Western Port Local Coastal Hazard Assessment

Report 4 (R04)– Inundation Hazards

October 2014

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GLOSSARY

Australian Height Datum (AHD)	A common national plane of level corresponding approximately to mean sea level
ARI	Average Recurrence Interval. The average or expected value of the periods between exceedances of a given event over a given duration.
AEP	Annual Exceedance Probability: The measure of the likelihood (expressed as a probability) of an event equalling or exceeding a given magnitude in any given year
Artesian Aquifer	A confined aquifer containing groundwater under positive pressure
Astronomical tide	Water level variations due to the combined effects of the Earth's rotation, the Moon's orbit around the Earth and the Earth's orbit around the Sun
Calibration	The process by which the results of a computer model are brought to agreement with observed data
Chart Datum (CD)	Common datum for navigation charts. Typically relative to Lowest Astronomical Tide
Coastal Hazard	A term to collectively describe physical changes and impacts to the natural environment which are significantly driven by coastal or oceanographic processes.
Dependence	Referring to a numerical relationship between variables and the extent to which one can be predicted solely from a knowledge of the other(s)
Diurnal	A daily variation, as in day and night.
DTM	Digital Terrain Model, a three dimensional representation of the ground surface
Ebb Tide	The outgoing tidal movement of water resulting in a low tide.
Eustatic Sea Level Rise	A rise in mean sea level at the global scale, for example as a result of melting ice-caps
Exceedance Probability	The probability of an extreme event occurring at least once during a prescribed period of assessment is given by the exceedance probability. The probability of a 1 in 100 year event (1% AEP) occurring during the first 25 years is 22%, during the first 50 years the probability is 39% and over a 100 year asset life the probability is 63%
Flood Tide	The incoming tidal movement of water resulting in a high tide
Foreshore	The area of shore between low and high tide marks and land adjacent thereto
GIS	Geographical Information System
HAT	Highest Astronomical Tide: the highest water level that can occur due to the effects of the astronomical tide in isolation from meteorological effects
Hydrodynamic Model	A numerical model that simulates the movement of water within a defined model area
Independence	The complete lack of dependence between two variables, even if time lag is permitted.
Intertidal	Pertaining to those areas of land covered by water at high tide, but exposed at low tide, eg. intertidal habitat
Inverse Barometric Pressure Effect	The inverse response of sea level to changes in atmospheric pressure.
Inundation	The area of land covered in water either through flooding from elevated coastal

	water levels or catchment generated flows.
Isostatic Sea Level Rise	A rise in sea level relative to a fixed position, for example as a result of land subsidence.
Joint Probability	Referring to the distribution and extremes of two (or more) related variables. In this study the variables assessed are storm surges and catchment generated stream flows.
Levee	Raised embankment along the edge of a coastal or riverine environment
LiDAR	**Li**ght **D**etection **a**nd **R**anging – also known as airborne laser scanning, is a remote sensing tool that is used to generate highly accurate 3D maps of the Earth's surface
MHHW	Mean Higher High Water: the mean of the higher of the two daily high waters over a long period of time. When only one high water occurs on a day this is taken as the higher high water
MHWM	Mean High Water Mark, i.e. the mean of high water over a long period of time
MHWS	Mean High Water Springs, i.e. the mean of spring tide water levels over a long period of time.
MLWM	Mean Low Water Mark, i.e. the mean of low water over a long period of time
MSL	Mean Sea Level
Neap Tides	Neap tides occur when the sun and moon lie at right angles relative to the earth (the gravitational effects of the moon and sun act in opposition on the ocean).
Nearshore	The region of land extending from the backshore to the beginning of the offshore zone.
Non Tidal Residual Water level	The non-tidal residual water level is the total water level minus the astronomical tidal component of the water level. The remaining non-tidal residual water level comprises of all or a combination of wind setup, wave setup, the inverse barometric effect, coastally trapped waves etc.
Peaks over Threshold (POT)	A method of preparing data for extreme analysis, in which independent maxima above a threshold are identified and extracted.
Pleistocene	The period from 2.5M to 12,000 years before present that spans the earth's recent period of repeated glaciations and large fluctuations in global sea levels
Sea Level Rise (SLR)	A long-term increase in the mean sea level
Semi-diurnal	A twice-daily variation, eg. two high waters per day
Significant wave height	The mean wave height of the one third highest waves
Spectral wave model	A numerical model used to simulate the sea state as it varies with time based on wind and/or swell conditions.
Spring Tides	Tides with the greatest range in a monthly cycle, which occur when the sun, moon and earth are in alignment (the gravitational effects of the moon and sun act in concert on the ocean)
Storm Surge	The increase in coastal water levels caused by the barometric and wind set-up effects of storms. Barometric set-up refers to the increase in coastal water levels associated with the lower atmospheric pressures characteristic of storms. Wind set-up refers to the increase in coastal water levels caused by an onshore wind driving water shoreward and piling it up against the coast
Storm Tide	Coastal water level produced by the combination of astronomical and

	meteorological (storm surge) ocean water level forcing
Synoptic Chart	A weather chart showing the distribution of meteorological conditions over a wide region at a given moment.
Tidal Constituents	The different components that make up the astronomical tide, based on the relative influence of the sun and the moon.
Tidal Planes	A series of water levels that define standard tides, eg. 'Mean High Water Spring' (MHWS) refers to the average high water level of Spring Tides
Tidal Prism	The volume of water moving into and out of an estuary or coastal waterway during the tidal cycle.
Tidal Range	The difference between successive high water and low water levels. Tidal range is maximum during Spring Tides and minimum during Neap Tides
Tides	The regular rise and fall in sea level in response to the gravitational attraction of the Sun, Moon and Earth
Vulnerability	Vulnerability is a function of exposure to climatic factors, sensitivity to change and the capacity to adapt to that change. In this report is means the degree to which a natural system is or is not capable of adapting or responding to the impacts of coastal hazards to which they are physically susceptible and exposed.[1]
Wave Setup	In shallow waters, the dynamics of waves in shallow depths including wave breaking process can result in an additional setup of water levels shoreward of the surf zone due to balance of the wave-induced shoreward momentum fluxes
Wind Shear	The stress exerted on the water's surface by wind blowing over the water. Wind shear causes the water to pile up against downwind shores and generates secondary currents
Wind Setup	The action of wind on the water surface creates shear stresses that can drag water in the downwind direction. In shallow depths and/or intertidal areas, the rate at which water is transported downwind exceeds the rate at which it can return under gravity and an elevation of water levels is observed at the downwind location.

[1] Definition taken from the Smartline Glossary http://www.ozcoasts.gov.au/coastal/smartline_terms.jsp
[2] Definition taken from the Smartline Introduction http://www.ozcoasts.gov.au/coastal/introduction.jsp

1. INTRODUCTION

1.1 Background

Melbourne Water commissioned Water Technology to undertake the Western Port Local Coastal Hazard Assessment (WPLCHA) project. The project has come about through a partnership between Melbourne Water, the Department of Environment and Primary Industries, South East Councils Climate Change Alliance, Bass Coast Shire Council, Cardinia Shire Council, City of Casey and Mornington Peninsula Shire Council.

The WPLCHA is a component of the Department of Environment and Primary Industries Future Coasts program, and Western Port is one of four priority sites in which local coastal hazard assessments have or are currently being undertaken.

1.2 Scope

As detailed in the project brief, the scope of the WPLCHA is to provide information on the extent of coastal hazards and their physical impacts for the Western Port coastal environment. The WPLCHA is focussed on assessing the physical hazards of erosion and inundation. It does not include any subsequent assessment of impacts of the hazards on built, economic or social infrastructure, assets or values and does not include preparing adaptation responses to the physical hazards.

The information developed by the project will assist in planning for and managing coastal hazards. It will allow management agencies and other key stakeholders to identify and define triggers as the basis for short, medium and long term management responses. Specifically, the information will provide information, data and mapping to inform consistent policy and practice and support agencies in identification and management of risk, and undertake; strategic planning, statutory planning, infrastructure maintenance and replacement schedules, natural asset management, and business planning and budgetary processes that are responsive to a changing climate, its impacts and opportunities.

The boundaries of the study area for the WPLCHA project are shown in Figure 2-1, and described as follows:

- Cape Schanck to West Head, along the shoreline of Western Port to the bridge at San Remo
- Inland from the Western Port shoreline will remain undefined enabling the assessment to be as far into the catchment as relevant
- All of the coast of French Island and the north side of Phillip Island from the bridge at Newhaven to the western extremity of Phillip Island (Seal Rocks), but excluding the south side of Phillip Island from Seal Rocks to the Bridge at Newhaven.

The study itself was split into two components:

- Part A - a broad scale Western Port wide coastal hazard assessment, and
- Part B - four local scale coastal hazard assessments.

1.3 Inundation Hazard Assessment Overview

The WPLCHA has broadly identified key coastal processes and hazards within the study area through the application of various techniques including detailed hydrodynamic modelling.

This report details the analysis undertaken to assess the potential impact of projected mean sea level rise this century on the extent of inundation hazards associated with storm surge events and catchment streamflows within the Western Port study area.

Various assets, ranging from economic to environmental, are located adjacent to the shorelines of Western Port and lie at relatively low elevations making them vulnerable to inundation associated with elevated water levels. Extreme elevated water levels within Western Port are a function of a number of different physical forcings and hydrodynamic processes including coastally driven water levels, wind and wave set-up and catchment generated streamflows. Detailed hydrodynamic modelling has been used to integrate these processes to enable estimates of extreme water levels to be assessed and identify how the processes may be impacted by increased mean sea levels.

This report describes the Part A broad scale assessment of potential coastal inundation hazards undertaken for this study, and incorporates the following components:

- Summary of the physical processes and dynamics that cause extreme elevated water levels, and an overview of historic inundation and flooding in Western Port;
- Overview of existing protection works and structures which influence inundation and flooding;
- Discussion of the analysis undertaken to identify representative design storm surge, wind and wave scenarios;
- Modeling analysis of the impact of sea level rise on extreme water levels in Western Port; and
- Analysis of major sources of uncertainty that could impact the inundation hazard assessment

In addition to the inundation hazard assessment associated with coastal and catchment derived surface waters, the project brief also requested a high level assessment of groundwater change hazards to identify areas where groundwater change hazards are likely to be key issues warranting further assessment. Due to the impact of surface water changes on potential groundwater hazards, this high level groundwater hazard assessment has been incorporated into this report.

1.4 Reporting & Outputs

This document is part of a series of reports produced as part of the Western Port Local Coastal Hazard Assessment project. It should be read in conjunction with the following:

- Report 1: Summary Report (R01)
- Report 2: Data Review (R02)
- Report 3: Methodology Overview (R03)
- **Report 4: Inundation Hazards (R04)**
- Report 5: Erosion Hazards (R05)
- Report 6: Critical Locations (R06)

Accompanying these documents is a project geographical information system (GIS), which includes the following outputs from the inundation assessment:

- Digital geo-referenced data, including shape files of inundation hazard areas for the present mean sea level situation and for future sea level rise events.
- Digital field data acquired for the study, including location, elevation and summary output.
- Relevant model set-up and run files.

2. INUNDATION PROCESSES AND DYNAMICS

2.1 Overview

The following sections provide an overview of the physical processes and mechanisms which influence water levels within Western Port and thus drive coastal inundation and flooding. These processes include tides, storm surge, wind and wave setup, and catchment generated streamflows. Following this, a brief summary of historical inundation around the coastal areas surrounding Western Port is described.

Figure 2-1 Project Study Area in Western Port Bay

Flooding and inundation within Western Port and low lying regions adjacent to Western Port can result from a complicated interaction between a number of physical forcings and hydrodynamic processes, as shown in Figure 2-2. The following summarises the main physical forcing that can give rise to extreme water levels in Western Port.

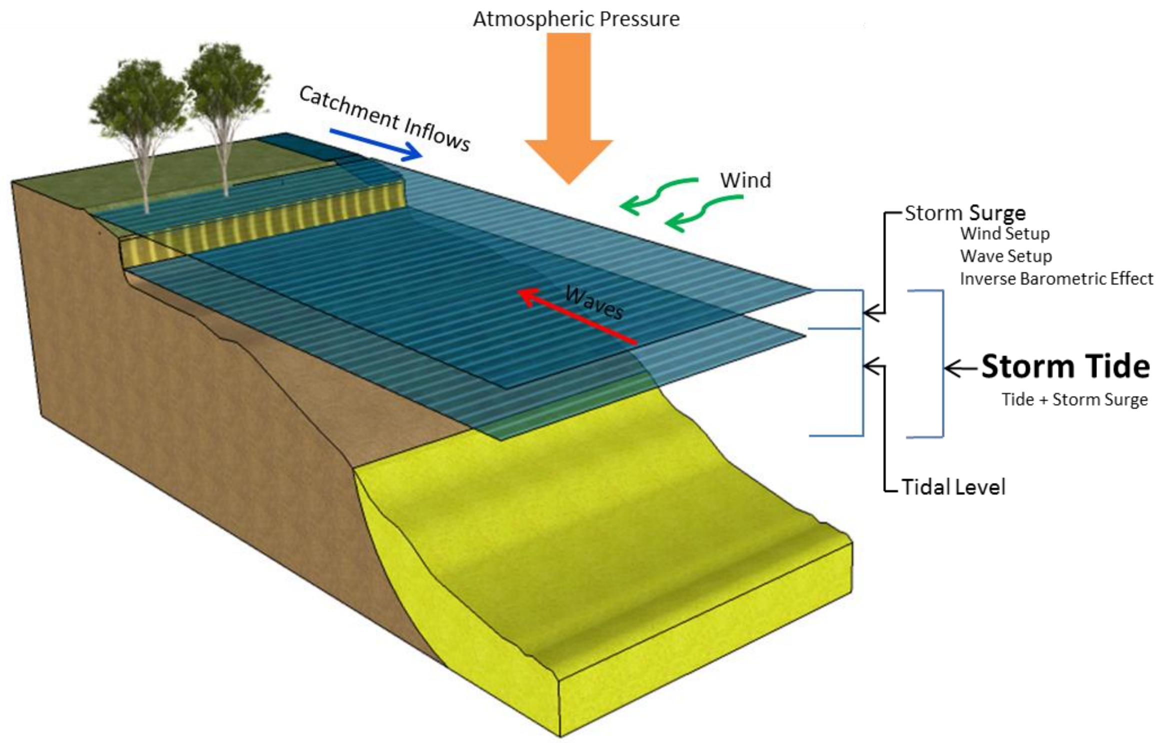

Figure 2-2 Processes Producing Elevated Water Levels in Western Port

2.1.1 Coastal Driven Water Levels

Western Port experiences water level variations associated with astronomical tides caused by the gravitational attractions between the Earth, Moon and Sun. Astronomical tides result in relatively high frequency diurnal (daily) and semi-diurnal (twice daily) water level variations that propagate through the Western and Eastern entrance of Western Port. The astronomical tides undergo further modifications within Western Port due to interactions with the planform and bathymetry.

Additional coastal water level variations propagate into Western Port associated with meteorological forcing due to the inverse barometric pressure effects and the interaction of weather systems and coastal waters which generate coastally trapped waves along the southern margin of the continental Australian landmass. Extreme meteorologically driven water level events are generally referred to as storm surges, and are further described in Section 2.2.3. The combined elevated water level due to the astronomical tide and storm surge is generally referred to as a storm tide.

2.1.2 Wind & Wave Setup

The action of wind on the water surface creates shear stresses that can drag water in the downwind direction. In shallow depths and/or intertidal areas within Western Port, the rate at which water is transported downwind exceeds the rate at which it can return under gravity and an increased elevation of water levels is observed at downwind locations.

The action of wind on the water surface also generates waves which propagate in the downwind direction. Along a shoreline, the dynamics of waves in shallow depths including wave breaking process can result in further increase in water levels shoreward of the surf zone.

2.1.3 Catchment Streamflows

A number of major streams and drains enter Western Port, including the Bunyip, Lang Lang and Bass Rivers (Figure 2-8). Intense and/or prolonged rainfall in the catchment produces flood flows which can affect low-lying areas adjacent to the coast. Many of these low-lying areas were swamps prior to European settlement, and are naturally flood-prone areas. They rely on constructed drains to reduce the severity and duration of flooding. Elevated coastal water levels can exacerbate catchment generated flood events in these areas.

2.2 Coastal Inundation Processes

The following sections provide an overview of the main components of coastal water level variations relevant for assessing inundation hazards within Western Port.

2.2.1 Mean Sea Level

A small difference between mean sea levels and 0.0 m AHD is observable from the long term sea level observations at Stony Point in Western Port. The difference is generally of the order of 0.03-0.05 m.

2.2.2 Astronomical Tides

The lunar semi-diurnal tide is the principal mechanism driving water level variations observed in Western Port. The spring tidal range is approximately 2.0 m at Stony Point. Figure 2-3 displays the frequency of observed water levels at Stony Point relative to key tidal planes.

Astronomical tidal ranges increase towards the head of the bay to a maximum of approximately 1.3 times the range at the entrances. Amplification of the tide towards the northern and eastern shorelines is due to the one quarter wavelength resonance of the lunar semidiurnal tide within Western Port.

Table 2-1 displays the change in amplitude of the main astronomical tidal constituents between Flinders and Tooradin noting the significant increase in the amplitude of the lunar semi diurnal (M_2) constituent.

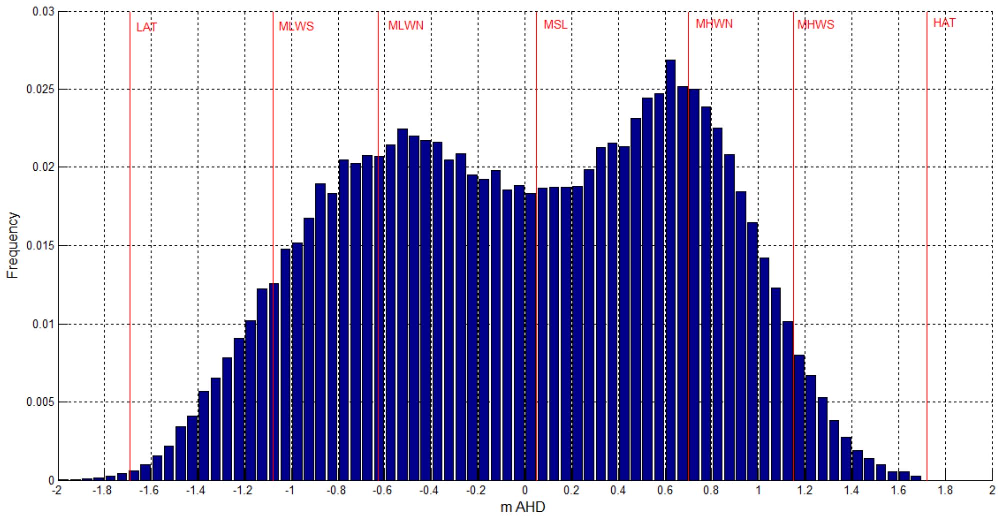

Figure 2-3 Frequency Histogram of Stony Point Water Levels Relative to Key Tidal Plane (Stony Point IDO71004, 1993 – 2012)

Table 2-1 Change in Amplitudes of Main Tidal Constituents between Flinders and Tooradin (Hinwood & Jones, 1979)

Astronomical Tidal Constituent	Amplitude (m)		Difference (cm)
	Flinders	Tooradin	
M2 (semi-diurnal)	0.804	0.987	+0.183
S2 (semi-diurnal)	0.209	0.263	+0.054
K1 (diurnal)	0.227	0.22	-0.05
O1 (diurnal)	0.148	0.142	-0.006

2.2.3 Storm Surges

Storm surge is the common term used to describe variations in coastal water levels that exceed that which can be attributed solely to the astronomical tide. Storm surges are generated by meteorological processes and generally comprise of a combination of the inverse barometric pressure affect, coastally trapped waves and wind setup.

Large storm surges in Western Port are generally associated with the passage of strong cold fronts and associated low pressure systems along the southern margin of the Australian continental land mass.

Figure 2-4 displays an example of the synoptic analysis charts from a strong cold front and associated low pressure system progressing west to east from the Great Australian Bight to Bass Strait. The figures to the left of the synoptic charts show the predicted sea level anomaly at approximately the same time from the results of the Bureau of Meteorology's BLUElink Ocean Model, Analysis and Prediction System. The significant sea level anomaly is associated with a coastally trapped wave generated by interaction of the low pressure system and the coastal waters along the southern continental margin of the Australian landmass. The coastally trapped wave propagates along the southern coastline of Australia and subsequently into Western Port (Figure 2-5).

The annual exceedance probability of extreme storm surges and storm tide (astronomical tide plus storm surge) has previously been estimated for Western Port by the CSIRO and is displayed in Table 2-2. From Table 2-2 it can be seen that storm surges in Western Port can exceed 0.8 m on rare occasions. It should however also be noted that the difference between the 10 % AEP and 1% AEP storm surge is only estimated at approximately 0.08 m. The very tight absolute distribution of storm surge levels is a characteristic of storm surges and an important consideration when evaluating the inundation vulnerability of locations within Western Port.

For the purposes of this project only the 1% AEP storm tide has been considered. However, the increased frequency of inundation associated with more frequent storm tide events may pose a higher hazard in some areas. Recommendations for future assessment of more frequent storm tide events are noted in Section 7.2.

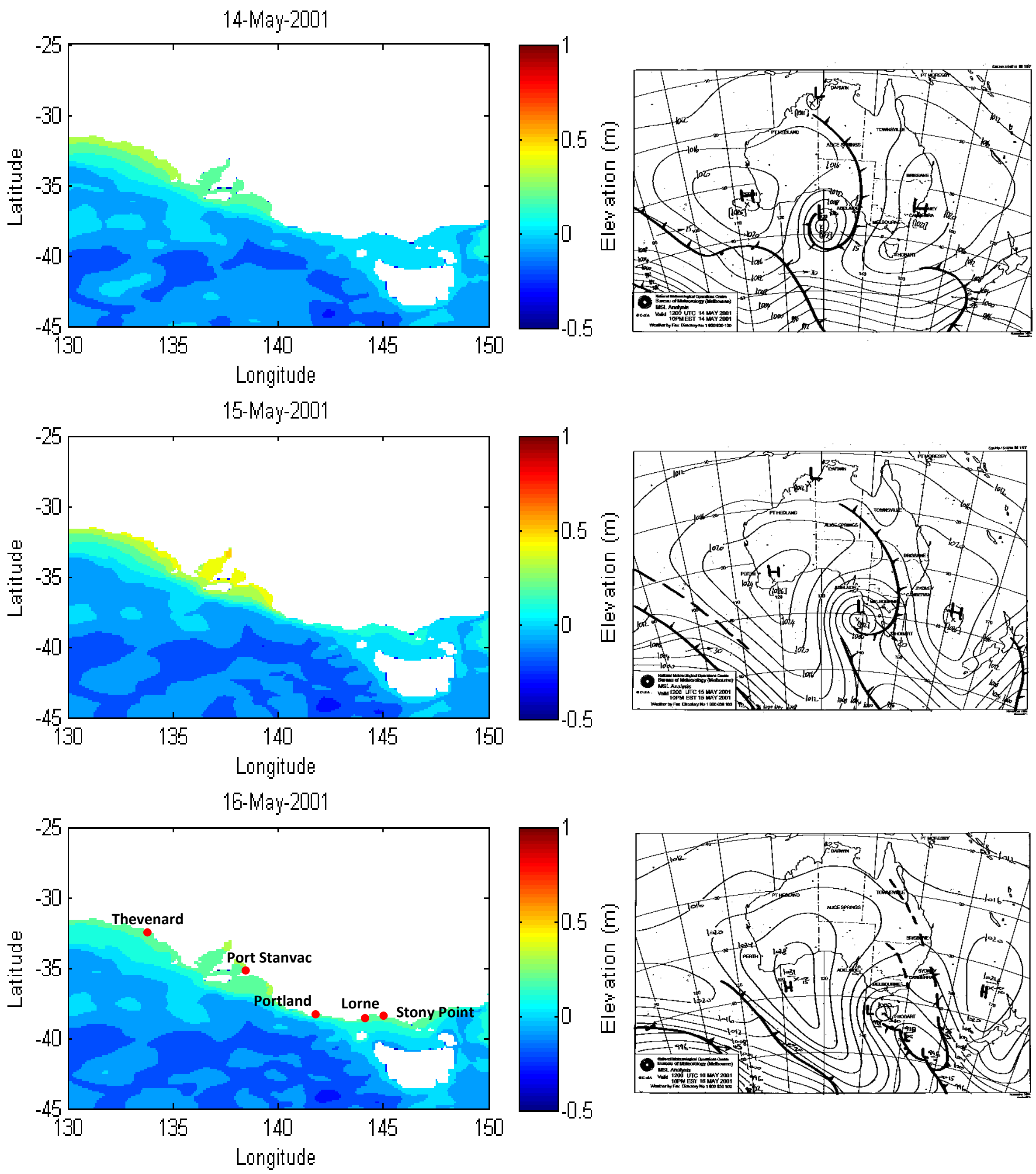

Figure 2-4 **Comparison of Synoptic Analysis Charts and Bluelink Ocean Sea Level Anomaly Model Results and Approximate Location of the Sites shown In Figure 2-5**

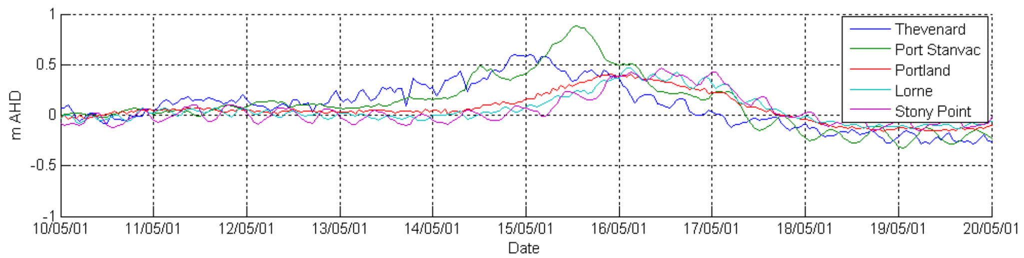

Figure 2-5 **Time Series of Non-tidal Residual Water Levels at Five locations along the South-Eastern Australian Coastline, Showing the Propagation of a Coastally Trapped Wave**

Table 2-2 Estimated Storm Surge and Storm Tide Height Present Day Average Recurrence Intervals at Stony Point (McInnes K L, 2009)

Average Recurrence Interval (years)	Average Exceedance Probability (%)	Storm Surge (m)	Storm Tide (m)
10	**10%**	0.74 ±0.05	1.62 ±0.19
20	**5%**	0.77 ±0.05	1.79 ±0.20
50	**2%**	0.80 ±0.05	1.94 ±0.21
100	**1%**	0.82 ±0.05	2.08 ±0.22

2.2.4 Ocean Swell and Wind Generated Waves

When waves break on a beach they can produce an increase in the mean water level, known as wave set-up. Waves can therefore contribute to peak water levels at the shoreline and thus require consideration as part of the coastal inundation hazard assessment. Two sources of waves occur within Western Port; ocean swell waves and locally generated wind waves.

