George Main Road until the cul-de-sac at Kent Drive is all classified as either high or extreme flood hazard, meaning that anyone still remaining in the Victor Harbor Holiday & Cabin Park would be unable to leave.

5.3.8 0.2% AEP Flood Hazard

In the 0.2% AEP map, the extent of high and extreme hazard is much larger, as substantial water inundates the region roughly bounded by George Main Street and Encounter Lakes and the Inman River channel, which was beyond the floodplain in the 1% AEP storm event. Throughout this region there is frequently high or extreme flood hazard along roads, with low to moderate flood hazard throughout the majority of affected properties. This makes it so that vehicles attempting to evacuate during the peak of the flooding will have difficulty, especially along Matthew Flinders Drive and Maude Street, where there is extreme flood hazard.

The whole region surrounding the Stock Road crossing experiences very dangerous conditions, including extreme hazard from the intersection of Stock Road and Swans Crossing Road to almost the Stock Road crossing. A substantial portion of the golf course is classified as extremely hazardous, and more still is moderate to high hazard such that any golfers at holes close to the inside of the oxbow will be trapped. Throughout the high school there is moderate to high flood risk, and the whole of the Civic Centre site sees extreme flood risk in this large flow event.

5.4 Hindmarsh River Floodplain Mapping Commentary

The Hindmarsh River is the other major river flowing through Victor Harbor out to the ocean, and is a defining feature of the landscape. It is characterised by high overbanks surrounding a deep main channel, with many bridges connecting the land on either side of the river. There is a major channel which, for the purposes of floodplain modelling and mapping, stretches from near Appaloose Drive at an elevation of 15 mAHD through to the ocean. There is also a smaller tributary, which in the model begins upstream of Panorama Drive, and flows through a series of basins and road culverts before converging with the main channel after crossing Waggon Road.

The Hindmarsh River, as modelled in this study, is comprised of a main channel to the east of Waggon Road, and a minor channel flowing into the main channel across Waggon Road from the west. Across all modelled AEPs excluding the PMF, the major channel had a critical storm duration of 18 hours, while the smaller channel had a critical storm duration of 12 hours. In the PMF event, the critical duration for the main channel was 12 hours, while in the minor channel it was 3 hours. As with the Inman River floodplain maps, the output from both critical storm durations has been combined, such that the final floodplain maps show the largest extent of flooding across all simulations.

Detailed flood depths at bridges and other crossings has been provided, as this information is critical for determining suitable evacuation routes for use in flood events of different severity.

5.4.1 5% AEP

In a 5% AEP storm event, water mostly remained within the main channel and the overbank, not affecting many dwellings or businesses.

Within the minor tributary to the west of Waggon Road, water initially flows into a large dam, but quickly exceeds its capacity, and spills towards Panorama Drive through to a series of smaller dams, all of which have their capacities exceeded as well. Water then reaches the culvert beneath Armstrong Road, but due to its small diameter of 600 mm, water accumulates upstream of the road, reaching a maximum water depth of 3.8 m before spilling across to the other side. This water then flows towards Waggon Road, passing through a road culvert into a small channel, overflowing a dam before connecting back to the main river channel. At the low point of Armstrong Road there is a culvert underneath it, and water pools on the upstream side of the culvert to a depth of up to 600 mm, and forms a pond of up to 400 mm deep on the downstream end.

Houses along Lamont Court, Lamont Road and Wattle Drive are affected by flood waters, with water spilling from the river at the intersection of Wattle Drive and Lamont Road, reaching depths of over 800 mm along roads. Seven dwellings are affected in this way, with water depths within the building parcels varying from 100 mm to over 700 mm.

The oval off The Parkway is badly affected, with depths of up to 1 m on the oval and up to 250 mm within this hall. The sheds on the perimeter of the primary school are also within the floodplain and see water of over 600 mm deep. Part of the school staff car park is reached by flood waters, however only a small portion of this flooding exceeds a depth of 250 mm.

