Glenelg Hopkins CMA

Surry River Estuary Flood Study
Study Report

Report No. J543/R03

July 2008

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WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

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GLOSSARY OF TERMS

Term	Description
Annual Exceedance Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1%AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level. Introduced in 1971 to eventually supersede all earlier datums.
Cadastre, cadastral base	Information in map or digital form showing the extent and usage of land, including streets, lot boundaries, water courses etc.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design flood	A significant event to be considered in the design process; various works within the floodplain may have different design events. e.g. some roads may be designed to be overtopped in the 1 in 1 year or 1 00%AEP flood event.
Development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or overland runoff before entering a watercourse and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.
Flood fringe	The remaining area of flood-prone land after floodway and flood storage areas have been defined.
Flood hazard	Potential risk to life and limb caused by flooding.
Flood-prone land	Land susceptible to inundation by the probable maximum flood (PMF) event, i.e. The maximum extent of flood liable land. Floodplain Risk Management Plans encompass all flood-prone land, rather than being restricted to land subject to designated flood events.
Floodplain	Area of land which is subject to inundation by floods up to the probable maximum flood event, i.e. flood prone land.
Floodplain management measures	The full range of techniques available to floodplain managers.
Floodplain management options	The measures which might be feasible for the management of a particular area.

Flood planning area	The area of land below the flood planning level and thus subject to flood related development controls.
Flood storages	Those parts of the floodplain that are important for the temporary storage, of floodwaters during the passage of a flood
Floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often, but not always, aligned with naturally defined channels. Floodways are areas which, even if only partially blocked, would cause a significant redistribution of flood flow, or significant increase in flood levels. Floodways are often, but not necessarily, areas of deeper flow or areas where higher velocities occur. As for flood storage areas, the extent and behaviour of floodways may change with flood severity. Areas that are benign for small floods may cater for much greater and more hazardous flows during larger floods. Hence, it is necessary to investigate a range of flood sizes before adopting a design flood event to define floodway areas.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.
High hazard	Possible danger to life and limb; evacuation by trucks difficult; able-bodied adults would have difficulty wading to safety; potential for significant structural damage to buildings.
Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.
Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
IFD	Intensity Frequency Duration, method of determining design rainfalls according to procedures in Australian Rainfall and Runoff. This includes total rainfall for a given design (ARI) storm event and the pre-determined temporal pattern over which this rainfall is distributed.
Low hazard	Should it be necessary, people and their possessions could be evacuated by trucks; able-bodied adults would have little difficulty wading to safety.
Mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of the principal watercourses in a catchment. Mainstream flooding generally excludes watercourses constructed with pipes or artificial channels considered as stormwater channels.
Management plan	A document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, special features and values of the area, the specific

	management measures which are to apply and the means and timing by which the plan will be implemented.
Mathematical computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff, pipe and overland stream.
Peak discharge	The maximum discharge occurring during a flood event.
Probable maximum flood	The flood calculated to be the maximum that is likely to occur.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedance Probability.
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. For this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Stage	Equivalent to 'water level'. Both are measured with reference to a specified datum
Stage hydrograph	A graph that shows how the water level changes with time. It must be referenced to a particular location and datum.
Stormwater flooding	Inundation by local runoff. Stormwater flooding can be caused by local runoff exceeding the capacity of an urban stormwater drainage system or by the backwater effects of mainstream flooding causing the urban stormwater drainage system to overflow.
Topography	A surface which defines the ground level of a chosen area

1 INTRODUCTION

This report summarises the hydrological and hydraulic investigations undertaken as part of the Surry River Estuary Flood Study.

The report has been prepared for the following purposes:

- Document the hydrologic analysis of the Surry River catchment
- Document the level of uncertainty in the development of design flows for the Surry River
- Document the structure and development of the hydraulic model
- Document the sensitivity analysis of the hydraulic model parameters
- Document the design flood conditions modelling

2 AVAILABLE INFORMATION

This section documents the various sources of information utilised for the study

2.1 Previous Studies

Previous hydrologic and/or hydraulic studies relevant to the present project and region include:

- Report on the Western District Floods of March 1946 (SR&WSC 1946) – This report documented and examined the severe flooding that occurred on the 16th to 19th March 1946. This flood event is the largest on record and hence this information is particularly beneficial to the hydraulic model calibration process.
- South Warrnambool Flood Study (Water Technology, 2007) – This study investigated flooding of the Merri River and Kelly and Salt Swamp. The investigations into design ocean water levels undertaken in this study have been applied for the Surry River estuary

2.2 Topographic and Cadastral Survey Data

2.2.1 Overview

Topographic and cadastral data have been collected from a number of sources including:

- Aerial Survey
- Field Survey
- Bathymetric Survey

The location and extent of the various sources of topographic data gathered as part of the study are illustrated in Figure 2-1. A listing of survey sources, along with the nominal accuracy of the data is provided below in Table 2-1.

2.2.2 Aerial Photogrammetry

Low level aerial photogrammetry of the Surry River estuary and the immediate surrounds was undertaken by QASCO in January 2007. This low-level photogrammetry has a derived vertical accuracy of +/- 100mm to one standard deviation. A full metadata description of this information is provided in Appendix A. The photogrammetry data consisted of a 10 metre grid of spot elevations and breaklines defining linear features in the topography.

Validation of the accuracy of the photogrammetry was undertaken by comparing elevations from a digital terrain model developed from the photogrammetry with four permanent survey marks within the area captured by the photogrammetry. The comparisons made against the permanent survey marks are provided in Appendix A and are considered to validate the levels developed from the aerial photogrammetry.

2.2.3 Field Survey

Field survey was conducted by Berry & Whyte Surveyors Pty Ltd. The field survey was undertaken to supplement the aerial photogrammetry. The field survey included the following:

- Defining the topography of the wetland area to the immediate west of the estuary that was inundated at the time the aerial photogrammetry was undertaken.
- Princess Highway Bridge Structural Details
- Wades Road Bridge Structural Details

- Culvert dimensions and invert details at a number of locations within the study area.

General arrangement drawings for Wades Road and Princess Highway Bridge's are attached in Appendix B.

2.2.4 Bathymetric Survey

Acoustic sounding survey of the estuary bathymetry was provided by the Glenelg Hopkins CMA. The low frequency (30KHz) return soundings were employed for the study as these levels are considered to represent the denser substrate as opposed to very soft muds and vegetation mats.

Figure 2-1 Topographic Survey

Table 2-1 Topographic Survey Sources and Nominal Accuracy

Data	Estimated Nominal Accuracy	Source
10 m base contour data (from 1:25,000 state mapping)	Vertical +/- 5 m Horizontal +/- 10 m	Land Victoria
Photogrammetric points and breaklines	Vertical +/- 0.1 m (1 Standard Dev.) Horizontal +/- 5 m	(QASCO, 2007)
Field Survey	Vertical +/- 0.05 m Horizontal +/- 1 m	Berry & Whyte Surveyors Pty Ltd (2007)
Bathymetric Data	Vertical +/- 0.10 m (>0.6m depths) Horizontal +/- 0.5 m	Redborough Mapping Pty Ltd

Note: As appropriate meta-data is not available for most data sources, reasonable estimates of survey accuracy have been made based on the capture techniques used and experience with previous, similar data sets.

2.3 Streamflow Data

Two streamflow gauging stations were employed in the study. Details of the streamflow gauges are listed below in Table 2-2 and their locations are displayed in Figure 2-3..

Table 2-2 Details of Streamflow Gauge

Station No.	Station Name	Catchment (km^2)	Period of Record
237207	**Surry River at Heathmere**	**310**	**1970 – 2006**
237202	**Fitzroy River at Heywood**	**234**	**1948 - 2006**

The gauge on the Surry River at Heathmere is located on a reasonably well confined section of the river. Former local resident John Fyfe has witnessed a minor overland flow path originating from the Surry River that bypassed the gauge during the 1976 flood. However, based on John Fyfe's description of the depth and width of the overland flow witnessed, this flow was considered a minor (<5%) proportion of the overall flow in the Surry River. Hydrographic flow measurements have been undertaken up to 2.21 metres on the gauge, above this level the rating has been extrapolated. The rating is considered reasonably reliable given the accuracy limitations of the underlying hydrographic flow measurements used to develop the rating. The rating for the Surry River Gauge at Heathmere is displayed in Figure 2-2.

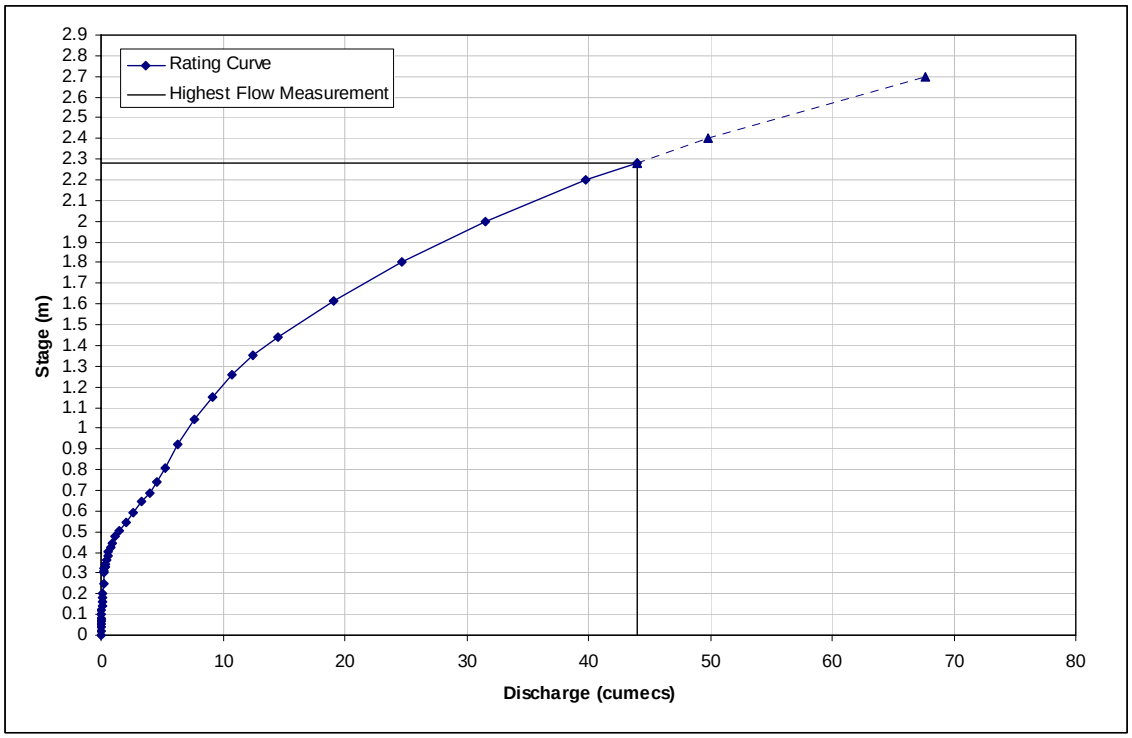


Figure 2-2 Surry River Gauge at Heathmere Rating (237207)

2.4 Rainfall Data

An analysis of Bureau of Meteorology records shows that no rainfall stations are located within the Surry River catchment. However, there are a number of daily rainfall stations with significant periods of record close to the catchment boundary. There are no pluviographic (rainfall intensity) stations within or in the near vicinity of the catchment, the closest being at Casterton or Mortlake which are approximately 80 km and 100 km to the north and east respectively. These are considered too distant from the catchment to be reliable for use in this study.

Table 2-3 Details the daily rainfall stations in the immediate vicinity of the Surry River catchment.

Table 2-3 Details of Rainfall Stations

Station No.	Station Name	Type	Period of Record
0900150	**Gorae**	**Daily**	**1905 - 1920**
090048	**Heywood Forestry**	**Daily**	**1949 - Present**
090070	**Portland**	**Daily**	**1872 - Present**
090038	**Tyrendarra**	**Daily**	**1907 – Present**
090124	**Narrawong**	**Daily**	**1892 – 1919**

090013	**Cape Bridgewater**	**Daily**	**1905 – Present**
090050	**Kentbruck**	**Daily**	**1940 - Present**

Catchment averaged rainfalls were developed from Thiessen polygon weightings to allow a record of daily average catchment rainfalls to be developed from the Kentbruck, Portland and Heywood Forestry rainfall stations. These three stations were found to have the most complete historical record and allowed a catchment averaged rainfall record to be developed for the subsequent hydrologic analysis. The location of the rainfall stations and the Thiessen polygon weightings are displayed in Figure 2-3.

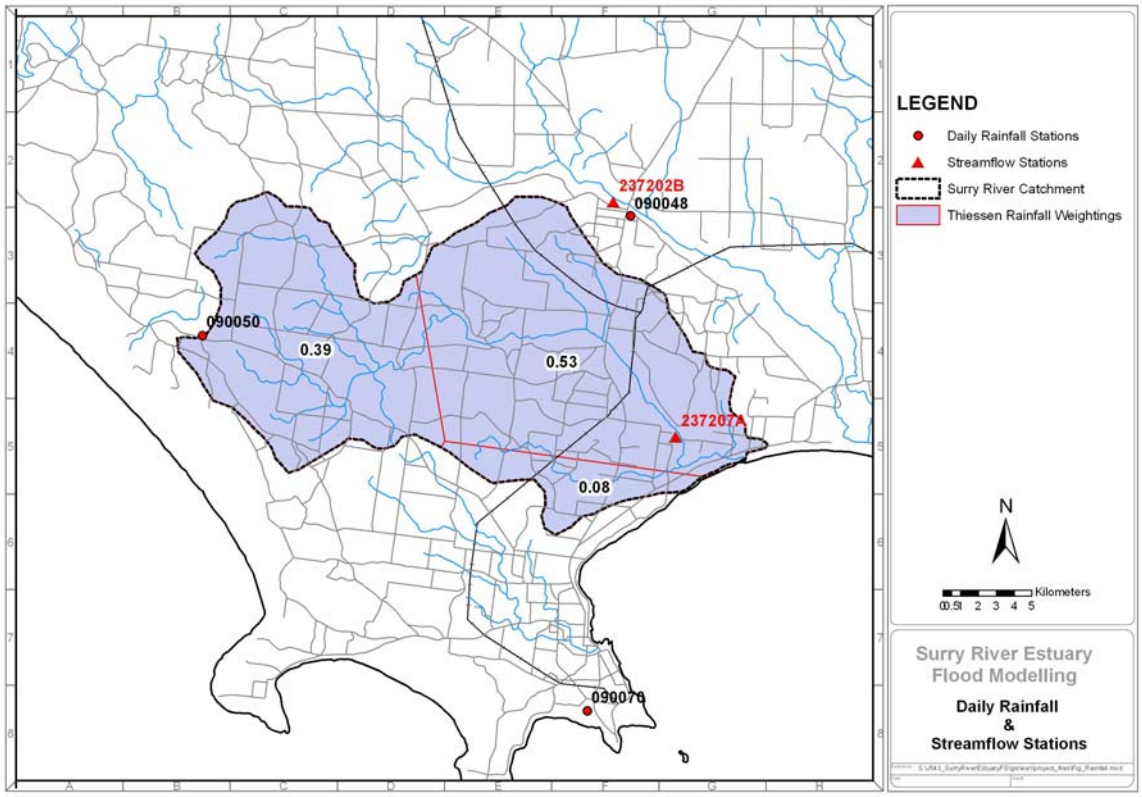


Figure 2-3 Daily Rainfall Stations and Streamflow Stations

2.5 Historic Flood Level Data

In the preparation of this study no historic flood level or extent data was located for any major historical floods. However during the course of the study, a small flood was observed in early November 2007. Some flood level data was captured from this flood and was employed in the model validation described in Section 8.

3 HYDROLOGIC ANALYSIS

3.1 Overview

Design flood hydrographs were required for the 20, 10, 5, 2 & 1 % Annual Exceedance Probability floods and the Probable Maximum Flood (PMF) for the Surry River at Narrawong.

It was recognised early in the study inception that the relatively short period of observed streamflow record at Heathmere and the lack of representative pluviographic rainfall observations would limit the reliability of the design flood estimates from a conventional flood frequency analysis and/or calibration of a rainfall-runoff-routing model.

A number of methodologies have therefore been explored in order to improve confidence in the reliability of design flow estimates for the Surry River.

The following three approaches have been used to develop design flow estimates for the Surry River:

- Flood Frequency Analysis (FFA) on the Surry River at Heathmere
- Scaled FFA estimates from the Fitzroy River at Heywood
- RORB rainfall-runoff model development and "calibration" to design flow estimates at the Surry River gauge at Heathmere

3.2 Flood Frequency Analysis for the Surry River at Heathmere

An annual series flood frequency analysis on the recorded streamflow data at Heathmere has been undertaken. Thirty years of instantaneous streamflow records between 1976 and 2006 are available. An additional 6 years of historical mean daily flow data between 1970 and 1975 also exists for the Surry Gauge at Heathmere. A regression analysis comparing maximum annual average daily flows to maximum annual instantaneous peak flow for the thirty years of record showed a very strong correlation at the Surry River gauge between these two values (Appendix C). The regression relationship was therefore used to convert the additional six years of historical daily flows to instantaneous peak flows. This allowed the flood frequency analysis to be undertaken with a combined total of thirty-six years of streamflow record for the Surry River at Heathmere.

The recorded data was fitted to a Log Pearson III Distribution. Six low-flows were omitted from the analysis to reduce the skewness to acceptable limits. The flood frequency analysis curve is shown in Figure 3-1 and design flow estimates are summarised in Table 3-1.

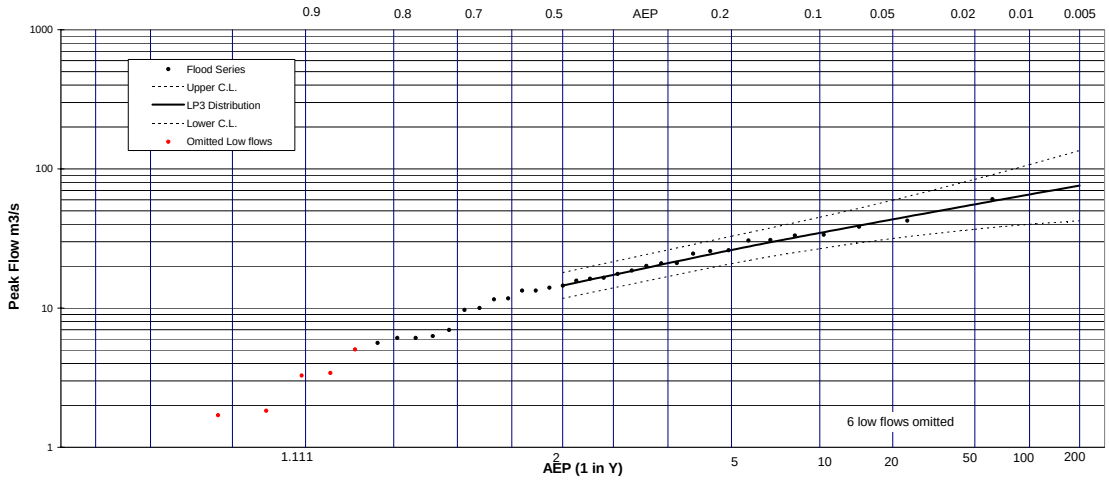


Figure 3-1 Flood Frequency Analysis of the Surry River at Heathmere

Table 3-1 Summary of Flood Frequency Analysis

AEP (%)	Peak Design Flow (m^3/s)	5% & 95% Confidence Limits (m^3/s)
20	26	21 – 33
10	34	26 – 45
5	43	31 – 59
2	54	35 – 82
1	63	38 – 105

The use of a relatively short length of streamflow record is reflected in the large confidence intervals around the design flow estimates. Anecdotal evidence also suggests that a number of significant floods occurred before streamflow was recorded in the catchment (March 1946 for example).

Additional analysis in the following sections has therefore been undertaken to provide additional estimates of the magnitude of the design flows on the Surry River.

3.3 Scaled Flood Frequency Analysis for the Fitzroy River at Heywood

Design flow estimates for the Surry River at Heathmere have been developed through a relationship between the annual series for the Fitzroy River at Heywood and the Surry River at Heathmere.

The Fitzroy River catchment to Heywood is an adjoining catchment to the Surry River Catchment. The Fitzroy River catchment area to Heywood is approximately 234 km^2 compared to the Surry River catchment area to Heathmere of approximately 310 km^2.. The two catchments are similar in size, geographical location, land use and topographic relief. The two catchments could therefore be expected to display similar catchment runoff characteristics.

The Fitzroy River at Heywood has 37 years (1969 – to date) of continuous instantaneous streamflow record. An additional 21 years (1948 – 1968) of daily historical flow data is also available. A regression analysis comparing maximum annual average daily flows to maximum annual instantaneous peak flow for the 37 years of record showed a strong correlation at the Fitzroy River gauge (Appendix B). The regression relationship was used to convert the additional 21 years of maximum annual historical daily flows to instantaneous peak flows. This allowed a flood frequency analysis to be undertaken over a combined total of 58 years of streamflow record for the Fitzroy River at Heywood. The use of a 58 year annual series results in smaller confidence intervals for the 1% AEP flood for the Fitzroy River at Heywood.

Five low-flows were omitted from the analysis to improve the fit between the recorded data and the Log Pearson III Distribution. The flood frequency analysis curve over the 58 year annual series for the Fitzroy River at Heywood is displayed in Figure 3-2.

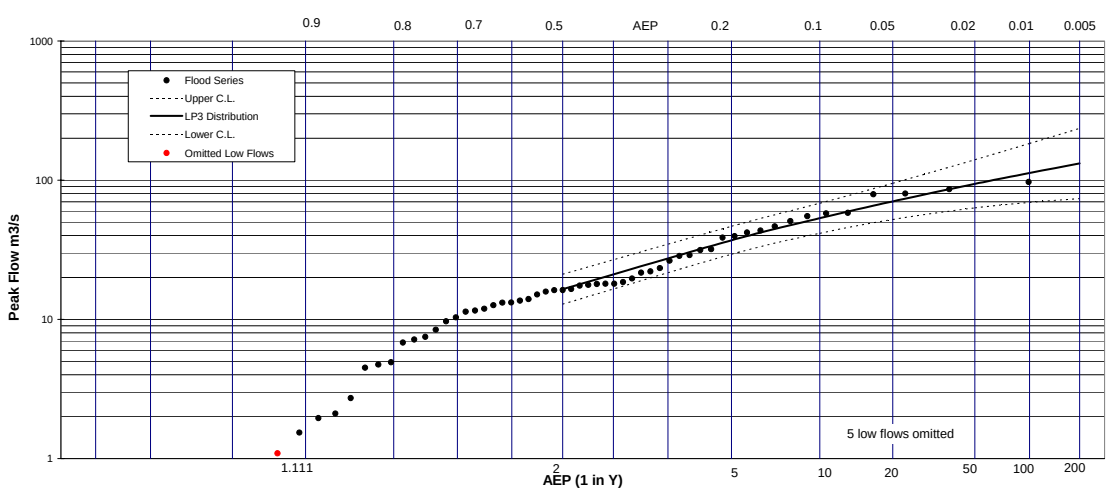


Figure 3-2 Flood Frequency Analysis of Fitzroy River at Heywood

A comparison of the coincident annual flood series between the Surry River at Heathmere and the Fitzroy River at Heywood is displayed in Figure 3-3.