Ocean Swell Waves

Figure 2-6 shows an exceedance probability curve of significant wave heights and peak wave periods at the Point Nepean wave buoy in Bass Strait, which is situated approximately 20 km north-west of the Western Entrance to Western Port bay, in approximately 30 m of water.

Significant wave heights at the Point Nepean wave buoy are generally 1.36 m or less 50% of the time, and significant wave heights of 2.2 m or larger are only exceeded 10% of the time. Peak wave periods are typically between 5 and 20 seconds, with a median of 12.5 seconds. The ocean swell waves at the Point Nepean wave buoy are also characterised by a narrow directional distribution, with waves predominantly coming from between 180 and 220 degrees.

Ocean swell waves generated in the Southern Ocean propagate through Bass Strait and into Western Port, where they undergo a range of transformations such as refraction, shoaling and breaking. These transformations result in spatial variations of wave conditions along the swell wave influenced shorelines of Western Port.

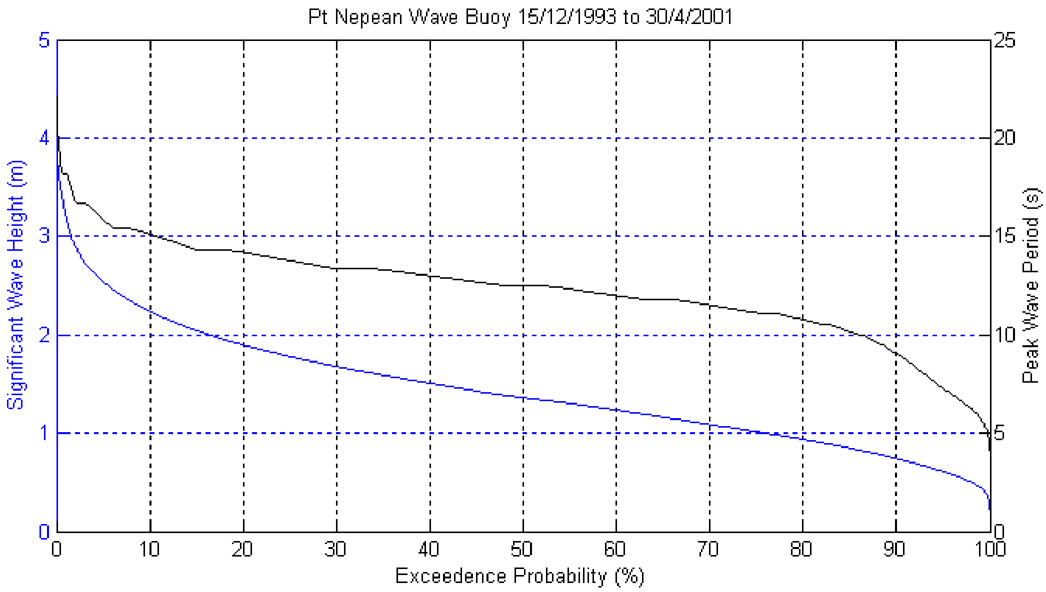

Figure 2-6 Significant Wave Height Exceedance Probabilities for the Point Nepean Wave Buoy (1993 – 2001)

Locally Generated Wind Waves

Ocean swell waves are essentially confined to the shorelines adjacent to the western entrance of Western Port, with locally generated wind waves dominating the wave climate of the remainder of Western Port. Locally generated wind waves are characterised by their generally smaller wave height, shorter wave period and highly variable directional distribution in comparison to ocean swell waves.

Western Port Wave Characteristics

Both ocean swell waves and locally generated wind waves were modelled for one representative year as part of study. Figure 2-7 displays the modelled average significant wave heights for the representative year of 2003 (further described in Appendix A of Report 05 – Erosion Hazards). It highlights the large spatial variation in wave heights within Western Port. The shorelines adjacent to the Western Entrance of Western Port experience average wave heights an order of magnitude larger than the remaining shorelines of Western Port which are only influenced by locally generated wind waves.

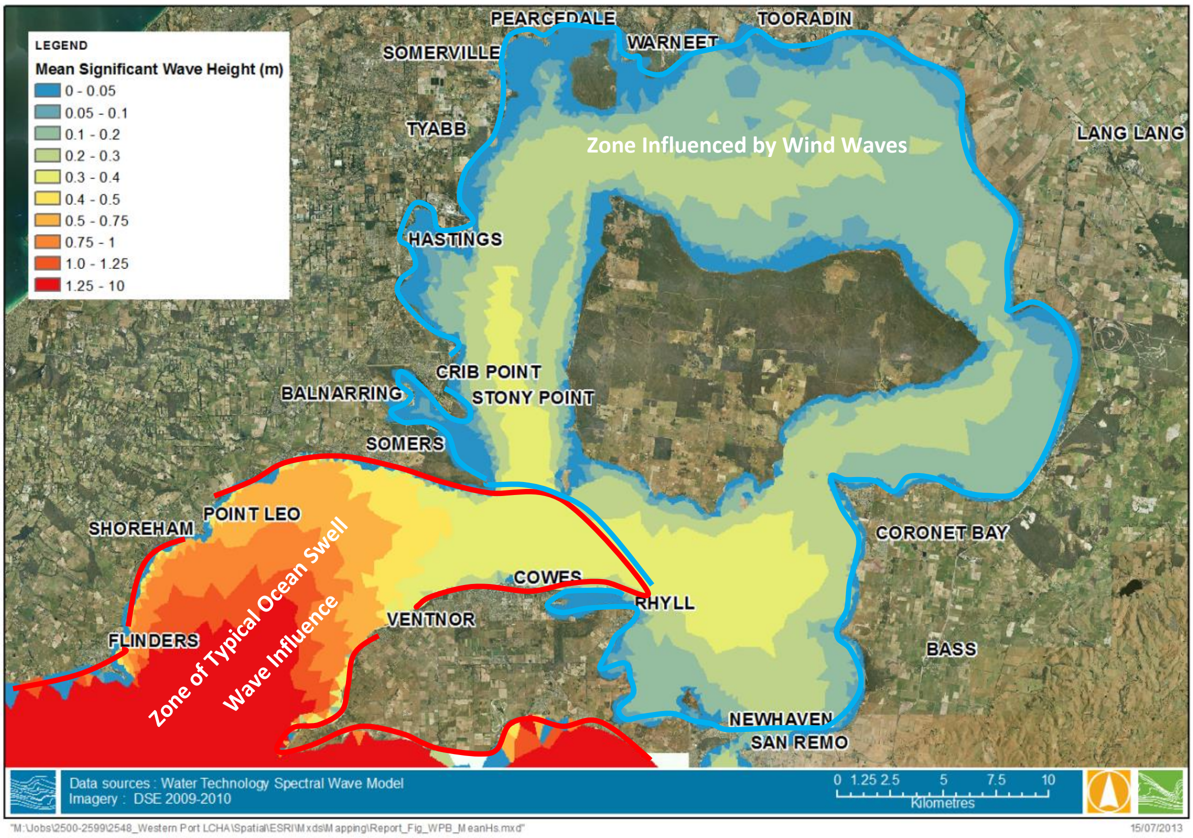

Figure 2-7 **Modelled Average Significant Wave Heights throughout Western Port for the Representative Year, 2003.** *The red line indicates the zone of typical ocean swell influence, and the blue line indicates the area influenced by wind waves.*

2.3 Catchment Inundation Processes

2.3.1 Catchment Description

The total catchment area of Western Port is approximately 3240 km^2. The seven largest sub-catchments, which make up approximately 70% of the catchment, are listed below:

- Bunyip River
- Lang Lang River
- Bass River
- Yallock Creek
- Cardinia Creek
- Deep Creek
- Toomuc Creek

The largest sub-catchment is the Bunyip River, which makes up over 30% of the Western Port catchment area. The Bunyip, Lang Lang and Bass Rivers combined account for over 50% of the Western Port catchment area. The remainder of the catchment is made up of small coastal sub-catchments. The major catchments are shown in Figure 2-8. The larger catchments are generally located to the north and east of the catchment, with the larger rivers entering the bay to the east of Tooradin.

The catchments have been largely cleared for agricultural land use. The northern Bunyip catchment, parts of French Island and the northern Cardinia Creek catchment are the only extensive forested areas within the catchment. The catchment lies on the fringe of Greater Melbourne, with urban development occurring in Pakenham, Beaconsfield, Kooweerup, Lang Lang and other areas.

Prior to European settlement many of the rivers terminated in broad swamps surrounding Western Port, particularly the Koo Wee Rup, Lang Lang and Tooradin areas. To enable agricultural production, these swamps were drained with a network of cut channels which have permanently altered the hydrology of the catchment. Inundation in the low-lying areas has been improved, but outflows to Western Port have increased, with quicker response times to catchment rainfall. The drainage of the low-lying areas depends on the ongoing maintenance of the capacity of the cut drains.

Flood flows in low-lying areas near the coast are typically generated by long duration rainfall in the catchment, with storm duration of 9 to 48 hours. Flooding in the larger catchments tends to be driven by longer duration storms, while the smaller coastal catchments respond to shorter duration rainfall.

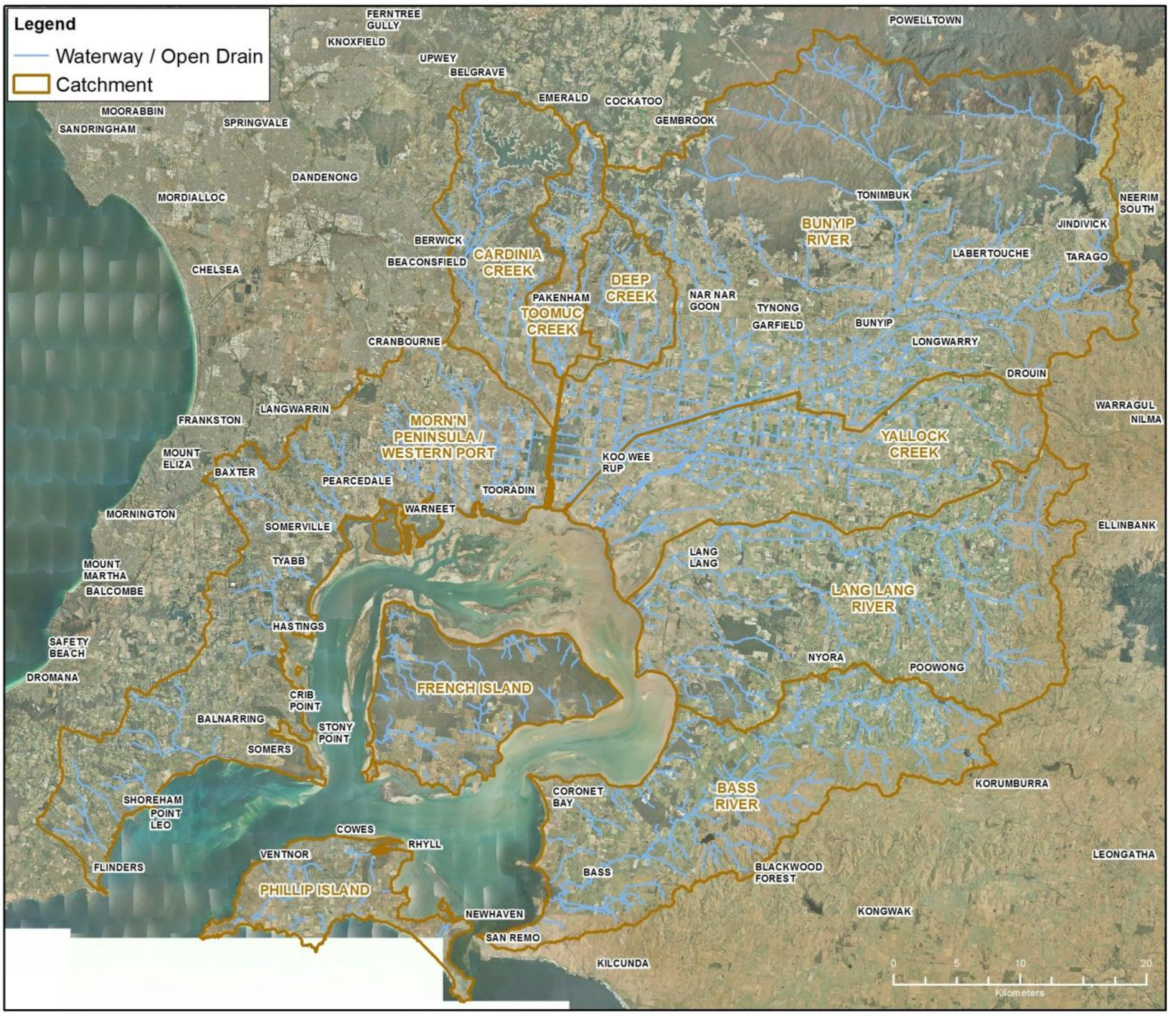

Figure 2-8 **Western Port Catchments**

2.3.2 Catchment Inundation Flows

There has been a long history of flooding within the Western Port catchment; however, there is limited flow data available, especially during large inflow events. Available flow, rainfall and anecdotal data has been reviewed and summarised in Appendix A, along with photography highlighting the impacts of flooding within the catchment.

2.4 Existing Inundation Protection Works and Structures

The following sections provides an overview of the extent of the existing inundation protection works and structures which limit or influence inundation extents and water levels on the low lying plains surrounding Western Port during periods of either extreme coastal water levels and/or high catchment streamflows. These structures are also discussed further in Report 5 (R05) and more detailed consideration and discussion of structures relevant to the Representative Locations (Part B of the study) is presented in Report 6 (R06).

2.4.1 Coastal Levees

Formal and informal embankments, referred to as coastal levees in this study, currently surround approximately 20% of the shoreline of Western Port (excluding French and Phillip Island). The structures predominantly exist along the north/north-eastern shorelines as shown in Figure 2-9 and many are not formally engineered structures. The heights of these embankments are generally around 3.0 m AHD. They have been built primarily to prevent the ingress of saline coastal water onto agricultural land during large storm tide events in Western Port.

The importance of the coastal levees and embankments for controlling coastal inundation in Western Port is demonstrated in Figure 2-10 which shows the low and vulnerable nature of the surrounding coastal plains landward of the levees and shoreline. These levees are generally built to a height sufficient to provide protection from a 1% AEP storm tide under existing mean sea levels. However, breaks in the levees are visible in LiDAR data collected in August 2010 and aerial imagery taken over December-January 2009/2010 south-west of Lang Lang. An example of one these breaks is displayed in the inset map of Figure 2-9, where the neighbouring levees are approximately 2.7 m AHD, the shoreline along the break is 1.8–2 m AHD, which is below the current 1% AEP storm tide (2.20 m) for Grantville (McInnes, *et al.,* 2008).

For the purposes of the inundation hazard assessment, it is assumed that all embankment or coastal levees that are currently in place remain in place at their current extent and configuration. However, given uncertainties as to how these structures will be maintained and/or rebuilt into the future, an additional modelling run was undertaken without the structures present to provide an indication of potential changes to inundation extents. This is described further in Section 6.

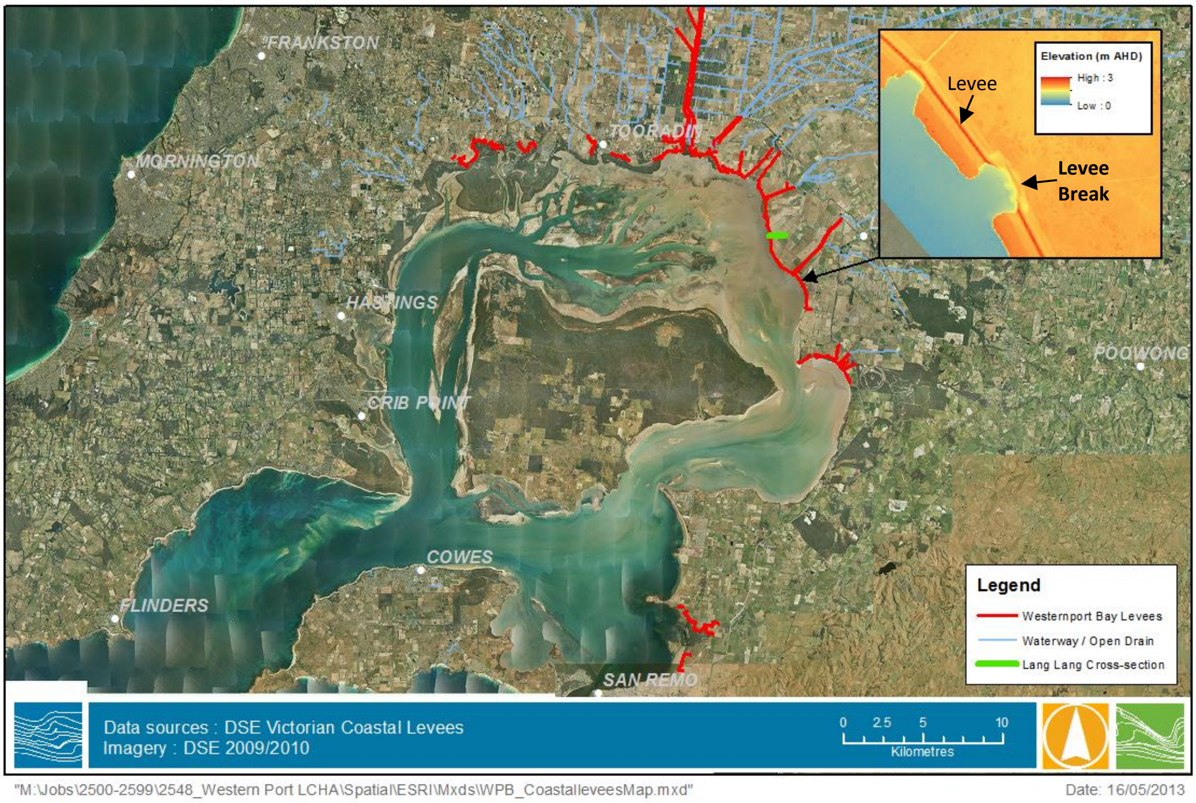

Figure 2-9 **Existing Embankments or Levees Surrounding Western Port Bay and the Location of the Lang Lang Cross-section shown in Figure 2-10 (Inset map shows a break in a levee).**

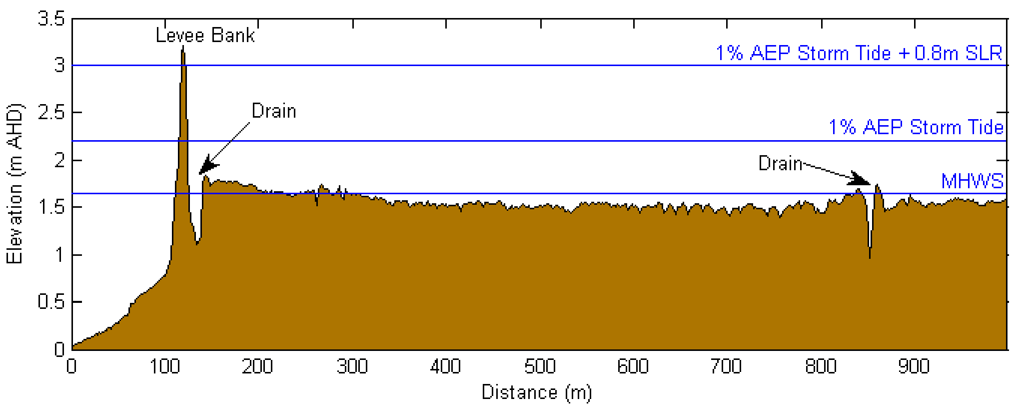

Figure 2-10 **Cross-section Extracted from the Terrestrial LiDAR near Lang Lang (location shown as the green line in Figure 2-9)** *The MHWS level is based on hydrodynamic modelling undertaken as part of this project and the 1% AEP Storm Tide level is from McInnes et. al., (2009).*

2.4.2 Sawtells Inlet Tidal Gates

A tidal gate structure exists at Sawtells Inlet (Tooradin), beneath the South Gippsland Highway (Figure 2-12), which throttles tidal flows upstream of the gates. The tidal gate structure comprises five individual gates, lying across three cells. The left and right cells contain two one way flap gates (2650 x 2700 mm). The central cell has a larger 2650 x 2700 mm one way flap gate above a 70 x 600 mm open sill; the sill allows flows in both directions (Water Technology, 2009).

To the west of the tidal gates a set of dual 900mm diameter culverts run beneath the South Gippsland Highway connecting the upstream and downstream sections of the inlet. These culverts do not have any flap gates on them and allow flows in both directions (Water Technology, 2009).

The one-way tidal gates regulate the volume of tidal water that can flow upstream north of the South Gippsland Highway, along Sawtell Inlet, and therefore limit the extent and elevation of coastal inundation.

Figure 2-11 Sawtells Inlet Tidal Gates beneath the South Gippsland Highway (Photograph: Water Technology, 2009)

2.4.3 South Gippsland Highway

The South Gippsland Highway wraps around the northern and eastern shorelines of Western Port. At several locations along the length of the Highway, the carriageways are comprised of elevated causeways relative to the surrounding land. The hydraulic connectivity through the causeways is limited to a small number of culverts and/or bridge structures in these areas.

The causeways associated with the South Gippsland Highway in these areas therefore exert a significant control on catchment flooding and/or coastal inundation. Figure 2-12 and Figure 2-13 highlight the elevation of the South Gippsland Highway causeway in relation to key coastal water levels.

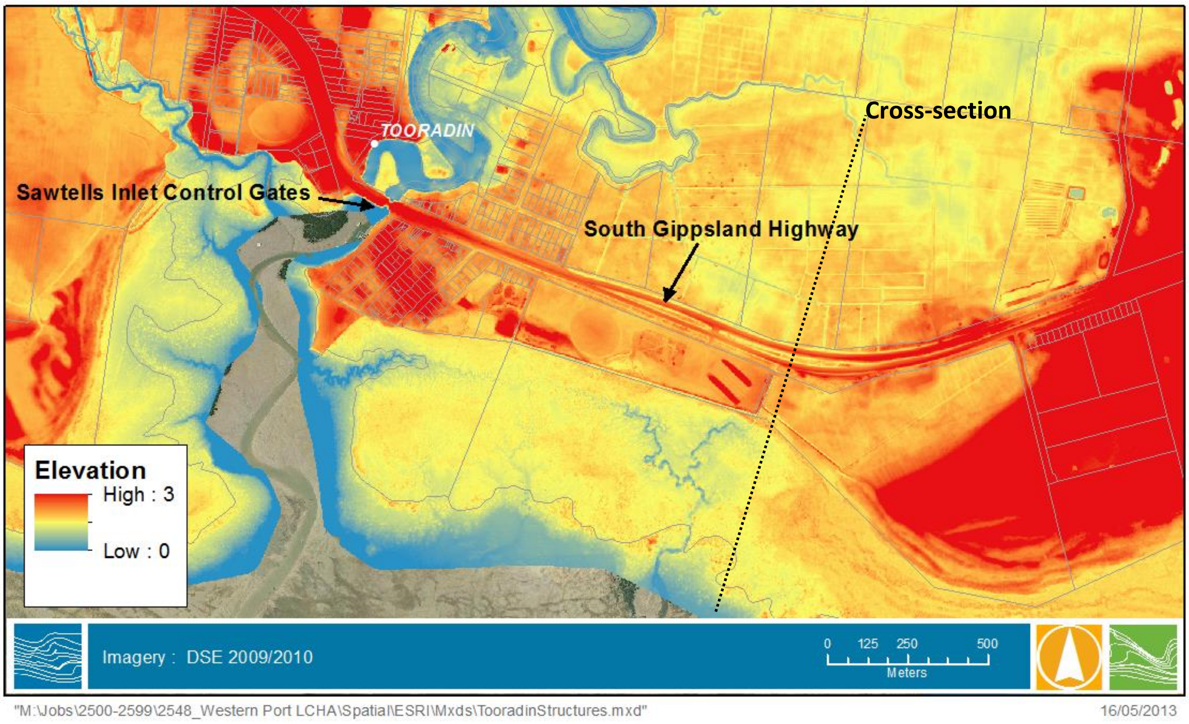

Figure 2-12 Elevations Surrounding Tooradin, Highlighting the Prominence of the South Gippsland Highway

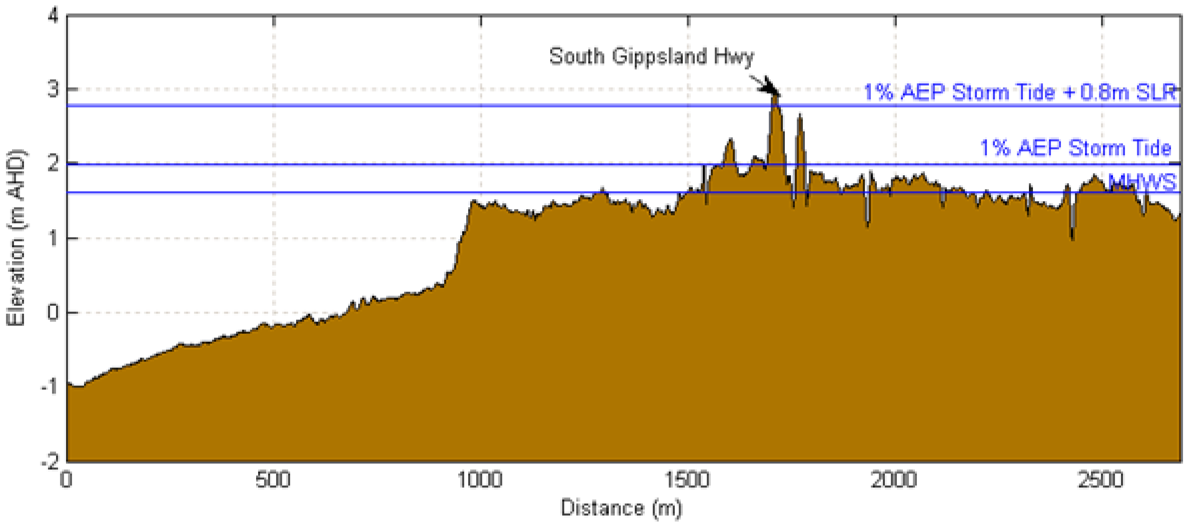

Figure 2-13 Cross-section through the Plain East of Tooradin and over the South Gippsland Highway

3. INUNDATION HAZARD ASSESSMENT METHODOLOGY

3.1 Overview

To integrate the impact that changes in mean sea level could have on the different inundation processes in Western Port, a detailed model of was developed of the study area.