A number of the houses broadly bounded by Wattle Drive, Pearsons Road and Hindmarsh Road are also within the floodplain, due to this region's very close proximity to the main river channel, and that it is just upstream of three bridges which further constrict the flow. Water inundates a large number of properties, and reaches a depth of up to 600 mm in some properties.

Flood Characteristics at Key Roadways and Bridges

Panorama Drive and Armstrong Road both see very shallow inundation of around 150 mm at the location of the road culverts as their flow capacities are exceeded. There is negligible flood depth over Waggon Road near the large culvert.

Welch Road Bridge: Freeboard of 3.90 m from bridge deck to water

Bashams Road Bridge: Water depth of 1.36 m

Wattle Drive Bridge: Water depth of 740 mm

Hindmarsh Road Bridge: Freeboard of 2.05 m from bridge deck to water

Willats Crossing: Water depth of 70 mm

Railway Bridge: Freeboard of 2.12 m from bridge deck to water

5.4.2 2% AEP

The majority of the floodplain of the 2% AEP flood is similar in extent to the 5% AEP event, with only a handful of locations newly experiencing inundation. Generally, the water depth throughout the model is slightly deeper in this 2% AEP flood than it was in the 5% AEP flood, with this difference decreasing towards the downstream edge of the model.

One location where conditions have noticeably worsened is the area surrounding Lamont Court and Lamont Road. Where previously the maximum water depth in dwellings was 700 mm, it is now 1 m, with a number of additional properties being affected. There is a similar change further downstream, in the region within Wattle Drive, Pearsons Road and Hindmarsh Road, with additional properties affected, as well as overall flood depths increasing.

Flood Characteristics at Key Roadways and Bridges

Panorama Drive again has shallow flow across it to the order of 150 mm, however Armstrong Road is inundated to up to 180 mm. At the large Waggon Road culvert there is flooding of up to 150 mm across the road as water bypasses the road culvert.

Welch Road Bridge: Freeboard of 3.63 m from bridge deck to water

Bashams Road Bridge: Water depth of 1.69 m

Wattle Drive Bridge: Water depth of 1.05 m

Hindmarsh Road Bridge: Freeboard of 1.78 m from bridge deck to water

Willats Crossing: Water depth of 180 mm

Railway Bridge: Freeboard of 1.90 m from bridge deck to water

5.4.3 1% AEP

As with the change from 5% AEP to 2% AEP, the increase to 1% AEP has only a minor impact on the extent of the floodplain. This is due to the high elevations surrounding the river channel, which prevents most sections of the river from spilling, even in rare events such as this.

The main differences are again in the areas of Lamont Court and Wattle Drive, where the floodplain extent and flood depths are both increased.

Flood Characteristics at Key Roadways and Bridges

Panorama Drive has water of up to 150 mm on the road, and Armstrong Road further downstream is below up to 200 mm of water. The water level on Waggon Road is largely unchanged, as the extent of flooding very slightly increases from the 2% AEP event.

There is much more significant flooding in the area near Lamont Court, with depths of up to 2.5 m through some houses.

Welch Road Bridge: Freeboard of 3.33 m from bridge deck to water

Bashams Road Bridge: Water depth of 2.09 m

Wattle Drive Bridge: Water depth of 1.46 m

Hindmarsh Road Bridge: Freeboard of 1.43 m from bridge deck to water

Willats Crossing: Water depth of 500 mm

Railway Bridge: Freeboard of 1.61 m from bridge deck to water

5.4.4 0.2% AEP

In the 0.2% AEP flood event, a number of new areas are affected by floodwaters. Near the upstream edge of the model, water breaks out of the channel and spills into part of the Victor Harbor Harness Racing Club closest to the river. There is water of up to 500 mm deep, however the majority of buildings and the track remain outside of the floodplain.

A part of the primary school is directly within the 0.2% AEP floodplain, with water spilling from the river towards The Parkway causing shallow flooding through the school of up to 300 mm.

The depth of water on The Parkway out the front of the school is over 500 mm on both lanes of the road.