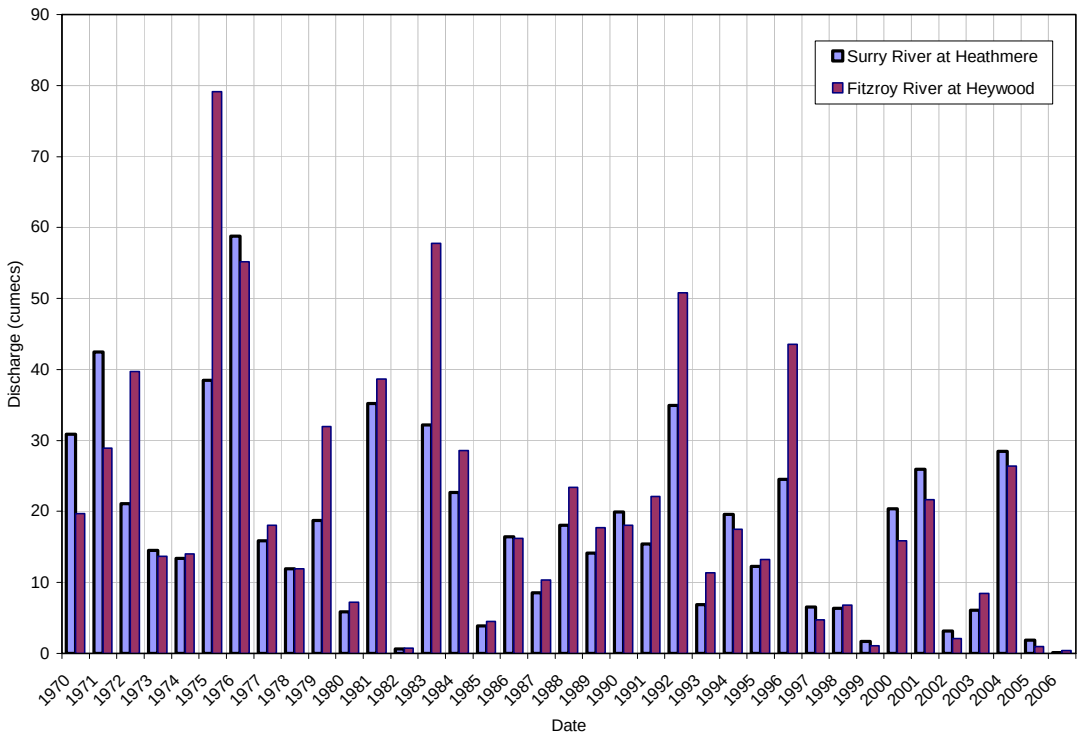


Figure 3-3 Comparison of the coincident annual flood series between the Surry River at Heathmere and the Fitzroy River at Heywood

The following observations can be made relating to the differences in the catchment runoff characteristics between the two catchments displayed in Figure 3-3.

- In broad terms it is reasoned that a strong correlation exists between the two catchments, with the pattern of high and low flood-flow years reproduced throughout the coincident record.
- For the majority of floods, the magnitude of the flood flows is similar, despite the Fitzroy River catchment area to Heywood being approximately 20% smaller than the Surry River catchment area at Heathmere. This could possibly be reasoned to indicate that the Fitzroy River catchment displays slightly higher catchment runoff characteristics than the Surry River catchment given similar rainfall inputs.
- There is some evidence to suggest that for larger floods, there is a significant departure in the nature of the runoff characteristics between the two catchments. The evidence available in the record is however not definitive, and the differences observed could possibly be due to variations in rainfall intensity and depths occurring across the two catchments during the same storm event.

Despite the differences observed in the catchment runoff characteristics displayed in Figure 3-3 and discussed above, a relationship has been applied to develop design flow estimates for the Surry River at Heathmere based on the flood frequency analysis of the Fitzroy River at Heywood.

This relationship is based on the methodology outlined in Hydrological Recipes (CRC-CH, 1996) for extending a short flow record.

The design flow estimates for Heathmere were determined using the following parameters:

Surry River at Heathmere – 300 km^2

Fitzroy River at Heywood --234 km^2

Multiplier Function (F)

F = (Ac/Ag)^0.7

Where
Ac = catchment area of the ungauged catchment (km^2)
Ag = catchment area of the gauged catchment (km^2)

*Surry River at Heathmere Q_y = 1.19*Fitzroy River Q_y*

Table 3-2 Flood Frequency Analysis of Fitzroy River and Scaled Surry River Design Flows

AEP (%)	Fitzroy River at Heywood		Surry River at Heathmere
	Design Flow (m^3/s)	**5% & 95% Confidence Limits (m^3/s)**	**Scaled Design Flow (m^3/s)**
20	**37**	**29 - 47**	**44**
10	**53**	**42 - 68**	**63**
5	**70**	**52 - 95**	**83**
2	**94**	**63 – 140**	**112**
1	**113**	**74 - 236**	**134**

Based on the observed differences in the catchment runoff characteristics between the Fitzroy River and the Surry River, the design flows derived in Table 3-2 are expected to overestimate the magnitude of the design flows for the Surry River.

3.4 RORB Model Application to the Surry River Catchment

3.4.1 Background

The runoff-routing model RORB, developed by Laurenson and Mein (1975), was used to estimate design flood hydrographs for the Surry River at Heathmere. RORB is a general runoff and streamflow routing program that calculates flood hydrographs from rainfall and other catchment characteristics. The model subtracts losses from rainfall to determine surface runoff which is then routed through a network of storages to produce flood hydrographs at points of interest. RORB is an areally distributed, non-linear model that is applicable to both urban and rural catchments. The model can account for both temporal and spatial distribution of rainfall and losses.

The model is based on catchment geometry and topographic data. RORB has two principal parameters, k_c and m. The parameter m describes the degree of non-linearity of the catchment's response to rainfall, while the parameter k_c describes the storage available within

the catchment. The rainfall loss parameters relate to the conversion of rainfall into surface runoff. The RORB model can represent these losses either by the initial-loss/continuing-loss model, or by the initial-loss/volumetric-runoff-coefficient model. The catchment is divided into sub-areas based on topographical features. This catchment sub-division allows for spatial variation of catchment characteristics and rainfall inputs.

3.4.2 RORB Model Development

A RORB model of the Surry River catchment to Narrawong was developed by dividing the catchment into a number of sub-areas based on the topography and drainage characteristics of the catchment. For design flood estimation purposes all reach types within the catchment were designated as natural. The sub-catchment delineation in the RORB model is presented in Figure 3-4

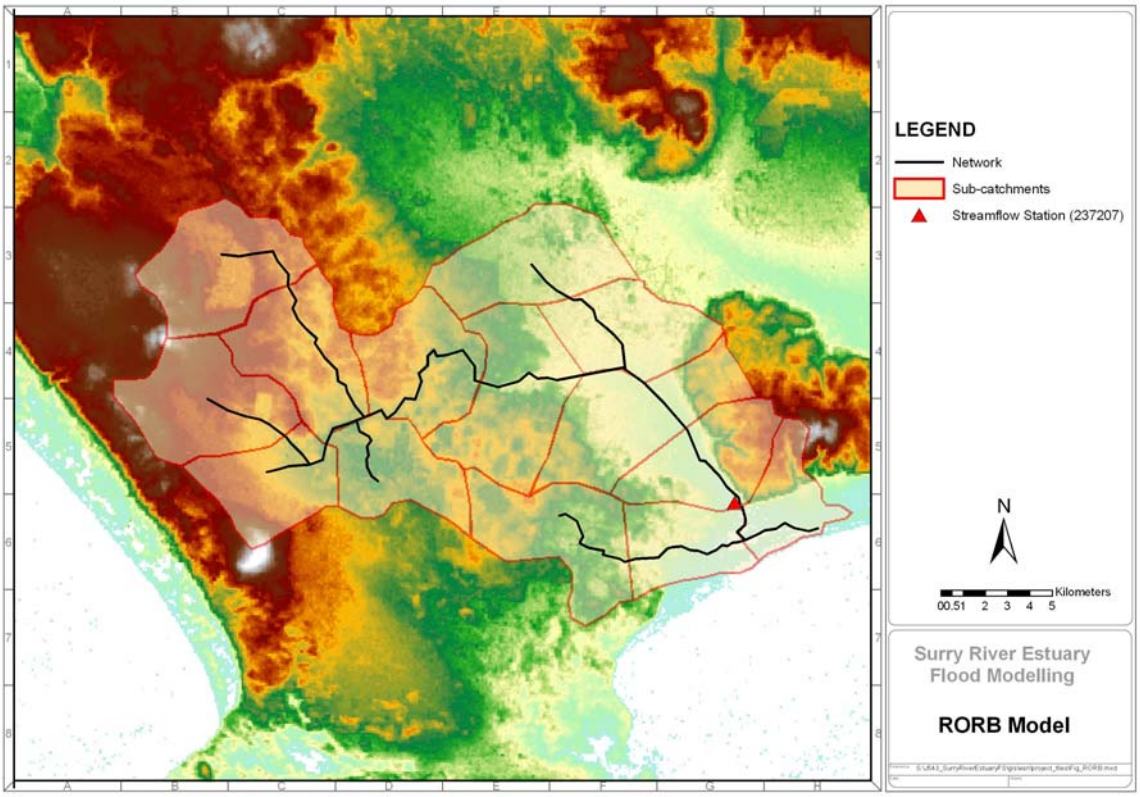


Figure 3-4 RORB Model Subcatchment Delineation

3.4.3 RORB Model Parameter Selection Approach

The selection of appropriate RORB model parameters ideally requires calibration through the comparison of the modelled flood hydrographs with observed flood hydrographs at streamflow gauge(s) throughout the catchment. The selection of suitable historical flood events for RORB model calibration is, however, also dependent on the availability of concurrent streamflow and pluviographic rainfall data. As no representative pluviographic rainfall data is available for the Surry River catchment, RORB model parameters and rainfall losses have been derived through analysis of the historical daily rainfall and streamflow records for the Surry River.

The flood frequency analysis on the 36 year annual series for the Surry River is considered to provide a good estimate of the magnitude of the 10 % AEP flood (Refer to Section 3.2).

Historical flood hydrographs with peaks flows of similar order to the 10% AEP flood have therefore been used in combination with the daily rainfall records to assist in the selection of appropriate RORB model parameters. The 10% AEP storm IFD rainfall from ARR was used to develop 10% AEP design hydrographs in the RORB model.

The RORB model parameters were adjusted to ensure the modelled hydrograph shapes and critical durations were broadly representaitve with those observed in the Surry River during floods with magnitudes similar to the 10% AEP peak flow.

Rainfall losses have been estimated for the Surry River catchment by ensuring the losses adopted in the model produce a similar ratio of rainfall to rainfall excess based on an analysis of the daily rainfall and streamflow records.

3.4.4 Rainfall Loss Estimation

The selection of rainfall losses has a significant impact on the magnitude of flood estimates. An analysis of the historical daily rainfall data and streamflow records has been undertaken to estimate the magnitude of these losses. A total of five significant historical floods have been analysed. For each flood, the total hydrograph volume, expressed as depth in millimetres over the entire catchment was determined. This depth represents the rainfall excess converted to runoff.

A comparison of the catchment averaged rainfall depths and resulting flood hydrograph volume (as a function of depth over the entire catchment) are presented in Figure 3-5 and summarised in Table 3-3. This analysis indicates that in general, approximately 40-60 % of rainfall is converted to rainfall excess. An average of 48 % of rainfall was converted to rainfall excess over the five floods analysed.

Rainfall temporal patterns and antecedent catchment conditions would be expected to influence the losses occurring for individual floods. However, the analysis undertaken is considered to provide a reasonable estimate of the magnitude of the rainfall losses expected for the Surry River catchment and provides a useful validity check of the losses adopted in the RORB model for design flood modelling purposes.

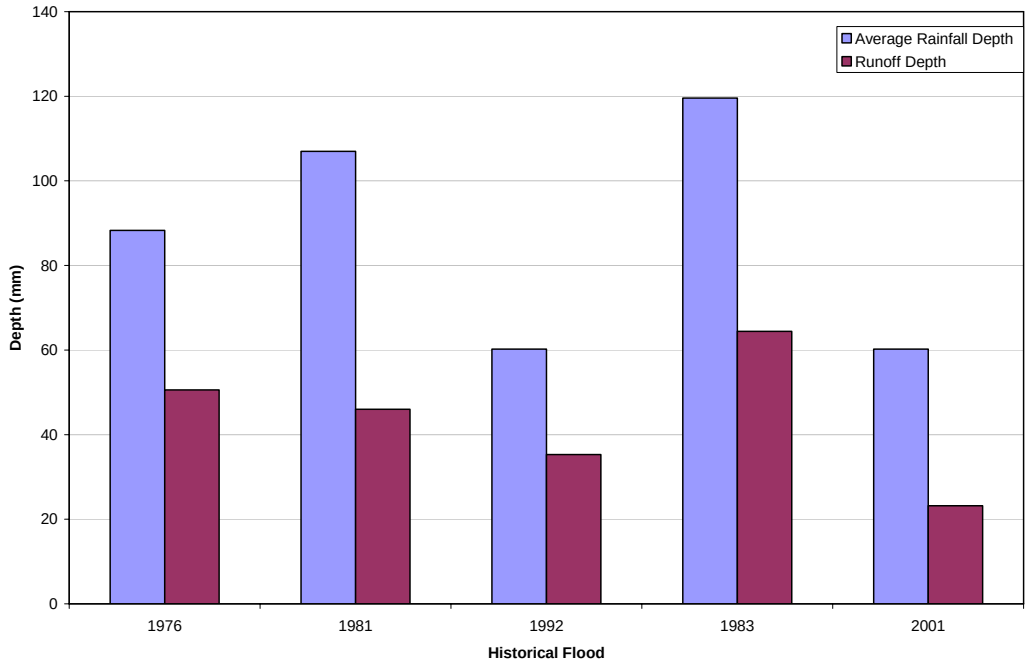


Figure 3-5 Comparison of Rainfall and Rainfall Excess for Historical Floods

Table 3-3 Summary of Rainfall and Rainfall Excess for Historical Floods

Flood	Rainfall (mm)	Runoff (mm)	% Runoff
1976	**88**	**45 - 56**	**51 – 64**
1981	**107**	**41 - 51**	**38 – 48**
1992	**72**	**32 -39**	**44 – 54**
1983	**120**	**58 – 71**	**48 – 59**
2001	**60**	**21 - 26**	**35 - 43**
		Average	43 – 54

3.4.5 Design Rainfall Depths

Design rainfall depths were calculated for the 1 in 20, 50 and 100 year events using the IFD procedures outlined in ARR87. The IFD parameters are provided in Table 3-4.

Table 3-4 Surry River Catchment IFD parameters

IFD Parameter	Value
1 hour duration 2 year ARI	15.22
12 hour duration 2 year ARI	3.5
72 hour duration 2 year ARI	0.98
1 hour duration 50 year ARI	30.0
12 hour duration 50 year ARI	5.5
72 hour duration 50 year ARI	1.6
Regional skew G	0.62
Geographic factor F2	4.34
Geographic factor F50	14.58
Zone	6

3.4.6 Areal Reduction Factor

The areal reduction factor is the ratio between the areal average catchment rainfall intensity and the point rainfall intensities provided in AR&R (1987). The areal reduction factor allows for the fact that larger catchments are unlikely to experience high intensity storms over the whole catchment area. The Siriwardena and Weinmann (AR&R, 1999) areal reduction factors have been applied. These factors were developed from the empirical analysis of data in south eastern Australia and provided satisfactory results on 11 representative catchments in Victoria.

3.4.7 Design Temporal Patterns

The AR&R (1987) design filtered temporal patterns for Zone 6 were used.

3.4.8 Design Spatial Patterns

The low relief of the Surry River catchment precludes any significant orographic influence on rainfall distribution in the catchment. Also, for large rainfall events, no consistent pattern in the spatial variation of rainfall depths was considered evident in the historical rainfall records. For these reasons, a uniform spatial rainfall pattern (i.e. same rainfall depths applied to the entire catchment) was adopted.

3.4.9 RORB Model Parameter Selection

The following approach was adopted to determine the RORB model parameters (k_c & m) and design loss parameters, initial loss (IL) and continuing loss (CL):

- The October 1992, September 1983 and August 1981 historical floods were analysed as they had similar peak flows to the 1 in 10 ARI flood and the flood peak could be reasonably isolated and associated to a discrete rainfall event

- Catchment averaged rainfalls derived from the Thiessen polygon weightings were analysed for the same three historical flood events to provide an indication of the approximate rainfall depths and storm durations.
- Rainfall losses were adjusted to provide a similar rainfall to rainfall-excess ratio as developed in Section 3.4.4.
- The RORB model parameter k_C was adjusted to provide a broadly similar initial response, peak, volume and recession in the modelled hydrograph to that observed in the historical hydrographs for 10% AEP storm. The parameter *m* was adopted as 0.8. This value is a generally accepted value for the degree of non-linearity of catchment response (ARR 1987).
- A constant baseflow component of 2 m^3/s was adopted based on the approximate baseflow rate apparent in the historical flood hydrographs.

Available rainfall and streamflow data for the catchment was employed to inform the selection of appropriate RORB model parameters and rainfall losses. The study team considers that the approach adopted provides for more representative design flood hydrographs for the Surry River catchment than would be provided by the adoption of solely regional-based estimation methodologies. In support of this approach it is noted that for the three historical floods analysed, the RORB model reproduced the following catchment responses:

- For a given depth of rainfall, a similar ratio of rainfall is converted to streamflow in the model as is observed for the catchment.
- Similar critical storm durations observed in the catchment are reproduced by the model.
- Similar hydrograph durations observed in the catchment are reproduced by the model.
- For a given depth of rainfall and approximate storm duration, the modelled peak flow is similar to that observed for the analysed historical floods.

The RORB model parameters and rainfall losses developed from the comparison of the historical flood hydrographs and daily rainfall records are displayed in Table 3-5.

Table 3-5 Developed RORB Model Parameters

RORB Model Parameter	k_C	*m*	IL(mm)	CL(mm)
Value	**75**	**0.8**	**4.0**	**1.3**

Figure 3-6 displays the modelled flood hydrographs compared to the historical flood hydrographs for 24, 36 and 48 hour storm durations. Table 3-6 summarises the comparison between the historical flood rainfall depths and rainfall excess and the 10 year IFD rainfall depths and modelled rainfall excess.

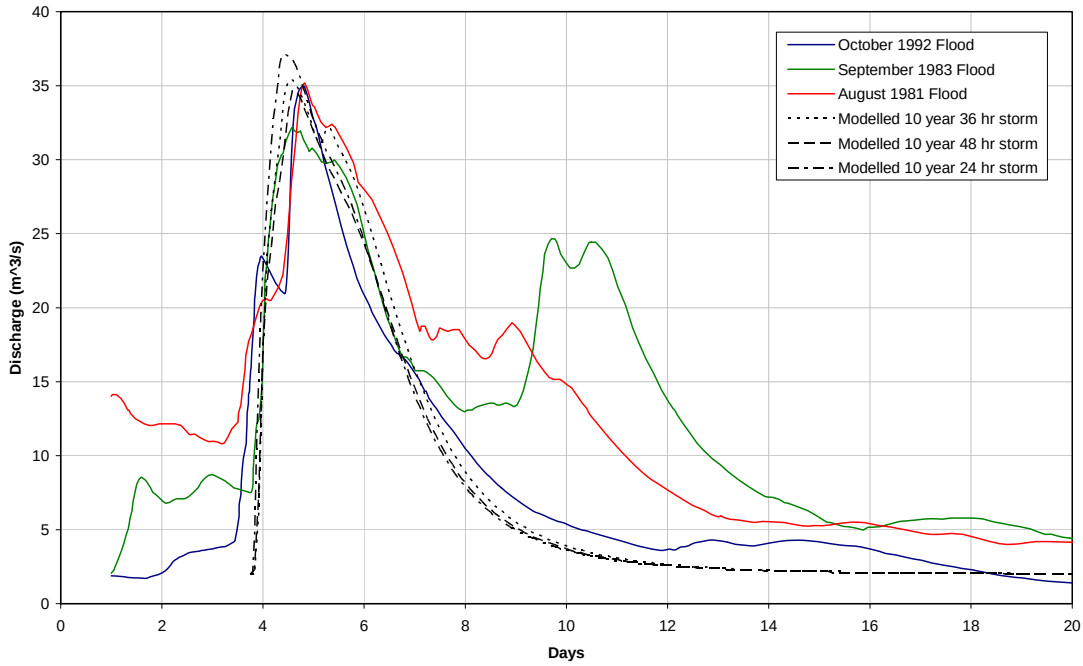


Figure 3-6 Comparison of Modelled and Historical Flood Hydrographs

Table 3-6 Rainfall Excess Comparison between Historical Floods and RORB Model

Historical Floods	9:00AM Cumulative Rainfall Depths (mm)			Rainfall Excess (%)
	24hr	**48hr**	**72hr**	
August 1981	**32**	**46**	**60**	**39 - 47**
September 1983	**27**	**49**	**49**	**49 – 59**
October 1992	**16**	**22**	**63**	**44 - 54**
Modelled Floods	Total Rainfall Depth (mm)			
10 Year 24 Hr	**61**			**46**
10 Year 36 Hr	**71**			**40**
10 Year 48 Hr	**78**			**34**

3.4.10 RORB Model Parameter Selection Verification

It was considered constructive to assess the validity of the RORB model parameters developed in Section 3.4.9 through a validation exercise on the largest flood in the streamflow record at Heathmere.

The October 1976 flood is the largest recorded and has an estimated peak flow of 61 m^3/s at the Heathmere gauge. The catchment-averaged rainfall depth has been estimated at 71.7 mm in the 24 hours to 9:00 am on the 16th October. The rainfall resulting in the flood fell almost exclusively on the 16 October. It is therefore not considered unreasonable to assume the storm duration was of the order of 12 to 24 hrs. A similar rainfall depth is provided by the 2% AEP

storm for these durations. Despite differences in the storm temporal patterns and uncertainties in the initial loss for this flood, it is reasoned that modelling of the 2% AEP storm with the developed RORB model parameters should produce a flood hydrograph broadly similar to that observed during October 1976. The 12 hr and 18 hr, 2% AEP storms were therefore modelled in RORB with the same RORB model parameters developed during the calibration. The comparison of the modelled and observed flood hydrographs are displayed in Figure 3-7. The rainfall depths and rainfall excess comparison between modelled and observed floods is displayed in Table 3-7.