The model comprised coupled hydrodynamic and spectral wave components representing the full extent of the study area, including adjacent floodplain areas up to approximately 4m AHD and the coastal offshore area. This model was calibrated to measured water levels, current velocities, discharge transects, and wave conditions. Calibrating the model consisted of testing the model with physical input data such as, winds, tide, coastal ocean levels and waves, and comparing the modelled water levels to measured data at a number of locations. (Refer to Appendix B of this report, and Appendix A of Report 05 for more details of the model development and calibration).

Available water level, wind, wave and catchment streamflow data was analysed in order to derive model boundary conditions to represent a range of inundation hazard design scenarios. These inundation hazard conditions were then simulated using the coupled hydrodynamic and spectral wave model. The model simulations were then repeated for a range of sea level rise scenarios.

Two additional scenarios were also simulated in the hydrodynamic model to test the sensitivity of the predicted inundation hazard extents, due to uncertainty relating to the following:

- The impact of failure/removal of coastal levees; and
- Changes to catchment flood hydrology due to climate change;

Figure 3-1 displays the conceptual methodology used to assess inundation hazards in Western Port.

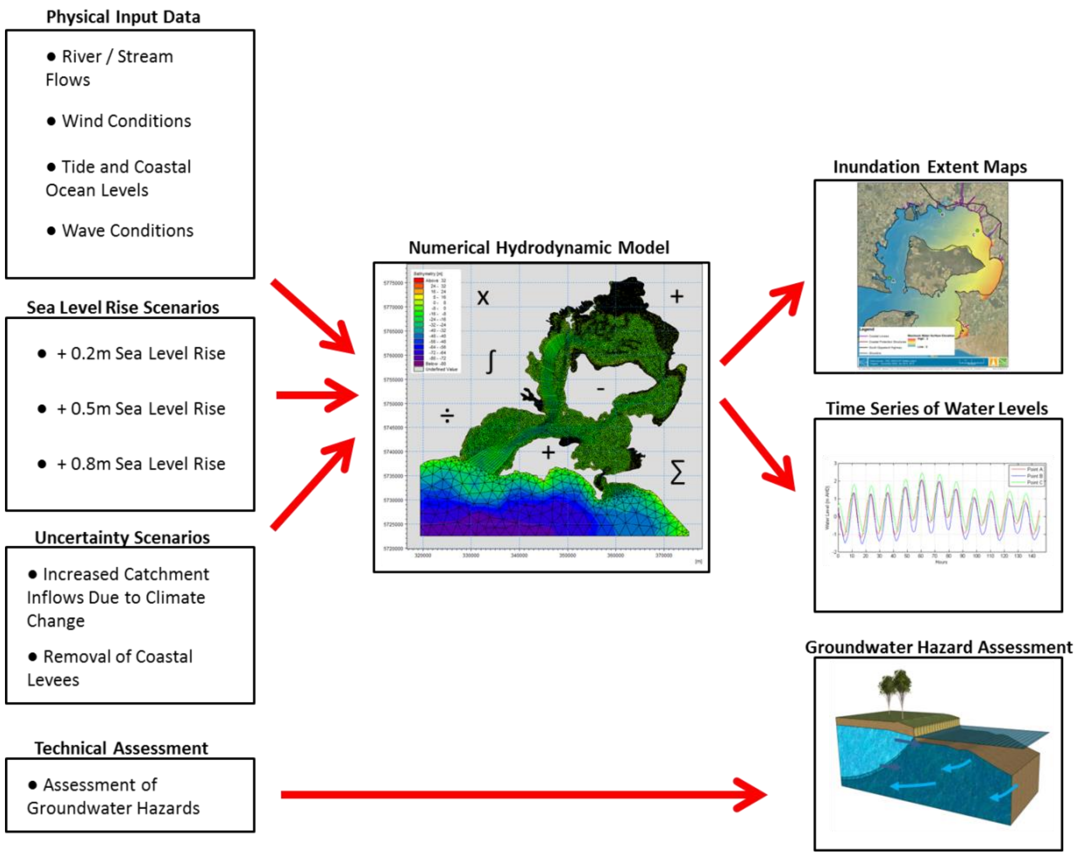

Figure 3-1 **Conceptual Process by which the Inundation Hazards throughout Western Port were assessed**

3.2 Design Scenario Description

The coastal hazard assessment was based on modelling a range of sea level rise, storm tide and catchment flow design events. Table 3-1 displays the combinations of events that were prescribed in the project brief which form the basis of the inundation hazard assessment. The details of the design boundary conditions for each of the events and sea level rise scenarios, summarised in Table 3-1, are described in the following sections.

Table 3-1 **Coastal Inundation Hazard Scenarios**

Present	2040	2070	2100	Combination of events to assess coastal hazards	Scenario
Likely	Virtually Certain			1% AEP storm tide and wave height with 10% AEP catchment flows	Base
Unlikely	About as likely as not	Likely	Virtually Certain	0.2 m of sea level rise plus 1% AEP storm tide and wave height with 10% AEP catchment flows	1
Very Unlikely	Unlikely	About as likely as not	Likely	0.5 m of sea level rise plus 1% AEP storm tide and wave height with 10% AEP catchment flows	2
		Very unlikely	About as likely as not	0.8 m of sea level rise plus 1% AEP storm tide and wave height with 10% AEP catchment flows	3

For each scenario the extent of inundation was assessed separately for the 1% AEP storm tide with each increase in mean sea level and 10% AEP catchment flows with each increase in mean sea level. The resultant inundation extents were then combined within the GIS to produce the overall inundation extent for each scenario.

This approach was adopted following an assessment of the probability of a storm tide and catchment generated flow event occurring at the same time indicated little to no correlation between these two flood generating processes. This is discussed further in Section 3.3.3.

3.3 Physical Inputs

3.3.1 1% AEP Design Storm Tide Conditions

Extreme storm tide conditions in Western Port can potentially be generated from a large range of different tidal, storm surge and wind- wave water level combinations. The following sections summarise the analysis undertaken to develop representative water level and forcing scenarios for the 1% AEP design storm tide conditions.

Design Water Levels

In order to develop a representative storm tide scenario that captured the critical temporal and spatial characteristics of storm tides in Western Port, analysis of available storm surge and meteorological data was undertaken.

To develop an understanding of typical storm surge durations in Western Port that have been observed, the available water level gauge data at Stony Point was analysed. The Stony Point tide gauge was analysed rather than the Tooradin tide gauge, as it provided a significantly longer continuous data set of water levels (1993 to 2011). All storm surge events greater than 0.4 m were extracted from the residual water level records. The duration of each storm surge greater than 0.4 m at Stony Point was then calculated and a histogram of these durations is shown in Figure 3-2. It can be seen that the water levels in the majority of large storm surge events in Western Port persist above 0.4 m for approximately 12-24 hours.

To develop a design storm surge scenario with similar temporal characteristics as is observed from the recorded data at Stony Point, a simple cosine function was used to develop a synthetic storm surge with a maximum height equivalent to the estimated 1% AEP storm surge height of 0.82 m (McInnes, 2009), and a duration above 0.4 m of 30 hours, which represented the 75th percentile of storm surge durations in the above assessment. The design storm surge was then directly added to an astronomical tidal time series derived from the tidal constituents calculated from the Stony Point tide gauge, creating a design 1% AEP storm tide boundary condition, which is shown in Figure 3-3.

As discussed in Section 2.2.3, the difference between the 10 % AEP and 1% AEP storm surge is only estimated at approximately 0.08 m. The very tight absolute distribution of storm surge levels is a characteristic of storm surges and an important consideration when evaluating the inundation vulnerability of locations within Western Port.

For the purposes of this project only the 1% AEP storm tide has been considered. However, with increases in sea level the increased frequency of inundation associated with more frequent storm tide events may pose a higher hazard in some areas. Recommendations for future assessment of more frequent storm tide events are noted in Section 7.2.

The 1% AEP design storm tide condition adopted for this study is representative of conditions for Western Port as a whole and may not provide a ‘worst case’ estimate for all locations along the shoreline. To refine the design storm tide the use of a statistically based approach such as a Monte Carlo analysis may be required.

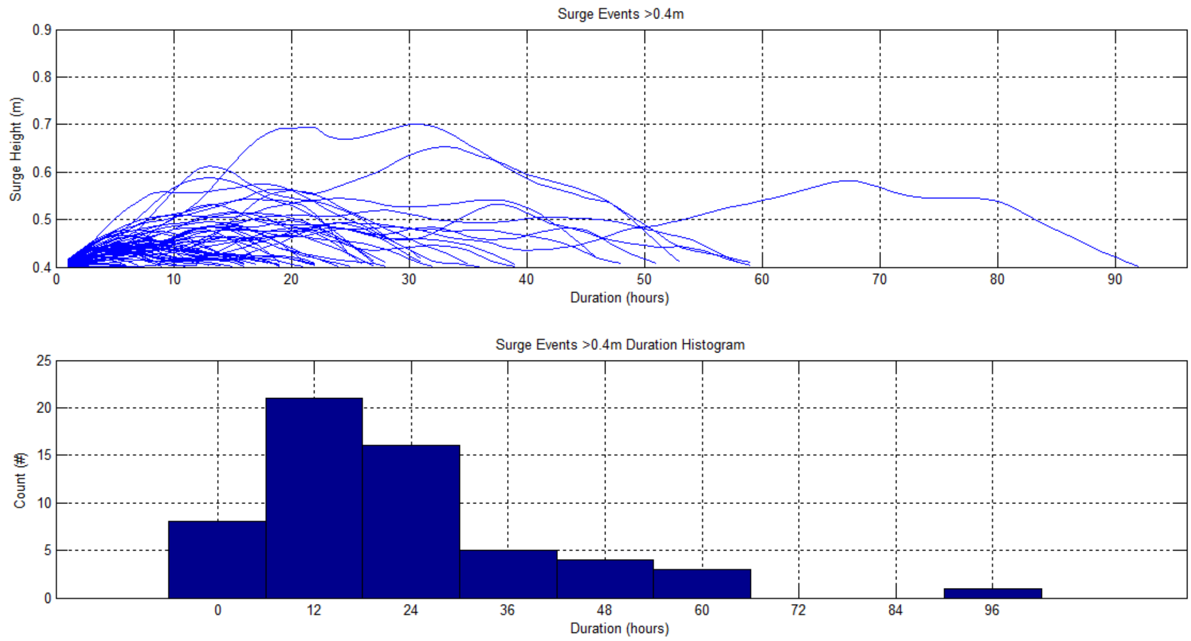

Figure 3-2 Analysis of Storm Surge Durations >0.4m at Stony Point between 1993 and 2012

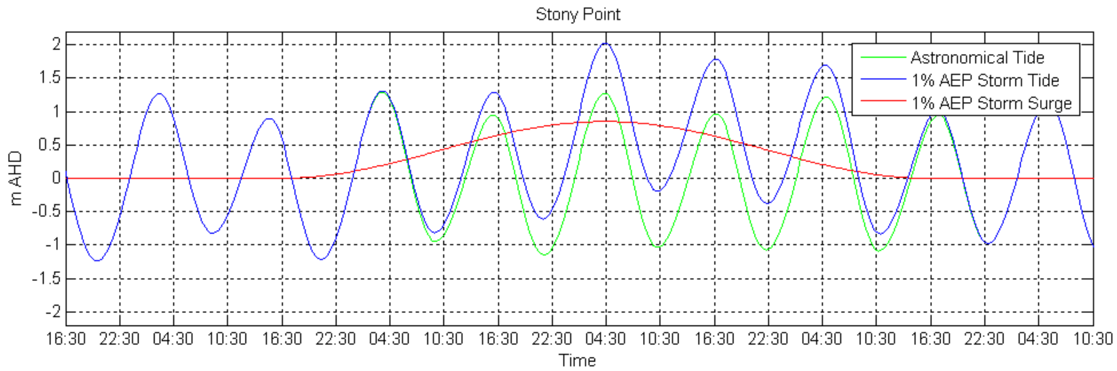

Figure 3-3 Design 1% AEP Storm Surge and Storm Water Levels at Stony Point

Design Wind Conditions

The typical wind and wind-wave conditions that generally accompany large storm surges in Western Port were reviewed by assessing the maximum wind speeds within the periods in which storm surges greater than 0.4 m also occurred. Figure 3-4 displays a comparison of the maximum storm surge height (for storm surges greater than 0.4 m) and maximum wind speed. It can be seen that a weak linear relationship between storm surge magnitude and maximum wind speeds appears to exist for Western Port. Maximum wind speeds of approximately 18 m/s or greater tend to accompany the storm surges greater than 0.6 m.

A similar analysis comparing the weighted mean wind direction and large storm surges was performed and is displayed in Figure 3-5. This shows that large historical storm surges are associated almost exclusively to west to north-westerly wind conditions.

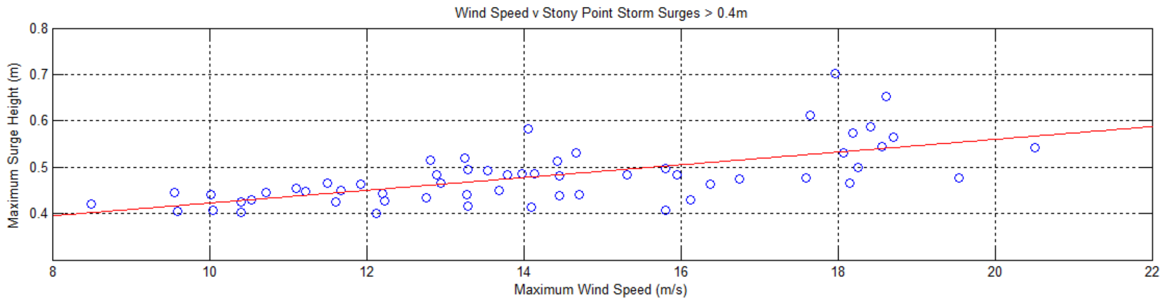

Figure 3-4 **Comparison of Maximum Storm Surge Levels above 0.4m at Stony Point against the Maximum NCEP Wind Speed that Occurred During the Storm Surge**

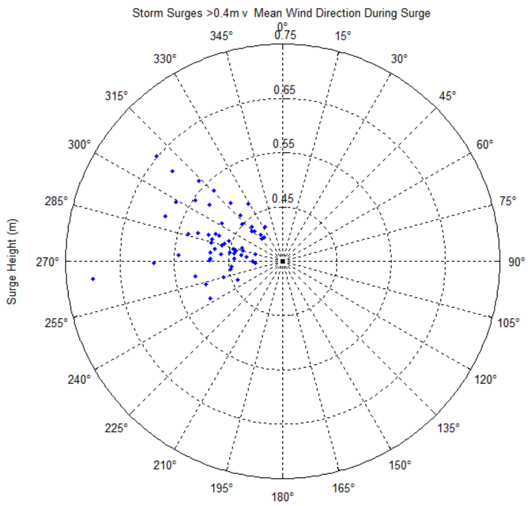

Figure 3-5 **Comparison of Maximum Storm Surge Levels above 0.4m at Stony Point against the Average NCEP Wind Direction that Occurred During the Storm Surge**

For the 1% AEP design wind conditions, the 1% AEP north westerly wind speed for the Melbourne region of 25.1 m/s was adopted for all scenarios. The design wind speed is derived from the Australian Standard, AS 1170.2 – 1989 "SAA Loading Code, Part 2: Wind Loads". This compares well with the measured maximum wind speeds described previously. The design north westerly wind speed of 25.1 m/s was applied in the form of a spatially and temporally constant wind field over the model domain during the storm tide scenarios.

Although this wind direction correlates to the measured storm surge conditions at the Stony Point gauge and is representative of extreme conditions for Western Port as a whole based on historical storm surge events, it may not represent the 'worst case' for some locations along the northern and western shoreline of Western Port. As discussed for storm tide, further refinement of these design event conditions for specific locations could be undertaken using a Monte Carlo type statistical approach.

Design Wave Conditions

An assessment of the potential relationship between large storm surge events at the Stony Point tide gauge and coincident wave heights in Bass Strait was completed in order to determine if there was any apparent correlation. Figure 3-6 displays the results of the assessment and shows that no significant correlation between these two processes.

Therefore the 1% AEP significant wave height for the Point Nepean wave buoy, estimated to be 5.2 m (Water Technology, 2011), was adopted. To simulate wave set-up within Western Port the hydrodynamic model was dynamically coupled to the spectral wave model. The spectral wave model ocean boundary conditions included the significant wave height of 5.2 m, a spectral peak period of 12 s, mean wave direction of 210 degrees, and the design wind conditions of 25.1 m/s.

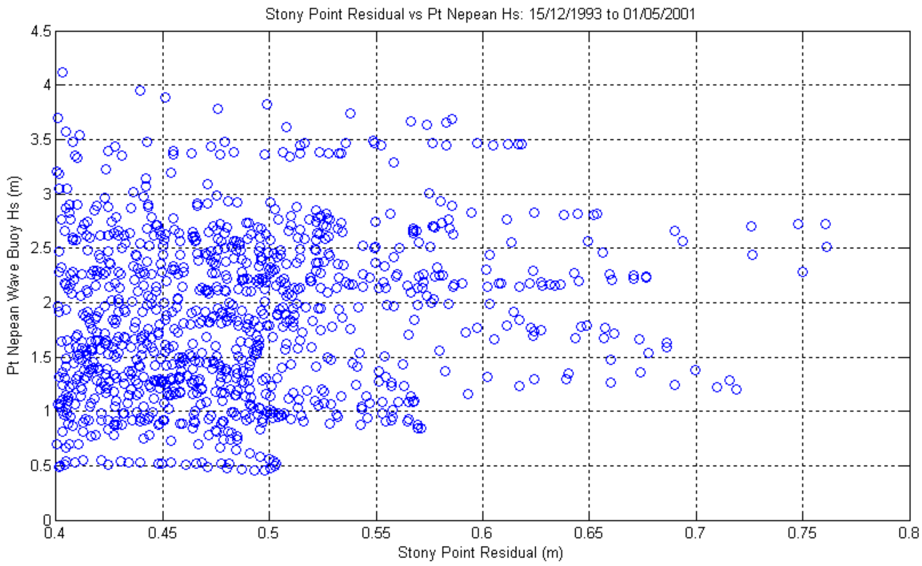

Figure 3-6 Comparison of Maximum Storm Surge Levels above 0.4m at Stony Point against the Pt. Nepean Wave Buoy Significant Wave Heights

3.3.2 10% AEP Design Catchment Inflows

The following section summarises the 10% AEP catchment inflows for the 8 main rivers and drains in Western Port. Detailed descriptions of the catchment hydrology flood frequency analysis, hydrologic modelling and design hydrographs are provided in Appendix C – Catchment Hydrology Analysis.

Design Catchment Inflows

The 10% AEP flow hydrographs were estimated using hydrologic models reconciled to flood frequency analysis and rational method peak flow estimates. A summary of the adopted 10% AEP peak flows for each catchment is provided in Table 3-2. The locations of the inflows adopted for this study are shown in Figure 3-7. The inflow locations were chosen based on existing sea level rise inundation extents provided by DEPI for this project.

Table 3-2 Summary of 10 year ARI Peak Flow Estimates

Location	10% AEP Catchment Flow (m^3/s)
Bunyip River	103
Lang Lang River	196
Bass River	63
Yallock Outfall	145
Cardinia Creek	47
Toomuc and Deep Creek	76
Tooradin Inlet Drains	19.5
Muddy Gates Drain	9.5

Figure 3-7 Location of Catchment Streamflow Inputs to the Hydrodynamic Model

3.3.3 Joint Probability of Storm Tides and Catchment Streamflows

Overview of Joint Probability

The project brief specified a number of scenarios to be analysed in this assessment, which are detailed in Section 3.2. Each scenario incorporates a component of sea level rise along with a 1 % AEP storm tide and 10 % AEP catchment generated flood. If the storm tide and the catchment generated flood are independent events from a statistical viewpoint then the chance of occurring is not changed by the occurrence of the other event. The probability of both events occurring at the same time is therefore the product of their individual probabilities. In this case, assuming the storm tide and catchment flood are independent, the probability of them both occurring at the same time is then 0.1 % (1 in 1000). This represents an extremely rare event.

However, if the storm tide occurs as a result of the same processes that result in a catchment generated flood then the events are termed conditional. The conditional probability of both events occurring is then different than the product of their individual probabilities and more complicated to estimate. However, the implication is particularly important in relation to coastal flooding. For example, if a particular weather event results in high river flows it may or may not also produced storm tide conditions. Assuming independence between the events could underestimate the likely flooding and result in higher risk to the coastal community.

For this project, an initial joint probability analysis was undertaken to establish if there was any dependence between storm tide events and catchment generated floods, and therefore whether it was necessary to apply both conditions within the one model simulation. If they are independent it is appropriate to model the events separately and then combine the results. If they are dependent they should be incorporated together within each simulation.

Comparison between Storm Tide and Catchment Streamflows

The historical relationship between catchment streamflow events and storm tides in Western Port has been reviewed through an analysis of coincident residual coastal water levels and peak flows in the major catchments of Western Port. The objective of the analysis was to develop an understanding of the probability of coincident large storm tide and catchment streamflow conditions in Western Port. As mentioned above, depending on the characteristics of the individual catchments in Western Port, coincident storm tide and catchment floods could result in inundation hazards that are significantly greater than the sum of the individual inundation processes.

To undertake the analysis, the Stony Point coastal residual water level record was compared to available catchment flow data from the most downstream flow gauges on the three largest catchments of Western Port; Bunyip, Lang Lang and Bass Rivers. The residual water level is the recorded water level minus the astronomical tide component. The astronomical tide was taken out of the analysis as it is independent of both storm surge and catchment flows and therefore any dependence relationship would depend only on the storm surge and catchment flow variables.

A partial series of peak flows was constructed for each of the stream flow gauges. Peak flows were deemed to be independent if they were more than seven days apart. For each of the peak flows, the maximum residual water level occurring on the same day was extracted from the Stony Point water level record. The peaks-over-threshold series (as defined in Table 3-3) were plotted against the coincident maximum residual water level at Stony Point and are displayed in Figure 3-8, Figure 3-9 and Figure 3-10 for the Bunyip, Lang Lang and Bass Rivers respectively.

The 10% AEP catchment generated flow magnitude (Table 3-2) can be compared to the water level comparisons shown in the figures.

From these results it can be seen that there are a wide range of coastal water levels that have occurred for a given catchment flow magnitude, with no discernible correlation between the peak of the catchment flood event and the peak of the storm surge. Furthermore, for the peak storm tide to be coincident with the peak of the catchment flow it must occur with the peak of the astronomical tide, further decreasing the likelihood of the events occurring together. However, the available dataset upon which this assessment is based is limited and additional data would be required to more accurately assess any dependence between these events.

Based on this information the joint probability of large storm surges and the associated storm tides occurring coincidently with major catchment streamflow events in Western Port appears extremely low (0.1 % AEP). This is in keeping with a general understanding of the synoptic weather setups that generate prolonged heavy rainfall across the study area compared to those required to generate large coastal trapped wave events in Western Port. In addition, the very significant differences in catchment areas and time of concentration between the different catchments in Western Port is such that the likelihood of all large catchment streamflows being coincident with the peak of a storm tide in Western Port is considered very low probability.

For these reasons, this assessment of the inundation hazards in Western Port assumes that the inundation hazards resulting from extreme storm tide events and catchment streamflows occur independently. The inundation extents generated by each of the two inundation processes have therefore been combined to provide an overall estimate of the inundation hazard extent for each scenario.

The impact of more frequent storm tide events in prolonging or exacerbating catchment generated flooding has not been assessed in the current study. This could be examined further through additional sensitivity analyses.