As with the properties surrounding Lamont Court, those near Pearsons Road and Wattle Drive are affected much more significantly, with all properties bounded by these two roads and Hindmarsh Road being inundated. The water depth exceeds 2 m through most properties, up to a maximum of 2.7 m.

Flood Characteristics at Key Roadways and Bridges

Flood behaviour is again largely the same at Panorama Drive, Armstrong Road and Waggon Road, with minor increases to flow depth. There is 200 mm deep water at Panorama Drive, water across Armstrong Road to a depth of 270 mm, and 150 mm across Waggon Road.

Welch Road Bridge: Freeboard of 2.41 m from bridge deck to water

Bashams Road Bridge: Water depth of 3.31 m

Wattle Drive Bridge: Water depth of 2.53 m

Hindmarsh Road Bridge: Freeboard of 210 mm from bridge deck to water

Willats Crossing: Water depth of 1.65 m

Railway Bridge: Freeboard of 680 mm from bridge deck to water

5.4.5 PMF

In the PMF event, there is much more widespread flooding, as water breaks out of the channel in many more locations.

At the upper end of the model, water spills onto the roads and properties of Palomino Court and Appaloosa Drive, with many dwellings seeing depths of over 1 m. Flood water does not cross Lipizzaner Drive to the east, but flows down towards the Victor Harbor Harness Racing Club track. There is water of over 1 m depth throughout the centre of the track, with deeper water of over 4 m in the fenced area within the track.

Further downstream near the Welch Road crossing, the open land between the river channel and Waggon Road is badly flooded, both upstream and downstream of Welch Road. Some properties close to Waggon Road are within the floodplain, while others are surrounded by deep water.

Water spills across Coromandel Drive, affecting properties in the vicinity of Goshawk Court and Isabella Court, with max water depths of 500 mm through properties. Water also flows over Basham Road, flooding 8 properties with a maximum water depth of 2.7 m.

Where in the 0.2% AEP event water was restricted to a small number of properties along Lamont Court and Wattle Drive, in the PMF the breakout is far more extensive, almost reaching Coleman Avenue. The flood depth in Lamont Court exceeds 5 m, while within Ives Crescent it is between 2 and 2.4 m.

Nearer to the mouth of the river, water breaks out to the south, flooding the primary school and some of the properties along The Parkway, Riverview Road, Bond Avenue and Brand Avenue. The water level in the region bounded by Hindmarsh and Pearsons Roads ranges from 2.5 m to 5 m closer to the river.

Flood Characteristics at Key Roadways and Bridges

There is 500 mm deep water at Panorama Drive, water across Armstrong Road to a depth of 550 mm, and 350 mm across Waggon Road.

Welch Road Bridge: Water depth of 220 mm

Bashams Road Bridge: Water depth of 5.99 m

Wattle Drive Bridge: Water depth of 4.28 m

Hindmarsh Road Bridge: Water depth of 1.45 m

Willats Crossing: Water depth of 3.31 m

Railway Bridge: Water depth of 840 mm

5.4.6 2% AEP Flood Hazard

Regions of high and extreme flood hazard are largely kept to the main river channel in the 2% AEP event, however in a few locations there are dangerous conditions.

Across Panorama Drive there is moderate to high flood hazard, across Armstrong Road as the culvert capacity is exceeded there is extreme flood hazard, and across Waggon road there is moderate flood hazard.

There is high flood hazard through Lamont Court including three properties. Between Hindmarsh Road and Pearsons Road there is high flood hazard through a number of properties.

5.4.7 1% AEP Flood Hazard

The 1% AEP flood hazard map is mostly similar to the 2% AEP for regions outside of the main channel, however there are still a few key differences.

There is extreme flood hazard on Panorama Drive instead of moderate flood hazard, while Armstrong and Waggon Roads remain mostly similar.

There is now extreme flood hazard through parts of Lamont Court, and high flood hazard in a much wider area. A similar worsening of conditions is observed in the region between Hindmarsh and Pearsons Roads.

5.4.8 0.2% AEP Flood Hazard

The 0.2% AEP flood hazard map is again similar to that of the 1% AEP, with high and extreme flood hazard at Panorama Drive, Armstrong Road and Waggon Road.