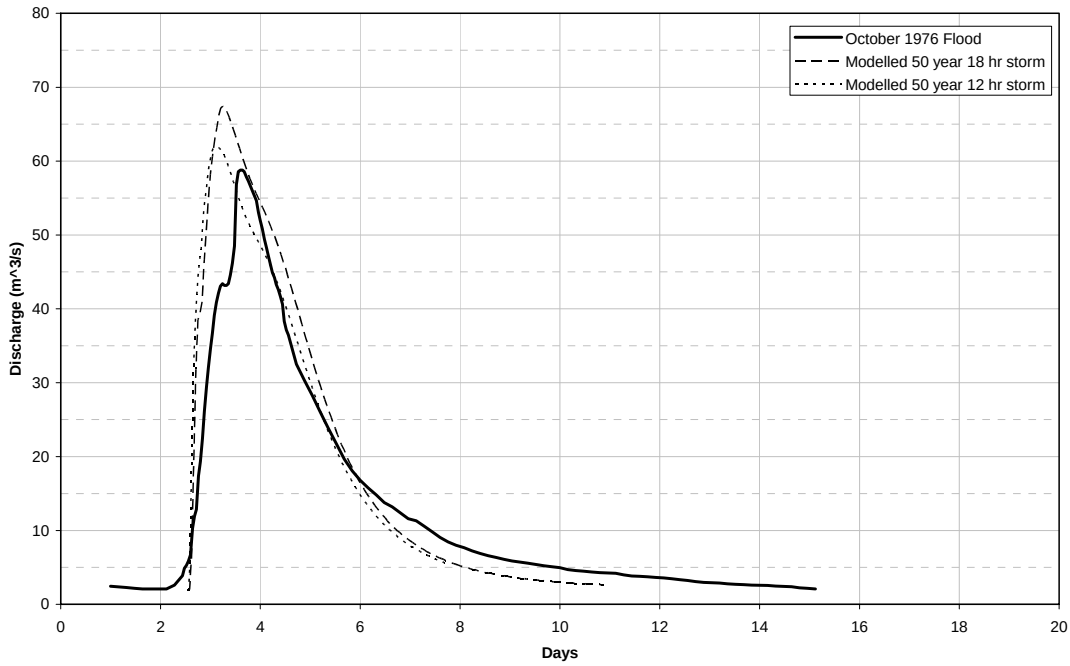


Figure 3-7 October 1976 Flood – RORB Model Validation

Table 3-7 Rainfall Excess Comparison between the October 1976 Flood and the RORB Model

Flood	Total Rainfall Depth (mm)	Rainfall Excess (%)
October 1976	**72 (24 hrs)**	**51 - 63**
50 year 18 hr storm	**72**	**63**
50 year 12 hr storm	**61**	**69**

Considering the degree of uncertainty inherent in this exercise, the level of agreement between the observed October 1976 flood hydrograph and the 2% AEP storm hydrographs is considered satisfactory. Despite the fact that considerable differences in the storm temporal

patterns are likely to have occurred and uncertainties in the initial-loss for this flood, the RORB model parameters developed have resulted in a modelled flood hydrograph in good agreement with that observed for a similar depth and duration of rainfall on the Surry River catchment. This is considered to provide an increased level of confidence that the RORB model parameters developed are reasonably representative of the particular storage and routing characteristics occurring in the Surry River catchment.

3.4.11 Discussion

The lack of representative pluviographic rainfall data for the Surry River catchment has prevented conventional calibration of the RORB model parameters. Additional analysis of the daily rainfall records and historic flood hydrographs provides useful information on the rainfall runoff characteristics of the catchment. The rainfall runoff characteristics developed from this analysis have been used to inform the selection of reasonable RORB model parameters for the Surry River catchment.

The k_C developed during the calibration is approximately 2 to 3 times the k_C values developed from regional estimate techniques for the Surry River catchment (as provided in ARR). It is noted however that within the Surry River catchment extensive broad swampy areas occur along tributaries and the Surry River itself is poorly defined in sections. This suggests that significant flood storage exists within the catchment and provides some justification for the adoption of a larger k_C value. It is also noted that adoption of a smaller k_C would produce design hydrographs significantly different in shape to any of the observed flood hydrographs.

Despite the analysis undertaken, considerable uncertainty remains in the adoption of appropriate RORB model parameters for the Surry River catchment. Due to the imprecise nature of the parameter selection approach, various RORB model parameters could be modified to produce broadly similar results as has been produced with the parameters developed. However it is considered that the calibration process has provided a reasonable set of parameters for describing the runoff characteristics of the Surry River catchment. The adoption of the RORB model parameters for design flood modelling is therefore considered to provide reasonably reliable design flood estimates for the Surry River catchment.

3.4.12 RORB Design Flood Estimates

The RORB model parameters developed in Section 3.4.9 were used to estimate design flows for the Surry River over a range of recurrence intervals. The design flood estimates produced by the RORB model are presented in Table 3-8.

Table 3-8 RORB Design Flood Estimates

Design Flood	Kc value	Rainfall loss parameters (mm)		Surry River at Heathmere	
		IL	CL	Peak flow (m^3/s)	Duration (hrs)
20% AEP	75	5	1.3	31	18
10% AEP	75	5	1.3	39	18
5% AEP	75	5	1.3	51	18
2% AEP	75	5	1.3	68	24
1% AEP	75	5	1.3	83	24

3.4.13 Review of March 1946 Flood

Exceptional rainfall totals was recorded in the western district of Victoria between 15 and 18 March 1946. The rainfall resulted in major flooding in the district with the magnitude of the flooding on the Merri River estimated to have exceeded the 1% AEP flood (Water Technology, 2007).

The impact of flooding in the western district during March 1946 was documented by the State Rivers and Water Supply Commission (Rural Water Corporation, 1993). Only fleeting references to the impact of the event on the Surry River are made in the report. The local Shire Engineer at the time reported that no undue complaints about the flooding on the Surry River had been received.

The lack of any substantial data on the impact of the March 1946 on the Surry River estuary, limits the ability to undertake a formal analysis to estimate this floods recurrence interval. Any such analysis would therefore be considered largely speculative and unlikely to inform the adoption of appropriate design flood conditions for the estuary.

3.5 Design Flood Estimates Comparison and Discussion

The hydrological analysis undertaken for the Surry River has highlighted the degree of uncertainty that exists in determining appropriate design flows for the Surry River. A number of approaches for estimating the magnitude of design flows on the Surry River have been undertaken to help inform the likely range of design flood estimates expected. The approaches employed to estimate the magnitude of the design flows on the Surry River have been compared directly in Figure 3-8. From Figure 3-8 the following observations can be drawn:

- The design flow estimates for the Surry River, derived from scaling of the Fitzroy River design flow estimates, result in design flows for the Surry River falling well outside the confidence limits developed from the flood frequency analysis of the Surry

River streamflow data. This is consistent with the observed differences in the catchment runoff characteristics between the two catchments.

- The RORB model produces peak flow estimates broadly in agreement with the flood frequency analysis, although for the 2% and 1% AEP flows, the RORB model is producing floods approximately 20% and 25 % larger respectively. This is consistent with flood frequency analysis under-predicting the magnitude of more extreme floods due to the relatively short period of the streamflow record available. The design flows developed from the RORB model fall within the confidence limits developed from the flood frequency analysis of the Surry River streamflow data.

Based on the observations above and results of the parameter selection and validation exercises in the development of the RORB model, the study team consider the design flood estimates produced by the RORB model as the best available estimate of design flow magnitude (both in terms of peak flow and volume) for the Surry River at Heathmere.

Given the uncertainty inherent in their development, analysis of the sensitivity of the design flood estimates on the study outcomes has been undertaken as part of the hydraulic analysis. It is considered, however, that while the analysis of the sensitivity of hydrologic model parameters will provide an indication as to the possible range of design flood estimates for the Surry River, it will not in itself provide a good indication of the reliability of the study outcomes. The floodplain geometry characteristics and influence of the sea-level boundary conditions may negate the sensitivity of flood levels and extents due to even moderate variations in design flow estimates.

For these reasons it is considered that the development of an unsteady, two-dimensional hydraulic model is likely to provide a more precise tool for quantifying the sensitivity of the design flows on the study outcomes than can be provided by the hydrologic model.

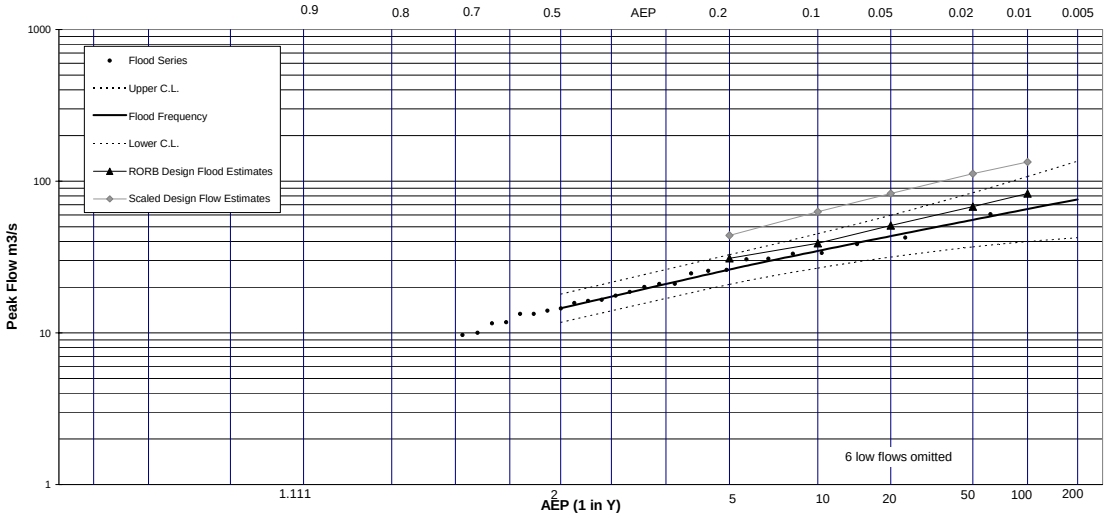


Figure 3-8 Comparison of Design Flow Estimates

3.6 Probable Maximum Precipitation Estimate

The Probable Maximum Flood (PMF) has been estimated for the contributing catchments for the Surry River at Narrrawong based on a regression equation for PMF's in South Eastern Australia as outlined in Hydrological Recipes (CRC-CH, 1996). Triangular hydrographs were

developed based on the methodology outlined to provide boundary conditions for the hydraulic model. The peak flow PMF estimates are displayed below in Table 3-9.

$$Q_{PMF} = 129.1A^{0.616}$$

Where
A = catchment area (km^2)

Table 3-9 PMF Peak Flow Estimates

Catchment	Catchment Area (km^2)	Peak Flow (m^3/s)	Volume (m^3)	Time to Peak (hr)
Surry River at Heathmere	**300**	**4,336**	**136287**	**6.8**
Western Sub Catchment	**48**	**1,395**	**22454**	**3.5**

4 OCEAN CONDITIONS

By definition the Surry River estuary implies a coastal influence, as such, the tide, storm surge and predicted sea level rise are likely to influence flood levels in the Surry River estuary.

This section documents an investigation into tidal and storm surge levels in the Southern Ocean used to establish appropriate ocean water levels for a flood assessment on the Merri River at Warrnambool South (Water Technology, 2007). As this investigation largely drew upon the long period of observed water level observations available at Portland, the findings of this investigation are considered directly transferable to the establishment of design ocean water levels for the Surry River estuary

4.1 Astronomical Tide

Astronomical tide refers to the rise and fall of the ocean surface due to gravitational attraction between the Earth, Moon and Sun. In coastal areas, an empirical approach can be used with observed water level data to predict the astronomical tide levels at any one time.

The significant tidal constituents, which together can be used to form the predicted tide, are available for Portland Harbour from the Australian National Tide Tables (2003). The tidal constituents for Portland are based on recorded water levels available since 1982 and are therefore considered very reliable. They have been used to establish a statistical representation of high water as a percentage occurrence plot shown in Figure 4-1. This plot illustrates the probability that the ocean high tide water level is expected to exceed a given height due to the influence of astronomical tides alone. It shows, for example, that a high tide greater than or equal to 0.2 m MSL can be expected approximately 50% of the time, and that a high tide greater than or equal to 0.45 m MSL can be expected approximately 10% of the time.

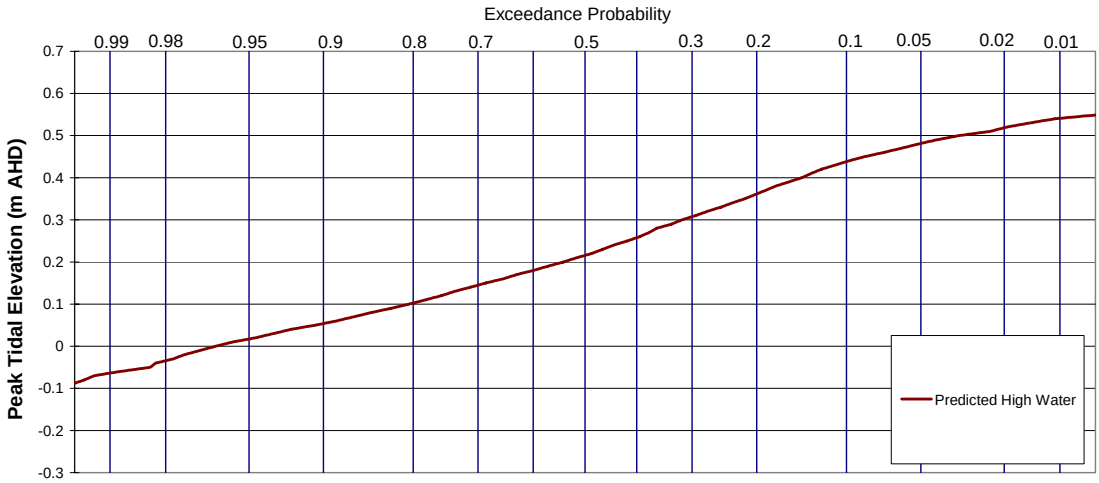


Figure 4-1 Probability of Occurrence of High Water Levels at Portland

4.2 Storm Surge

The term *storm surge* is generally used to collectively describe the variation in coastal water levels in response to atmospheric pressure fluctuations, wind setup and wave setup.

Variations in atmospheric pressure load and unload the ocean surface such that under high pressure systems, water levels are lower than those that would be expected and under low pressure systems, water levels are elevated. This *inverse barometric effect* generally results in a 10 hectopascal (hPa) variation in atmospheric pressure producing a 0.1m change in observed sea level. Super-elevation of water levels due to low pressure systems is commonly referred to as storm surge.

Wind setup is the term used to describe the super elevation of coastal water levels due to wind induced shear stresses pushing water against the coast. Wave setup is the term used to describe the super elevation of coastal water levels caused by the hydrodynamic forces associated with waves resulting in a net drift of water in the direction of wave travel.

The storm surge level can be derived from observed water level observations by subtracting the predicted astronomical tidal water level. A dataset of 24 years of ocean water level observations are available for Portland Harbour. The predicted astronomical tide for this period has been subtracted from the observed water levels, resulting in a *tidal residual.* Positive tidal residual values provide a measure of the storm surge.

A frequency analysis of 34 individual storm surge events identified in the Portland Harbour water level record was undertaken to determine the probability of occurrence of storm surge levels greater than 0.4m.

The results of this analysis are shown below in Figure 4-2. The x-axis evaluates the period associated with the AEP to identify the probability of occurrence of a positive tidal residual (shown on the y-axis). For example, the 1% (1 in 100 year) positive tidal residual is evaluated as 0.702m.

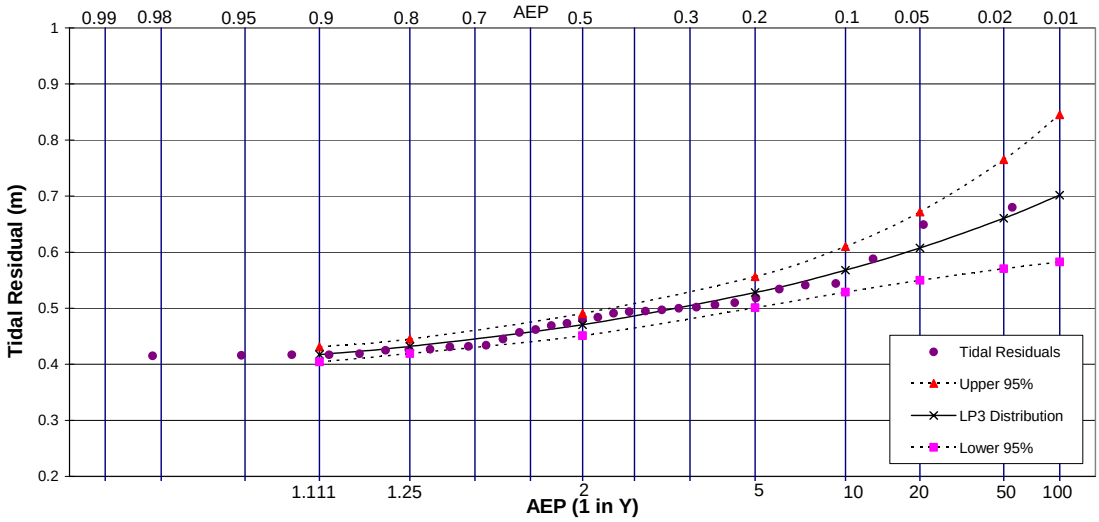


Figure 4-2 Probability of Occurrence of Storm Surge Levels at Portland

The analysis presented in Figure 4-2 indicates that storm surge of 0.4 m is a reasonably common occurrence, at least annually. For frequent events the residual-probability curve is fairly flat with only relatively small changes in residual between the 20% AEP levels. For events in excess of a 10% AEP , the level of uncertainty increases (illustrated by the divergent 5% and 95% confidence limits). However, it is considered that sufficient data exists to allow reasonable assessment of the 1% AEP storm surge level.

4.3 Combined Tide and Storm Surge

Storm surge events on the western Victorian coast occur over several days associated with the passage of low pressure systems across southern Australia. The pressure systems are independent of the tide. As such, they can occur at any time during the tidal cycle, neap or spring.

An analysis of tidal records from Portland has been undertaken to estimate the 1% AEP combined tide and storm surge level. It has been assumed that tide and surge are independent and that any given peak astronomical tide may coincide with the peak in a surge event. Since the peak levels of a surge event typically has a duration of many hours, this assumption is reasonable and results in slightly conservative (over) estimate of 1% AEP tide + surge levels. A monte-carlo type analysis has been undertaken, statistically sampling from the peak tide and peak surge levels to develop an extended "history" of theoretical tide + surge peak levels. Statistical analysis of the history has been conducted to develop an exceedance probability curve, shown in Figure 4-3. Also shown are the exceedance probability curves for tide and surge for comparison.

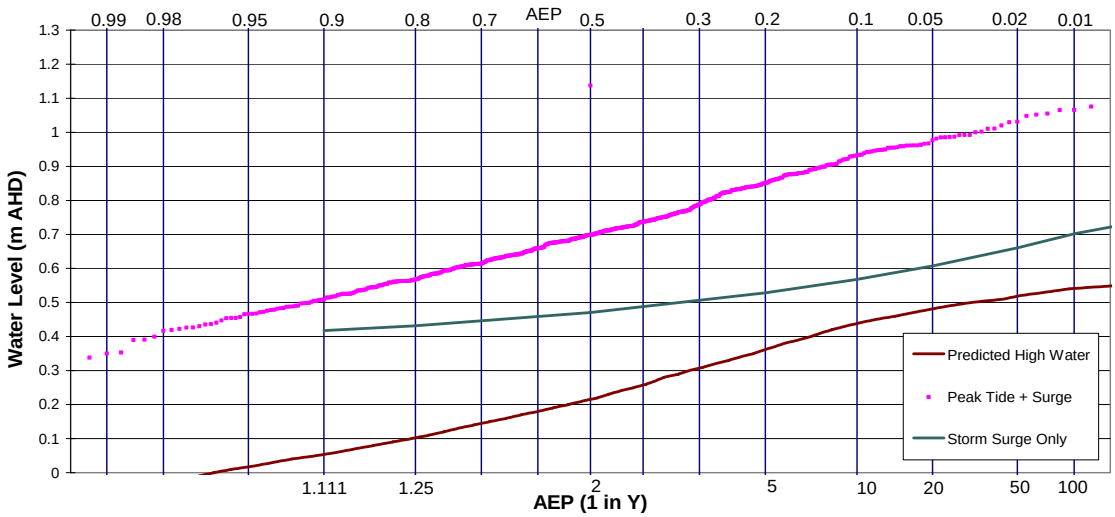


Figure 4-3 Portland Tide and Surge Statistics

Figure 4-3 indicates a 1% tide + surge level of 1.07 m AHD at Portland. This is somewhat lower than previous estimates as this analysis has excluded an extreme level of 1.3 m AHD measured during the 1971 flood event as discussed above in Section 4.2. Peak tide + surge levels are summarised in Table 4-1 below.

Table 4-1 Tide + Surge Levels - Portland

AEP (%)	ARI (1 in Yr)	Level (m AHD)
50	**2**	**0.70**
20	**5**	**0.85**
10	**10**	**0.93**
5	**20**	**0.98**
2	**50**	**1.03**
1	**100**	**1.07**

The above analysis is based on available data from Portland. The relatively close proximity of Narrawong to Portland is considered to allow this analysis to be directly applicable to determining design storm surge levels at Narrawong.

4.4 Design Ocean Water Level

The 100 year ARI sea-level at Narrawong under existing conditions has been estimated, based on data available from Portland, as 1.07 m AHD. The CSIRO (refer Kathleen McInnes) has provided an estimate of the 100 year water levels at Portland of 1.12 m AHD ±0.06 m. The estimates provided by this study team and those of the CSIRO are considered consistent within the error bounds of the analysis.

The design 1% AEP sea level applicable to Portland for planning purposes is determined to be 1.1 m AHD as shown in Table 4-2.

Table 4-2 Design Ocean Level Summary

Source	100 Year ARI Ocean Level	Ocean Level for Planning Purposes	Ocean Level for Planning Purposes (Rounded)
	(m AHD)	(m AHD)	(m AHD)
Water Technology	**1.07**	**1.07**	**1.1**
CSIRO (Kathleen McInnes)	**1.12**	**1.12**	**1.1**

5 HYDRAULIC ANALYSIS

The complicated interaction of the estuary and floodplain geometry, flood flows and ocean water levels requires detailed hydraulic modelling analysis to determine appropriate flood inundation extents and levels and frequencies within the study area.

A dynamic and flexible hydraulic modelling approach has been applied that incorporates both one and two-dimensional hydraulic models of the estuary.

An overall, one-dimensional model of the Surry River from Heathmere to the ocean has been developed. The one-dimensional model is computationally fast and easy to modify. The one-dimensional model has been used to undertake sensitivity analysis on a number of model parameters and to develop an understanding of the system behaviour.