Table 3-3 Summary of Flow Gauges and Peaks-Over-Threshold Flow Series at Each Gauge

Gauge number and name	Coincident years with tide gauge	Flow threshold (m^3/s)	No. peaks over threshold
228213 Bunyip River at Iona	1993-2004 (12)	20	31
228209 Lang Lang @ Hamiltons Bridge	1993-2000 (8)	20	20
227231 Bass @ Glen Forbes South	1993-2012 (20)	20	51

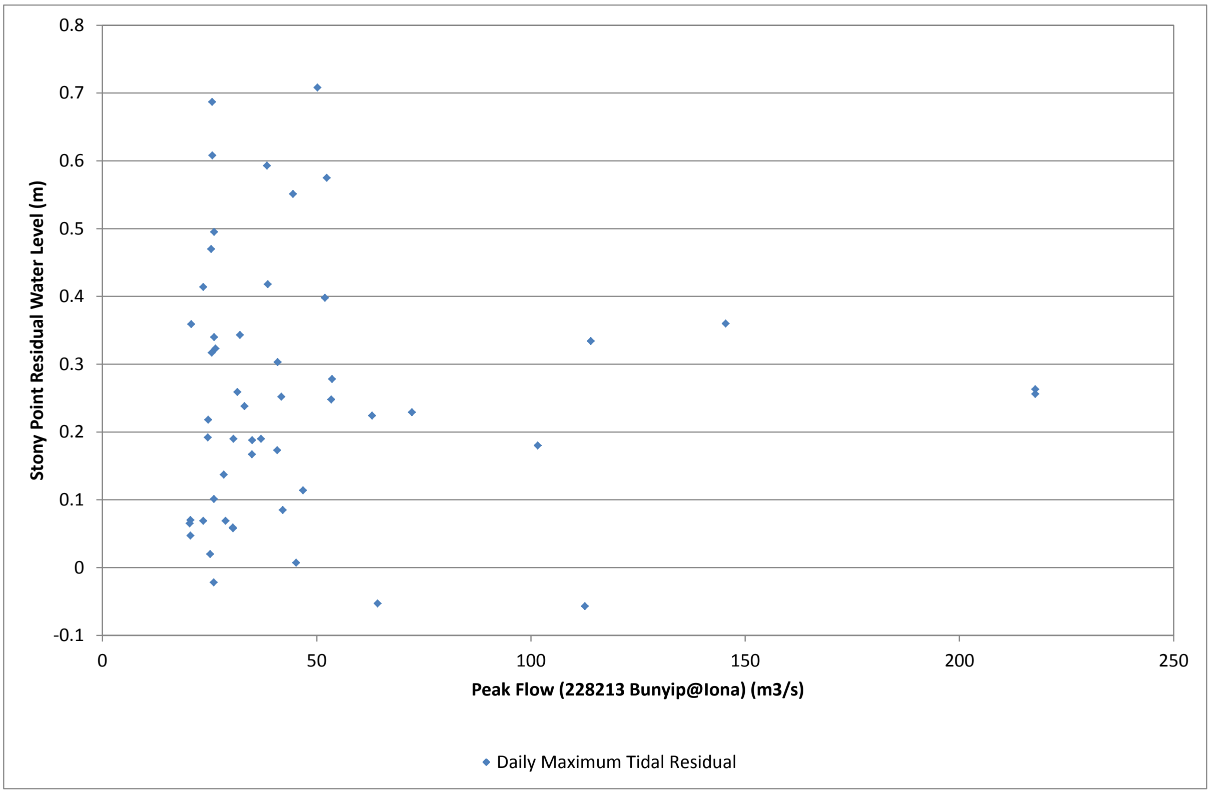

Figure 3-8 **Relationship between Coastal Water Level Residual and Peak Flows over 20 m^3/s for Bunyip River (10% AEP flow = 103 m^3/s)**

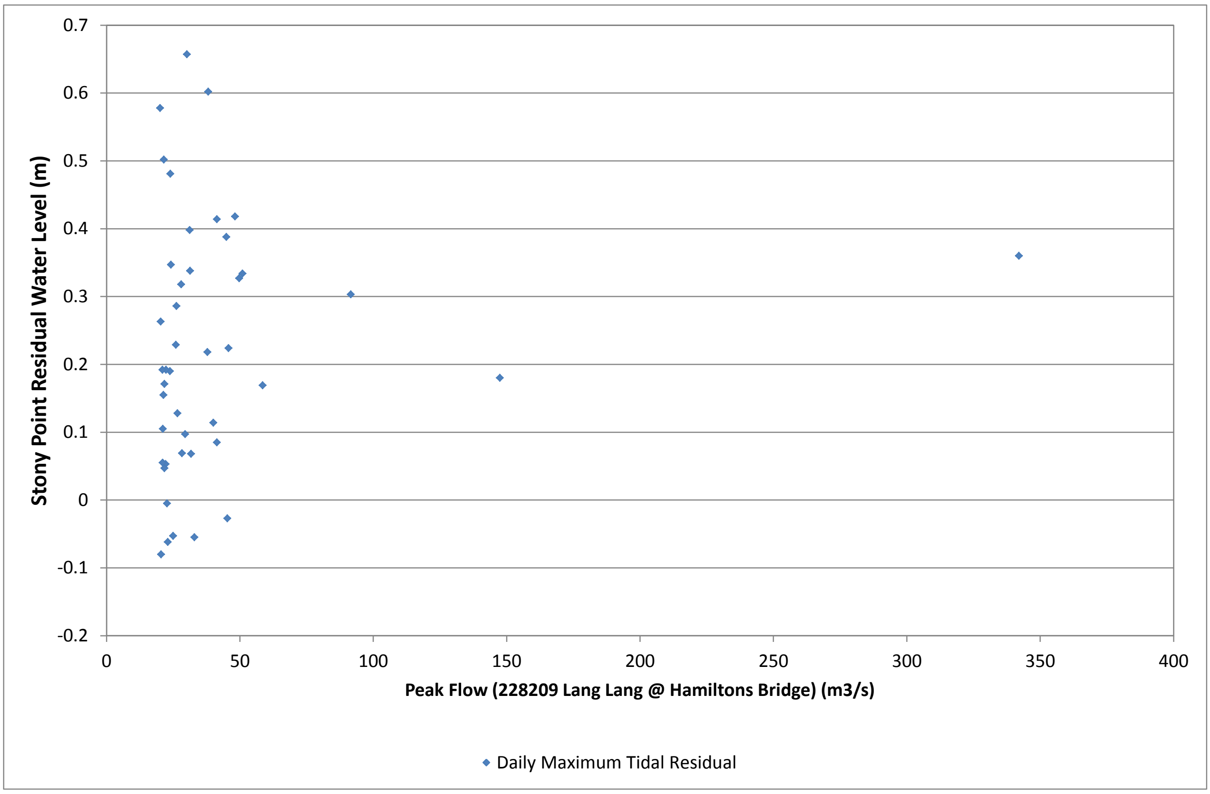

Figure 3-9 **Relationship between Coastal Water Level Residual and Peak Flows over 20 m^3/s for Lang Lang River (10% AEP flow = 196 m^3/s)**

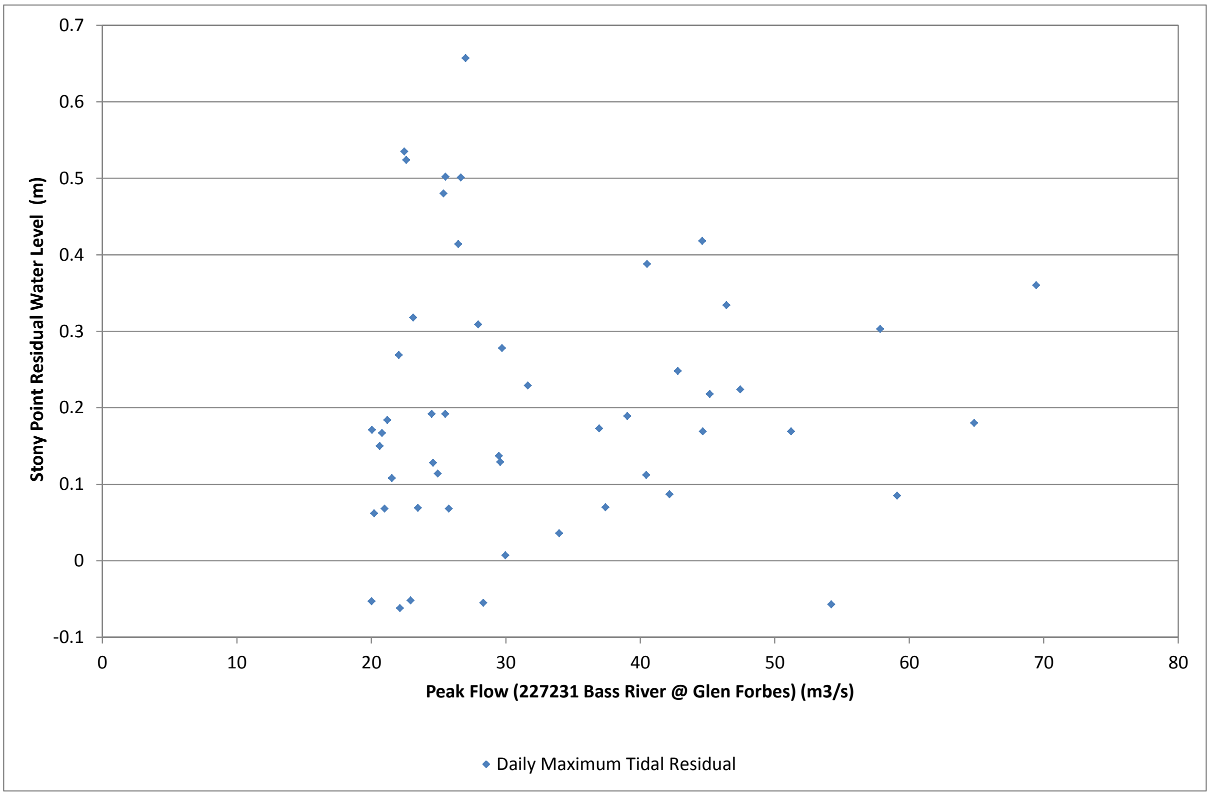

Figure 3-10 Relationship between Coastal Water Level Residual and Peak Flows over 20 m^3/s for Bass River (10% AEP flow = 63 m^3/s)

3.4 Sources of Uncertainty

The analysis and modelling of the impact of sea level rise and climate change on inundation hazards within the Western Port study area has a number of potential sources of uncertainty. These sources of uncertainty require evaluation to determine the sensitivity they may have on the study outcomes.

From the literature review and the parallel assessments of the coastal erosion hazards, the assessment of the inundation hazards in Western Port due to sea level rise and climate change is considered potentially sensitive to the following major sources of uncertainty:

- Ownership and future management and adaptation of coastal levees and embankments;
- Significant changes to catchment flood hydrology due to climate change and associated increases in rainfall intensity.

3.5 Outputs

In addition to the detailed project reports, a project GIS dataset has been provided as an output from the project. This contains:

- Inundation extent polygons for each of the modelled scenarios; and
- Water surface elevation contours at 0.1m intervals for each of the modelled scenarios;

The hydrodynamic model used as part of this project to assess inundation hazards was based upon a combined triangular and rectangular mesh. Therefore the model outputs were post-processed in ArcGIS to transform the model result files into a uniform rectangular grid.

The inundation extent was defined by intersecting the modelled water surface elevations with a 5 m grid of the terrain data (using the terrestrial LiDAR). Following the intersection, all grid cells are converted to an extent polygon. The extent is smoothed to remove the sharp edges of the grid cells for cartographic / presentation purposes.

The water surface elevations were contoured at 0.1 m intervals. The automatic contouring procedures can create erroneous flood elevation contours, therefore manual refinement of the flood contours was undertaken to improve their interpretability.

The inundation extent polygons are considered accurate to a “group of properties” scale rather than at the scale of individual properties, similar to that produced for a rural township flood investigation.

4. INUNDATION HAZARD ASSESSMENT RESULTS

4.1 Overview

The following sections present the results of the inundation hazard modelling of the storm tide and catchment streamflow design events and sea level rise scenarios.

The shoreline displayed on the inundation extent maps is a study specific shoreline that has been developed by mapping the modelled elevations and extent of the MHWS tidal plane in Western Port. The MHWS tidal plane was chosen to define the shoreline in the inundation hazard mapping as it provides a common representative shoreline for both the inundation and erosion hazard components of this study.

For the purposes of the scenario assessment, it was assumed that all embankment or coastal levees that are currently in place remain in place at their current extent and configuration. Therefore the inundation extent mapped outputs include this assumption. However, given uncertainties as to how these structures will be maintained and/or rebuilt into the future, an additional modelling run was undertaken without the structures present to provide an indication of potential changes to inundation extents. This is described further in Section 6.

Other coastal structures within the study area were typically associated with major marine infrastructure such as ports, harbours, or boat ramps. The elevation and configuration of these structures was also not altered in the scenario assessments, in order to provide an indication of their vulnerability to inundation in the future.

4.2 1% AEP Storm Tide Simulations

The impact of the representative 1% AEP storm tide, wind and wave conditions on coastal inundation was assessed by simulating the representative 1% AEP storm tide, wind and wave conditions under each of the different sea level rise scenarios. The resulting inundation extents are displayed in **Error! Reference source not found.**, and descriptions of the inundation at key locations round Western Port are given in Section 4.4 , Table 4-2.

The conservative north-westerly design wind scenario creates a significant water level gradient from north-west to south east across Western Port due to the combined effects of wind and wave setup. Along the eastern shorelines of Western Port, the resulting water levels were in general approximately 0.3 to 0.6 m higher than the western shorelines in these scenarios. The gradient in water level across Western Port is evident in the time series of water levels displayed in Figure 4-1 and the spatial variation in maximum water surface elevation is shown in Figure 4-2. The conservative north-westerly design wind-wave scenario effectively results in relatively conservative (high) storm tide inundation extents on the eastern shorelines of Western Port and slightly less conservative (lower) storm tide inundation extents on the western shorelines of Western Port.

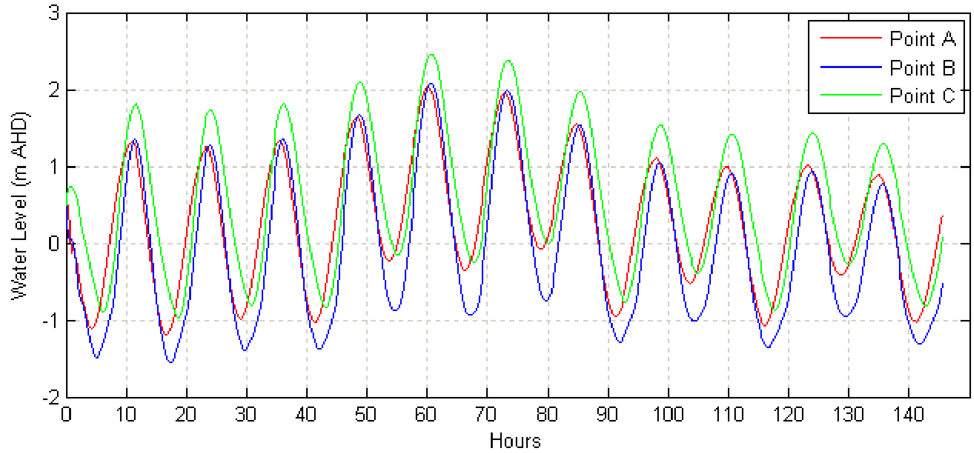

Figure 4-1 Time series of Modelled Water Levels from the 1% AEP Storm Tide Scenario under the Existing Mean Sea Level

The Locations of Points A, B and C are displayed in Figure 4-2, and Were Extracted Offshore of Stony Point, Tooradin and Lang Lang Respectively.

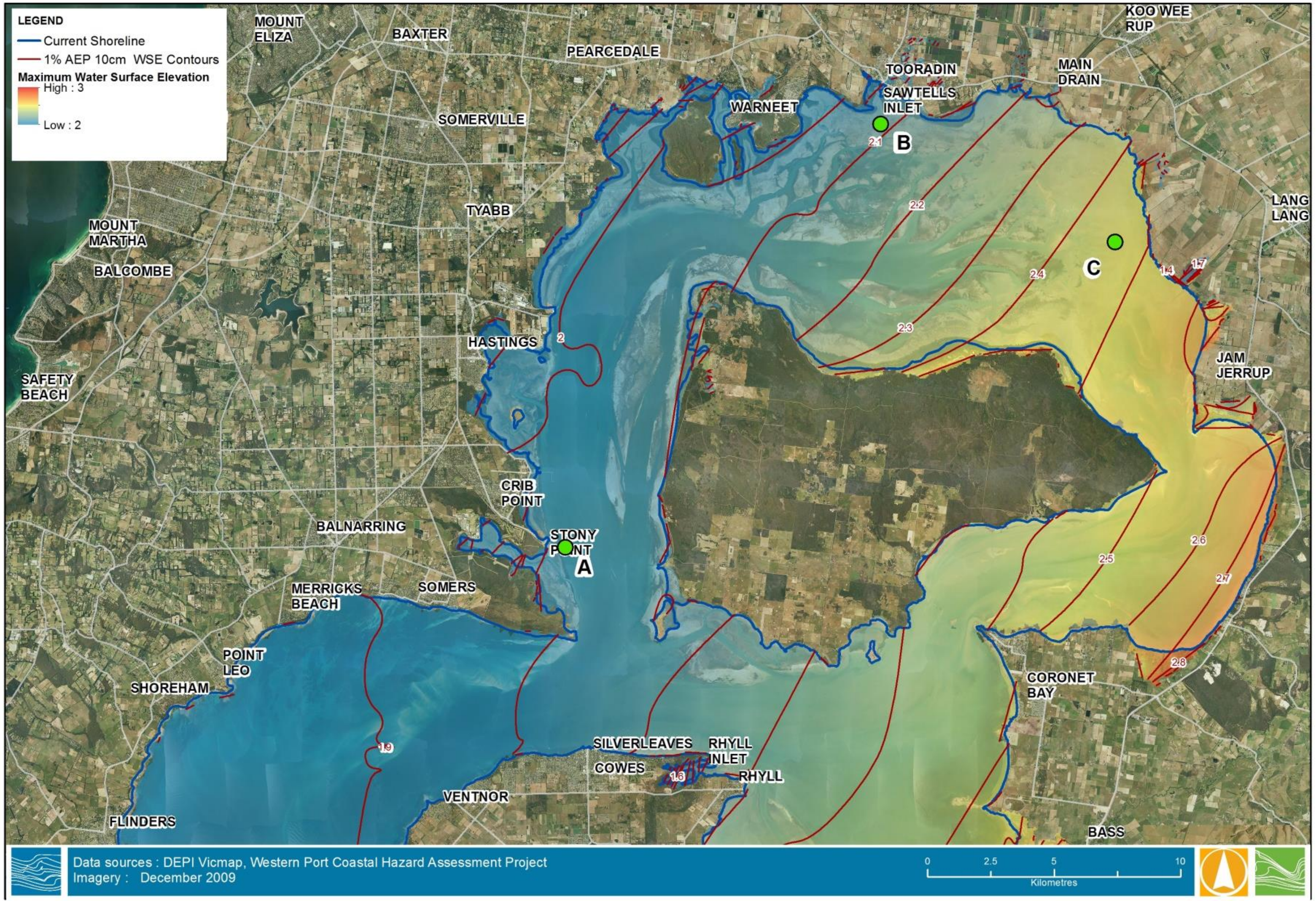

Figure 4-2 **Gradient in Maximum Water Surface Elevation across Western Port under the 1% AEP Storm Tide Scenario under Existing Mean Sea Level. Contour Values Represent the Maximum Water Surface Elevation during the Simulation, Referenced to AHD.**
Points A, B and C Indicate the Locations from which the time series in Figure 4-1 were extracted

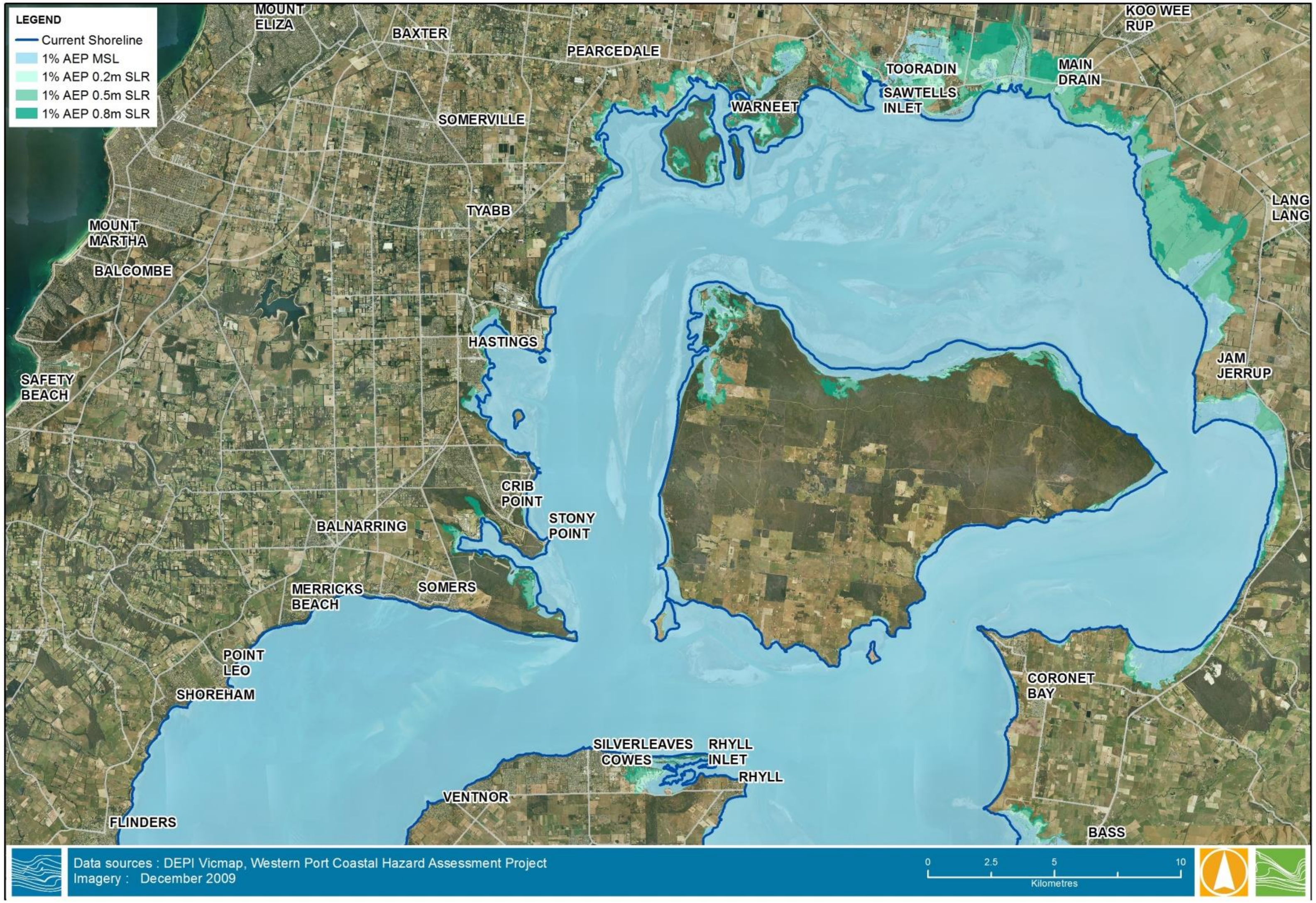

Figure 4-3 **Modelled 1% AEP Storm Tide and Wave Condition Inundation Extent under the Existing, +0.2m, +0.5m and +0.8m Sea Level Rise Scenarios**

4.3 10% AEP Catchment Streamflow Simulations

The impact of catchment streamflows on inundation extents have been assessed by simulating the 10% AEP catchment streamflow events for all catchments under the different sea level rise conditions, and are displayed in Figure 4-4. These simulations were run with the design hydrographs described in Appendix B in conjunction with a typical spring tidal cycle. A summary of inundation extent features around the main section of Western Port are given in Table 4-2.

As noted in Section 3.3.3, storm surges and catchment generated floods are independent events in Western Port and have therefore been modelled as independent processes. The resultant inundation extents from both the storm tide and stream flow simulations are combined in the project GIS to give an envelope of maximum water levels and inundation extents across the study area.

4.4 Descriptions of Inundation Hazard Extents

Table 4-1 lists the total inundated area in square metres for the base case and each of the sea level rise scenarios for the 1% storm tide conditions. It can be seen that the inundated area more than doubles under Scenario 3 (+0.8m SLR) for the storm tide relative to existing sea levels.

A general description of the results of the inundation modelling is provided in Table 4-2 and Figure 4-5, Figure 4-6 and Figure 4-7.