Around Lamont Court there is much worse flooding, with extreme flood hazard through a much larger number of properties, due to the increased flood depth throughout. A similar increase can be seen in the streets and properties situated between Pearsons Road, Hindmarsh Road and Wattle Drive.

5.5 Port Elliot West Model Floodplain Mapping Commentary

The Port Elliot West model area is comprised of a mixture of land usage, including sections of residential land to the south and commercial land to the north of Waterport Road. Flows enter from channels to the north of Waterport Road and flow through a series of detention basins and culverts before exiting across the Council boundary under Ocean Road. There is also water entering the City of Victor Harbor Council land from Alexandrina Council to the east.

All model inflow locations for all storm events modelled besides the PMF were found to have a critical duration of 2 hours, meaning that only a single model run was needed for each AEP selected. For the PMF event, the critical duration for each channel was 1 hour.

5.5.1 5% AEP

In the 5% AEP flood event, water pools at the corner of Strawberry Hill Road and Waterport Road upstream of the road culvert due to a flow capacity limitation, reaching a depth of over 1.1 m. There is shallow water along most of Waterport Road between Strawberry Hill Road and Lincoln Road, remaining below a depth of 250 mm.

There is shallow flooding in the order of 100 mm to 300 mm through the Commercial land between Waterport Road and Buchanan Court. Buchanan Court, Lincoln Park Drive and Pit Lane are flooded to a depth of 200 mm. Properties on the corner of Lincoln and Waterport Roads are also inundated with shallow water of maximum depth of 250 mm. Water entering from Alexandrina Council flows along Lincoln Road and then across Waterport Road, resulting in widespread sheet flow to the east of the Fleurieu Aquatic Centre.

There is shallow inundation of the carparks of the Fleurieu Aquatic Centre, with the main building remaining outside of the floodplain. There is shallow water along the east lane of Ocean Road, from Waterport Road until the culvert connecting the two detention storages.

5.5.2 2% AEP

The 2% AEP flood event is very similar to the 5% AEP event flood, with only minor increases to depth and extent throughout the model area.

5.5.3 1% AEP

The 1% AEP flood event is again very similar to the 2% AEP flood event. The most significant change from the 2% AEP flood event is some shallow flooding across both lanes of Ocean Road above the large road culverts. There is also very shallow water encroaching on the building of the Fleurieu Aquatic Centre.

5.5.4 0.2% AEP

Both the extent and depth of flooding in the 0.2% AEP event are significantly increased as compared to the 1% AEP event.

At the upstream end of the culvert beneath Strawberry Hill Road, there is a large backwater effect resulting in a large pond of between 700 mm and 800 mm along Strawberry Hill Road. At the intersection of Strawberry Hill Road and Waterport Road there is water of minimum depth 250 mm across both lanes of the road, reaching up to maximum depth of 500 mm towards the side of the road.

Through the commercial region there is flooding of up to 400 mm through some buildings, and up to 350 mm along the roads. Along Ocean Road there is water of up to 350 mm, and shallow water along both lanes of the whole road. There is shallow water of a depth of up to 200 mm through the Fleurieu Aquatic Centre, and deeper water of up to 1 m through the surrounding carparks.

5.5.5 PMF

In the significantly larger PMF flood, the extent of flooding is much more widespread, coupled with much greater flood depths throughout the region. Along Waterport Road and Ocean Road there is water of up to 1.5 m deep.

There is widespread flooding throughout the majority of the commercial region north of Waterport Road, affecting all but a handful of properties along Trade Court and Commerce Crescent.

At the site of the Fleurieu Aquatic Centre there is up to 1.6 m deep water, and some spillage to the south onto Sun Orchid Drive. The properties to the south of the detention basins at the corner of Waterport and Ocean Roads are barely outside of the floodplain, with water at the boundary fences of some properties.

5.5.6 2% AEP Flood Hazard

In the 2% AEP event, there is significant flood hazard outside of the dedicated channels and basins, affecting businesses and nearby residents.