A detailed, two-dimensional model (including one-dimensional components) has also been developed from Heathmere to the ocean to allow the flood behaviour of the estuary to simulated in greater resolution.

The results from the two models were verified against each other to ensure they were both producing similar flood behaviour.

5.1.1 Hydraulic Model Software

Hydraulic modelling of the study area has been undertaken utilising the Danish Hydraulic Institute's (DHI) MIKE FLOOD modelling software. MIKEFLOOD is a state of the art tool for floodplain modelling that has been formed by the dynamic coupling of DHI's well proven MIKE 11 river modelling and MIKE 21 fully two-dimensional modelling systems. Through this coupling it is possible to extend the capability of the 2D MIKE 21 model to include:

- A comprehensive range of hydraulic structure (including weirs, culverts, bridges, etc);

- ability to accurately model sub-grid scale channels;
- ability to accurately model dambreak or levee failures.

For the present study, a two-dimensional (2D) MIKE 21 model has been set up to model the overall floodplain flows. A coupled one dimensional (1D) MIKE 11 model has also been utilised to explicitly model waterway bridge and culvert crossings within the study area.

More information on MIKE FLOOD can be found at:

http://www.dhigroup.com/Software/WaterResources/MIKEFLOOD.aspx

5.1.2 Two Dimensional Model Structure

The development of a detailed terrain model and subsequent construction of a hydraulic model of the study area enables the behaviour of Surry River flood flows and ocean storm surge conditions to be simulated in great detail. Flow conditions varying from historical flood events to the simulation of hypothetical "design" events can be modelled to investigate the pattern of flooding behaviour within the study area. These flow conditions can be applied to both the existing topography, and topographies that have been altered to represent changes eg. flood mitigation measures or proposed developments.

The basis of the two-dimensional model is the topographic grid which is based on the aerial photogrammetry, bathymetric data and field survey. A 5m grid, rotated 16^0 anticlockwise from true north was used for hydraulic model. The grid was rotated to ensure computational efficiency, allowing a smaller grid size to be used overall while also aligning the grid parallel to major topographic features such as the coast and Princess Hwy.

The bridge and culvert crossings within the study area were modelled as MIKE 11 structures and dynamically coupled with the two-dimensional model. Head loss through the structures could therefore be modelled explicitly within the model.

Figure 5-1 displays the two-dimensional hydraulic model topography, the location of one-dimensional hydraulic structure elements and the model boundary conditions.

The variation in hydraulic roughness within the study area has been schematised as a hydraulic roughness grid, representing various hydraulic roughness e.g. open grassland, reeds, thick vegetation. The hydraulic roughness grid was based principally on the aerial orthophoto (QASCO 2002) and visual inspection undertaken during field visits. Hydraulic roughness values adopted for the two-dimensional hydraulic model are summarised in Table 5-1. The sensitivity of these adopted values was tested as part of the sensitivity analysis described in Section 6.

Table 5-1 Hydraulic Roughness Parameters

Topography Class	Manning's "n"
Open Floodplain (Pasture)	0.04
Vegetated	0.05
Estuary	0.035
Roads	0.02
Thick ground-cover (Reeds)	0.06
Developed areas	0.15

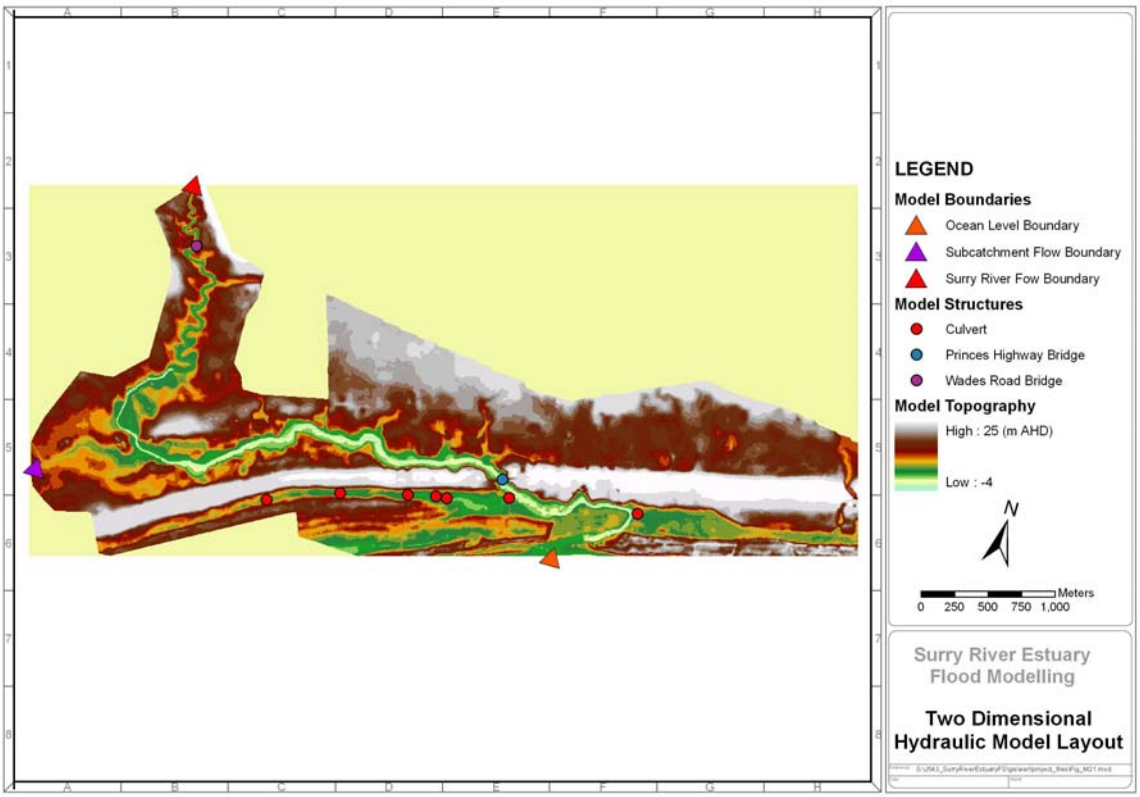


Figure 5-1 Two Dimensional Hydraulic Model Layout

5.2 Entrance Dynamics

5.2.1 Historical Aerial Photographic Record

The morphology of the estuary entrance under flood conditions could be expected to change significantly due to the dynamic changes of the sand bar under flood flow, storm surge and tidal forces. Flood levels within the estuary could therefore be expected to be sensitive to the controlling geometry used to describe the estuary's connection with the ocean.

To help improve the understanding of the dynamics of the estuary entrance, historical aerial photography of the Surry River estuary was sourced from the Land and Survey Information Centre. A total of five historical aerial photographs of the estuary entrance at reasonable scales were obtained between 1947 and 1992 (Appendix E). The photos were scanned and geo-referenced in a GIS. The edge of the dune vegetation was mapped out for each of the photos to provide an indication of the quasi-stable position of the dunes at the time of each photo. This allowed for a relative comparison of the entrance morphology to be undertaken with photos from other periods. The comparison of the vegetated dune edge developed over the aerial photographic record is presented in Figure 5-2.

Of particular interest was the morphology of the entrance following significant flow events in the Surry River at Heathmere. Two historical photos are considered to have captured the morphology of the entrance within a reasonable period following significant flood flows on the Surry River.

Anecdotal evidence suggests that large flows occurred in the Surry River during the western district floods of March 1946. The photo obtained in January 1947 shows some moderate changes to the vegetated dune edge compared with subsequent years, particularly on the

western side of the entrance. The differences are however considered relatively minor and may not necessarily be attributed to the actions of flood flows at the entrance.

The photo obtained for February 1977 captures the entrance morphology following the largest flood on the historical gauge record at Heathmere which occurred approximately four months earlier in October 1976. This flood has been estimated as having a recurrence interval of approximately 1 in 30 years. No discernable change in the entrance morphology can be seen outside the natural variability apparent in the historical photographic record.

The review of the historical aerial photography available for the Surry River estuary entrance is considered to show that the lateral extent of the entrance is relatively stable and that even following apparent periods of significant flood flows, little change in the estuary morphology can be noted visually.

The historical aerial photography does not however provide a good indication of the change in the level of the sand bar following flood flows. Bed shear stresses associated with flood flows across the sand bar entrance can be expected to result in scour of the estuary entrance bar. For this reason, a non-cohesive sediment transport formulation has been used to estimate the sediment transport and morphological changes to the estuary entrance under flood flow conditions. This analysis is documented in Section 5.2.2.

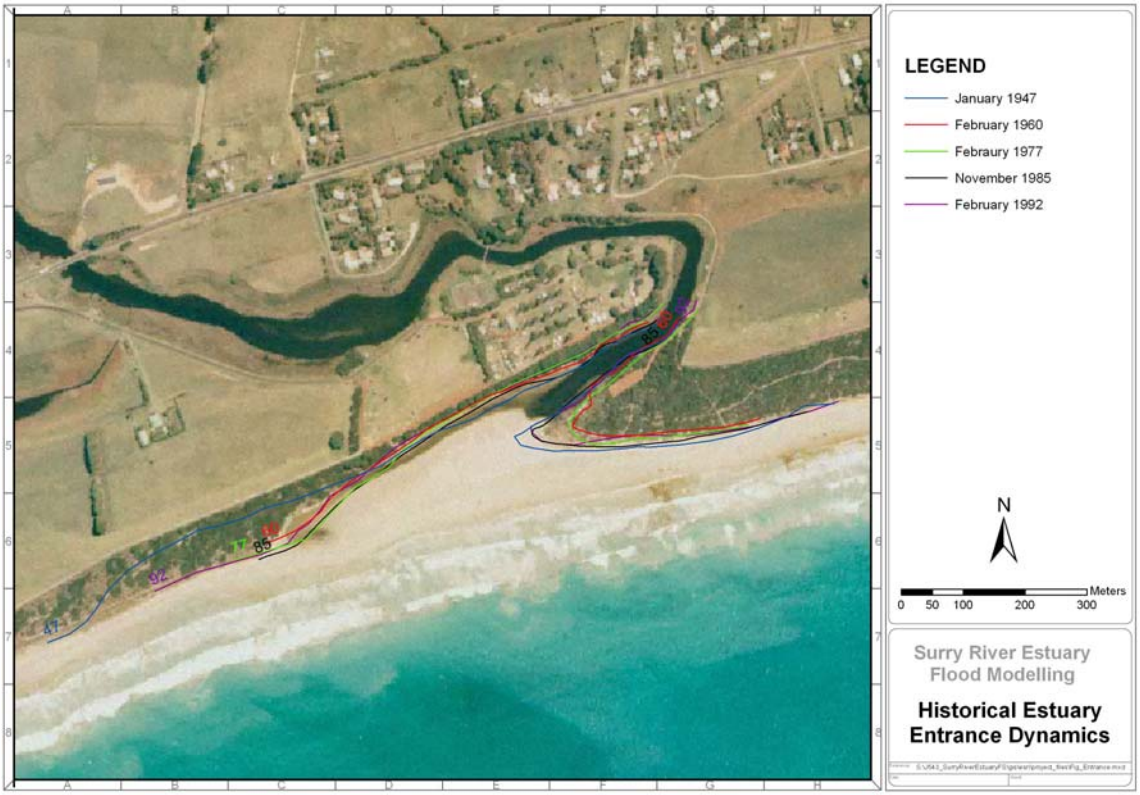


Figure 5-2 Historical Estuary Entrance Dynamics

5.2.2 Sediment Transport Modelling

A non-cohesive sediment transport formulation has been used to estimate the depth of scour of the sand bar entrance under flood flow conditions. As the historical aerial photographic record was considered to show that the lateral extent of the estuary entrance remains relatively stable, a one-dimensional analysis of the sediment transport through the entrance was

considered appropriate. With this analysis, shear stresses on the bar associated with flood flows are only considered to scour the bar vertically.

The GHCMA Estuary Projects team provided a profile of the bedrock depth across the estuary entrance. This profile was used to define the maximum depth of scour in the sediment transport. The bedrock cross section provided by the GHCMA is attached in Appendix F.

Sieve analysis was undertaken on the sand at the estuary entrance to determine the particle size distribution. The sieve analysis was undertaken by Anacon Laboratory Services and the results are attached in Appendix G.

The cumulative particle size distribution for the estuary entrance sand is presented in Figure 5-3. The median grain size diameter was determined as approximately equal to 0.21mm.

Consideration has been given to the adoption of an appropriate initial entrance bar cross section for the modelling in recognition that the bar geometry captured by the photogrammetry may not necessarily be representative of antecedent conditions during large flood events in the catchment. The sensitivity of the antecedent bar geometry to the modelled flood levels was tested as part of the sensitivity analysis undertaken and described in Section 6.

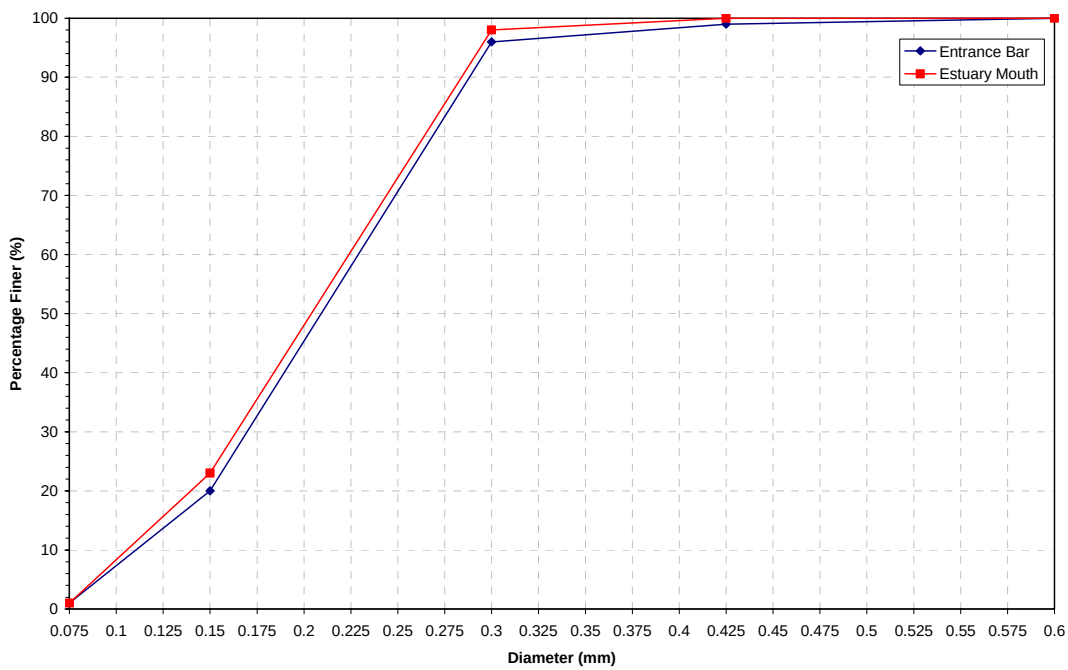


Figure 5-3 Cumulative Particle Size Distribution of Estuary Entrance Sand

The Van Rijn (1984) sediment transport formulation was applied to undertake sediment transport simulations for the 1% AEP flood hydrograph with a representative spring-neap tidal cycle ocean boundary derived from tidal constituents at Portland.

Under a 1% AEP flood the sediment transport modelling predicts of order 700m^3 (assuming a porosity of 0.35) of material could potentially be transported from within the estuary and entrance bar to the shoreward slope of the ocean beach. Peak water levels upstream of the entrance bar are predicted to occur before the maximum depth of scour is achieved. Figure 5-4 displays the predicted water level immediately upstream of the entrance bar, the ocean tidal boundary and the variation in the entrance bar level under a 1% AEP flood.

Estimation of sediment transport rates and hence timing and depth of scour of the estuary entrance under flood flow conditions is considered extremely difficult to reliably quantify. Sediment transport rate formulations are a function of the depth averaged velocity up to the 5th and 6th power and are therefore extremely sensitive to grain size, bed roughness, bedforms and the controlling geometry and discharge.

Due to the sensitivity of the sediment transport rates and hence timing and depth of scour of the entrance bar it was considered prudent to adopt conservative values for the sediment transport modelling parameters for this analysis. However, as can be seen from Figure 5-4, peak flood levels are predicted to occur before the maximum depth of scour, meaning the antecedent conditions of the bar are important in determining flood levels directly upstream of the bar. For this reason the sensitivity of predicted flood levels was tested for various antecedent entrance bar heights as described in Section 6.

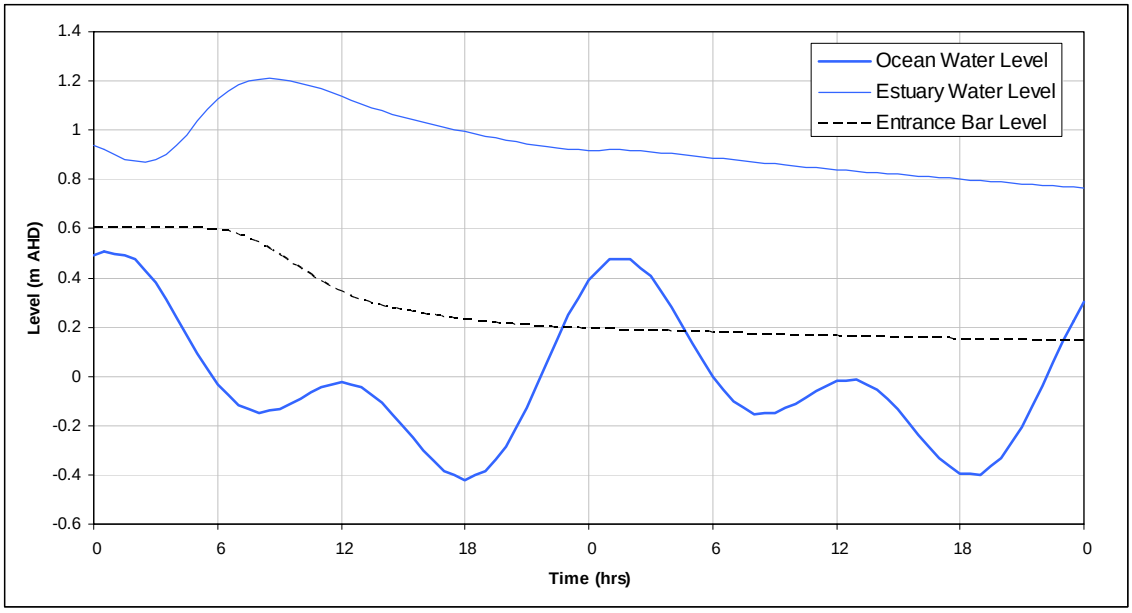


Figure 5-4 1% AEP flood estuary entrance dynamics

6 SENSITIVITY ANALYSIS

Due to the uncertainties inherent in the hydrologic analysis and the inability to undertake conventional calibration of the hydraulic model (see 2.5 above), the sensitivity of a number of the major model inputs has been investigated to provide an indication as to the sensitivity of the modelled flood levels to various model inputs and assumptions.

The sensitivity analysis has been undertaken on the model inputs for the 1% AEP design flood hydrographs developed from the RORB model.

The following sensitivity scenarios have been tested with the one-dimensional hydraulic model:

- Impact on flood levels when simulating the scour and sand transport of the entrance sand bar adopting the bar geometry captured by the photogrammetry
- Impact on flood levels when simulating the scour and sand transport of an entrance sand bar adopting the geometry captured by the photogrammetry but with antecedent scouring of 0.3m.
- Impact on flood levels with design flows scaled up by 20%. This produces a peak flow at the Heathmere gauge on the Surry River approximately equal to the 95% confidence limit flow developed from the flood frequency analysis.
- Impact on flood levels with Manning's 'n' values increased by 20%

The impact of the sensitivity scenarios on the maximum water surface profile predicted by the one-dimensional hydraulic model from the Heathmere gauge to the estuary entrance is displayed in Figure 6-1.

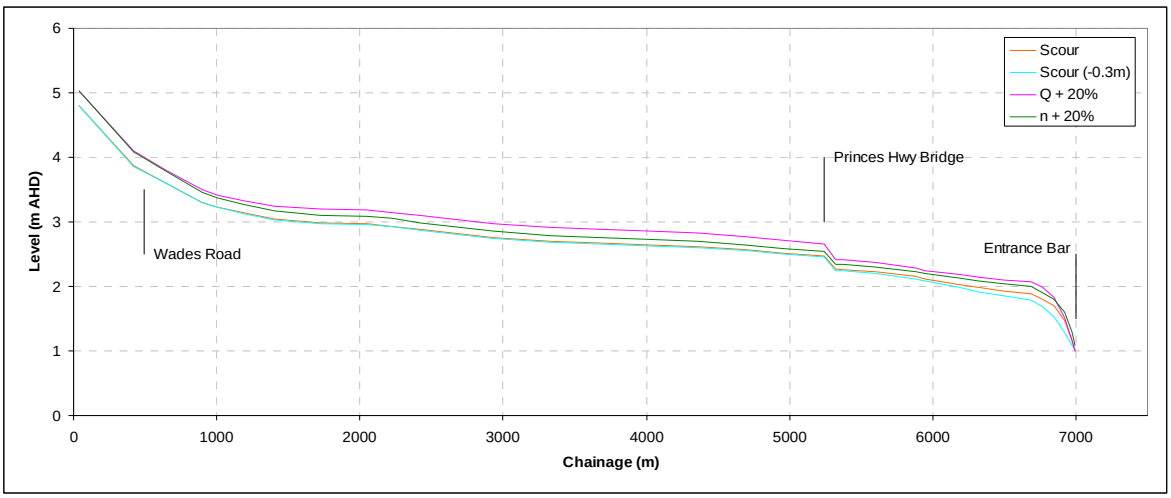


Figure 6-1 Impact of Sensitivity Scenarios on Maximum Water Surface Profile

From Figure 6-1 the following observations are made:

- The sandbar entrance dynamics and antecedent conditions have a relatively localized effect on flood levels back through the estuary with negligible difference observed upstream of the highway bridge. The adoption of sea level rise and concurrent storm surge are also expected to negate the

sensitivity of the sandbar entrance dynamics on flood levels for design flood modelling.

- Scaling the design flows up to the 95% confidence limit flows at Heathmere results in flood level increases of approximately less than 200mm through the study area.
- Scaling the Manning's 'n' coefficients up by 20% results generally in flood levels increases of approximately less than 100mm.

Given the magnitude of the increase in the design peak flow and volume when scaling the 1% AEP flood hydrographs up to the 95% confidence limit peak flows at Heathmere, the flood level increases predicted by the model are considered modest. It is also noted that the increase in flooding extents are also expected to be very minimal as flooding extents are quite confined by the floodplain geometry.