Table 4-1 Areas of inundation extent above MHWS for the base case and three sea level rise scenarios under the 1% AEP storm tide

Scenario	1% AEP storm tide wind and wave conditions	
	Area Inundated (m^2)	Relative Increase from Base Case (%)
Base (Existing MSL)	43045000	-
1 (+0.2m SLR)	54175000	126
2 (+0.5m SLR)	76612500	178
3 (+0.8m SLR)	95890000	223

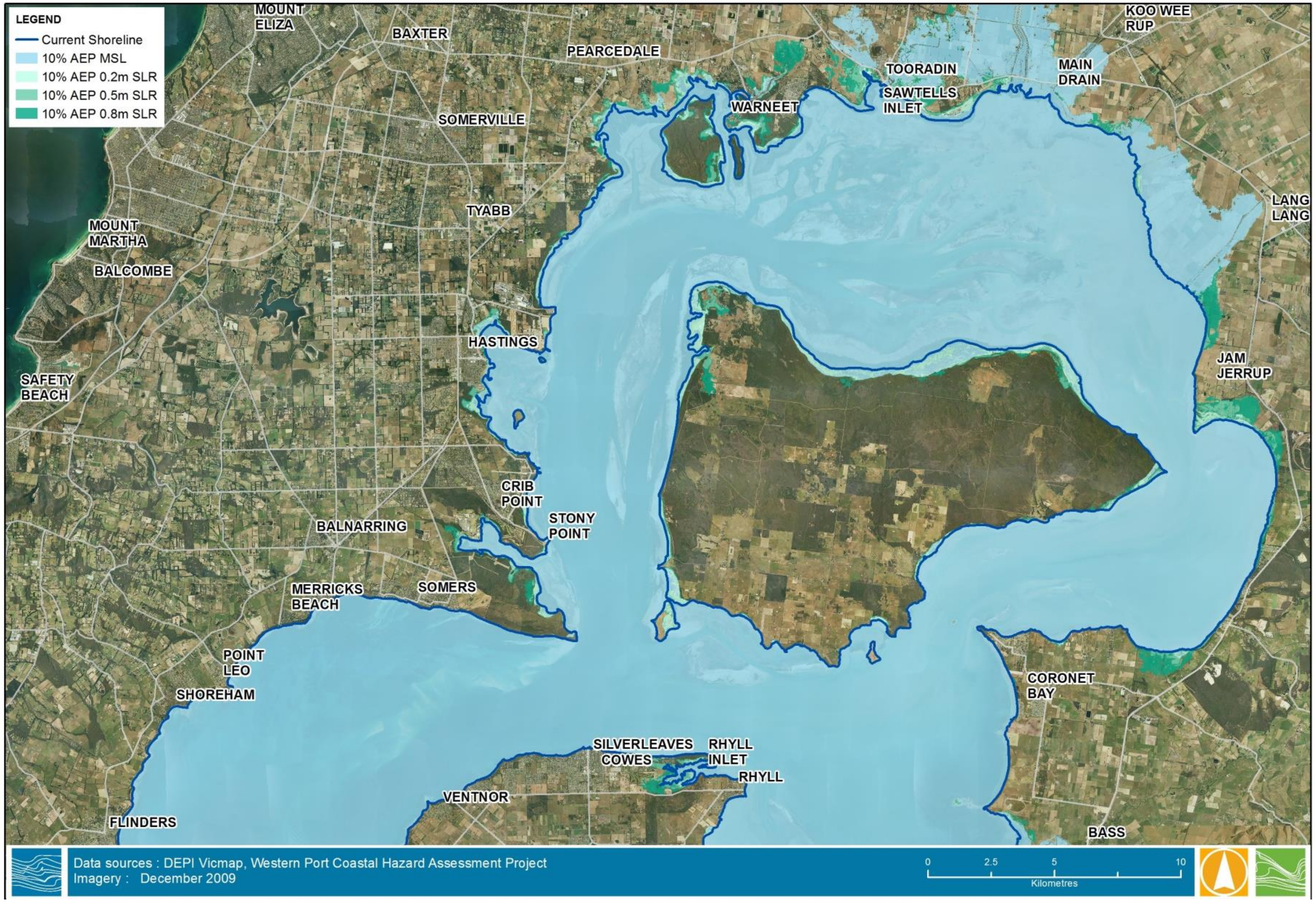

Figure 4-4 **Modelled 10% AEP Catchment Streamflow Inundation Extent under the Existing, +0.2m, +0.5m and +0.8m Sea Level Rise Scenarios**

Table 4-2 Comparison of Inundation Extents under Existing Mean Sea Level and Sea Level Rise Scenarios

Scenario	Locations						
	Flinders to Sandy Point	**Sandy Point to Warneet**	**Warneet to Main Drain**	**Main Drain to Jam Jerrup**	**Jam Jerrup to Corinella Point**	**Bass River and Phillip Island**	**French Island**
Existing Mean Sea Level, 1% Storm Tide & Waves (Base Case)	No significant inundation due to geomorphology of this section of shoreline	• Inundation of Hanns Inlet and coastal wetlands and low lying areas. • Flooding limited by coastal structures	• Inundation of Sawtells inlet upstream of South Gippsland Highway • Inundation of low lying areas to the west of Tooradin Airfield	• Inundation contained by coastal levees along drains and waterways.	• Inundation of low lying areas, particularly north of Grantville and at Queensferry • Only limited protection provide existing coastal levees	• Inundation of the Bass River delta, with inundation limited in some locations due to existing coastal levees. • Minor increase in flood extent at Rhyll inlet.	• Inundation of predominantly the northern shoreline and low lying wetland areas
+0.2m Mean Sea Level, 1% Storm Tide & Waves (Scenario 1)	As above	• Minor increase in the inundation extent in low lying areas.	• Increased inundation upstream of South Gippsland Highway, with flow paths linking Sawtells inlet to Cardinia Creek and Main Drain.	• Inundation contained by coastal levees along drains and waterways but with limited overtopping of levees at some locations, particular along the Lang Lang shoreline	• Minor increase in the inundation extent in low lying areas.	• Minor increase in the inundation extent in low lying areas at Bass River. • Increased inundation at Rhyll inlet particularly towards Silverleaves and the Cowes-Rhyll Road.	• Minor increase in the inundation extent in low lying areas.
+0.5m Mean Sea Level, 1% Storm Tide & Waves (Scenario 2)	As above	• Further increase in flood extent in low lying areas, particularly at Hastings, Warneet, and cannons Creek.	• Inundation of Tooradin and areas between Sawtells inlet and Cardinia Creek. • Overtopping of levees more widespread.	• Increasing overtopping of levees along drains and waterways. • Increased floodplain inundation.	• Further increase in flood extent towards Pioneer Bay and Grantville.	• Minor increase in the inundation extent in low lying areas at Bass River. • Inundation of Silverleaves and Cowes-Rhyll Road	• Further increase in flood extent along northern shore in particular

Scenario	Locations						
	Flinders to Sandy Point	Sandy Point to Warneet	Warneet to Main Drain	Main Drain to Jam Jerrup	Jam Jerrup to Corinella Point	Bass River and Phillip Island	French Island
			• Potential overtopping of South Gippsland Highway. • Tooradin Airport isolated.				
+0.8m Mean Sea Level, 1% Storm Tide & Waves (Scenario 3)	As above	• Further increase in flood extent in low lying areas, particularly at Hastings, Warneet, and cannons Creek. • Many existing coastal structures outflanked or overtopped. • Flooding upstream in Rutherford Creek towards the Baxter-Tooradin Road.	• Increased flooding at Tooradin and overtopping of South Gippsland Highway	• Increasing overtopping of levees along drains and waterways. • Extensive inundation of floodplain areas	• Further increase in flood extent towards Pioneer Bay and Grantville. • Flooding of Bass Highway.	• Increase in the inundation extent in low lying areas at Bass River. • Further increase in Inundation of Silverleaves and Cowes-Rhyll Road	• Further increase in flood extent along northern shore in particular
10% Catchment Inflows & +0.2m Mean Sea Level	No significant inundation as there are no major sources of catchment inflow along this section of coastline	• Similar to Scenario 3 but with slightly reduced inundation extent	• Similar to Scenario 3 but with increased inundation extent north of Manks Road due to overtopping of levees along the Drains and Waterways	• Similar to Scenario 3 but with increased inundation along the lower Lang Lang River, extending to the South Gippsland Highway	• Reduced flood extent compared to storm tide scenario. Inundation predominately due to increased mean sea level rather than catchment inflows	• Reduced flood extent compared to storm tide scenario • Inundation in Rhyll Inlet predominantly due to increased mean sea level rather than catchment	• Changes in inundation extent predominantly due to increased sea level, rather than catchment inflows

Scenario	Locations						
	Flinders to Sandy Point	**Sandy Point to Warneet**	**Warneet to Main Drain**	**Main Drain to Jam Jerrup**	**Jam Jerrup to Corinella Point**	**Bass River and Phillip Island**	**French Island**
						inflows	
10% Catchment Inflows & +0.5m Mean Sea Level	As above	• As above	• As above, but with increased inundation upstream of Sawtells inlet due to increased mean sea levels	• As above	• As above	• As above	• As above
10% Catchment Inflows & +0.8m Mean Sea Level	As above	• As above	• As above, but with further increased inundation upstream of Sawtells inlet due to increased mean sea levels	• As above	• As above	• As above	• As above

Figure 4-5 Broad Scale Inundation Mapping Results – Map 1

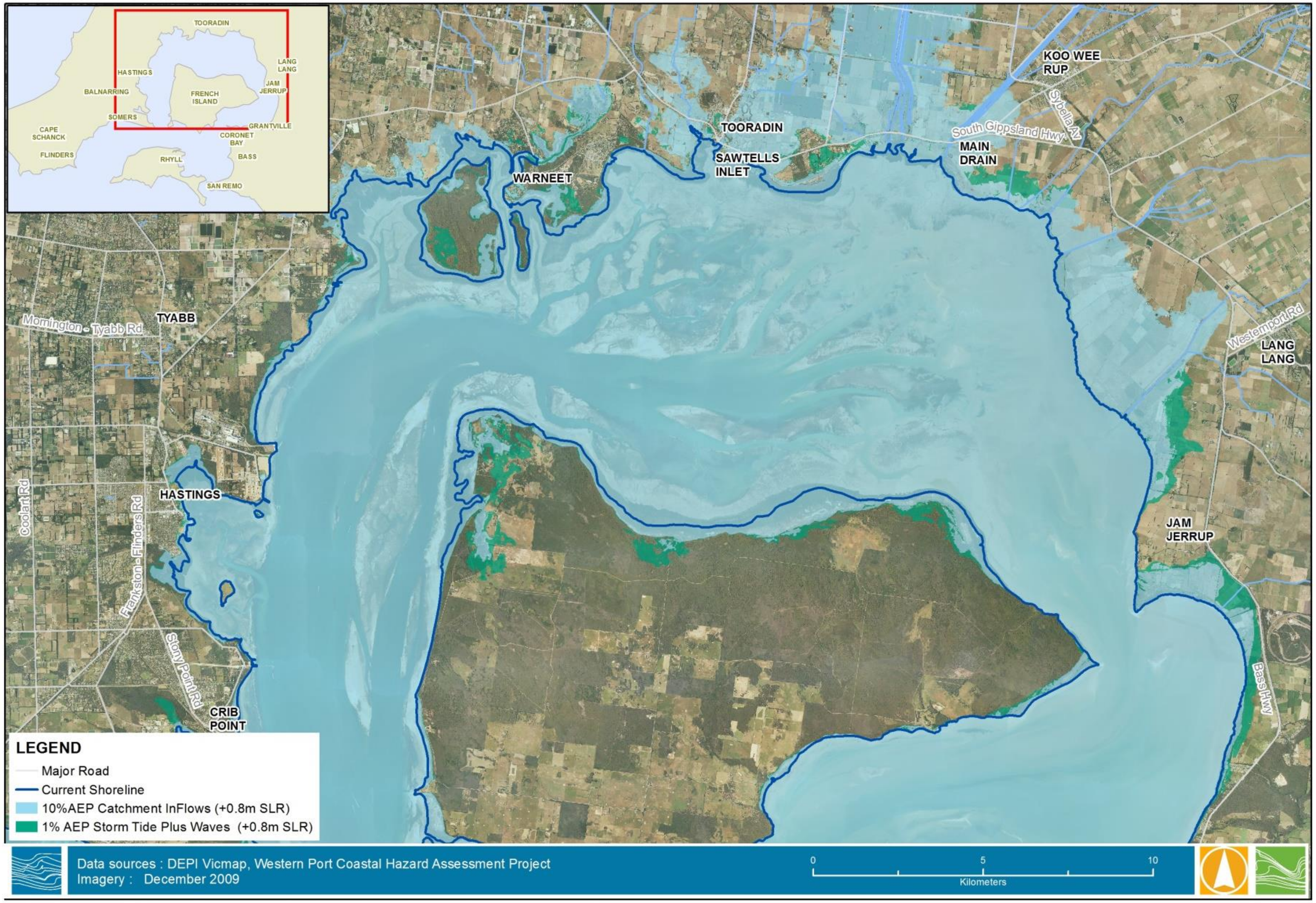

Figure 4-6 Broad Scale Inundation Mapping Results – Map 2

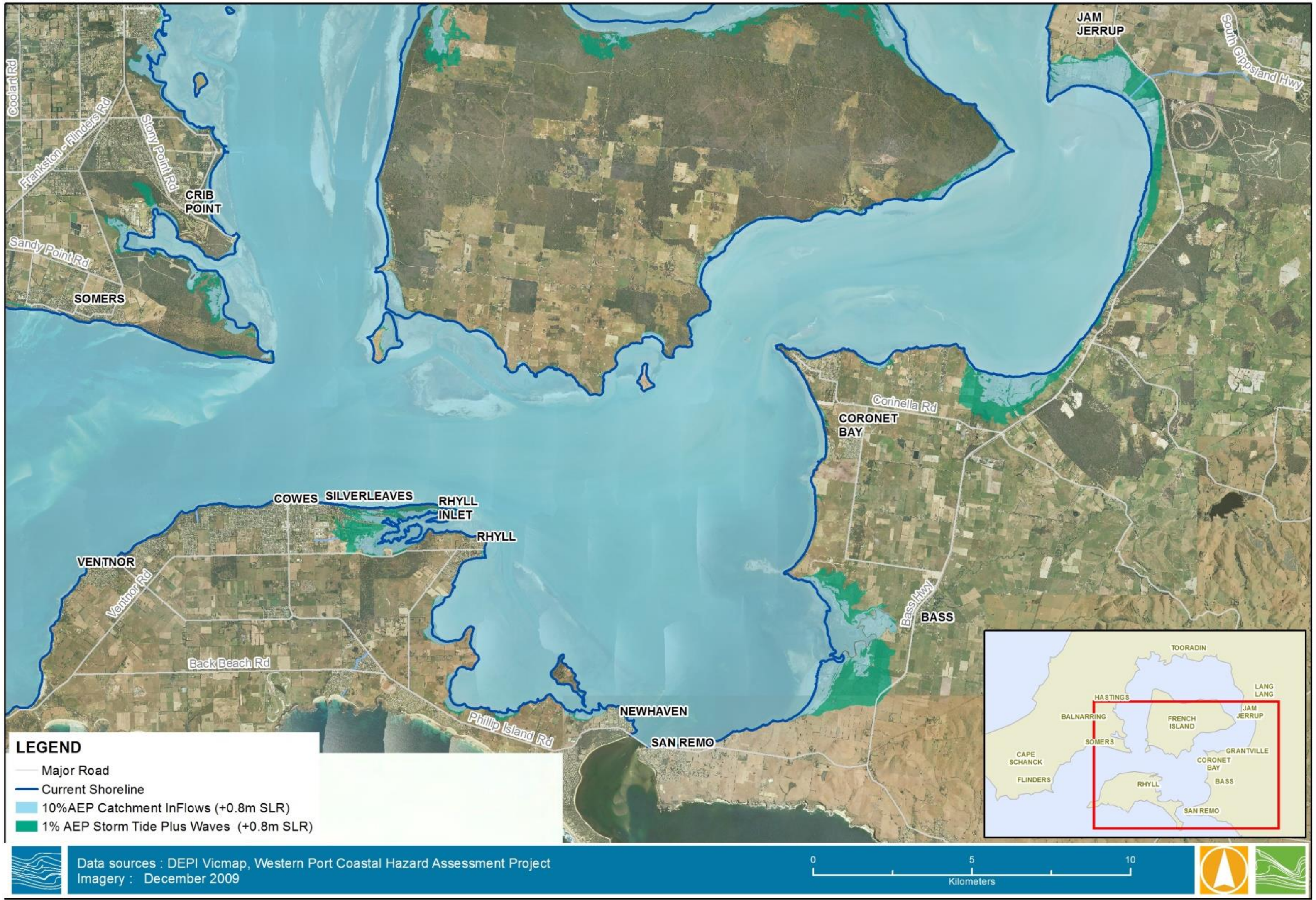

Figure 4-7 Broad Scale Inundation Mapping Results – Map 3

4.5 Comparison to Previous Storm Tide Mapping

Previous storm tide and sea level rise mapping provided by Melbourne Water is compared to the current study model results in Figure 4-8 and Table 4-3. It can be seen that the flood extent indicated in the previous work is significantly greater in the northern and north eastern areas of Western Port in particular. These differences are a result of three main factors:

- The current study has incorporated levees and structures such as culverts and drains, which in many instances limit the flood extent, particularly under existing mean sea level and the +0.2m and +0.5m SLR scenarios. As discussed on Section 3, the height of levee banks along the Western Port shoreline influences flood extents. The sensitivity of the inundation extents to the presence of coastal levees is discussed in the following Section.
- Dynamic effects of the storm tide are included in the current study. As discussed in Section 4, the duration as well as the magnitude of the 1% AEP Storm Tide has been considered. This is important when modelling the extent of coastal inundation as the volume of water within the storm surge along with tidal water levels will influence both the peak water levels and the ability of the surge to penetrate inshore. The effect of considering dynamic effects is typically to reduce flood inundation extent due to the storm tide compared to the previous static water level approach, as shown in Figure 4-8.
- For the 1% AEP design wind conditions, the 1% AEP north westerly wind speed for the Melbourne region of 25.1 m/s was adopted for all scenarios. Although this wind direction correlates to the measured storm surge conditions at the Stony Point gauge and is representative of extreme conditions for Western Port as a whole based on historical storm surge events, it may not represent the 'worst case' for some locations along the northern and western shoreline of Western Port.

Table 4-3 Comparison between the Dynamically Modelled Inundated Area (Water Technology) and the "Bath Tub" Modelled Inundated Area (Melbourne Water). *It should be noted that the Melbourne Water and DSE "Bath Tub" model did not include French Island, Phillip Island and Churchill Island, and therefore, underestimate the total inundated area in their mapped results*

Scenario	**Water Technology Dynamically Modeled Area Inundated (m^2)** 1% AEP storm tide wind and wave conditions + 10% AEP Catchment Inflows	**Melbourne Water "Bath Tub" Modeled Area Inundated (m^2)** 1% AEP storm tide
Base (Existing MSL)	43045000	93167509
1 (+0.2m SLR)	54175000	Not modelled
2 (+0.5m SLR)	76612500	Not modelled
3 (+0.8m SLR)	95890000	150202265

Figure 4-8 **Comparison of Different Modelled Inundation Extents Under 0.8m of Sea Level Rise; Water Technology (Dynamic Modelling Method), Melbourne Water 1% AEP Storm Surge Under 0.8m of Sea Level Rise (Static Modelling Method).**

5. GROUNDWATER HAZARDS

5.1 Overview

A preliminary high level assessment of climate change and groundwater hazards was undertaken to assess how sea level rise is likely to impact coastal groundwater aquifers and to identify any key issues warranting further assessment beyond the scope of this project. The review has involved the following components:

- Literature review of sea level rise impacts on coastal aquifer systems,
- A general review of the Western Port groundwater systems – extent and conditions,
- Identification of key potential groundwater hazards under present and future sea level conditions,
- Qualitative interpretation of results of inundation modelling on the groundwater hazards identified.

5.2 Groundwater Processes

5.2.1 Groundwater Description

A detailed overview of the groundwater flow systems of Western Port is provided in Dalhaus et al (2004). For each groundwater flow system the report provides a description of the landscape features, hydrogeology, salinity, and risks and management options. They delineate eighteen groundwater flow systems in the Port Phillip and Western Port region, of which the following interact with the coastline of Western Port. They included:

- Quaternary sediments – e.g. Sandy Point, Hastings, eastern shore of French Island.
- Gravel and sand sediments – e.g. surrounding Pioneer Bay, Lang Lang
- Swamps and back dune wetlands - e.g. Warneet, Tooradin
- Weathered older volcanics – e.g. south east Mornington Peninsula, Phillip Island, Corinella Point
- Brighton Group sediment – e.g. Tyabb, south eastern French Island
- Western Port plains – e.g. Koo Wee Rup, north western French Island

An overview of the various relevant groundwater systems is provided in Figure 5-1. Previous reports such as Carillo-Rivera (1975) and Lakey and Tickell (1981) characterise the groundwater systems of Western Port in terms of the hydrogeology. In general, the Western Port groundwater basin can be defined as a "leaky-confined, horizontally stratified groundwater system" (Lakey and Tickell, 1981) with hydraulic connections between the different geologic and hydrogeologic groups.

The groundwater systems of the Western Port plains are widely used for agriculture, industry and domestic supply. In 1971 Koo Wee Rup was declared the State's first Groundwater Conservation Area, and subsequently a Groundwater Management Plan was approved in 2010 by the Minister for Water for the Koo Wee Rup Water Supply Protection Area (WSPA). Currently within the WSPA there are groundwater licenses which entitle the license holders to extract 12,624.4 ML each year (SRW, 2013). Before groundwater was developed in the Western Port area the aquifers were generally artesian (free flowing) but during the peak of the irrigation season in the early 1970's the potentiometric surface was lowered to more than 25 m below mean sea level (Carillo-Rivera, 1975). The effect of depressing the groundwater table creates changing groundwater flow directions and gradients with the potential for intrusion of sea water or groundwater from saltier parts of the Western Port aquifer into the freshwater areas. The depression of the groundwater surface due to extraction pressures is an on-going issue for the system.

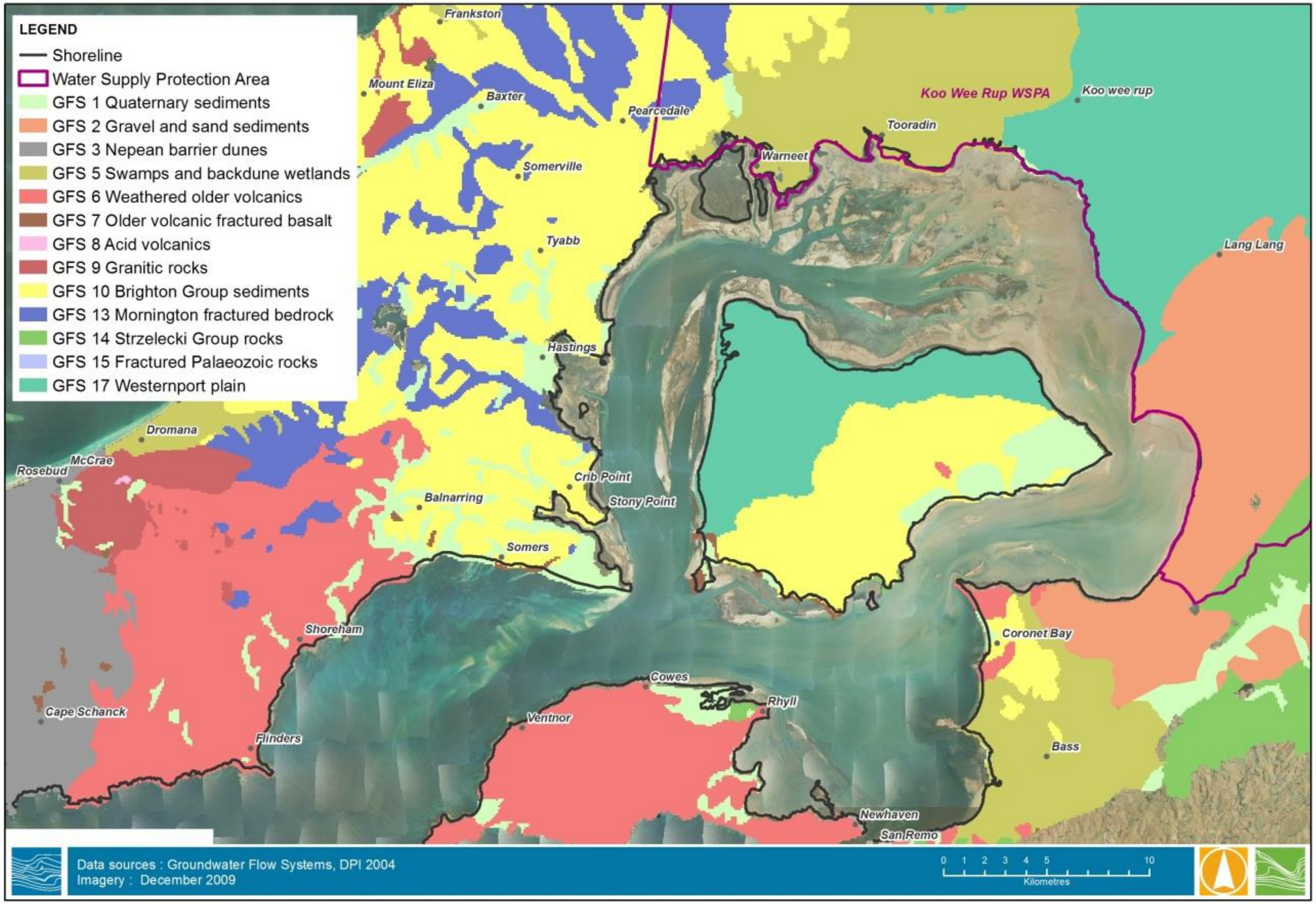

Figure 5-1 Groundwater Flow Systems of Western Port

5.2.2 Key Drivers and Processes

Coastal aquifer systems where there is a hydraulic connection with sea water exhibit an interface between the less dense freshwater siting above, and adjacent to, a wedge of saltwater. As the salt water is denser than the fresh water it moves in this form beneath the fresh water. This wedge of salt water can occur on the landward side of the coastline and can extend from metres to several kilometres beneath the freshwater system (Ivkovic et al, 2012). Mixing occurs between the freshwater and saltwater at the interface of the two, with the position and width of the interface zone depending on the particular hydrogeological and hydrological conditions. The key components and processes are displayed conceptually in Figure 5-2 and discussed below.

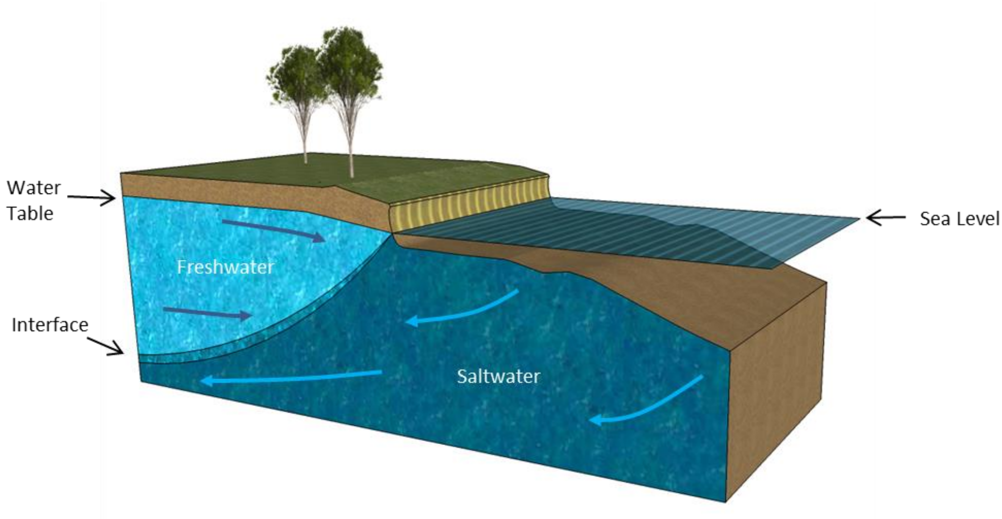

Figure 5-2 Simplified schematic showing the freshwater-saltwater interface in an unconfined coastal aquifer

During a tidal cycle the location of the interface can vary depending on the sea level; with the interface moving inland at the high tide and then retreating seaward on the ebb. The result is that on areas such as tidal flats, there can be a layer of saline groundwater beneath the near-shore vegetation communities during the high tide which is displaced by fresh groundwater as the tide retreats. The magnitude of these changes is strongly dependent on the level of hydraulic connection between the aquifer and the sea and the geological properties of the aquifer such as permeability.