At the intersection of Waterport Road and Strawberry Hill Road, where water spills onto the road, there is a wide region of high to extreme flood hazard, limiting travel along this key roadway. Pit Lane, Buchanan Way and Lincoln Road all also have high to extreme flood hazard along the road.

Through the commercial sites to the north of Waterport Road, the flood hazard is moderate at worst, and low for the majority of building parcels.

5.5.7 1% AEP Flood Hazard

The flood hazard for the 1% AEP event is very similar to that of the 2% AEP, with only minor increases to extent and severity.

5.5.8 0.2% AEP Flood Hazard

In the 0.2% AEP flood, there are a number of regions where the flood hazard has worsened significantly compared to the 1% AEP event.

The region of extreme flood hazard on all of Waterport Road, Pit Lane, Buchanan Court and Lincoln Court has greatly increased, making driving much more dangerous. There is also now moderate to high flood hazard along Ocean Road, with the worst conditions occurring above the culvert crossing.

Through the commercial land the flood hazard reaches moderate at a number of properties, however for the most part it is still considered low. There is also low flood hazard through the entire building of the Fleurieu Aquatic Centre.

5.6 Southern Encounter Bay Channels Floodplain Mapping Commentary

The flood model for the southern Encounter Bay region is made up of many model inflow points coming from subcatchments surrounding the channels, and from the rural catchments upstream. There are two main channels running through this urban zone, which both flow through natural channels and culverts beneath roads out to sea.

This area is characterised by narrow channels in valleys with steep side slopes up to streets and properties. There are two detention basins, both in the southernmost of the two channels, which can hold substantial water during infrequent storm events, limiting the extent of flooding. Throughout the model runs, water was generally held within the channels, with minimal spill over roads and very few properties affected.

5.6.1 5% AEP

Of the water entering the model from the southern boundary, the vast majority spills across Jagger Road following a channel that flows south and out to the ocean. Only a small percentage of this inflow reaches the drainage channel of interest through the culverts beneath Jagger Road. Water is seen to pond upstream of the culvert beneath Krill Court, reaching a depth of over 1 m in a small pond. The same behaviour occurs upstream of the culvert beneath Minke Whale Drive, however there is some spill across the road up to a depth of 150 mm.

Despite the pipe beneath the cul-de-sac of Pollard Court, substantial water spills out of the channel and onto the road, vastly exceeding the flow capacity of the culvert. Water pools to a depth of up to 300 mm on the road, and spills across the other kerb down to the channel. A similar situation occurs further along Pollard Court, near the intersection with Solway Crescent, where water flows across the road up to a depth of 250 mm.

The water depth within the basin in Hicks Reserve reaches 420 mm, and in the Russell Bird Reserve basin reaches 1.8 m, both comfortably below the level of the surrounding roads.

No dwellings are seriously impacted by flood flows.

5.6.2 2% AEP

As was the case in the 5% AEP event, a substantial amount of water exits the model across the southern boundary, flowing across Jagger Road. There is also very similar behaviour to the 5% AEP flood across Minke Whale Drive and Pollard Court, with slight increases to flood depths and extent.

The Hicks Reserve basin receives water to a depth of 700 mm, while the Russell Bird Reserve experiences up to 2 m deep water.

Again there is minimal impact on private property. 5 Dormer Court has up to 100 mm of water around the back fence.

5.6.3 1% AEP

Again flood behaviour is very similar to the 2% AEP storm event. Generally speaking, depths are slightly greater than in the 2% AEP event, however there are a few additional areas of inundation.

Shallow water encroaches into 2 Minke Whale Drive, coming from the densely vegetated space behind the row of houses.

The water level in the Hicks Reserve basin reaches 900 mm, still safely contained within the basin sides. The basin in Russell Bird Reserve has a maximum water depth of 2.3 m, also still below the top of the sides.

5.6.4 0.2% AEP

In the much larger 0.2% AEP event, there is more widespread flooding of roads and dwellings in the close proximity of the channels. Water flowing across Jagger Road has a maximum depth of 300 mm, and water extends across a much wider section of the road. At the location of the culvert underneath Krill Court, shallow water overflows the road as the pipe capacity is exceeded and pools to a depth of 150 mm.