7 ADOPTED DESIGN FLOWS

For the reasons discussed in the sensitivity analysis in Section 6, and in consultation with the GHCMA, the study team decided to adopt the 95% confidence limit peak flows at Heathmere for the design flood modelling to allow for the uncertainty in the design flood magnitudes and due to the inability to undertake a conventional calibration of the hydraulic model. It is however recognised that the 95% confidence limits themselves are derived from a statistically small population of annual floods and could potentially provide an under estimation of the actual confidence limits peak flows. This should be taken into consideration when determining appropriate freeboard provisions to apply to the relevant planning scheme overlays.

The final adopted design peak flows equivalent to the 95% confidence limit peak flows developed from the flood frequency analysis are presented in Table 7-1 below.

Table 7-1 RORB Design Flood Estimates

Design Flood	Kc value	Rainfall loss parameters (mm)		Surry River at Heathmere	
				Peak flow (m^3/s)	Duration (hrs)
		IL	CL		
20% AEP	75	5	1.3	31	18
10% AEP	75	5	1.3	39	18
5% AEP	75	5	1.3	51	18
2% AEP	75	5	1.3	68	24
1% AEP	75	5	1.3	83	24

8 MODEL VALIDATION

The hydraulic model development and calibration has been validated by simulating a small flood that occurred in the Surry River in early November 2007. The flood had a peak discharge of approximately 34 m^3/s at Heathmere. This flood magnitude has an approximate AEP of greater than 20% and is therefore at the low end of the flood magnitudes for which the hydraulic model was developed to simulate. Nevertheless, the flood and gauge data collected during the flood, provide a good opportunity to validate the hydraulic model results. The flood hydrograph at Heathmere is presented in Figure 8-1 below.

The flood hydrograph was first modelled in the one dimensional hydraulic model to test the sensitivity of the bar entrance and scour conditions on the water level record captured at the water quality monitoring site (237212) approximately 500m upstream of the Princes Highway bridge. The results of this analysis, simulating the flood with and without scour of the entrance bar is displayed in Figure 8-2. From Figure 8-2 it can be seen that by simulating the scour of the entrance bar during the flood, the hydraulic model is able to more accurately simulate the water level response in the estuary. It is noted however that the absolute impact on maximum flood levels in the estuary as a result of the entrance scour dynamics is relatively minor.

Following the results of the one dimensional modelling, the two dimensional hydraulic model topography was altered to reflect the level of scour of the entrance bar simulated by the one dimensional model. The November 2007 flood hydrograph was subsequently simulated in the two dimensional hydraulic model. Comparisons of observed water levels collected by the GHCMA at various locations along the estuary and the predicted water levels from the hydraulic model at the same location and time are presented in Figure 8-3.

The comparisons of the observed and modelled flood level observations presented in Figure 8-1 to Figure 8-3 are considered to validate the hydraulic model results. Some differences in observed and model levels are evident in the lower section of the estuary, however these are considered primarily due to uncertainties in the initial geometry of the entrance bar. Conservative assumptions as to the geometry of the entrance bar have been adopted in the design flood modelling to reflect this uncertainty in the flooding behaviour due to scour of the entrance bar.

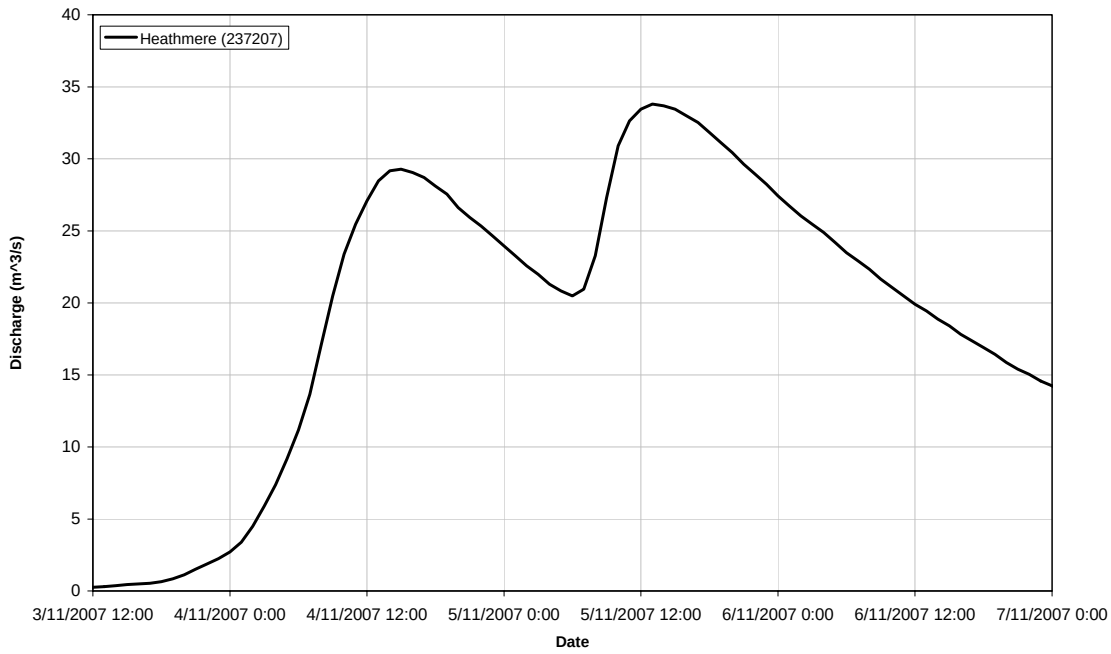


Figure 8-1 November 2007 Flood Hydrograph at Heathmere

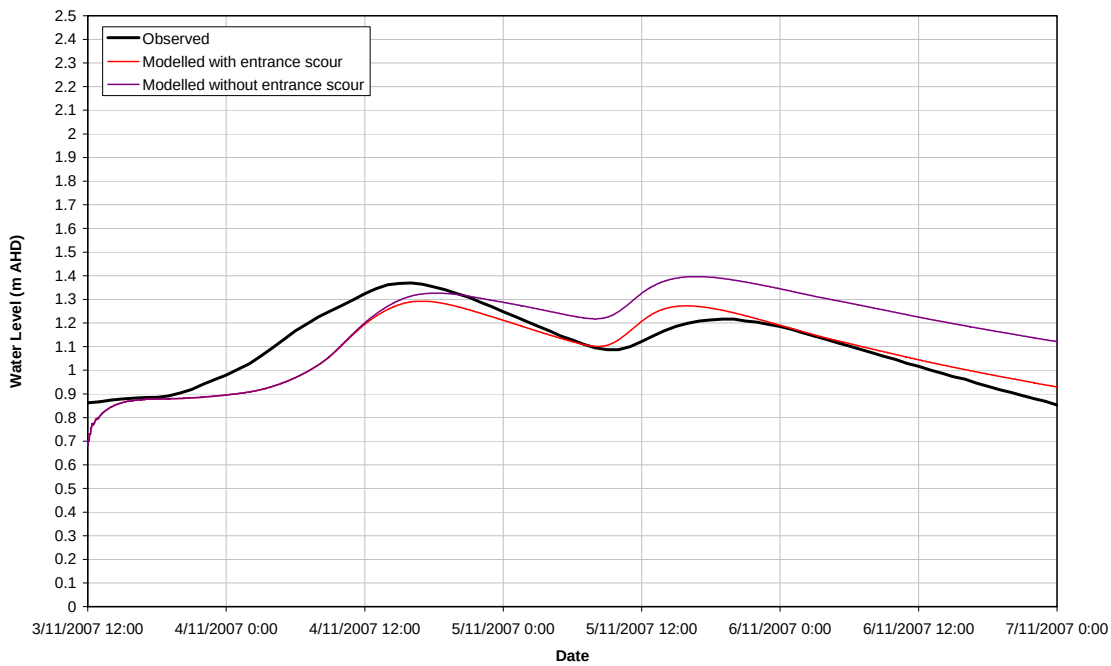
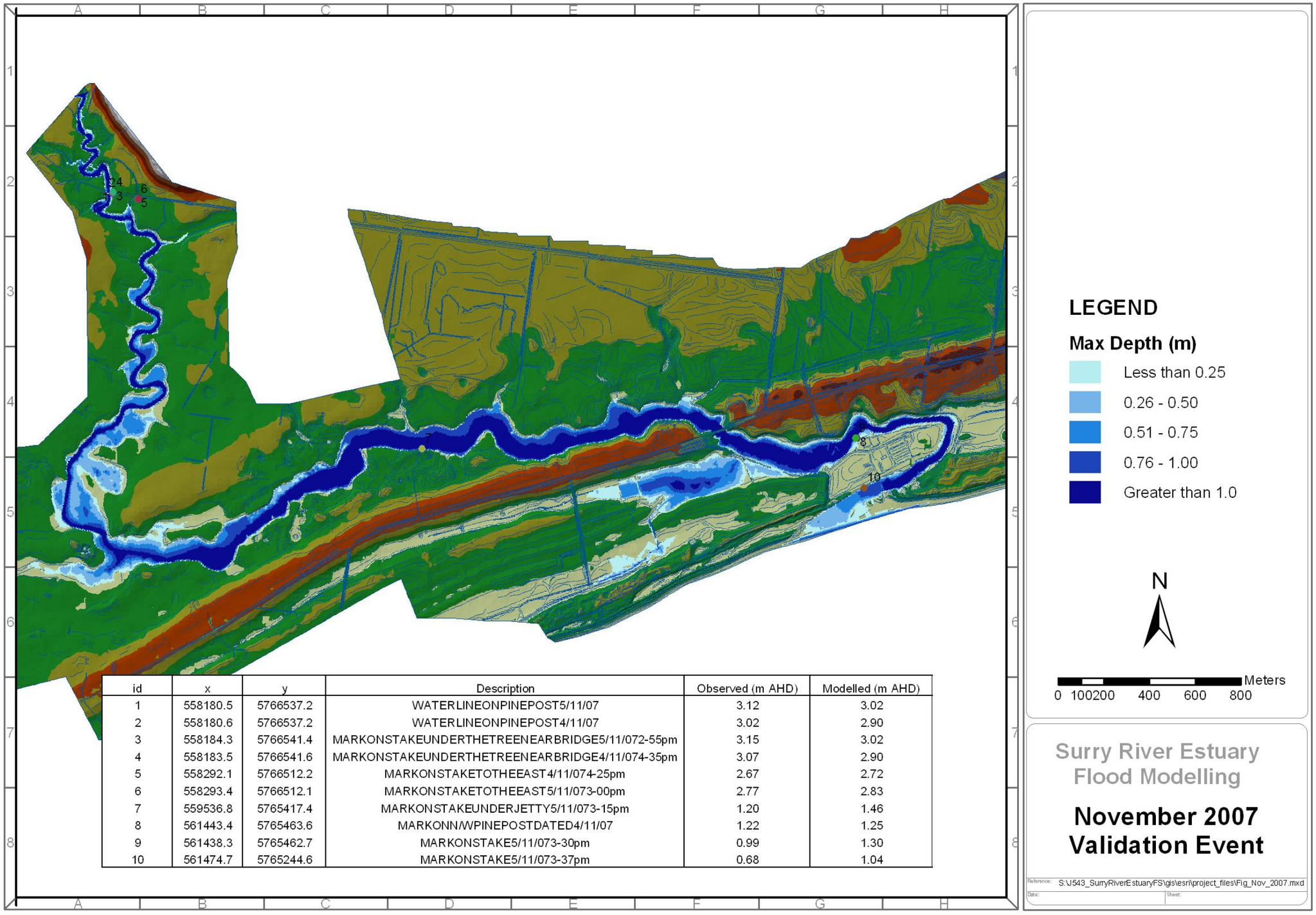


Figure 8-2 Comparison of Modelled and Observed Water Levels at 237212



id	x	y	Description	Observed (m AHD)	Modelled (m AHD)
1	558180.5	5766537.2	WATERLINEONPINEPOST5/11/07	3.12	3.02
2	558180.6	5766537.2	WATERLINEONPINEPOST4/11/07	3.02	2.90
3	558184.3	5766541.4	MARKONSTAKEUNDERTHETREENEARBRIDGE5/11/072-55pm	3.15	3.02
4	558183.5	5766541.6	MARKONSTAKEUNDERTHETREENEARBRIDGE4/11/074-35pm	3.07	2.90
5	558292.1	5766512.2	MARKONSTAKETOTHEEAST4/11/074-25pm	2.67	2.72
6	558293.4	5766512.1	MARKONSTAKETOTHEEAST5/11/073-00pm	2.77	2.83
7	559536.8	5765417.4	MARKONSTAKEUNDERJETTY5/11/073-15pm	1.20	1.46
8	561443.4	5765463.6	MARKONN/WPINEPOSTDATED4/11/07	1.22	1.25
9	561438.3	5765462.7	MARKONSTAKE5/11/073-30pm	0.99	1.30
10	561474.7	5765244.6	MARKONSTAKE5/11/073-37pm	0.68	1.04

Figure 8-3 November 2007 Validation Event

9 DESIGN FLOOD SIMULATIONS

9.1 Design Flood Conditions

The hydraulic model was simulated for the 10, 5, 2 and 1% AEP flood events as well as the nominal PMF event. The boundary inputs to the model consisted of:

- RORB predicted design flow hydrographs for the Surry River at Heathmere scaled up to the 95% confidence limit peak flow determined from the flood frequency analysis.
- Due to the difficulties in determining the joint probabilities of ocean water level and riverine flooding for design flood modelling on the Surry River estuary, the 10% AEP ocean water level of 0.93 m AHD was adopted and is considered a reasonable and prudent level to adopt for design flood modelling.
- An assumption at the estuary entrance of a bar height of approximately 0.3m AHD at the design flood peak. This is considered indicative of a relatively high antecedent bar height and provides a conservative controlling geometry for determining design flood levels.

9.1.1 Design Flood Simulation Results

Maximum design flood extent results from the design simulations are presented in Figure 9-1. From Figure 9-1 it can be seen that only very minor differences between the 10% AEP and the 1% AEP flood are predicted despite the significant difference in flow magnitude and volume. This is considered largely due to the confining nature of the floodplain geometry in the study area.

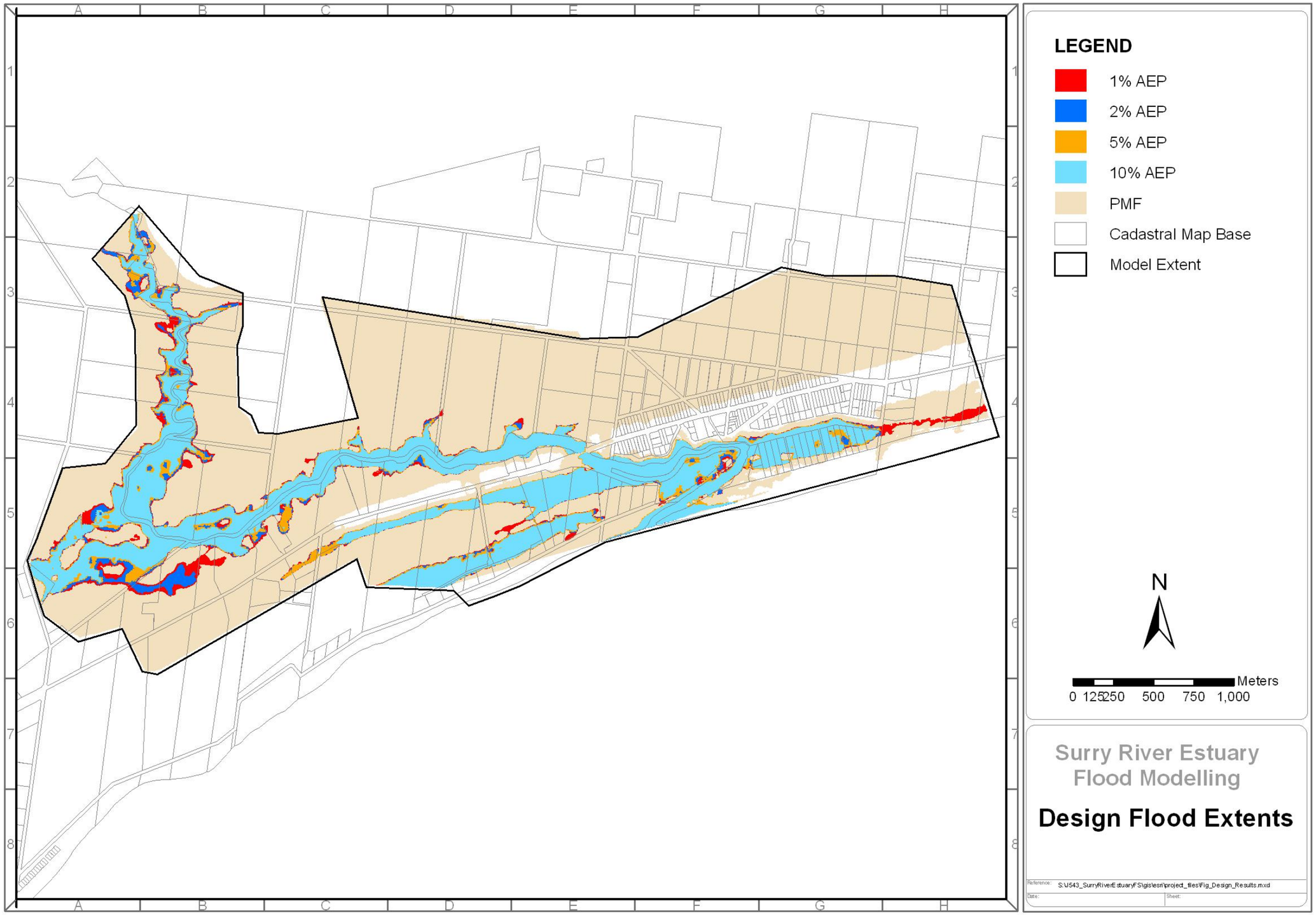


Figure 9-1 Design Maximum Flood Extents

The maximum water surface profile along the Surry River Estuary has been compared for the 1% and 10% AEP design floods (Table 3-8) in Figure 9-2. From Figure 9-2 it can be seen that the influence of the ocean water level boundary condition on flood levels upstream diminishes with increasing flood magnitude. For large floods, the conveyance capacity of floodplain features such as the Princess Highway Bridge and the bar entrance geometry are relatively more important in controlling flood levels within the estuary.

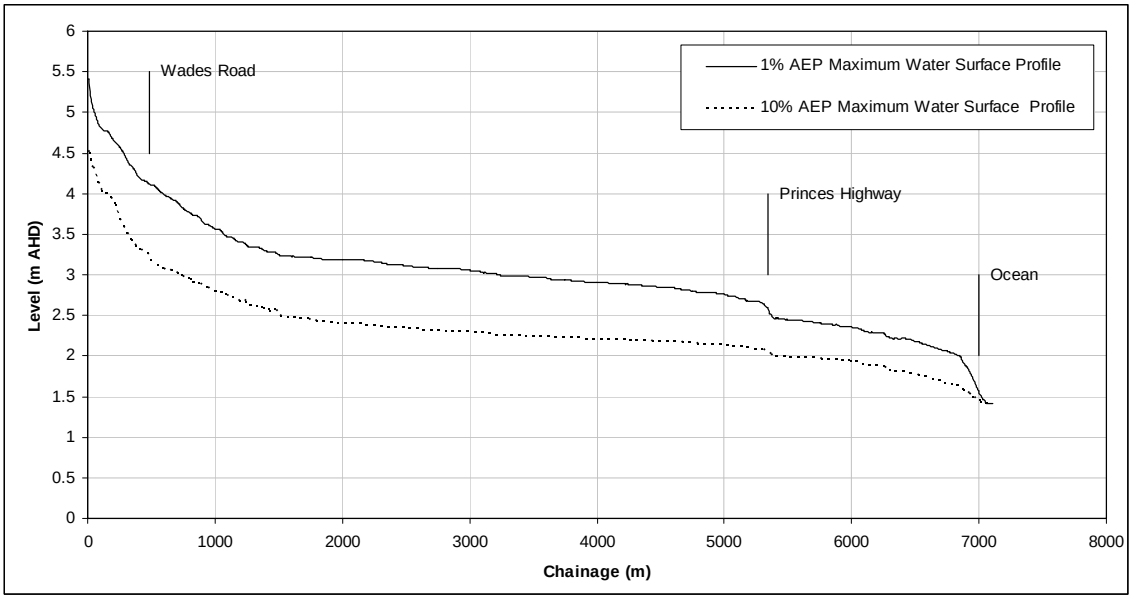


Figure 9-2 Comparison of Design 1% & 10% Maximum Water Surface Profile

Figure 9-3 displays the predicted maximum extent and depth of inundation and peak velocity and direction for the 1% AEP flood. From Figure 9-3 it can be seen that the majority of the predicted inundation occurs at depths exceeding 1.0 metres. There are considered to be no significant secondary overland flowpaths within the study area for flood magnitudes ranging between the 10% and 1% AEP. Through much of the lower section of the study area, maximum predicted velocities are generally less than 0.5m/s during a 1% AEP.

9.2 Mean Sea Level Rise Impact Assessment

The impact of a rise in mean sea level on the flooding behaviour within the study area has been undertaken on the design 1% AEP flood hydrographs and the existing 10% AEP ocean storm tide level for a range of mean sea level rise scenarios.

The three mean seal level rise scenarios investigated and the corresponding ocean boundary condition are displayed in Table 9-1.Figure 9-4 through to Figure 9-6 displays the difference in the predicted 1% AEP flood extent under the various mean sea level rise scenarios.

From Figure 9-4 through to Figure 9-6 it can be seen that the predicted impact on the 1% AEP design flood extent is very minor and confined to the lower reaches of the estuary even for the worst case sea level rise scenario considered. It is important to note the response of a sustained mean sea level rise on the morphology of the dune system and the estuary bar entrance and the resulting impact on flood behaviour has not been considered in this study.

It should also be noted that the sea level rise impact assessment undertaken has only considered the impact of a rise in mean sea level on the 1% AEP flood. The relative impact on flood extents and flood levels in the study area due to a rise in mean sea level may possibly be

greater for the more frequent flood flows than is indicated from the assessment of the 1% AEP flood.