Rivers, stream and drainage channels can provide freshwater inputs known as “recharge” to the coastal groundwater systems but they can also act as conduits for the more dense salt water to move inland with the tidal movements.

Another key process by which saline water migrates landward is as a result of groundwater extraction. The extraction of the freshwater through pumping lowers the groundwater table and can reverse the natural movement of fresh groundwater towards the coastline. This change in hydraulic gradient allows salt water to move more easily into the freshwater areas.

The movement of saline water from sea water sources into freshwater aquifer systems is known as sea water intrusion.

5.2.3 Climate Change Impacts

Climate change has the potential to affect groundwater systems through a number of different mechanisms, including the following (based on Barron et al, 2011):

- Changes in rainfall amounts and intensity,

- The effect of vegetation from changes in temperature and carbon dioxide,
- Alteration of groundwater-surface water interactions, such as sea water intrusion in coastal settings.

These impacts and the associated changes are predominantly recharge processes for the groundwater systems. Due to the coastal setting of Western Port and the scope of this coastal hazard assessment, the focus of this groundwater hazard review is on sea water intrusion into coastal groundwater systems and how the current groundwater hazards may be impacted by sea level rise.

The effect of changes to rainfall, catchment derived surface water, and vegetation on coastal groundwater recharge and water quality is beyond the scope of this study.

Sea Level Rise Changes

Sea level rise is predicted to have a number of potential impacts upon coastal aquifer systems. The following key potential hazards have identified:

- Landward migration of the freshwater-saltwater interface. The scale of the intrusion of sea water is likely to be dependent on the capacity of the groundwater table to rise at the same rate as sea level change. Surface controls such as drains, wetlands, streams/rivers, groundwater evapotranspiration, and groundwater abstraction may limit the water table rise that could occur (Werner and Simons, 2009). This is displayed conceptually in Figure 5-3.
- As well as sub-surface impacts, sea level rise may also result in the permanent surface inundation of low-lying coastal regions and/or increase the frequency and intensity of temporary inundation. This could result in the intrusion of salt water into freshwater reserves by movement of the interface or by downward seepage. It may also limit existing recharge zones (Ivkovic et al, 2012). This is shown conceptually in Figure 5-4.
- The time taken for the freshwater-saltwater transition zone to reach equilibrium can vary significantly. Highly permeable aquifers can have a quick response time from a geological point of view. Nevertheless, even in these rapid systems, the time scale will still be in the order of years to decades for a new dynamic state of equilibrium to be reached. Barlow (2003) found that sea water intrusion from past sea-level rise fluctuations have not yet reached equilibrium even after periods as long as 100,000 years (Ivkovic et al, 2012).

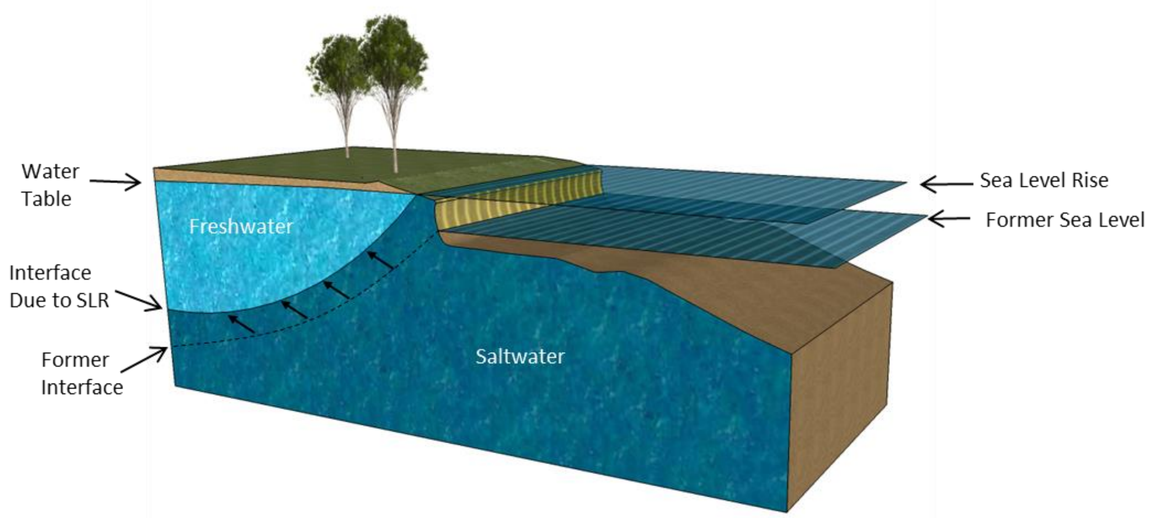

Figure 5-3 Upward and Landward shift in the Freshwater-Saltwater Interface as a result of SLR

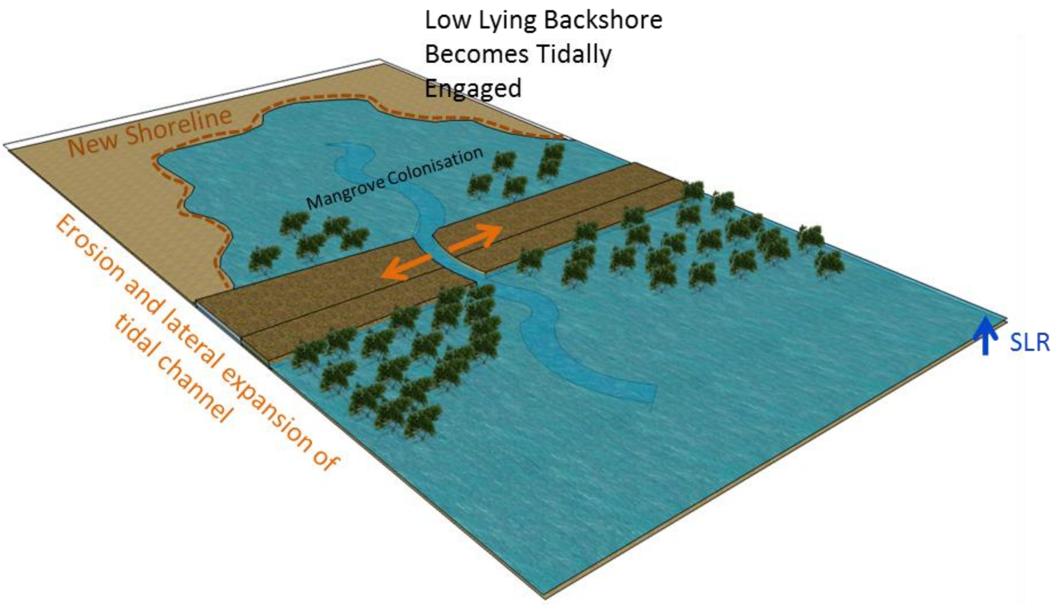

Figure 5-4 Increased Inundation of Backshore Areas with Sea Level Rise

5.3 Groundwater Hazards in Western Port

Present Conditions

Under present conditions the groundwater systems of Western Port are considered to be at risk of increased salinity due to changing groundwater flow directions and gradients, associated with groundwater extraction in the Western Port aquifers, particularly in the Koo Wee Rup area. The Koo Wee Rup Groundwater Management Plan considers the risks of sea water intrusion and requires monitoring of groundwater quality. It has been identified that the risks are (SRW, 2010):

- Intrusion of sea water from Western Port bay via slow vertical leakage through the overlying Quaternary sediments into the confined target aquifers; and/or
- Intrusion of lower quality groundwater from saltier parts of the Western Port aquifer.

To date monitoring data has not indicated that these conditions have or are occurring and the zone of potential hazard associated with these conditions has not been defined.

Ivkovic et al (2012) details a national scale vulnerability assessment of sea water intrusion on coastal aquifer systems. Although only the Werribee River delta site was assessed in detail in Victoria, Koo Wee Rup was identified as highly vulnerable to sea water intrusion based on the following characteristics:

- Over the period 2000-2009 borehole records indicated groundwater levels < 0 m AHD,
- Inter-decadal decline in minimum groundwater levels of between 2.5 to 5 m,
- Areas with decreasing groundwater level trends between 0.25 and 0.5 m/year,
- Inter-decadal increases in total dissolved solids (TDS) in the range 1000 to 3000 mg/L.

Sea Level Rise Impacts

Under the sea level rise scenarios used for this study, by 2100 the mean sea level is expected to have risen by +0.8m. For Western Port's coastal aquifers, this would likely result in the following:

- Increased sea water intrusion, the extent of which would be enhanced by current extraction and surface drainage systems across the Koo Wee Rup area.
- Reduced freshwater recharge through a reduction in the extent of recharge areas as a result of increased permanent inundation of low lying areas and increased frequency and intensity of temporary inundation.

For example, Werner and Simons (2012) found that for a simplified coastal groundwater system where the water table was controlled through surface drains or pumping, migration of the toe of the sea water - freshwater interface was in the order of 5 km inland for +0.5 m of sea level rise. In Australia, sea level rise is considered a contributing factor to sea water intrusion of freshwater meadows and billabongs in the Alligator River Region in the Northern Territory (e.g. Cobb et al, 2007).

It should however be noted, that the rate of change of groundwater systems can be significantly slower than surface waters due to the properties of the aquifer material. Over the time period of this assessment (to 2100) the coastal aquifer systems are unlikely to reach an equilibrium response to the expected +0.8m level of sea level rise.

In addition to the impacts identified above, increases in groundwater levels as a result of sea level rise may also have a significant impact on shoreline erosion processes for cliffed shorelines around Western Port (refer to Report 5 – Erosion Hazards for further discussion of shoreline types and erosion processes). Variations in groundwater flow and fluctuations can affect slope failure susceptibility. This occurs through a range of different processes, as described in Hampton and Griggs (2004) and Castedo et al (2012), resulting in a reduction in the maximum stable slope. However, the significance of groundwater on the cliffed coastal shorelines is uncertain and the impacts of sea level rise even more so. This potential hazard has not been considered further in the erosion assessment for this study due to the current lack of knowledge of both the processes and responses under present sea level or with sea level rise.

To provide an indication of potential changes in groundwater hazards associated with sea level rise, Figure 5-5 shows the water table salinity in Western Port with the 1% AEP +0.8 m sea level rise scenario inundation extent superimposed. The areas affected by inundation will experience an increase in groundwater salinity through sea water intrusion. This may have significant effects for the currently freshwater aquifer system around Lang Lang, and within the Koo Wee Rup WSPA as this area provides the main freshwater recharge for the WSPA.

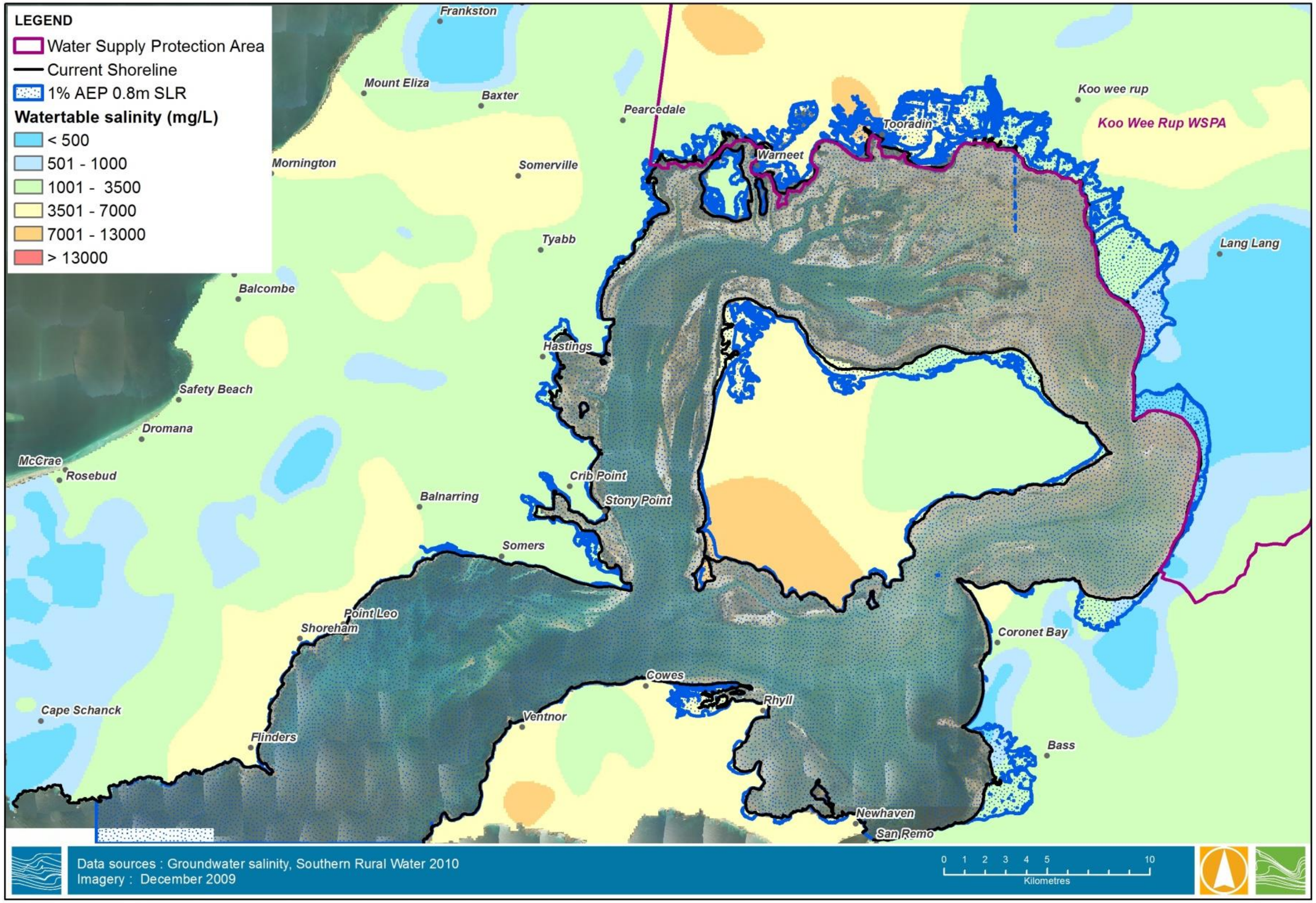

Figure 5-5 Superposition of 1% AEP +0.8m SLR Scenario Inundation Extents with Groundwater Salinity in Western Port

6. EVALUATION OF SOURCES OF UNCERTAINTY

6.1 Uncertainty in Future Changes

The analysis and modelling of the impact of sea level rise and climate change on the extent of inundation hazards in Western port has a number of potential sources of uncertainty. These sources of uncertainty require evaluation to determine the sensitivity they may have on the findings of the study.

From the review of relevant literature and the parallel assessment of shoreline erosion hazards, the assessment of inundation hazards due to sea level rise is considered potentially sensitive to the following future sources of uncertainty:

- Ownership and future management and adaptation of coastal levees and embankments;
- Changes to catchment flood hydrology due to climate change and associated increases in rainfall intensity;

Table 6-1 summarises the combination of events and the uncertainty scenarios assessed. The uncertainty assessment was only conducted for the +0.8 m sea level rise scenario as this was considered the worst case and most sensitive to future assumptions.

Table 6-1 Summary Table of Uncertainty Scenarios

Combination of Events to Assess Coastal Hazards	Uncertainty Assessed
0.8 m of sea level rise plus 1% AEP storm tide, wind and wave conditions	Importance of Coastal Levees – Scenario was simulated with the coastal levees removed
0.8 m of sea level rise plus 10% AEP catchment flows	Influence of Increased Catchment Inflows Associated with Climate Change – Scenario was simulated with all catchment inflows scaled up 20%.

6.2 Ownership and Future Management of Coastal Levees

Ownership and management responsibilities of shoreline embankments and coastal levees around Western Port, (small levees parallel to the shoreline in Figure 2-9) is currently unclear, with many of the structures apparently being informal and not engineered to any particular standard.

Significant works to adapt and maintain these structures will be required in the future if they are to prevent inundation hazards extending across low lying backshore regions behind the structures. In order to gain an understanding of the sensitivity of the inundation extent to the absence of the embankments and levees the hydrodynamic model has been modified to create conditions where the structures have been removed. This has then been tested for Scenario 3, assuming the +0.8 m sea level rise and 1% AEP storm tide conditions.

Figure 6-1 displays a comparison between inundation extents for the 1% AEP storm tide with +0.8m sea level rise, with and without the structures. The most significant change in inundation extent was observed inland between Lang Lang and Yallock Drain, where coastal inundation extended approximately one kilometre further inland. Inundation extents did not increase around Tooradin, Jam Jerrup or the Bass River delta as the existing structures were already overtopped in Scenario 3. However, some increases in inundation depth were still observed in these areas with the structures removed.

6.3 Changes in Flood Hydrology Associated with Climate Change

Although mean annual rainfall is likely to decline due to climate change, extreme precipitation is likely to increase in intensity. Abbs and Rafter (2008) used a high-resolution regional atmospheric model to predict the likely changes in extreme rainfall intensity in the Western Port catchment in 2030 and 2070. Their research found that:

- An increase in the magnitude of future extreme rainfall events is expected across most of the Western Port catchment.
- The largest increases occur for the short durations events.
- By 2030, short duration (2 hour) extreme rainfall is expected to increase while long duration (72 hour) rainfall is expected to decrease. Mid-duration (24 hour) rainfall is expected to increase in the southern parts of the catchment and decrease in the northern part of the catchment.
- By 2070 an increase in extreme rainfall is expected across all durations.

The report presents specific results for the likely change in extreme rainfall intensity for the Bunyip and Lang Lang catchments. Flooding in these catchments is driven by long-duration (24–48 hours) rainfall events. The median predicted changes in the Bunyip and Lang Lang catchments for 24 and 72 hour durations for 2030 and 2070 are given in Table 6-2. By 2070, long-duration extreme rainfall intensity is expected to increase by 13 % in the Bunyip catchment and 17-20 % in the Lang Lang catchment.

Although there are available predictions for changes in extreme rainfall intensity for the northern and southern parts of the Bunyip and Lang Lang catchments, there is no available literature on how catchment inflows for the whole Western Port region will respond to these predicted changes in extreme rainfall intensity. Therefore, to assess the sensitivity of inundation associated with the 10% AEP catchment inflow conditions, all flows were scaled up by 20 % and re-run through the hydrodynamic model under the +0.8 m sea level rise scenario, to provide a worst case – conservative sensitivity analysis.

Table 6-2 Average Percentage Change in Extreme Rainfall Intensity for Bunyip and Lang Lang Catchments (Abbs and Rafter 2008)

Duration (hrs)	Catchment	2030 Median % Change	2070 Median % Change
24	Bunyip	-4	13
	Lang Lang	3	17
72	Bunyip	-16	13
	Lang Lang	-9	20

Figure 6-2 presents the results of the changes in flood hydrology associated with the climate change uncertainty assessment. In general, a 20 % increase in catchment inflows resulted in relatively minor increases in inundation extent. The largest increase in inundation extent occurred to the east of Cardinia, Toomuc and Deep Creeks, due to the broad low lying plains surrounding these water courses. An increase in inundation extent was also observed between the Bunyip Main Drain to Yallock Drain.

Although only relatively minor increases in inundation extents occurred as a result of the scaled up catchment inflows, the inundation depths increased to accommodate the additional streamflow volumes.

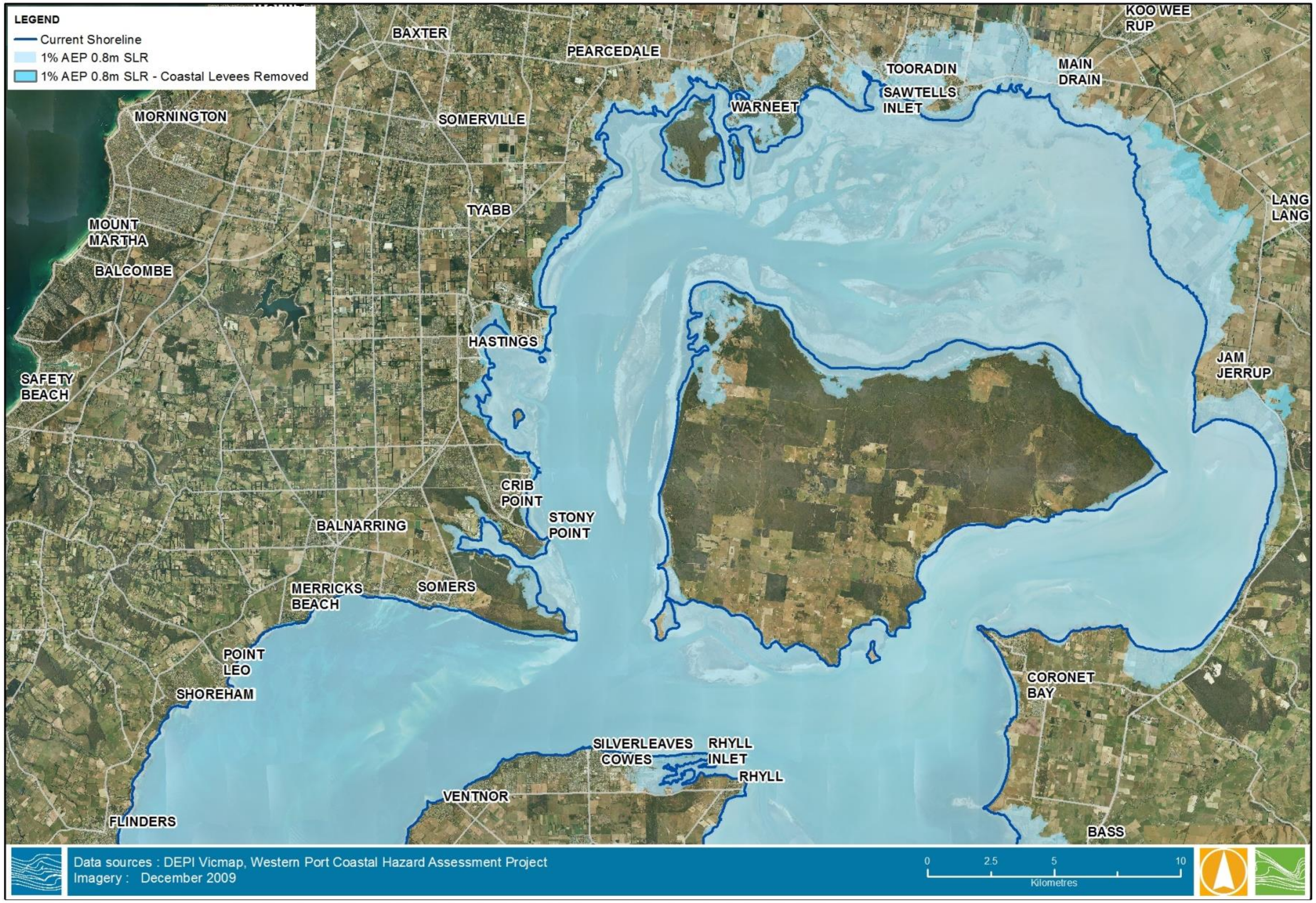

Figure 6-1 Modelled Change in Inundation Extent for the 1% AEP Storm Tide, Wave and Wind Conditions Under the +0.8m SLR Scenario, With and Without the Coastal Levees

Figure 6-2 **Modelled Change in Inundation Extent for the 10% AEP Catchment Inflows Simulation and the +0.8m Sea Level Rise Scenario, with Catchment Streamflows Scaled up by 20%**

6.4 Additional Sources of Uncertainty

Several limitations and sources of uncertainty remain, which were unable to be evaluated either due to lack of data or being beyond the scope of the project, and need to be acknowledged when considering the results presented in the above assessment. They are briefly summarised as follows:

- No detailed bathymetry data was available for river and drainage channels, and therefore channel depths were assumed based on the available terrestrial LiDAR. In addition, fine scale schematization of all the rivers and drains in the hydrodynamic model is inappropriate at the Western Port wide scale, without compromising computational run times. However, considerable care and effort was taken in including all major rivers and drains in the hydrodynamic model mesh at an appropriate level for a Western Port wide assessment
- As above, inclusion of all culverts and waterway structures was not possible at the Western Port wide scale. However, the major structures controlling inundation extents (i.e. the control gates at Sawtell's Inlet) were included.
- The assessment required the analysis and design of a representative 1% storm tide for Western Port. The design storm surge was based on an assessment of the historic water level at Stony Point and return period storm surge and storm tide levels from McInnes *et. al.,* (2009). The assessment revealed a wide range of possible large storm surge duration lengths, and a representative duration was adopted for the design storm surge used in the assessment. However, it is possible that a longer duration storm tide event, which matched the 1% AEP storm tide height, could occur and may result in greater inundation extents than modelled in this study. This could be addressed through further sensitivity analyses of storm surge characteristics in Western Port.
- The representative design 1% storm tide event used for this assessment has not been optimised for all combinations of tide, wind and wave set-up or for all locations around the Western Port shoreline. To refine the design storm tide further the application of a statistically based approach such as a Monte Carlo analysis may be required
- There is limited long term streamflow gauging for the major and minor inflowing rivers, streams and drainage channels.
- There is a lack of data and analysis of current groundwater conditions across Western Port and general knowledge gaps in understanding the likely impacts of sea level rise on coastal aquifers.

7. SUMMARY AND RECOMMENDATIONS

7.1 Summary

This report has detailed the inundation component of the broad scale Part A local coastal hazard assessment for Western Port.

Coastal inundation around Western Port is function of a number of different physical forcings and hydrodynamic processes, including; astronomical tides, storm surge, wind setup, wave setup and catchment inflows. A detailed hydrodynamic and wave model of Western Port was developed and applied to integrate these processes and provide a dynamic analysis of extreme water levels within and around Western Port.

Two major inundation processes were assessed as part of this study; a representative 1% AEP storm tide and a 10% AEP catchment inflow event occurring in all major catchments within Western Port. Both inundation processes were modelled under existing mean sea level conditions and projected increased mean sea levels of +0.2 m, +0.5 m and +0.8 m.