Across Minke Whale Drive and Pollard Court, in the locations mentioned above, there is more extensive inundation, but the flood behaviour is largely the same. At the location of the culvert beneath Three Gullies Road, a small volume of water spills across the road, with a maximum depth of 250 mm observed. Further downstream, Joy Court is overtopped, as water pooling at the culvert inlet spills onto the road and flows along Wallage Court down towards the cul-de-sac. Both sides of the road have flooding of over 50 mm, reaching almost 250 mm within the culvert. As a result of this flow path, properties 13, 14 and 15 Crystal Court are within the 0.2% AEP floodplain, and are inundated with water up to 200 mm deep.

The flood storage capacity of Russell Bird reserve is exceeded in this storm event, and water spills out onto Pamir Street and Solway Crescent. Properties 6 through 18 Solway Crescent are affected by the floodwater, with driveways below as much as 200 mm of water, and some houses seeing 150 mm deep water. Water moving along Solway Crescent flows along Investigator Crescent and then onto Franklin Parade, where both lanes of the road have water depths of approximately 100 mm.

The water depth in the Hicks Reserve basin is 1.5 m, and in the Russell Bird Reserve basin, which ends up spilling, is 2.7 m at the deepest point.

5.6.5 PMF

The PMF event shows much more extensive flooding than the 0.2% AEP event, as would be expected of a flood of this magnitude. Water of maximum depth 750 mm flows across Jagger Road before leaving the model area to the southern boundary. Water exceeds the capacity of the culvert beneath Jagger Road and flows across it, with shallow depths of up to 200 mm on the road. Two properties along Minke Whale Drive see shallow flooding of maximum depth 200 mm.

Water flows across Minke Whale Drive and Pollard Court, with depths of over 500 mm on the road. Water flows across Krill Court up to a maximum depth of 450 mm across both lanes of the road before flowing into the Hicks Reserve detention basin. Water reaches a depth of 2.8 m in the basin, exceeding its capacity and flowing into the overflow route towards Russell Bird reserve. The capacity of the reserve is also exceeded, resulting in widespread flooding along Solway Crescent and through the properties on this street, with a maximum depth on the road of over 500 mm.

Limited by the capacity of the culvert, water flows over Three Gullies Road and through the back of three properties, with a maximum depth of over 1 m on both lanes of the road. Further downstream at Joy Street, water overtops the road and flows down Wallage Court, inundating a number of properties to the side of the road. Water then spills from Wallage Court back to the channel, passing through a number of properties around Crystal Court in the process. Bayview Grove sees moderate flood depths on the road, with water of over 400 mm depth through a house nearby the channel.

Along Franklin Parade there is varying water depth, reaching upwards of 800 mm near the northern edge of the model.

5.6.6 2% AEP Flood Hazard

In the 2% AEP storm event, there is minimal flood hazard observed along roadways and through properties, with only a handful of locations beyond the channels being vulnerable.

Across Jagger Road where water flows towards the model boundary there is moderate to high flood hazard, primarily caused by high flow velocities as the flood depth is only 200 mm.

Across Minke Whale Drive and Pollard Court the flood hazard is a combination of low and moderate, with water flowing across the roads where culvert capacities are exceeded.

5.6.7 1% AEP Flood Hazard

In the 1% AEP flood, the conditions at Jagger Road are quite dangerous, with much more widespread high flood hazard covering both lanes of the road.

Again Minke Whale Drive near the culvert faces low to moderate flood hazard, however at the cul-de-sac of Pollard Court there is high flood hazard across the road. Elsewhere the flood hazard is well contained within the main channels.

5.6.8 0.2% AEP Flood Hazard

The conditions are significantly more dangerous in the 0.2% AEP event than in the 1% AEP event, with increased flow velocities and depths throughout the model domain.

Across Jagger Road there is extreme flood hazard across both lanes of the road, effectively preventing vehicle passageway, as well as high flood hazard for a much longer section of the road.