Table 9-1 Sea Level Rise Scenarios

M.S.L (m AHD)	10% AEP Storm Tide (m)	Ocean Boundary (m AHD)
0	0.93	0.93
0.49	0.93	1.42
0.8	0.93	1.73
1.2	0.93	2.13

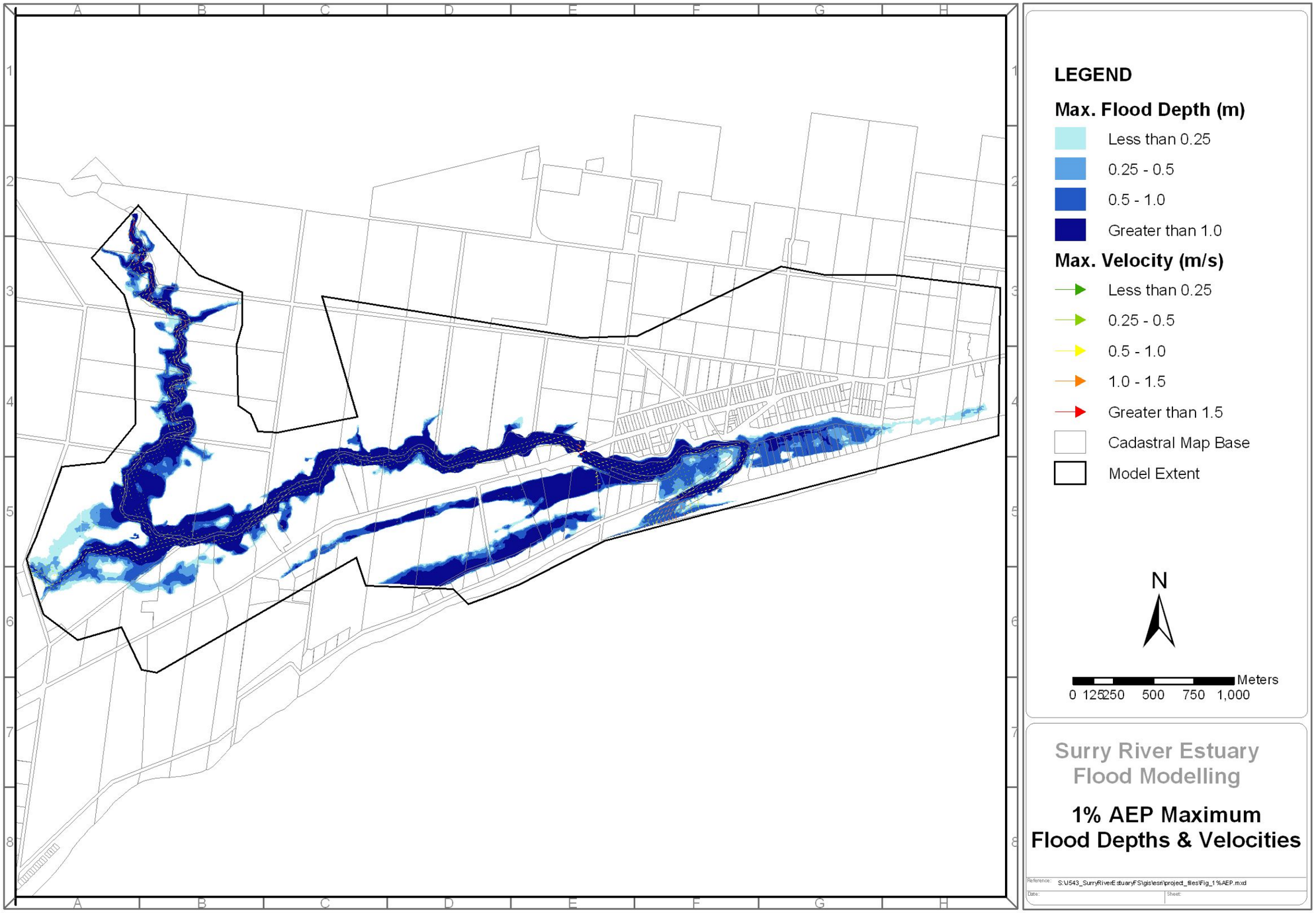


Figure 9-3 1% AEP Maximum Flood Depths & Velocities

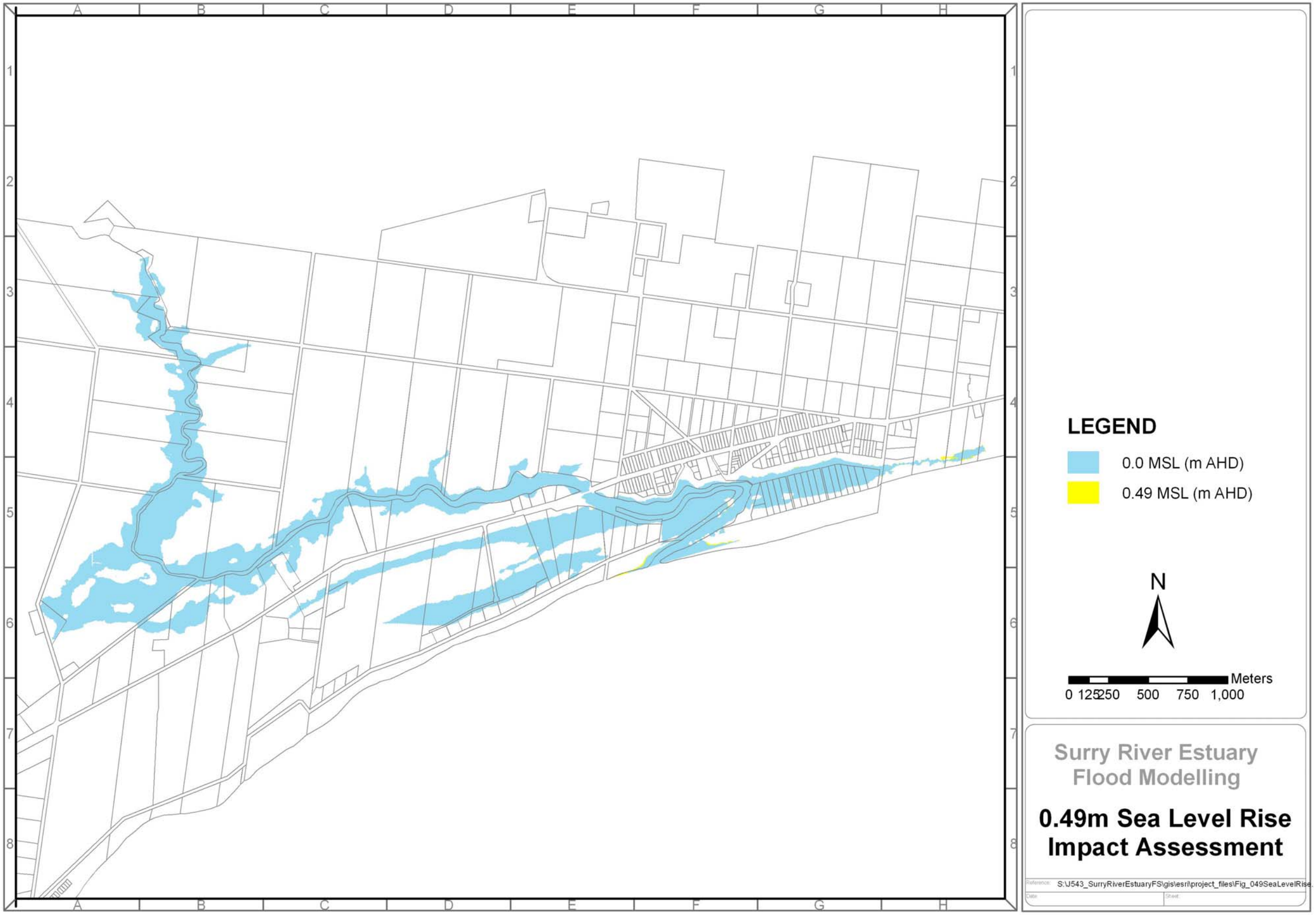


Figure 9-4 1% AEP Flood, 0.49m Sea Level Rise Impact Assessment

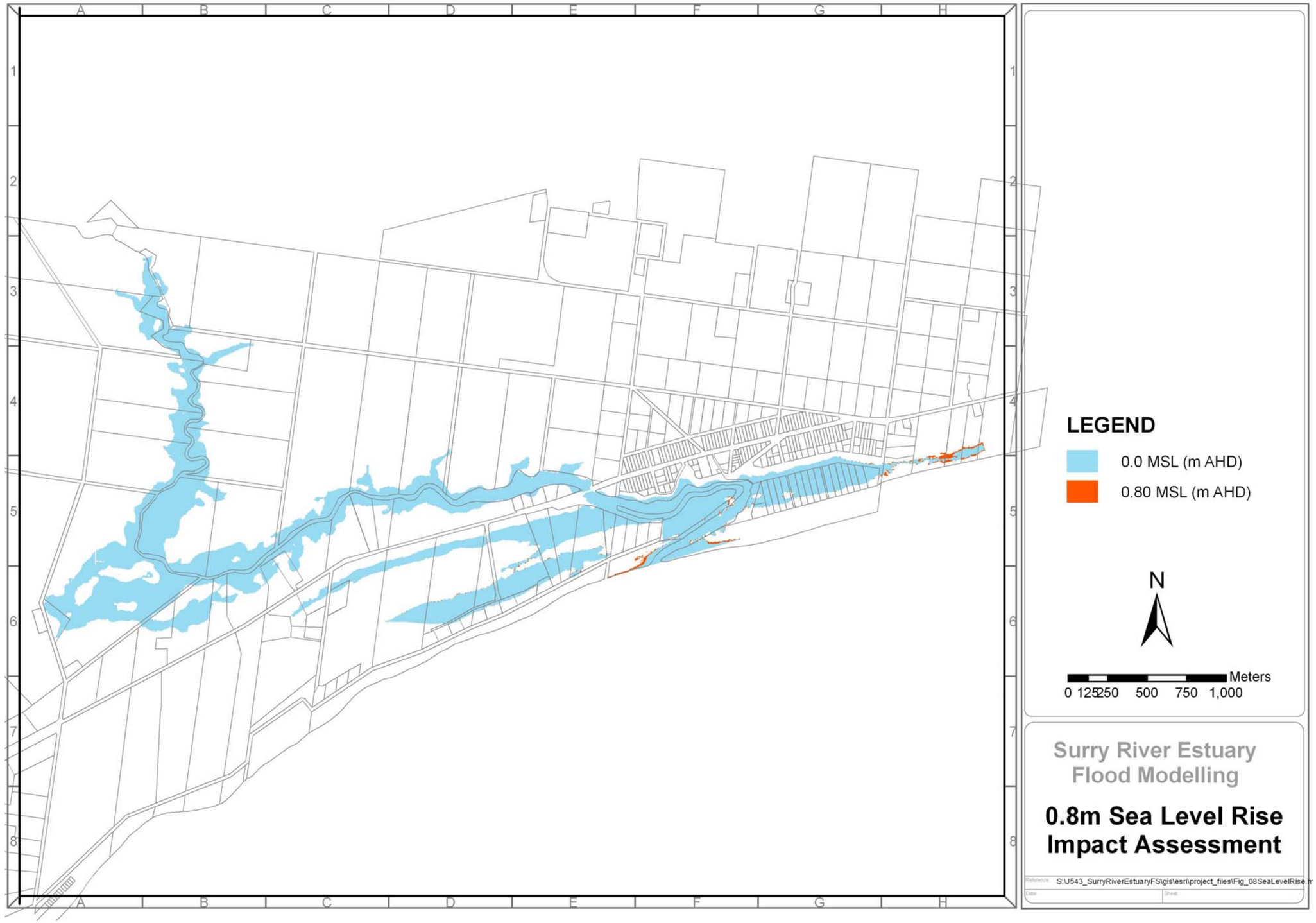


Figure 9-5 1% AEP Flood, 0.8m Sea Level Rise Impact Assessment

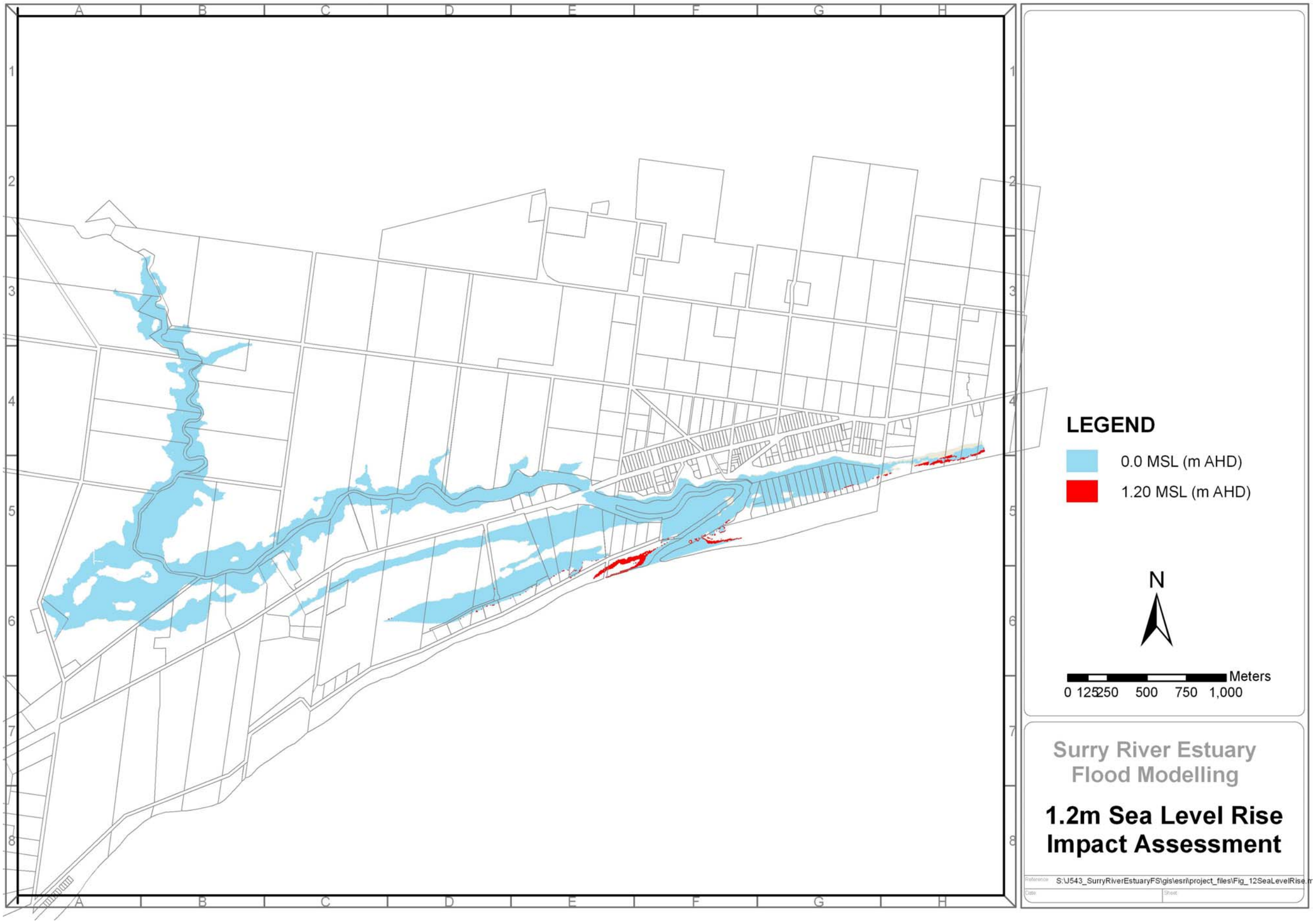


Figure 9-6 1% AEP Flood, 1.2m Sea Level Rise Impact Assessment

10 DATASETS AND MAPPING

10.1 Overview

Land use planning controls and building regulations provide mechanisms for ensuring appropriate use of land and building construction, given the flooding behaviour. Land use planning controls are aimed at reducing the growth in flood damages over time. The controls balance the likelihood of flooding with the consequences (flood risk).

10.2 Flood Related Planning Zones and Overlays

10.2.1 Land Subject to Inundation Overlay (LSIO)

The LSIO identifies land liable to inundation by overland flow, in flood storage or in flood fringe areas affected by the 1% AEP flood. The extent of the LSIO is displayed in Figure 10-2.

10.2.2 Floodway Overlay (FO)

The floodway overlay identifies waterways, main flood paths, drainage depressions and high hazard regions within rural areas. The identification of floodways was based on NRE's "Advisory Notes for Delineating Floodways." (NRE 1998). The advisory notes provide three approaches to the delineation of FO, as follows:

- Flood frequency
- Flood depth
- Flood hazard

For **flood frequency**, Appendix A1 of the advisory notes suggest areas which flood frequently and for which the consequences of flooding are moderate or high, should generally be regarded as floodway.

Flood hazard combines the flood depth and flow speed for a given design flood event. The advisory notes suggest the use of Figure 10-1 for delineating the floodway based on flood hazard. The flood hazard for the 1% AEP flood was considered for this study.

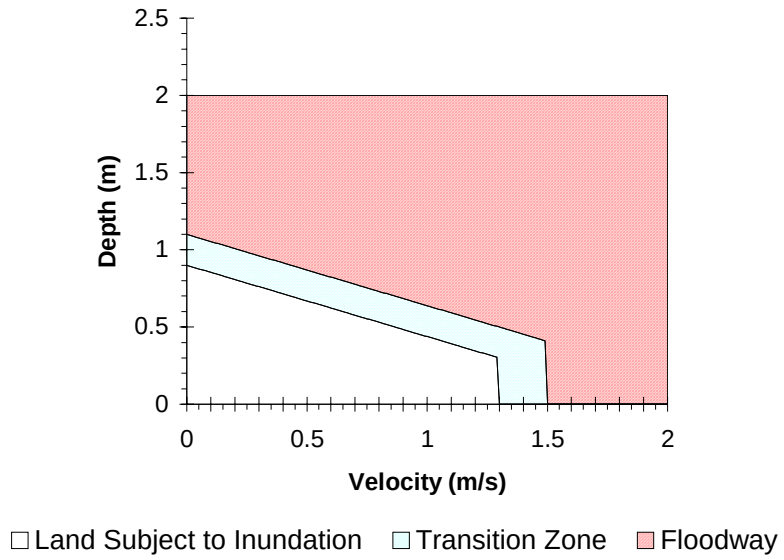


Figure 10-1 Floodway overlay flood hazard criteria

For **flood depth**, regions with a flood depth in the 1% AEP flood greater than 0.5 m were considered as FO based on the flood depth delineation option.

The three flood overlay delineation approaches were provided to the GHCMA from the design flood modelling results. The GHCMA determined that the 10% AEP flood extent was considered an appropriate floodway delineation option for the Surry River at Narrawong. It is noted that all three approaches produced quite similar FO extents within the study area.

The extent of the floodway overlay and LSIO are displayed in Figure 10-2.

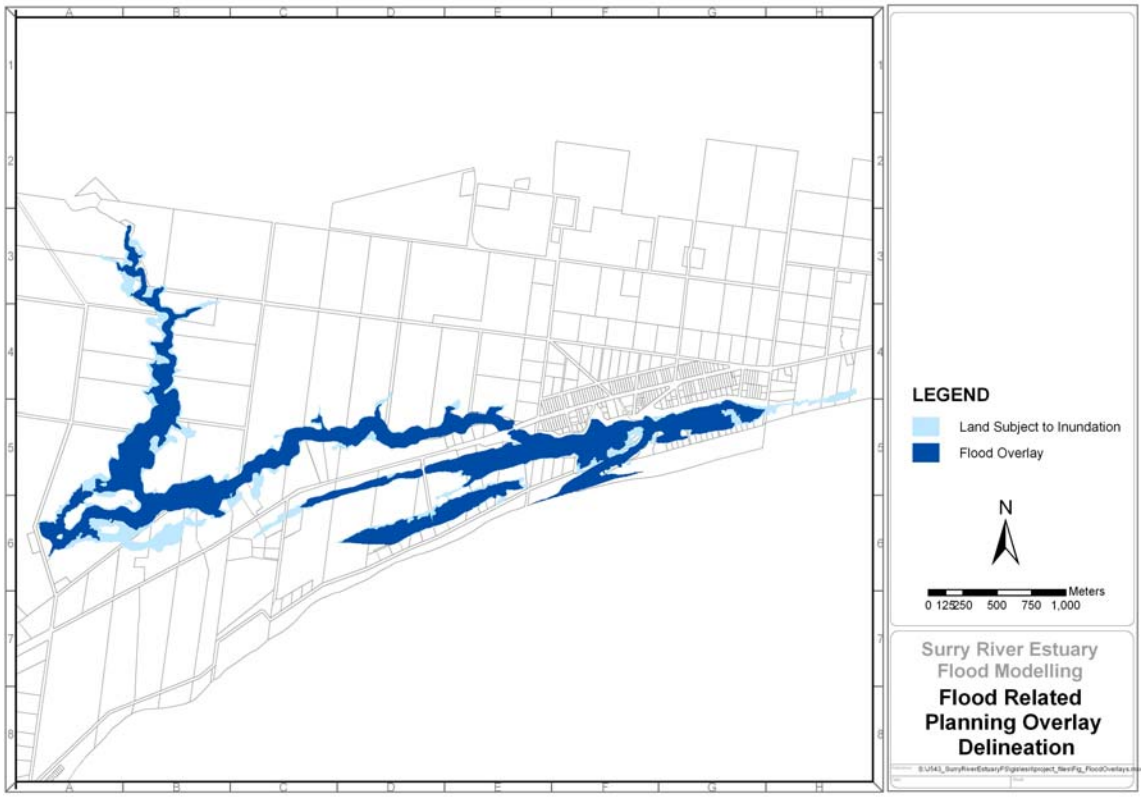


Figure 10-2 FO and LSIO Planning Overlay Delineation

10.3 Static Water Level Inundation Extents

Static water level inundation extents for the study area were developed from the DTM using GIS techniques. The inundation extents are required to assist the Glenelg Hopkins CMA in the management of water levels in the estuary during 'dry weather' flooding when the estuary mouth is closed. The inundation extents were determined at 100mm intervals between 1.0 and 2.0 m AHD.

The inundation extents developed assume that all culverts and minor channels in the study area are not obstructed and flows can pass through/along these features resulting in inundation to the same level. Some minor interpretive refinement of the inundation extents was required to take into consideration the uncertainties inherent in the resolution of very fine topographic details in the DTM.

The progressive inundation extents assuming a static water level along the entire length of the estuary are presented in Figure 10-3 from 1.0 to 2.0 m AHD.