7.1.1 Storm Tide Inundation (1% AEP storm tide)

The results of the representative 1 % AEP storm tide inundation assessment can be briefly summarised as follows:

- The inundation extent between Flinders and Somers is constrained by the steep sloping topography behind the shoreline and minimal changes in inundation extent were observed between all of the sea level rise scenarios.
- Significant increases in inundation extent were observed upstream of Sawtells Inlet at Tooradin, under each increased mean sea level scenario.
- Along the eastern shorelines, around Lang Lang, Pioneer Bay and Queensferry, the storm tide was largely contained where raised embankments (termed 'coastal levees' in this study) are present under existing mean sea level; however, some overtopping did occur along low areas or where there were breaks in the levees. Large increases in inundation extent were observed with each progressive rise in mean sea level scenario, with majority of the coastal levees being overtopped under the +0.8 m sea level rise (SLR) scenario.
- Scenario testing with and without the existing coastal levees showed the most significant increase in inundation extent when the levees were removed was between Lang Lang and Yallock Drain, where coastal inundation extended approximately one kilometre further inland.
- The Bass Delta was observed to be inundated during the storm tide under existing mean sea level conditions, however, only relatively small changes in inundation extent were observed under the subsequent increased mean sea level scenarios due to the steep topography behind the delta.
- Inundation extents increased under each of the increasing mean sea level scenarios inside Rhyll Inlet, with the inundation extent increasing towards the head of the inlet.

Each of these key findings was investigated in further detail during the local scale 'representative location' assessments (Part B) of the study, detailed in Report 6.

7.1.2 Catchment Derived Inundation (10% AEP flood)

The result of the inundation associated with the 10% AEP catchment inflow assessment can be briefly summarised as follows:

- Only relatively minor changes in inundation extent were observed under sea level rise scenarios during the 10% AEP catchment inflows inundation assessment, and where

inundation extents did increase, were primarily a result of increased tidal inundation rather than a result of catchment inflows.
- The largest inundation extents resulting from the 10% AEP catchment inflows were observed downstream of the Cardinia Creek, Deep Creek, Toomuc Creek, the Lang Lang and Bass Rivers and Yallock Drain. The inundation occurred where inflow water levels overtopped the drain/river banks and levees or embankments (if present), onto the surrounding low lying land, resulting in inundation further inland than was observed during the storm tide assessment. This was particularly noticeable to the north-east of Tooradin.

7.1.3 Groundwater

A preliminary high level assessment of climate change and groundwater hazards was undertaken to assess how sea level rise is likely to impact coastal groundwater aquifers and to identify any key issues warranting further assessment beyond the scope of this project.

For Western Port's coastal aquifers, the review identified the following potential impacts due to sea level rise:

- Increased sea water intrusion, the extent of which would be enhanced by current extraction and surface drainage systems across the Koo Wee Rup area.
- Potential for reduced freshwater recharge through a reduction in the extent of recharge areas as a result of increased permanent tidal inundation of low lying areas and increased frequency and intensity of temporary inundation.

It should however be noted, that the rate of change of groundwater systems can be significantly slower than surface waters due to the properties of the aquifer material. This means that over the time scale of this assessment (to 2100) any changes in the coastal aquifer systems are unlikely to have reached equilibrium and would continue to respond to the higher mean sea level conditions.

In addition to the impacts identified above, increases in groundwater levels as a result of sea level rise may also have a significant impact on shoreline erosion processes for cliffed shorelines around Western Port (refer to Report 5 for further details). This potential hazard has not been considered further in the erosion assessment for this study due to the current lack of knowledge of both the process and responses under present mean sea level or under sea level rise conditions.

7.1.4 Assumptions

Coastal Structures

For the purposes of the inundation hazard assessment, it is assumed that all embankment or coastal levees that are currently in place remain in place at their current extent and configuration. However, given uncertainties as to how these structures will be maintained and/or rebuilt into the future, an additional modelling run was undertaken without the structures present to provide an indicate of potential changes to inundation extents.

Storm Tide Scenarios

The modelled storm tide scenarios used in this assessment considered only the 1% AEP storm tide event. This is however, a very tight absolute distribution between the 10% AEP and 1 % AEP storm surge levels (0.08 m) which is characteristic of storm surges and is an important consideration when evaluating the inundation and vulnerability of locations within Western Port.

With increases in sea level the increased frequency of inundation associated with more frequent storm tide events (> 1% AEP frequency) may pose a higher hazard in some areas. Recommendations for future assessment of more frequent storm tide events are in the following section.

The 1% AEP design storm tide condition adopted for this study is representative of conditions for Western Port as a whole and may not provide a 'worst case' estimate for all locations along the shoreline. To refine the design storm tide the use of a statistically based approach such as a Monte Carlo analysis may be required.

Storm Tide & Catchment Flooding

An analysis of peak coastal driven water levels and catchment flood flows indicated that these is little to no correlation between these events and hence the storm tide and catchment flood events for each sea level rise increment were modelled separately.

7.2 Recommendations

The following recommendations are drawn from the results and findings of this Part A Inundation Hazard Assessment. Part B of the project involves build upon this work and provides a detailed local scale assessment of coastal hazards at four critical locations.

7.2.1 Critical Locations (for Part B Assessment)

Based on the results of the inundation assessment, the following locations have been identified as possible critical locations for the Part B assessment stage of this study.

- Tooradin and Sawtells Inlet – inundation of roadways (particularly the South Gippsland Highway), Tooradin Airport and township of Tooradin. The current broad scale assessment could be refined further at a local scale by better resolving the drainage infrastructure and bathymetry upstream of the South Gippsland Highway.
- Main Drain – the presence of levees along the Main Drain constrains flood extents due to storm tide conditions, however significant flooding of areas such as the South Gippsland Highway occurs. The condition and continued maintenance of coastal levees will impact future flood extents. Catchment derived flood events, combined with higher mean sea levels will increase flood extents in this area. The current broad scale assessment could be refined further at a local scale by better resolving the levees and drainage channels for Main Drain and Cardinia Creek.
- North East Shoreline from Main Drain to Jam Jerrup – the presence of existing formal and informal coastal levees along this section of shoreline significantly impacts flood extents under existing mean sea level and SLR scenarios. As discussed in Section 6.1, the condition and maintenance of these structures impacts future flood extents in this area. Removal or failure of structures currently on private land may potentially result in storm tide inundation of public assets such as the South Gippsland Highway. Catchment derived flood events, combined with higher mean sea levels will increase flood extents in this area.
- Rhyll Inlet and Silverleaves – under SLR scenarios there is increased inundation of Rhyll inlet. Possible inundation of Cowes-Rhyll Road and other areas in the vicinity.
- Warneet - Rutherford Inlet, Rutherford Creek, and Cannons Creek – the increased inundation extent in this area is less than for the other critical locations identified under the SLR scenarios. However, there are a number of coastal levees which currently mitigate flood impacts and the condition and continued maintenance of these structures will impact future flood extents.

The following locations have been identified as experiencing increased potential inundation under sea level rise scenarios; however inundation impacts are considered less than for those locations discussed above.

- Northern shore of French Island.
- Shoreline to the south east of Jam Jerrup around Stockyard Point – it is also noted that catchment derived flooding combined with higher mean sea level conditions will increase the extent of flooding in this area.
- Queensferry (former delta of Bass River).

- Bass River delta.

7.2.2 Future Data Collection

Collection of the following data sets would reduce uncertainties for future inundation hazard assessments:

- Detailed bathymetric data in Sawtells' Inlet, upstream of the control structures, and within all of the main rivers and drains connecting into Western Port bay.
- An audit and condition assessment of the coastal levees surrounding Western Port bay.
- Collection of a comprehensive wave dataset throughout Western Port in order to further understand the spatial and temporal differences in the wave climate, and further calibrate existing and future wave models.
- Continued monitoring of coastal groundwater systems such as in the Koo Wee Rup Water Supply Protection Area. This could be expanded to other coastal aquifers in the Western Port area.
- Documentation and data collection after storm surge and catchment generated flood events including survey marks and photographs.

7.2.3 Future Inundation Assessments

Based on the results of the various sea level rise scenarios assessed in this project, the following future assessments are recommended:

- Refinement of the estimates of 1% AEP storm tide to provide 'worst case' estimates for all locations around Western Port. This may require the use of a statistically based approach such as a Monte Carlo analysis.
- Extension of the current inundation assessment to include consideration of more frequent storm tide events (e.g. 10% AEP storm tide) for each sea level rise scenario. Increased frequency of inundation may pose a higher hazard in some areas that that generated by more extreme events.
- While the peak of a storm tide and the peak of a catchment generated flood occurring together have been shown to very rare events (>1% AEP) within Western Port, the impact of more frequent storm tides on prolonging or exacerbating catchment generated flooding has not been assessed in the current study. This could be examined further through additional sensitivity analyses.
- A detailed assessment of vulnerability of the Western Port groundwater basin to sea water intrusion, based on the approach outlined in Ivkovic et al (2012).

7.3 How to Use the Study Outputs

The information contained in this report along with the inundation hazard GIS datasets can be used to provide a better understanding of inundation hazards in an area of interest, particularly the key processes and how these may be impacted by sea level rise. Figure 7-1 outlines the typical process for applying the inundation hazard assessment outputs to assess potential risks for a particular section of the Western Port shoreline.

Understanding inundation processes

- Use the information in **Chapter 4** to understand potential inundation hazard for a location
- What is the **key driver-** storm surge / catchment flooding?

Consider Future Inundation Hazard Zones

- Refer to the inundation hazard **GIS datasets** & **Chapter 4**
- **Visualise** the potential inundation hazard extent for the area of interest for each sea level rise scenario

Understand Uncertainties

- Review other issues/considerations for the location e.g. **existing shoreline structures, timescales** etc
- Consider uncertainties (**Chapter 6**) and how they may affect the location

Figure 7-1 How to Use the Inundation Hazard Assessment Outputs

The outputs from the inundation hazard assessment should also be considered in conjunction with the erosion hazard assessment detailed in the Erosion Hazard Report (R05). An overview of both the erosion and inundation hazard assessments is provided in the project Summary Report (R01).

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APPENDIX A INUNDATION PHOTOS

FLOOD HISTORY

There is a long history of flooding within the Western Port catchment and coastal areas. The following table summarises the known flood history for the coastal areas within the catchment.

Table 8-1 Summary of the Flood History of the Western Port Catchment – Coastal Areas

Date	Location	Description	Source
April 1900	Koo Wee Rup Swamp	Heavy rain over a two-day period in the Upper Beaconsfield and Jindivick catchment areas (90.9mm) caused severe flooding in Yallock and Yannathan, resulting in the loss of crops, homes and bridges.	Cardinia Website[2]
April 1901	Koo Wee Rup Swamp	Extremely heavy rain over a three-day period in the Upper Beaconsfield and Jindivick catchment areas (217.4mm) caused extensive flooding across the Koo Wee Rup swamp.	Cardinia Website[1]
June 1911	Koo Wee Rup Swamp	Heavy rains (resulting in flows of 9800 megalitres per day in the Bunyip River) at Bunyip caused flooding in the southern and central parts of the Koo Wee Rup swamp to depths of 1.5 metres.	Cardinia Website[1]
1916	Koo Wee Rup	N/A	Kooweerup then and now[3]
October 1923	Koo Wee Rup Swamp	Floods reaching a peak of 21,600 megalitres per day inundated the Koo Wee Rup swamp, completely destroying potato crops at Cora Lynn with water nearly 2 metres deep in the cheese factory. Newly planted crops at Modella were destroyed also.	Cardinia Website[1]
August 1924	Koo Wee Rup Swamp	Rain in the Upper Beaconsfield, Gembrook and Jindivick catchment areas (301mm) caused a flood flow of 27,900 megalitres per day in the Bunyip River resulting in most of the swamp area being covered in flood water to more than 1.5 metres. Approx 30 year ARI	Cardinia Website[1] Kooweerup then and now[2] Flood Victoria[4]
November and December 1934	Koo Wee Rup Swamp	The Super Flood saw excessive rainfall (794.1mm) in the Upper Beaconsfield, Gembrook and Jindivick catchment area and 170.2 millimetres falling at Koo Wee Rup peaked at 97,840 megalitres per day.	Cardinia Website[1] Flood Victoria[3]

[2] *http://www.cardinia.vic.gov.au/Page/Page.aspx?Page_id=2012 Sourced from From Swampland to Farmland: A History of the Koo Wee Rup Flood Protection District by David Roberts (1985).*

[3] Kooweerup then and now http://www.kooweebypass.com.au/koo-wee-rup-then-and-now.pd

[4] *http://www.floodvictoria.vic.gov.au/centric/learn_about_flooding/flood_history/*

Date	Location	Description	Source
		Almost the entire Koo Wee Rup swamp area was inundated causing more than 1,000 people to become homeless and surprisingly no-one died. > 150 year ARI. Largest Flood on record	
April 1935	Koo Wee Rup Swamp	Five months later the district was inundated with 24,460 megalitres per day.	Cardinia Website[1]
October 1937	Koo Wee Rup Swamp	Extreme rainfall (667.6mm) in the Upper Beaconsfield, Gembrook and Jindivick catchment area and 76.1 millimetres at Koo Wee Rup caused 48,920 megalitres per day to flood the district again. Approx 100 year ARI	Cardinia Website[1] "Casey Cardinia – links to our past" Blog[5] Flood Victoria[3]
1952	Koo Wee Rup	Approx 20 year ARI	Flood Victoria[3]
1958	Koo Wee Rup Swamp	N/A	"Casey Cardinia – links to our past" Blog[4]
September 1959	Koo Wee Rup Swamp	Heavy rainfall (258.1mm) in the Upper Beaconsfield and Jindivick catchment area and 79.4mm at Koo Wee Rup saw water rise to 7.3 metres at the Sixteen Mile Bridge and 3.9 metres at the Cora Lynn Bridge; however, due to significant works on the Main Drain, flooding was not widespread.	Cardinia Website[1]
1962	Koo Wee Rup Swamp	N/A	"Casey Cardinia – links to our past" Blog[4]
November 1971	Koo Wee Rup Swamp	Heavy rainfall (289.9mm) in the Upper Beaconsfield and Jindivick catchment area and 53.1 millimetres at Koo Wee Rup caused an outflow of 19,445 megalitres per day in the Main Drain resulting in minimal flooding due to the flood protection scheme.	Cardinia Website[1]
1990	Koo Wee Rup Swamp	N/A	"Casey Cardinia – links to our past" Blog[4]
1991	Koo Wee Rup Swamp	Significant flooding in the Koo Wee Rup Swamp area.	Cardinia Website[1]
1996	Koo Wee Rup Swamp	Significant flooding in the Koo Wee Rup Swamp area with overflow of the Main Drain levee bank and major overflow/flooding of creeks/drains.	Cardinia Website[1]
February	Koo Wee Rup	170 people registered at a local relief	News.com.au[6]

[5] http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

Date	Location	Description	Source
2011	Swamp	centre. Paramedics helped relocate 50 patients from the Koo Wee Rup Hospital to Melbourne. While several roads in the area were flooded, no homes were inundated.	
June 2012	Koo Wee Rup, Lang Lang, Bayles, Caldemeade, Silverleaves	Koo Wee Rup, Lang lang, Bayles and Caldemeade	Kooweerup then and now[2]

Photos showing flooding in the catchment for different flood events are provided in Appendix A.

[6] http://www.news.com.au/national/national-new/weather-promts-3500-calls-for-help/story-e6frfkvr-1226000805781

Storm Tide Flooding

Storm Tide Photos – April 21st 2011

Warneet Boat Hire – Photo Courtesy of the Casey City Council

Warneet Boat Hire – Photo Courtesy of the Casey City Council

Photo Courtesy of the Casey City Council

Catchment Inflow Inundation

PRE 2011 FLOOD PHOTOS

Dustings Garage/Kooweerup Veterinary Clinic – 272 Rossiter Raod - Date unknown - Kooweerup then and now http://www.kooweebypass.com.au/koo-wee-rup-then-and-now.pd - Photos courtesy of the Kooweerup Swamp Historical Society and C Wallis

Memorial Hall – 1924 - - Kooweerup then and now http://www.kooweebypass.com.au/koo-wee-rup-then-and-now.pd - Photos courtesy of the Kooweerup Swamp Historical Society and C Wallis

London Bank/English, Scottish and Australian Bank/ ANZ Bank – date unknown- Kooweerup then and now http://www.kooweebypass.com.au/koo-wee-rup-then-and-now.pd - Photos courtesy of the Kooweerup Swamp Historical Society and C Wallis

Railway Station - 1916- Kooweerup then and now http://www.kooweebypass.com.au/koo-wee-rup-then-and-now.pd - Photos courtesy of the Kooweerup Swamp Historical Society and C Wallis

Old Post Office and Newsagency – 1924- Kooweerup then and now http://www.kooweebypass.com.au/koo-wee-rup-then-and-now.pd - Photos courtesy of the Kooweerup Swamp Historical Society and C Wallis

Royal Hotel – 1924- Kooweerup then and now http://www.kooweebypass.com.au/koo-wee-rup-then-and-now.pd - Photos courtesy of the Kooweerup Swamp Historical Society and C Wallis

Cora Lynn in an early flood, perhaps in the 1910s. The building on the right is the E.S.& A bank and the building in the middle is Murdoch's General Store.
http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

Rossiter Road in Koo-Wee-Rup in the 1934 flood. The photograph was taken just near the Railway line, the building on the right is St George's Anglican Church

http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

Station Street, Koo-Wee-Rup, during the 1934 flood
http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

Corner of Main Drain Road and Dessent Road, under flood, in 1937
http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

Corner of Main Drain Road and Dessent Road, under flood, in 1958.
http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

This photograph was taken by Jim Rouse, in October 1962, before the official opening of the Cora Lynn spillway. The building, with the brown coloured roof, is the Cora Lynn Hall. The other buildings you can see in the background are the same as the ones on the other Cora Lynn photograph at the top of this post - the E.S.& A Bank and the general store, then Dillon's store. The road at the top left is the newly constructed spillway and you can see where flood waters have broken through the Main Drain bank and are spilling across it.

http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

Corner of Main Drain Road and Dessent Road, under flood, in 1990. The original house in the previous two pictures has been demolished and a new house built plus some potato sheds.

http://caseycardinialinkstoourpast.blogspot.com.au/2009/11/1934-flood.html

FEBRUARY 2011

Feb 2011 Photos – Iona – Heraldsun.com.au

Feb 2011 Photos – Iona – Heraldsun.com.au

JUNE 2012

ABC News– Koo Wee Rup

Theage.com.au – Koo Wee Rup

Theage.com.au– Koo Wee Rup

http://www.weeklytimesnow.com.au/article/2012/06/22/499301_latest-news.html

Awash: Homes in McDonalds Drain Rd, Koo Wee Up, are surrounded by water.

3AW - http://www.3aw.com.au/Melbourne_Flash_Floods_Images– Koo Wee Rup

3AW - http://www.3aw.com.au/Melbourne_Flash_Floods_Images– Koo Wee Rup

3AW - http://www.3aw.com.au/Melbourne_Flash_Floods_Images– Koo Wee Rup

3AW - http://www.3aw.com.au/Melbourne_Flash_Floods_Images– Koo Wee Rup

Silverleaves, 26 June 2012 http://petesflap.blogspot.com.au/2012_06_01_archive.html

Silverleaves, 26 June 2012 http://petesflap.blogspot.com.au/2012_06_01_archive.html

Koo Wee Rup, 26 June 2012 http://petesflap.blogspot.com.au/2012_06_01_archive.html

Koo Wee Rup, 26 June 2012 http://petesflap.blogspot.com.au/2012_06_01_archive.html

APPENDIX B DESCRIPTION OF HYDRODYNAMIC MODEL

1. HYDRODYNAMIC MODEL

The Danish Hydraulic Institute's (DHI) MIKE21 Flexible Mesh (FM) hydrodynamic model was used to enable the impacts of sea level rise and climate change on the extent of inundation hazards to be predicted within the study area. The MIKE21 FM model is a two-dimensional model based on the two-dimensional shallow water equations; the depth-integrated incompressible Reynolds averaged Navier-Stokes equations.

The discretization of the governing equations is performed using a cell-centred finite volume method, with an unstructured mesh in the geographical domain. An explicit scheme was used for the time integration.

1.1 Domain Schematisation

Two hydrodynamic model meshes were created; the hydrodynamic model mesh (Figure 1-1) and a detailed hydrodynamic inundation model mesh (Figure 1-2). The hydrodynamic model mesh consisted of all areas within Western Port Bay below high water. This mesh was used in calibrating the astronomical tides, storm surge events, ADCP current and flux calibration, and was used to simulate one year (2003) of hydrodynamic conditions at existing and a potential future mean sea levels (+0.2, 0.5 and 0.8m), to provide the hydrodynamic forcing conditions for the spectral wave model simulations of Western Port Bay.

The hydrodynamic model inundation mesh was identical to the hydrodynamic model mesh, but included the surrounding low lying coastal land up to approximately 3.5m AHD. The hydrodynamic model inundation mesh was used in the storm tide and catchment inflow inundation simulations to assess the changes in inundation levels and extents under a range of potential sea level rise scenarios.

The coastal boundary of both models extended approximately 10 km offshore, into the Bass Strait. The bathymetry for both models was derived from a combination of the following bathymetric data sets, and interpolated onto the meshes using a prioritization routine to ensure the most recent data was used where available.

- Terrestrial Coastal LiDAR survey captured as part of the Coordinated Imagery Program.
- Bathymetric multibeam survey data of Western Entrance Channel, Lower North Arm, the deep channel along the western Upper North Arm and the East Arm.
- Bathymetric LiDAR
- A 50m resolution bathymetric grid of Western Port Bay provided by the EPA

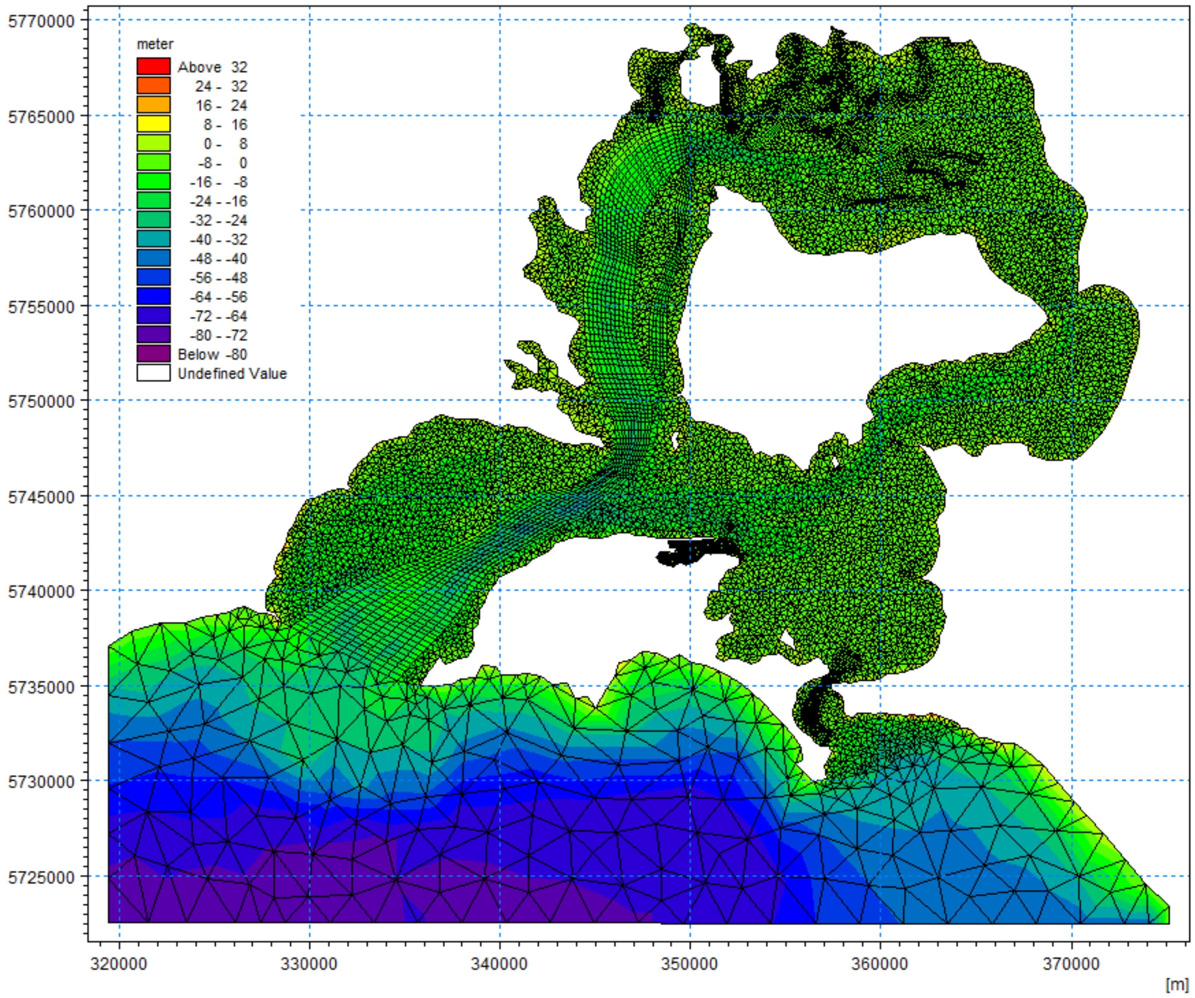


Figure 1-1 Hydrodynamic Model Domain and Mesh Schematization

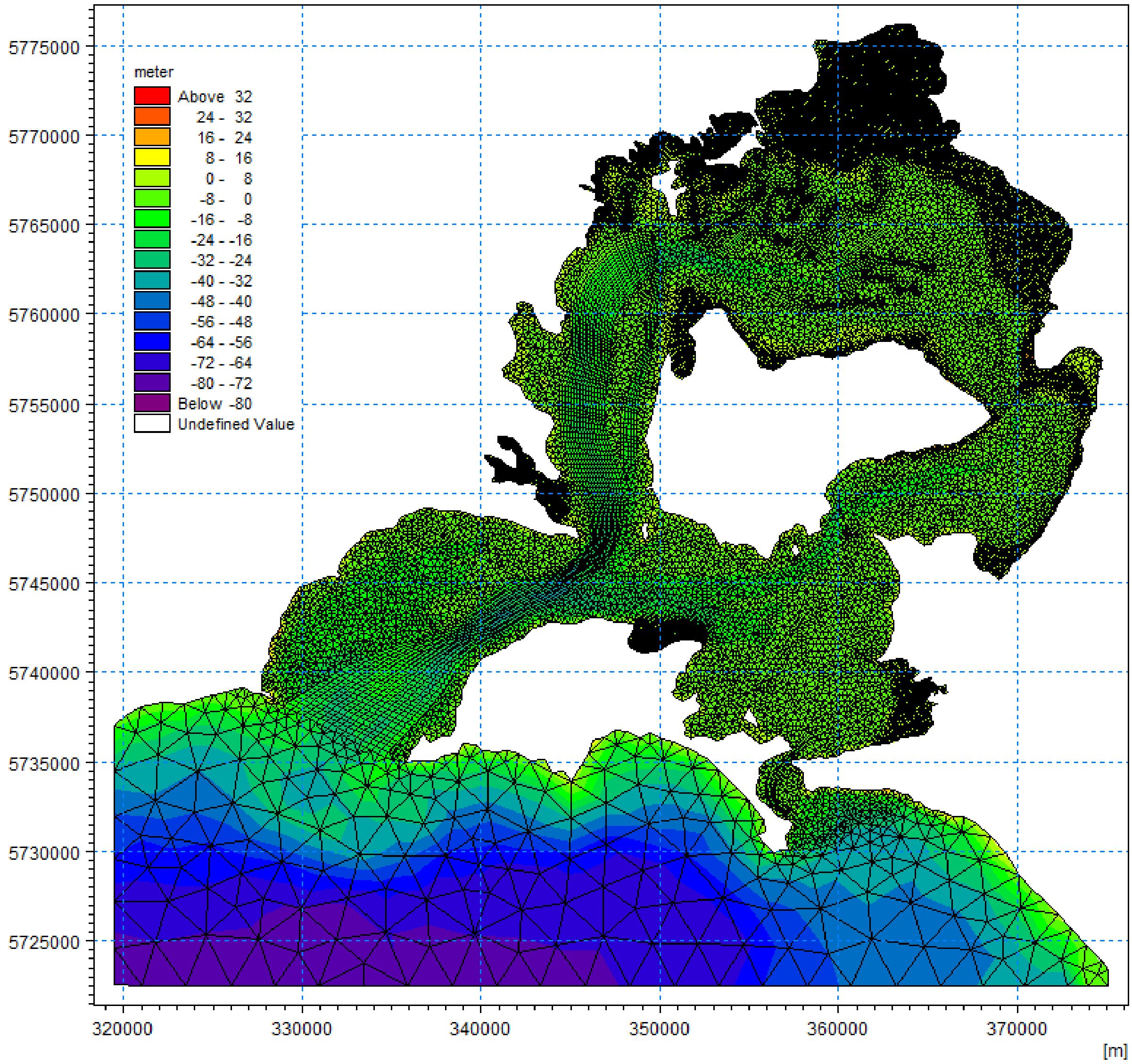


Figure 1-2 **Inundation Hydrodynamic Model Bathymetric and Model domain Schematization**