There is low flood hazard on Krill Court as water overflows the upstream storage area provided by the land around the culvert inlet. On Minke Whale Drive the flood hazard ranges from low to high, however only a small section of one side of the road shows high flood hazard. Across Pollard Court at both culvert crossings there is moderate to high flood hazard across the road.

Further downstream, water overflows from Russell Bird Reserve and there is high flood hazard along the sides of Solway Crescent. There is also low flood hazard through properties along Solway Crescent.

In the northernmost channel, there is low flood hazard on both sides of Three Gullies Road as water overtops the culvert. From there water remains within the channel until Joy Street, where there is moderate to high flood hazard across the road, and high to extreme flood hazard along both lanes of Wallage Court as water flows along the road back towards the channel.

6 Flood Management Strategies

In addition to floodplain and flood hazard mapping, an initial analysis of potential future flood management strategies has been investigated in isolated select locations. This has been undertaken qualitatively, and more detailed consideration would be required prior to proceeding with any of the recommendations.

Future catchment management and flood mitigation opportunities identified through this study have been separated into two categories:

- Engineering strategies (such as earthworks to modify the floodplain, raising finished floor levels of vulnerable properties and the construction of new stormwater infrastructure); and
- Planning and development control strategies (aimed at future policy changes to reduce overall flood risk and safeguard new dwellings and businesses).

6.1 Engineering Strategies

Structural flood prevention strategies include the construction or upgrade of levees, earthworks to modify the flow of water and hence the overall floodplain, as well as capital works such as dams, culverts or other stormwater infrastructure.

6.1.1 Stock Road Crossing

Following the flood modelling of the major rivers, some of the properties located around the Stock Road crossing over the Inman River were found to be vulnerable to flooding within the 5% AEP event. In events of greater magnitude, the depth of inundation is greater, and additional houses are affected. The cause of this inundation is water spilling out of the Inman River channel upstream of the crossing, where it flows across the ground towards these properties.

The height of Stock Road, both at the crossing and to the east of it, were such that water easily overtopped it and flowed along the road. The proposed arrangements involve redesigning this section of Stock Road, which dips down at the crossing, so that more water is retained within the river channel. Through the modelling, the existing culvert arrangement beneath Stock Road was found to be vastly insufficient, with just three 375 mm diameter pipes. Upon inspecting the DTM, it was observed that there is very little elevation difference between the bed of the channel and the road surface at the crossing, limiting the size of culvert that can be used.

Also of note is that just upstream of the Stock Road crossing, a driveway crosses directly over the Inman River channel, significantly restricting the natural flow path. It is possible that this driveway will need to be removed in order to construct the levee, however aerial imagery shows what appears to be an alternative driveway to these properties which is much further from the main channel, and would be protected by a new levee.

Flood management provisions to protect the affected houses from the 1% AEP flood event were investigated and modelled in TUFLOW. Levees were installed along both sides of the Inman River channel upstream of the crossing. These were aligned logically, considering both the existing surface elevation and the presence of obstructions such as trees and driveways. A longer levee was required downstream of the crossing, to separate the properties from the main Inman River channel, as well as a further two levees each protecting a single vulnerable property. The alignment and height of the levees can be seen in Figure 33 in Appendix A.

Given the significant levee height increases required to prevent water from spilling along Stock Road, the entire crossing, including the existing culverts, will likely need to be redesigned.

6.1.2 Port Elliot West Industrial Area

In the Port Elliot West model, there is a moderately sized industrial use precinct next to Waterport Road which sees significant flooding in the 1% AEP event. The main cause of this flooding is a channel which splits to the north and directs water from a large upstream catchment into the area as shown in Figure 6.1.



Figure 6.1 Watercourse with Split in Flow Path (1% AEP)

As shown in Figure 6.1, the natural watercourse splits, with some water diverting around the industrial area, but the majority of it flowing through the site. Some earthworks at the river channel upstream of the split could ensure that water naturally deviates around the site, avoiding properties. However, this land is privately owned, so negotiations with the property owner(s) would be required.