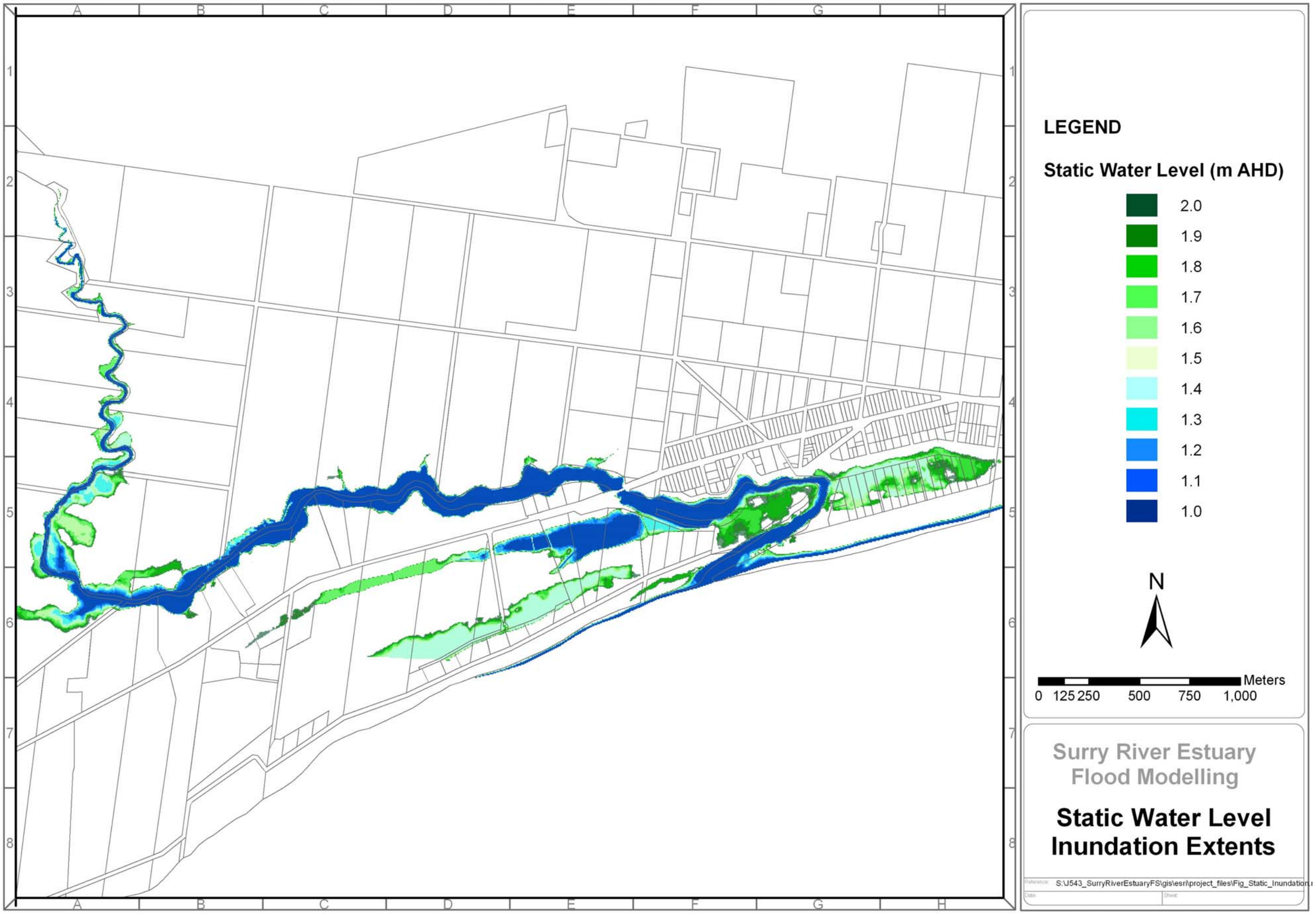


Figure 10-3 Static Water Level Inundation Extents

11 FLOOD RISK ASSESSMENT

11.1 Overview

A flood damages assessment has been undertaken for the study area under existing conditions. The flood assessment determined the monetary flood damages for design flood hydrographs as determined by the hydrologic and hydraulic models. The average annual damage (AAD) was also determined as part of the flood damage assessment.

Damages from flooding can be sub-divided into a number of categories. Figure 11-1 shows the various categories commonly used in flood damage assessments.

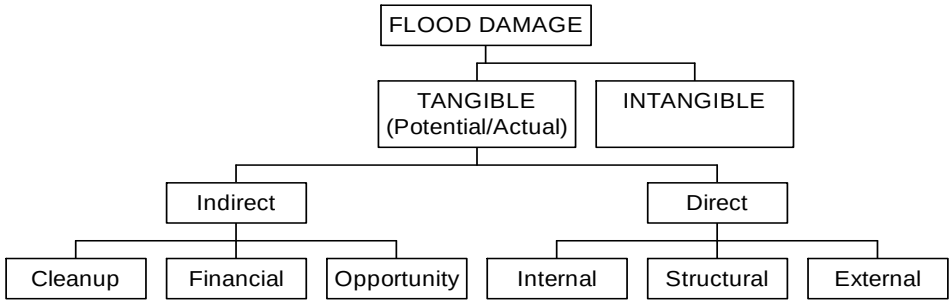


Figure 11-1 Flood Damage Categories

Tangible flood damages are those to which a monetary value can be assigned and include property damages, business losses and recovery costs. Intangible flood damages are those to which a monetary value cannot be assigned and include anxiety, inconvenience and disruption of social activities. Both are a function of flood magnitude. This flood damages assessment focuses on the tangible flood damages. Intangible damages are important but have not been directly accounted for in this flood damage assessment.

Tangible damages can be sub-divided into direct and indirect damages. Direct damages are those financial costs caused by the physical contact of flood waters and include damage to property, roads and infrastructure.

Property damages can be sub-divided into internal and external damages. Internal damages include damage to carpets, furniture and electrical goods. External damages include damages to building structures, vehicles and in rural areas, crops, fencing and machinery.

Tangible direct damages are further defined as either potential or actual damages. Potential damages are the maximum damages that could occur for a given flood event. In determining potential damages, it is assumed that no actions are taken (whether months or hours) prior to or during the flood to reduce damage by, for example, lifting or shifting items to flood free locations, shifting motor vehicles or sandbagging. Actual damages are the expected damages for a given flood event, allowing for some degree of community flood damage control. The actual damage is calculated as a proportion of the potential damage, based on the community's flood preparedness, a function of community awareness and the lead-time of flood warnings.

Indirect damages are those additional financial costs generally incurred after the flood during clean-up and include the cost of temporary accommodation, loss of wages, loss of production for commercial and industrial establishments and the opportunity loss caused by the closure or limited operation of business and public facilities. Indirect damages are often extremely hard to estimate.

The remainder of this section details the input data required and the methodology adopted for this flood damage assessment.

11.2 Available Information

This section outlines the range of information utilised within the flood risk assessment including property and floor level data, infrastructure data and flood data.

11.2.1 Property and Floor Level Data

The Narrawong Holiday Park caretaker's property was the only property identified as at flood risk under a 1% AEP flood. The floor level of the caretaker's property was surveyed at 1.92m AHD and was allocated a medium value class for the flood damage assessment.

11.2.2 Infrastructure Data

For this study, as detailed in the report '*Rapid Appraisal Method (RAM) for Floodplain Management*' (NRE, 2000), total damage to infrastructure was based on the length of road infrastructure inundated. NRE (2000) considers this assumption reasonable, as much of the service infrastructure follows the paths of road reserves and the quantity of other infrastructure might be expected to be broadly a function of the length of road. Damage to bridges is also incorporated into the NRE (2000) infrastructure damage cost estimates.

Roads were identified using the cadastral information supplied by GHCMA and by inspection of aerial photos.

11.2.3 Flood Data

The hydraulic analysis provides a regular grid of flood elevations and flood depths across the hydraulic model study area. By overlaying the flood elevations and depths onto the property data, a flood level can be assigned to each flood affected building, similarly lengths of road inundated can easily be calculated. The 10, 5, 2 and 1% AEP design floods were assessed in this study, with a 20% AEP flood assumed to result in no significant flood damage cost. This is discussed in further detail in Section 11.3.3.

11.3 Approach

The flood damage assessment was based on the RAM (NRE, 2000) and current best practice. The Bureau of Transport Economics report '*Economic Costs of Natural Disasters in Australia'* (BTE, 2001), provides an excellent source of information regarding methodology and cost estimates for flood damage assessments.

The flood damage assessment first estimated costs associated with direct flood damage (e.g. structural building, contents, external property, and infrastructure damage), then considered the costs associated with indirect flood impacts (e.g. emergency services, clean-up costs, alternative accommodation costs).

11.3.1 Direct Flood Damage

11.3.1.1 Property Damage

The ANUFLOOD stage-damage curves were factored up by 60% to bring them up to a 1999 flood damage cost level as recommended by the RAM (NRE, 2000). The ANUFLOOD stage-damage curves were further adjusted by the historical Consumer Price Index (CPI) ratio up to June 2007.

The stage-damage curves were applied to each inundated property and the costs summed to calculate the total direct potential flood damage cost.

Suggested damages for caravan parks are provided by the RAM (NRE, 2000) at $\$80m^2$. This figures includes internal, external and structural damages to the park infrastructure and

caravans. The Narrawong Holiday Park caretaker reported that the total park area is approximately 40,500m^2, however the actual park area dedicated comprising on-site vans, cabins and powered sites was estimated as approximately 6,750m^2. The density of caravan park sites is also considered low and the damage estimate from RAM would appear overly conservative.

The total direct potential flood damage cost is the cost that would be incurred if no mitigation measures are taken prior to or during a flood. Communities generally have at least some degree of warning, and particularly if a community has had previous flood experience, may reduce the effect of the flood significantly. Measures such as evacuation, doorstep sandbagging or the removal of valuable items to a safe level above flood waters have the potential to reduce the flood damage cost. As part of the community consultation process it was determined that there was almost no awareness or previous experience with flooding issues at Narrawong. To reflect this lack of awareness the potential to actual direct flood damage reduction factor from RAM (NRE, 2000) of 0.8 was adopted. This reflects the fact that the community has little or no flood experience and that they have only approximately 12 hours warning time, as shown in Figure 11-3.

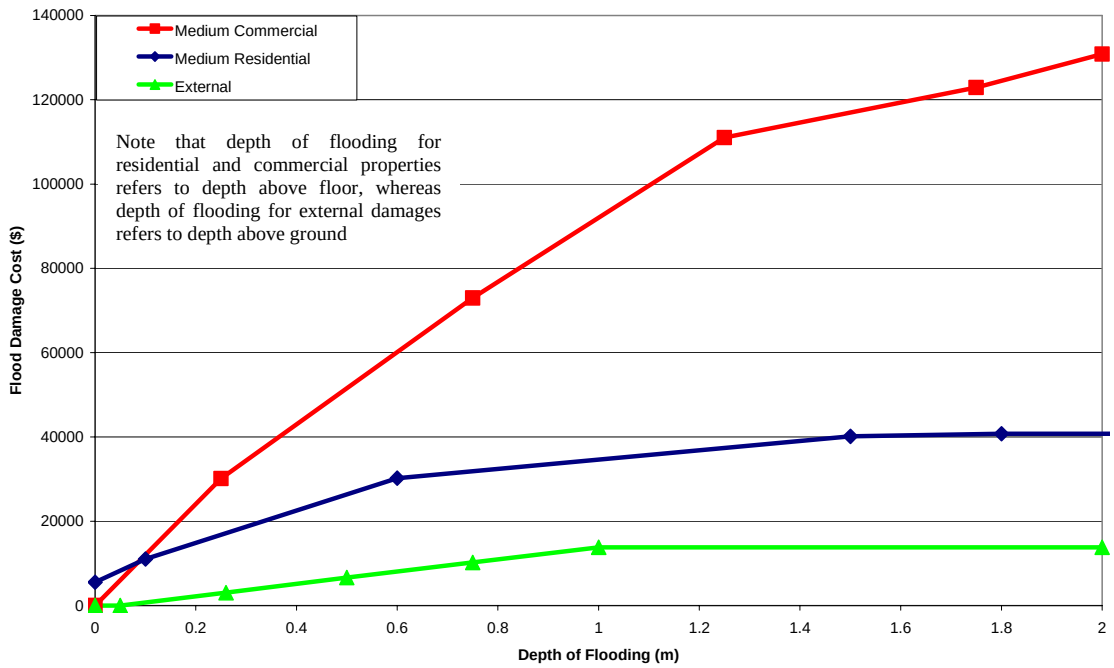


Figure 11-2 Adopted Stage-Damage Curves for Residential, Commercial and External Flooding

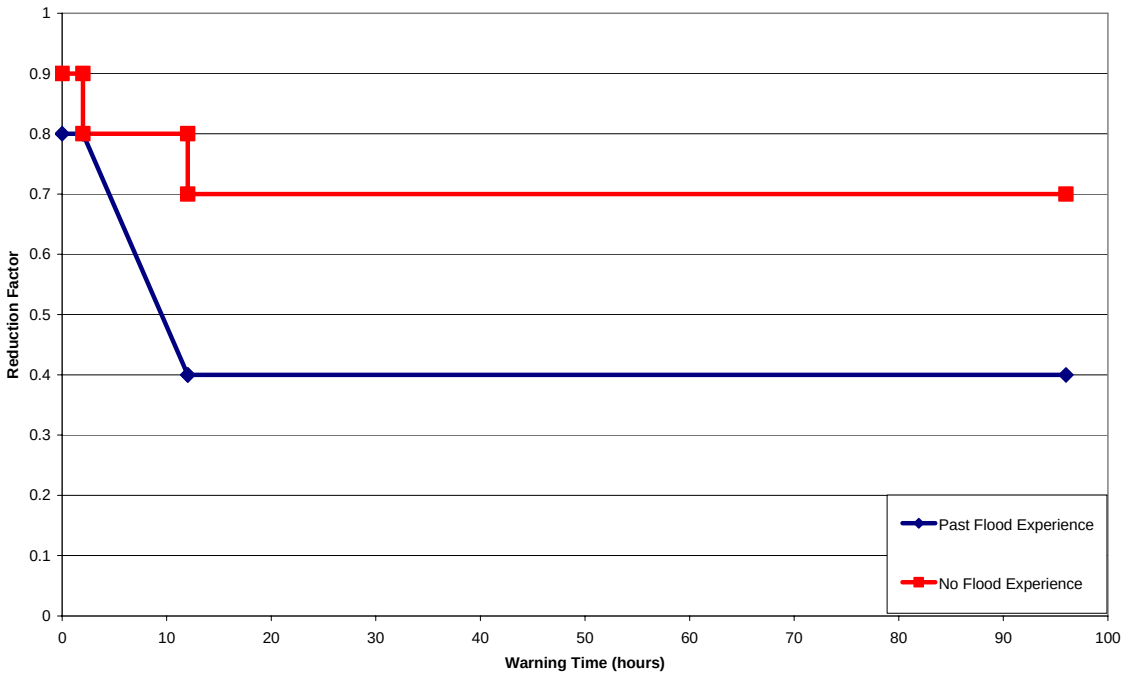


Figure 11-3 Reduction Factor Curves for Potential to Actual Direct Damage Ratio

11.3.1.2 Infrastructure Damage

Damage to infrastructure includes street and road repairs (including restoration of weakened subgrades), bridge repairs, telephone and telecommunications facilities, electrical connections, water supply and sewerage infrastructure and resulting higher maintenance costs.

For this study, as detailed in the RAM (NRE, 2000), total damage to infrastructure was based on the length of road infrastructure inundated. NRE (2000) considers this assumption reasonable, as much of the service infrastructure follows the paths of road reserves and the quantity of other infrastructure might be expected to be broadly a function of the length of road. Damage to bridges is also incorporated into the NRE (2000) infrastructure damage cost estimates by an approximation of damage to bridges per km of road inundated.

While it is appreciated that using the length of road inundated as the primary measure of total damage to infrastructure is a coarse approximation, it is considered reasonable, as it is the best estimate that we have due to lack of data and as it is only a small portion of the total damage cost.

Roads are subdivided into three categories in NRE (2000) – highway, sealed road and unsealed road. Roads inundated were identified as sealed roads from cadastral information supplied by GHCMA and by inspection of aerial photos.

The length of road inundated for the design flood events was calculated. The RAM (NRE, 1999) estimates of $10,000 per km for initial road repairs, $5,000 per km for road accelerated deterioration and $3,500 per km of road for bridge repairs were adjusted by a Consumer Price Index (CPI) ratio for 1999 to June 2007. The adopted flood damage rates for infrastructure are shown in Table 11-1. The length of inundated road for each design flood event was then multiplied by the adopted flood damage rates.

Table 11-1 Adopted Infrastructure Flood Damage Rates

Infrastructure	Flood Damage Rates (per km of road inundated)
Minor Sealed Road	**$23,435**
Unsealed Road	**$10,577**

June 2007.

11.3.2 Indirect Flood Damage

Indirect flood damages are damages incurred as a consequence of a flood but are not due to the direct impact of the flood itself (e.g. emergency services, clean-up costs, alternative accommodation, lost business opportunity, etc.). Indirect damages are extremely hard to estimate and are often calculated by assuming they equal 30% of the total actual direct flood damage cost (including damage to properties and infrastructure), as in the RAM (NRE, 2000). For rural areas with low population densities the RAM recommends an estimate of 20% ratio of indirect damages to actual direct flood damage costs and this estimate was considered applicable to the study area.

11.3.3 Total Flood Damage

The total flood damage cost was calculated as the sum of the direct actual property flood damage cost the direct infrastructure flood damage cost and the indirect flood damage cost.

The Average Annual Damage (AAD) was also calculated. The AAD is a measure of the flood damage per year averaged over an extended period. It is calculated from the area under the flood frequency and total flood damage curve as displayed in Figure 11-4. The AAD assumes that no flood damage is incurred at the 20% AEP flood event, and considers floods up to the 1% AEP flood.

As the total flood damages are very sensitive to the assumptions in the cost of flood damages to the caravan park it was decided to provide a range for the AAD that reflected the uncertainty in the flood damage estimate for the caravan park as discussed in Section 11.3.1.1.

The AAD for existing conditions for the study area is estimated at approximately **$33,000 - $42,000** assuming no damages at the 20% AEP flood, and considering floods up to the 1% AEP.

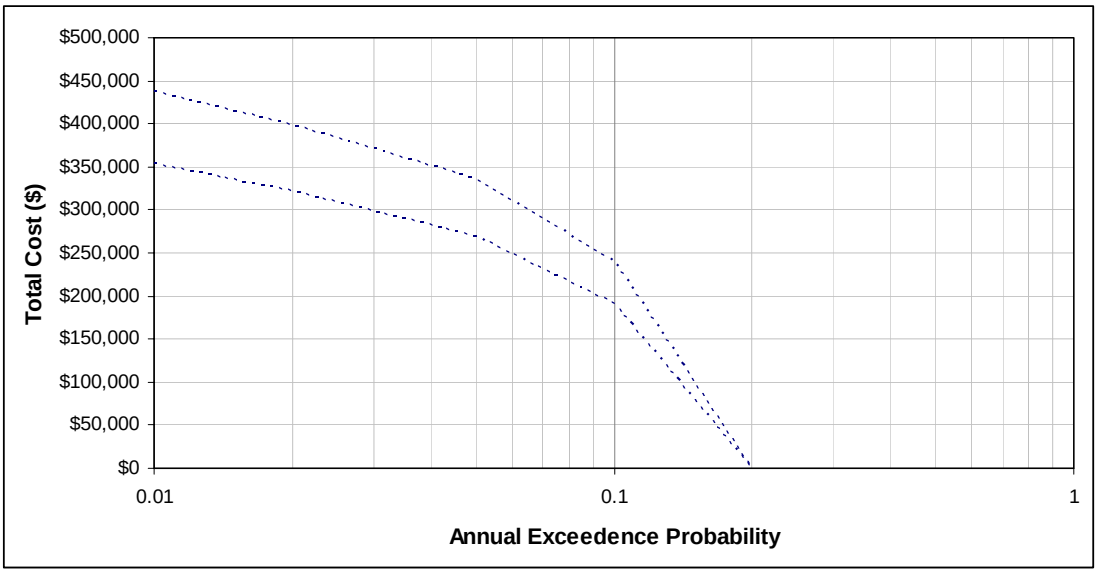


Figure 11-4 Average Annual Damages Curve

12 FLOOD WARNING, RESPONSE AND AWARENESS

The following are preliminary comments regarding flood warning and emergency response issues arising form the result of this study:

- The community consultation sessions held as part of the study highlighted the lack of awareness among the community of the flooding risks existing at Narrawong. This is likely due to the absence of significant flood events over the last decade and possibly due to a high population turnover. The Glenelg Hopkins CMA and Council should consider improving the communities flood awareness.
- Raising of the Caravan Park access road undertaken during the course of the study will provide greater lead time and improved safety in the event an evacuation of the caravan park is required due to flooding. Depths across the caravan park road are still however expected to exceed 0.5m during a 1% AEP flood.
- The flood peak travel time during a 1% AEP flood from the Heathmere Gauge to the Caravan Park is expected to be less than two hours making the Heathmere Gauge ineffective for providing practical flood warning for the Caravan Park residents. Given the size of the catchment it would not be considered feasible for the BOM to provide reliable flood warnings for Narrawong with the minimum 6 hour warning time.
- Council should develop a Flood Sub-Plan for Narrawong as part of the Council's Emergency Management Plan (EMP). The plan should identify the flood risks and document the response required to minimise risk to life and property.
- The EMP for the caravan park should be updated to reflect the improved understanding of the existing flood risks to the caravan park and residents. Guidelines as to the type of information and arrangements that should be included in the EMP for caravan parks are provided in the document, 'Victoria Caravan Parks Flood Risk Survey' (Bewsher Consulting Pty Ltd, 2006).

13 CONCLUSIONS AND RECOMMENDATIONS

The Surry River Estuary Flood Study has increased the understanding of flood behaviour and flood risks throughout the study area, leading to the following conclusions and recommendations.

Existing Flood Risks

Aside from the Caravan Park, it is considered that only limited risks are posed by flooding in the Surry River estuary up to the 1% AEP flood. Appropriate planning controls should be enforced to limit the increase in flood risks to property and lives in the future.

Land Use Planning

The hydraulic analysis enabled the delineation of revised FO and LSIO within the study area.

The study team recommends the GHCMA liaise in the preparation and adoption of a planning scheme amendment to enable the draft flood related planning zone and overlays.

Further, the study team recommends GHCMA declares the 1% AEP flood level for planning purposes under the Water Act (1989).

Flood Warning and Response

The study team recommends the GHCMA liaise with BOM and the Narrawong Holiday Park to consider feasible flash flood warning arrangements. The EMP for the caravan park should be updated to reflect the improved understanding of the existing flooding risks to the caravan park developed during the course of the study.

Improved community awareness of the flood risk can aid effective flood response. Using the study outcomes, the study team recommends material aimed at improving community flood awareness is prepared and distributed.

14 REFERENCES

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Laurenson E.M. and Mein R. G., (1992). RORB - *Version 4 Runoff Routing Program User Manual*. Monash University Department of Civil Engineering.

Schiller, P., Forbes I.G.(1946), *Report on the Western District Floods of March 1946*, SR&WSC

Meehl, G.A., T.F. Stocker, W.D. Collins, P. Friedlingstein, A.T. Gaye, J.M. Gregory, A. Kitoh, R. Knutti, J.M. Murphy, A. Noda, S.C.B. Raper,
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Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change [Solomon, S.,
D. Qin, M. Manning, Z. Chen, M. Marquis, K.B. Averyt, M. Tignor and H.L. Miller (eds.)]. Cambridge University Press, Cambridge, United
Kingdom and New York, NY, USA.

APPENDIX A

Aerial Photogrammetry MetaData and Validation

Qasco Victoria Pty Ltd

The information supplied in the data file(s) on this disc(s) is supplied subject to the following conditions :

1. That this file (Readme.txt) is always stored with the unaltered data file(s) contained on this disc(s), and shall accompany this file(s) if data is supplied to other parties.
2. That the data file(s) contained on this disc(s) are not altered in any way. The data may be copied to other storage and manipulated as required.
3. That the following information is noted and accepted:

. The topographic data contained on this disc(s) was derived for WaterTechnology from aerial photography at a scale of 1:6600 It is suitable for the generation of contours with a vertical interval of no less than 0.5 metres. It is not valid to reduce the contour interval from 0.5 metres by interpolation.

4. Data falling inside strings on layer "Boundary_Reliability" is of DOUBTFUL ACCURACY due to obstructions of some description (e.g. vegetation or shadow).

****

. Control for this project has been derived from ground survey.

The area falling under timber is fully obscured and is an area of very doubtful accuracy.

The contours in this area are formlines ONLY and should not be used for any final design work.

****

. Horizontal Co-ordinates are based on MGA Zone 54

. Vertical Datum is based on A.H.D.

. Vertical Accuracy on a Solid Point will be no more than 0.15m RMS

. Aerial Photography:	Qas 3598
. Date of Photography:	4th Feb 2007
. Flying Height:	1050 metres A.S.L.
. Date of Compilation:	March \ April 2007

4. That if the data on this disc(s) is presented in any hard copy format for use by other parties, then the details of Condition 3 shall be printed on the face of such a plan or map.

5. That Qasco Victoria Pty Ltd accept no responsibility for the data in the event of non-compliance with any of these conditions.

Qasco Victoria Pty Ltd maintains an archive copy of the files on this disc(s) and their validation codes are listed below.

For future Reference please Quote Job Number: V15104 Surry River

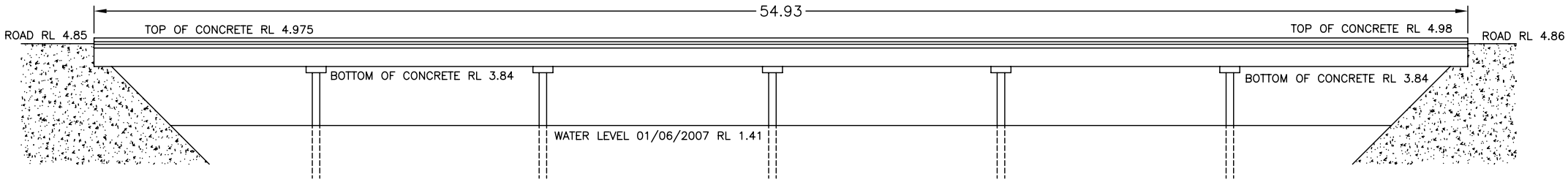
Any problems associated with the information in the data file(s) contained on this disc(s) should be reported to :

Qasco Victoria
171 Clarendon St.
South Melbourne VIC 3006
Ph: 03 9682 3330 Fx: 9682 3335
melbourne@qasco.com.au

Photogrammetry Validation Results

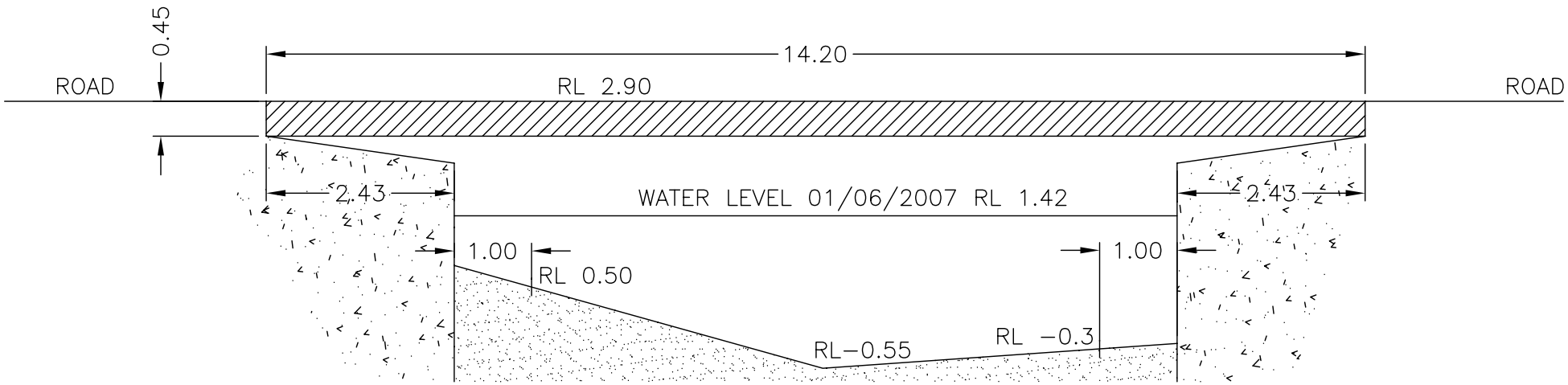
Permanent Marker	X	Y	Z (m AHD)	Photogrammetry Z (m AHD)	Difference (m)
58	**561981.514**	**5765650.11**	**7.793**	**7.743**	**0.050**
60	**561275.494**	**5765295.64**	**3.330**	**3.270**	**0.060**
27	**562722.924**	**5765912.92**	**10.840**	**10.716**	**0.124**
86	**560454.753**	**5765354.44**	**9.138**	**9.061**	**0.077**

APPENDIX B



PRINCES HIGHWAY BRIDGE

LOOKING NORTH

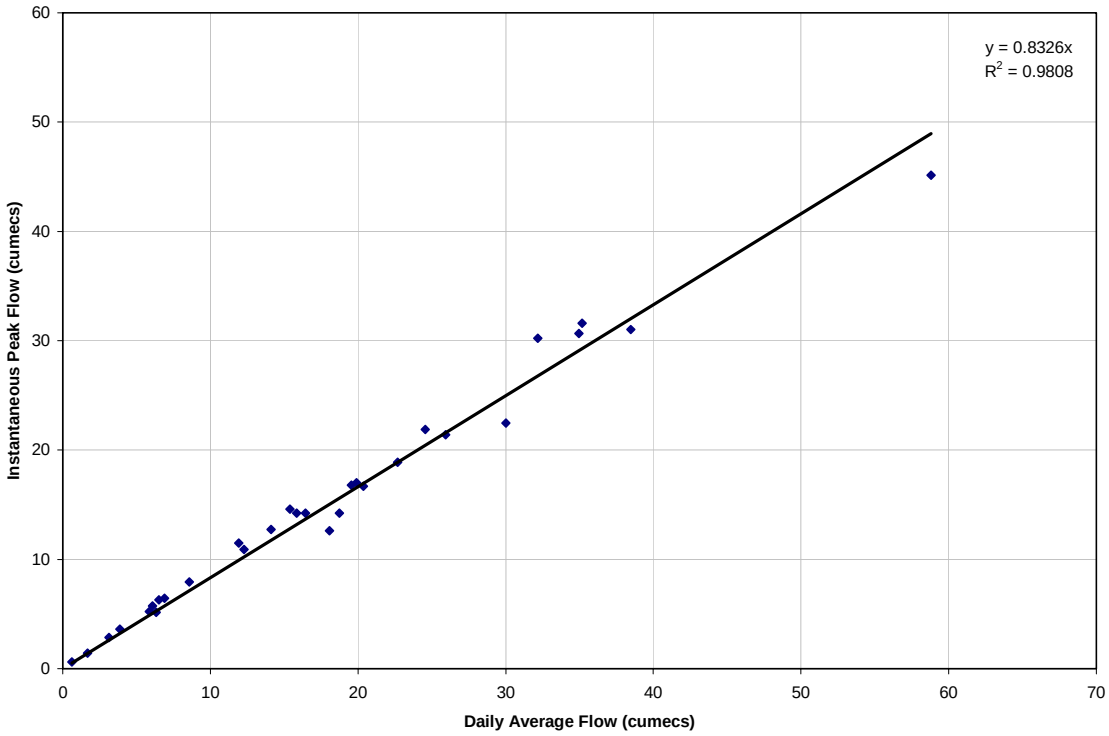


WADES BRIDGE

LOOKING SOUTH

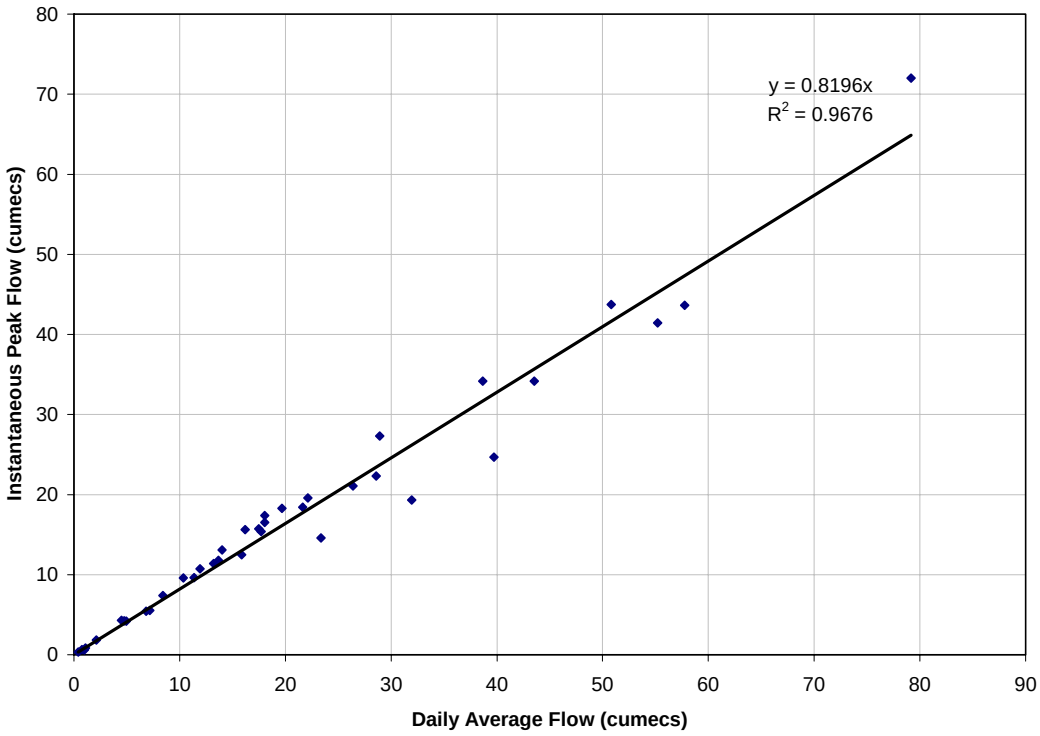
APPENDIX C

Surry River Maximum Annual Daily Flow and Maximum Annual Instantaneous Flow Correlation



APPENDIX D

Fitzroy River Maximum Annual Daily Flow and Maximum Annual Instantaneous Flow Correlation



APPENDIX E

HISTORICAL AERIAL PHOTOGRAPHY

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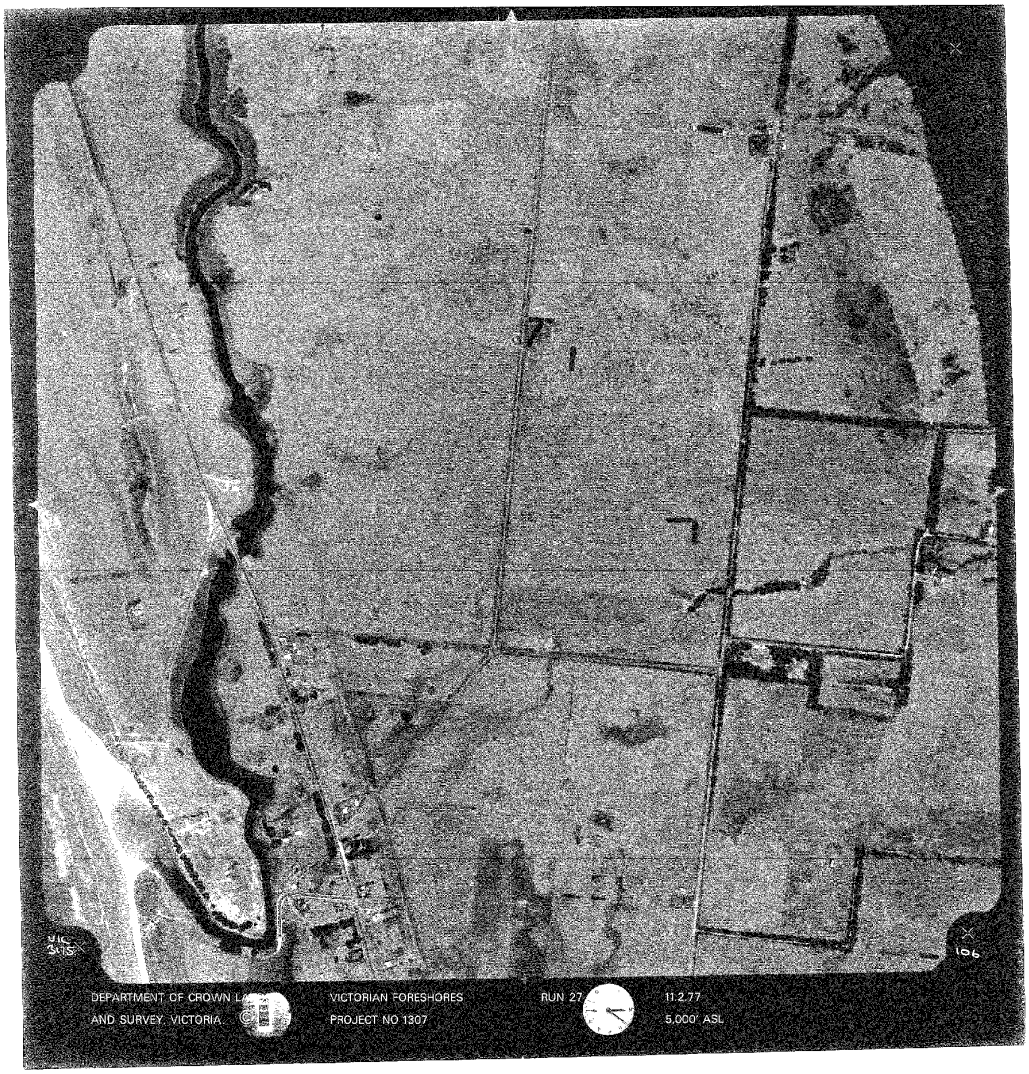
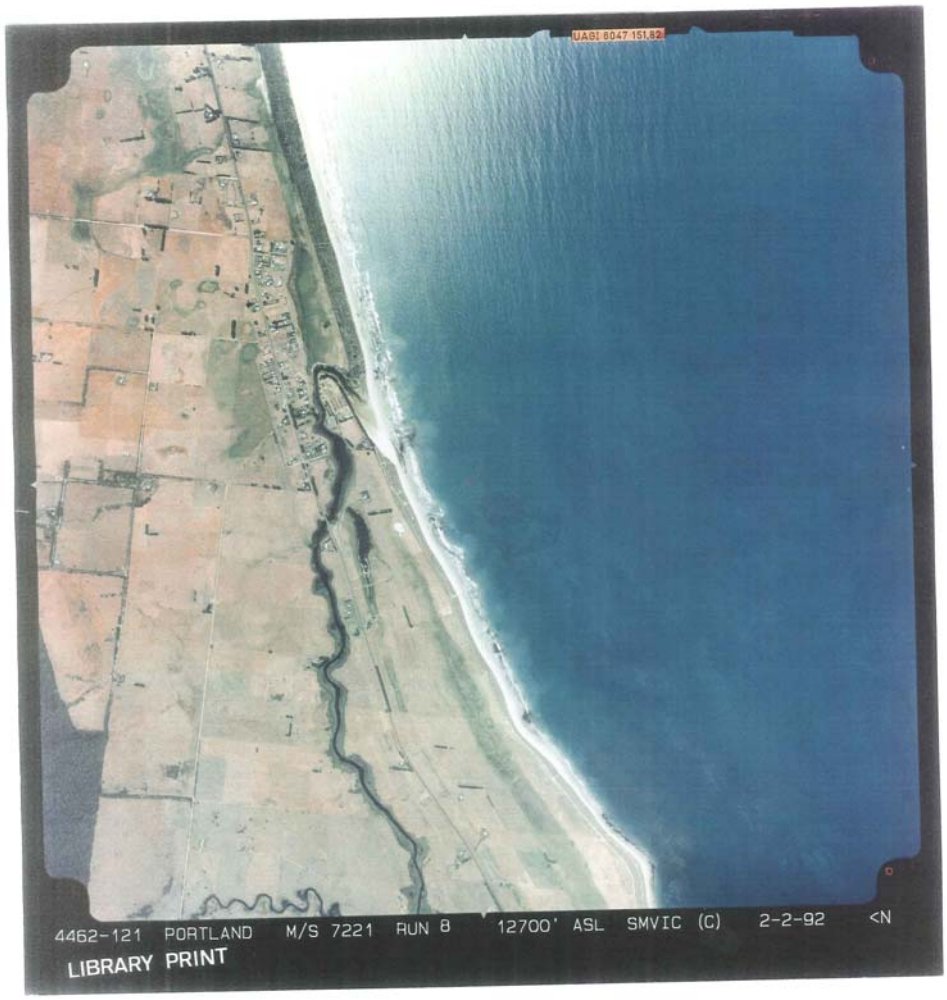
13 November 1985

19 January 1947



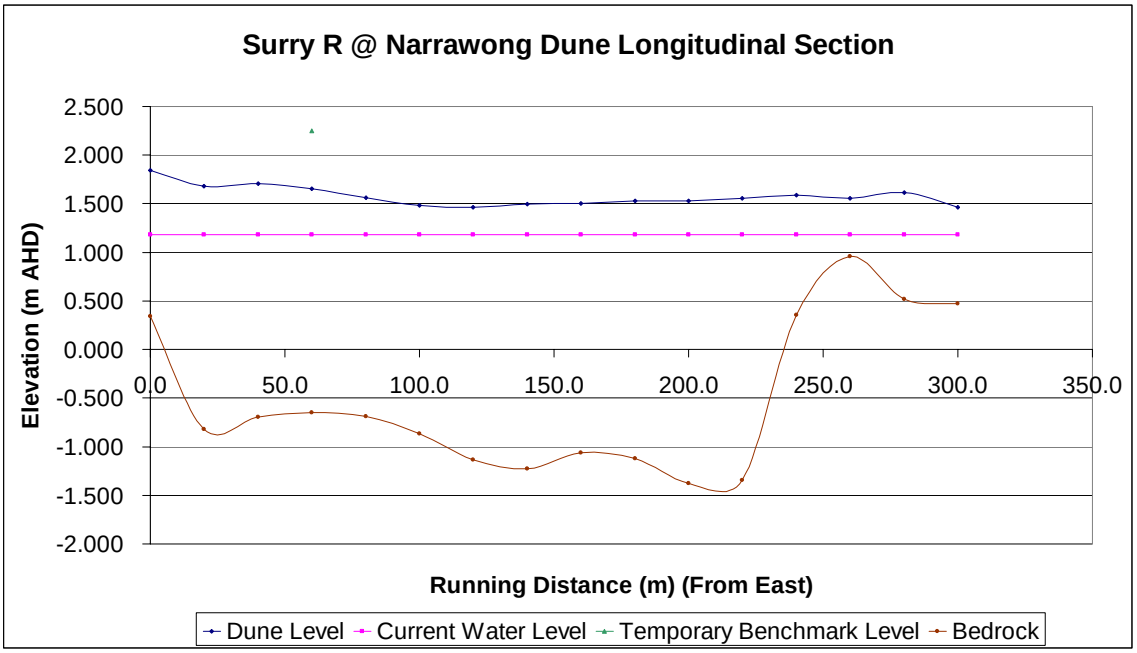
11 February 1977



2 February 1992

APPENDIX F

ENTRANCE BEDROCK PROFILE



APPENDIX G

PARTICLE SIZE DISTRIBUTION

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191 Drummond Street, Carlton VIC 3053
PO Box 663, Carlton South VIC 3053
Telephone: (03) 9663 1333 Facsimile: (03) 9663 2555

Material Test Report

Report No: MAT:WT--PM079506PM
Issue No: 1

Client: Water Technology

Project:

NATA

This laboratory is accredited by the National Association of Testing Authorities, Australia. The Test(s) reported have been performed in accordance with its terms of accreditation.

Approved Signatory: Danielle Hardwick

NATA Accredited Laboratory Number: 1235

Date of Issue: 7/06/2007

Sample Details

Sample ID: PM079506PM
Field Sample:
Date Sampled:
Source:
Material: Fine Sand
Specification:
Sampling Method: Unknown

Other Test Results

Description	Method	Result
Fineness Modulus		0.8
Moisture Content (%)	AS 1289.2.1.6	33.6

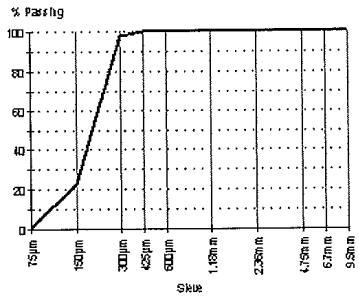
Particle Size Distribution

Method: AS 1141.11, AS 1141.12
Drying by: Hotplate

Note: Sample Washed

Sieve Size	% Passing
9.5mm	100
6.7mm	100
4.75mm	100
2.36mm	100
1.18mm	100
600µm	100
425µm	100
300µm	98
150µm	23
75µm	1
Finer 75µm	1

Chart



Comments
N/A

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PO Box 663, Carlton South VIC 3053
Telephone: (03) 9663 1333 Facsimile: (03) 9663 2555

Material Test Report

Report No: MAT:WT--PM079505PM
Issue No: 1

Client: Water Technology

Project:

NATA

This laboratory is accredited by the National Association of Testing Authorities, Australia. The Test(s) reported have been performed in accordance with its terms of accreditation.

Approved Signatory: Danielle Hardwick

NATA Accredited Laboratory Number: 1235

Date of Issue: 7/06/2007

Sample Details

Sample ID: PM079505PM
Field Sample:
Date Sampled:
Source:
Material: Fine Sand
Specification:
Sampling Method: Unknown

Other Test Results

Description	Method	Result
Moisture Content (%)	AS 1289.2.1.6	13.1

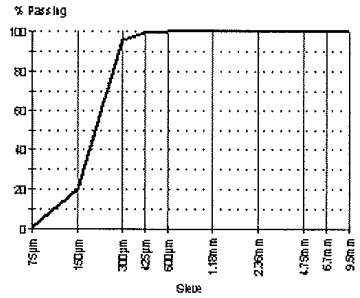
Particle Size Distribution

Method: AS 1141.11, AS 1141.12
Drying by: Hotplate

Note: Sample Washed

Sieve Size	% Passing
9.5mm	100
6.7mm	100
4.75mm	100
2.36mm	100
1.18mm	100
600µm	100
425µm	99
300µm	96
150µm	20
75µm	1
Finer 75µm	1

Chart



Comments
N/A

Form No: 18909.V1.00

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