A watercourse originating from within Alexandrina Council was identified, which produces a substantial amount of runoff that flows over Waterport Road in a 1% AEP storm event. Further north, water from this channel spills onto Lincoln Road and then towards Waterport Road. Installing a larger culvert beneath this road, coupled with a more formal channel connecting it with the upstream river channel, would help to alleviate the flood flows breaching Waterport Road. Given that this land lies within Alexandrina Council, and is privately owned, a collaborative approach would be required between all stakeholders to address this issue.

6.2 Planning and Development Control Strategies

Other non-engineered flood mitigation strategies that could be considered include:

- Running a flood awareness consultation program with the affected community, to aid residents and businesses in preparing flood action plans, identify opportunities to minimise existing flood risk of known vulnerable properties, and more generally to inform the community of the existing flood risk.
- Working with emergency services, government agencies and Council depot staff to ensure that the available floodplain maps are appropriately distributed for use in emergency response.
- Identifying any opportunities, present and future, to provide the affected residents and businesses with real-time or advanced flood warning. Given the long timeframe for the major rivers between the start of rainfall and the peak water levels downstream (greater than 15 hours for the Hindmarsh River and greater than 12 hours for the Inman River), this should be achievable in some capacity. A collaborative approach with the State Emergency Services (SES) and the BoM may be beneficial in this regard.
- Utilising flood plain mapping information to ensure that all new development is constructed well above known flood heights. This should serve to reduce overall flood risk over time, as new land is developed and older houses are demolished and reconstructed.
- Considering on-site detention for all new development nearby the major river channels. This would help to ensure that future development, and in particular land divisions, do not further worsen flood risk. So while this will not reduce existing flood risk, it should limit its increase into the future.

All of these measures should be considered in isolation of proceeding with any structural option.

7 Summary

- Flood inundation maps for storm events including 5%, 2%, 1% and 0.2% AEP, and the PMF event have been produced for the Inman River, Hindmarsh River, Port Elliot West and southern Encounter Bay regions. Flood hazard maps have also been produced for these regions for the 2%, 1% and 0.2% AEP storm events.
- Model peak flows (using the ARR 2016 procedures) were generally found to be slightly larger than those of the previous study.
- Existing flood risk has been evaluated and found to largely correlate with previous assessment of flood risk in this area, however additional inflow locations were added to the southern Encounter Bay and Port Elliot West regions when compared to the previous modelling, resulting in some additional areas of inundation.
- All models were found to be insensitive to climate change insofar as changes in ocean tide level are concerned, with negligible changes in floodplain extent.
- Structural flood mitigation measures have been identified and scoped at a preliminary level, sufficient to provide significant flood mitigation:
 - A levee to protect vulnerable houses from the 1% AEP floodplain in the region surrounding Stock Road and Swains Crossing Road was investigated, and a suitable design has been found which provides 300 mm freeboard to the maximum water height.
 - Some earthworks in the area north of Waterport Road could reduce the flood risk throughout the properties on Buchanan Court and Lincoln Park Drive.
- Non-structural flood mitigation measures that could also be considered include:
 - Leading a flood awareness consultation program with the affected community, to support residents and business owners in preparing flood action plans, identify opportunities to flood-proof vulnerable openings into dwellings and ensuring that the community is appropriately informed such that they can make informed insurance choices.
 - Utilising flood plain mapping information to ensure that all new development is constructed above known flood levels, such that over time flood risk to the area is reduced.
 - Working with SES and Council depot staff to ensure that the flood plain mapping information is appropriately distributed for use in emergency response.
 - Identify any opportunities to provide the affected community with real-time / advance flood warning, as technology in this area improves. This should be achievable given the long timeframe between rainfall onset and downstream flooding of the major rivers, and advances in radar technology, combined with flood warning systems, may provide further opportunities for greater warning times, and reduced flood exposure.
 - On-site detention for any new development within the upstream catchments. This would assist in ensuring that future development does not further exacerbate existing flood risk, although this is unlikely to reduce existing levels of flood risk.

Appendix A

Flood Plain Mapping