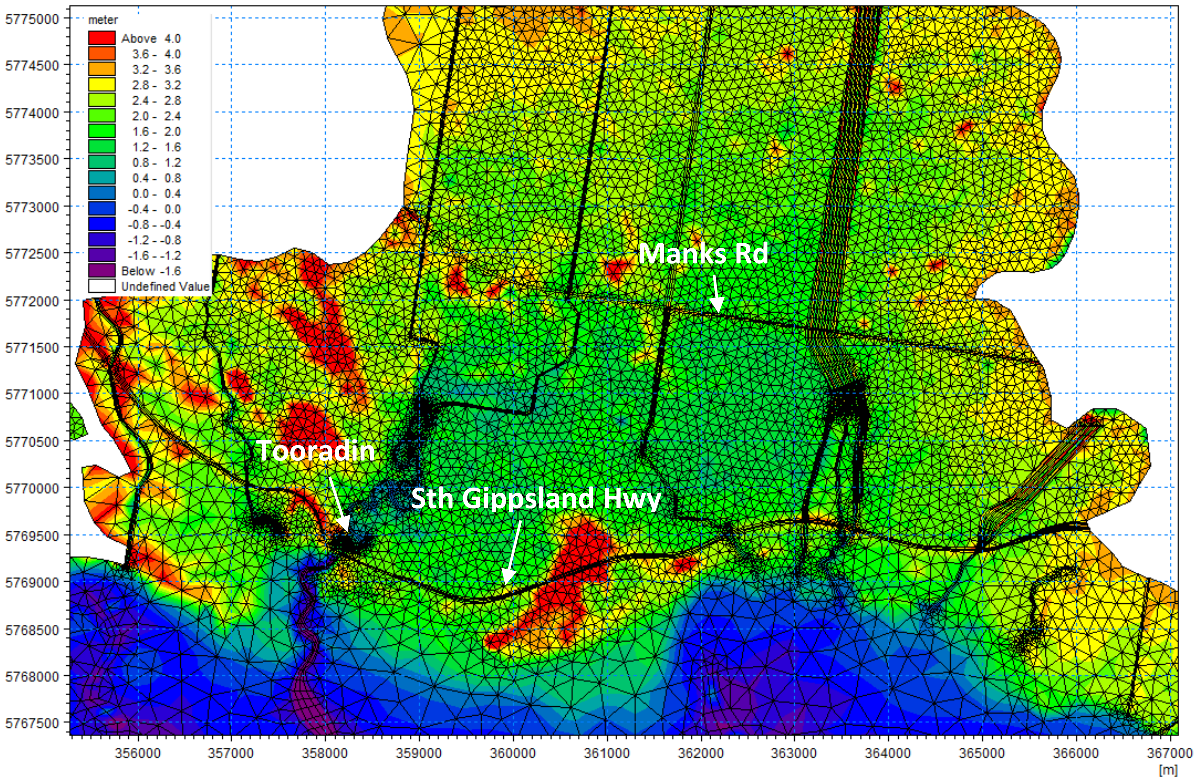


Figure 1-3 Close Up of the Hydrodynamic Model Mesh Covering the Low Lying Land to the North of Western Port Bay

The different survey data sets were projected to GDA coordinates and all elevations reduced to AHD. A prioritisation routine was used during the computational mesh interpolation to utilise more recent and or more detailed survey data sets where appropriate within the study area.

The computational meshes consisted of both triangular and quadrilateral elements, and varied in size depending on the complexity of the terrain and the need to resolve the hydrodynamic processes of interest at different locations within the study area. Figure 1-2displays an overview of the model domain and computation mesh, and Figure 1-3 displays a close-up of the model mesh covering the low lying land towards the north of the study area.

1.2 Boundary Conditions

Due to the multiple physical forcings giving rise to water level variations in Western Port Bay, the hydrodynamic model contains a number of different boundary conditions specifications. The hydrodynamic model boundary conditions and source data used to force the model boundaries are discussed below:

1.2.1 Astronomical Tides

An open tidal boundary was defined along the two offshore coastal boundaries in Bass Strait. The astronomical tidal component of the offshore open boundaries were derived from astronomical tidal constituents extracted from points along the western and southern open boundaries from South Australian and Bass Strait tide model developed by the Oregon State University (OSU, Tidal Data Inversion, 2010). Slight adjustments to the tidal constituent's amplitude and phase were made as part of the calibration processes, the results of which are presented in Section 1.3.

Table B1 List of Astronomical Tidal Constituents Used to Force the Western and Eastern Ends of the Southern Offshore Model Boundary.

Constituent Name	Amplitude (m) Western/Eastern	Phase (°) Western/Eastern
M2	0.788 / 0.867	328.480 / 329.660
S2	0.228 / 0.228	108.580 / 112.00
K1	0.181 / 0.186	76.360 / 81.360
O1	0.137 / 0.139	48.030 / 50.610
N2	0.147 / 0.154	286.360 / 258.000

1.2.2 Coastal Residual Water Levels

Time series of the meteorological forced component of coastal water level variations associated with barotropic effects, coastally trapped waves and local wind setup were extracted from the nearest open coastal tide gauge, at Lorne, and applied in combination with the predicted astronomical tidal component to the offshore open model boundaries.

1.2.3 Catchment Inflows

During calibration of the Hydrodynamic model for astronomical tides and coastal ocean level storm surge events catchment inflows were not considered.

Catchment flows are described in detail in Appendix C.

1.2.4 Wind Shear

Wind data extracted from the NCEP reanalysis global model was applied uniformly over the model domain. The wind drag coefficient was varied linearly with wind speed as follows:

$$C_D = \begin{cases} 7\ \frac{\mathrm{m}}{\mathrm{s}} & 1.255 \times 10^{-3} \\ 25\ \frac{\mathrm{m}}{\mathrm{s}} & 2.425 \times 10^{-3} \end{cases}$$

Comparisons of the NCEP reanalysis wind data against nearby wind gauges (Stony Point, Rhyll and Pound Creek) showed that the NCEP reanalysis wind data provided a good temporal and spatial representation of wind conditions over WPB. Therefore, the NCEP reanalysis wind data was used throughout the project as it provided long continuous record of wind conditions.

1.3 Model Calibration

1.3.1 Astronomical Tide

The astronomical tidal component of the hydrodynamics was calibrated by forcing the model solely by the offshore tidal boundary. Modelled water levels were then compared against predicted tidal time series derived from the same tidal constituents at a number of locations published by Hinwood and Jones (1979, all locations except Stony Point) and ANTT (2013, Stony Point), which list the principal harmonic tidal constituents.

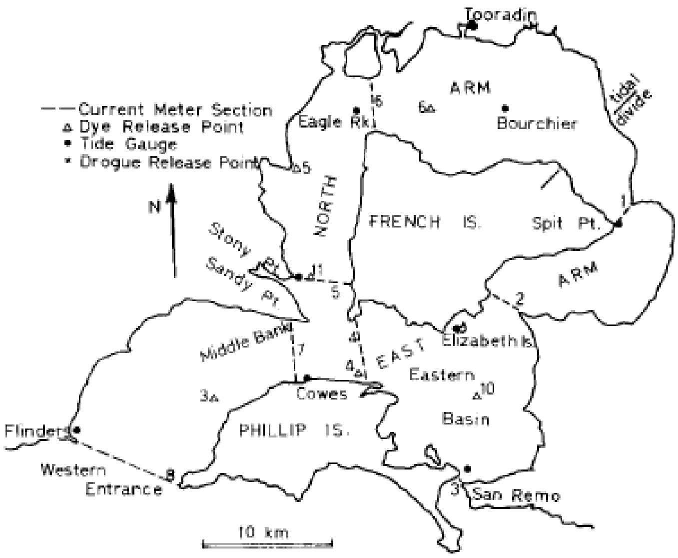
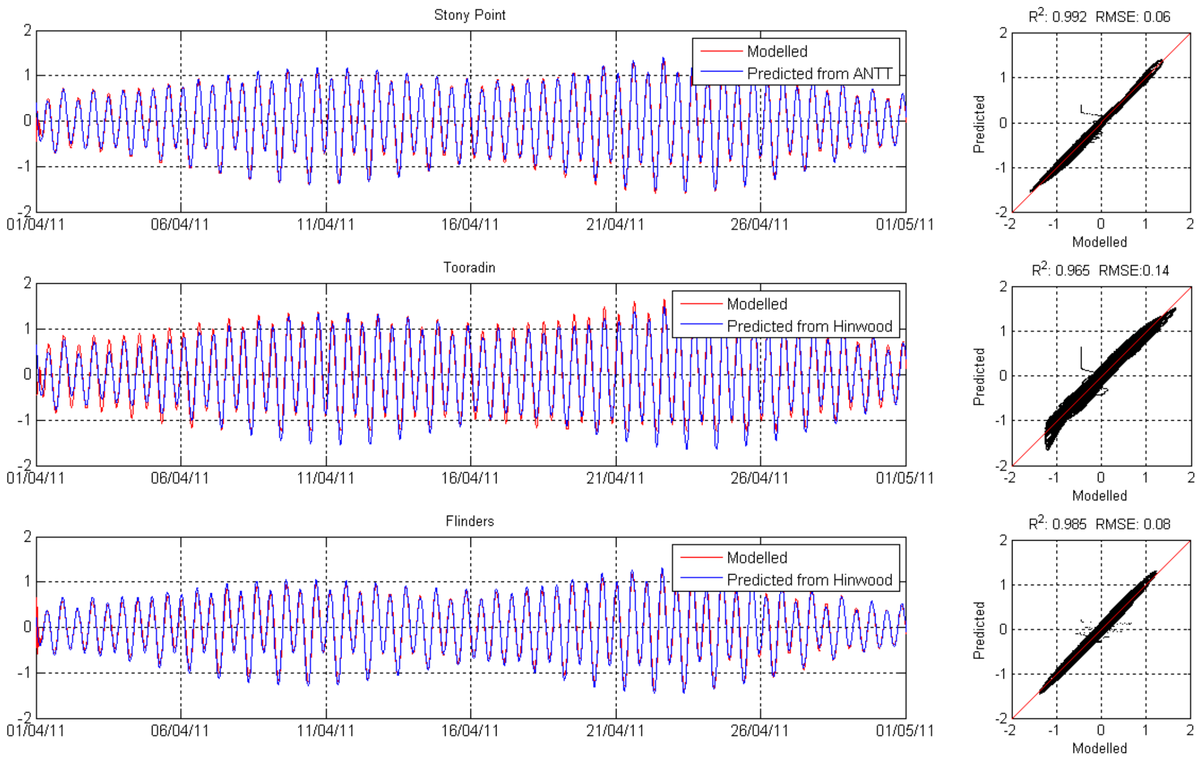


Figure 1-4 **Locations of where the Astronomical Tidal Constituents were Derived in Hinwood & Jones (1979) which were Used for the Hydrodynamic Model Calibration as Part of this Project (from Hinwood & Jones, 1979).**



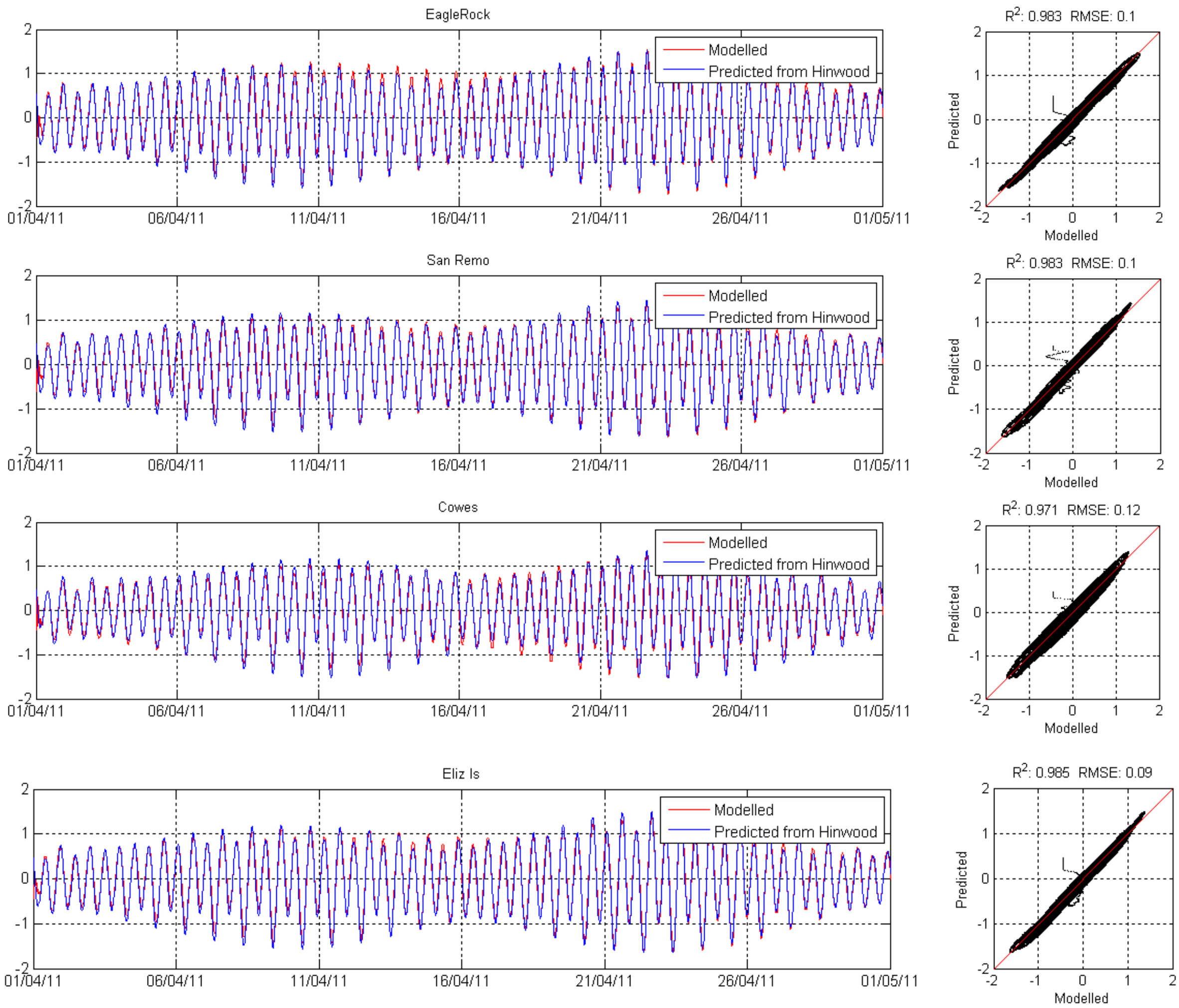


Figure 1-5 **Time Series Comparison of Modelled and Predicted (ANTT and Hinwood & Jones, 1979) Astronomical Tidal Water Levels**

Table 1-1 **Comparisons of Observed and Modelled Astronomical Tidal Constituent's Amplitude and Phase**

Location	Constituent	Hinwood & Jones (1979) or ANTT* (m) / (°)	Modelled (m) / (°)	Difference (m) / (°)
Stony Point (ANTT 2013)*	M2	0.89 / 352.1	0.88 / 352.79	-0.01 / 0.69
	S2	0.22 / 137.6	0.22 / 137.44	0.00 / -0.16
	K1	0.22 / 76.9	0.21 / 80.14	0.01 / 3.2
	O1	0.15 / 43	0.14 /40.28	-0.01 / -2.72
	N2	0.17 / 310.7	0.19 / 311.3	0.02 / -0.83
Tooradin	M2	0.987 / 17	1.00 / 13.8	0.01 / -3.23
	S2	0.263 / 178	0.23 / 166.8	-0.03 / -11.2

	K1	0.22 / 100	0.19 / 91.3	-0.03 / -8.7
	O1	0.142 / 46	0.12 / 49.3	-0.02 / 3.3
	N2	0.188 / 340	0.19 / 336.8	0.002 / -3.2
Flinders	M2	0.804 / 326	0.751 / 331.6	-0.05 / 5.6
	S2	0.209 / 88	0.197 / 111.4	-0.01 / 23.4
	K1	0.227 / 60	0.206 / 70.8	0.02 / 10.8
	O1	0.148 / 34	0.139 / 317.5	-0.008 / -2.25
	N2	0.170 / 291	0.176 / 286.7	0.006 / -4.3
San Remo	M2	0.926 / 352	0.893 / 355.2	-0.03 / 3.2
	S2	0.223 / 118	0.222 / 140.5	-0.001 / 22.5
	K1	0.232 / 63	0.213 / 81.4	-0.02 / 18.4
	O1	0.157 / 46	0.142 / 41.4	-0.015 / -4.6
	N2	0.177 / 321	0.187 / 312.0	0.01 / -9

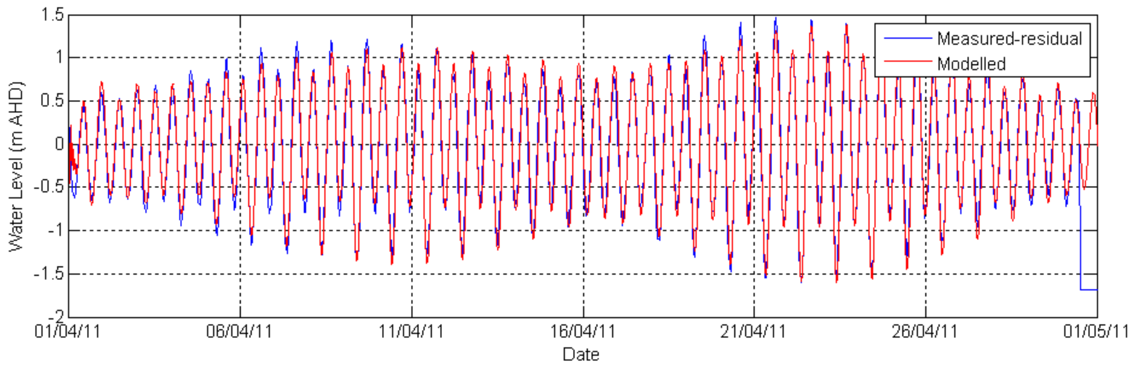


Figure 1-6 **Time Series Comparison of Modelled and Observed (Measured-residual) Astronomical Tidal Water Levels at the Stony Point Tide Gauge**

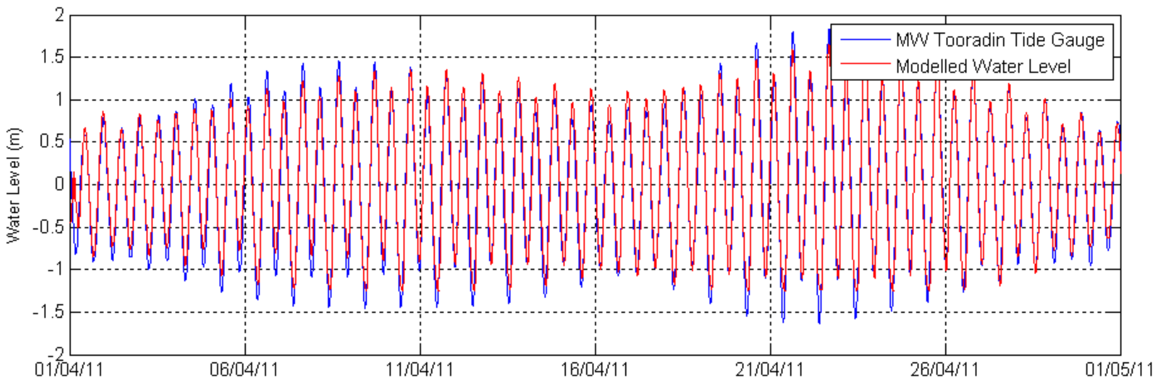


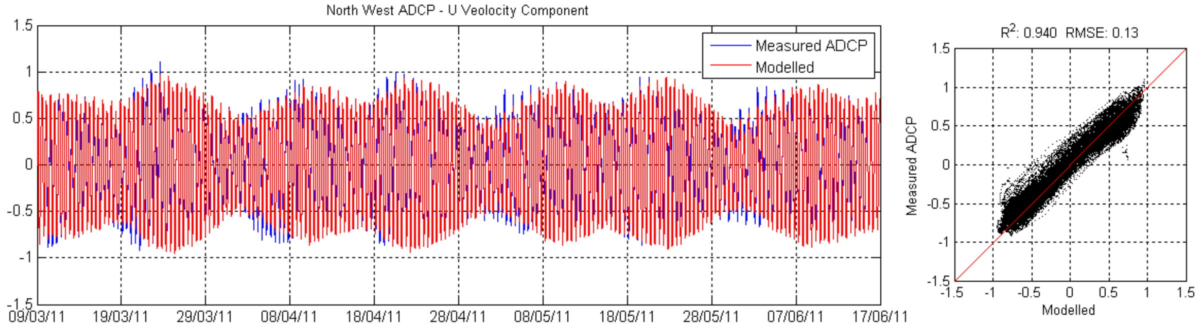
Figure 1-7 **Time Series Comparison of Modelled and Observed (Measured-residual) Astronomical Tidal Water Levels at the Tooradin Tide Gauge**

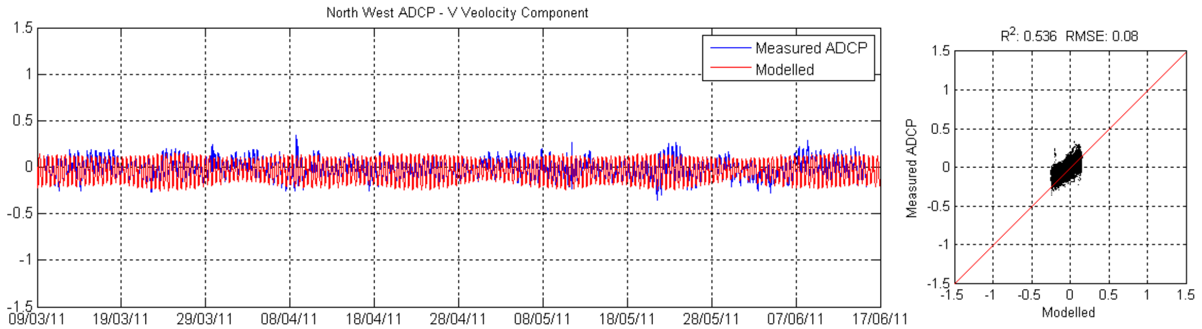
1.3.2 ADCP Data Collected March to May 2011

Four ADCP instruments were deployed by the EPA over March to May 2011, providing current velocities at 50cm vertical increments throughout the water column. Current velocities were converted to depth averaged velocities to allow for direct comparisons between the measured ADCP and the 2D model results. Analysis of the South (near San Remo) ADCP raw data indicated something had gone wrong with the ADCP deployment, and therefore current velocities and water levels from the South ADCP were not used during the model calibration. The locations of the three ADCP datasets used are shown in Figure 1-8

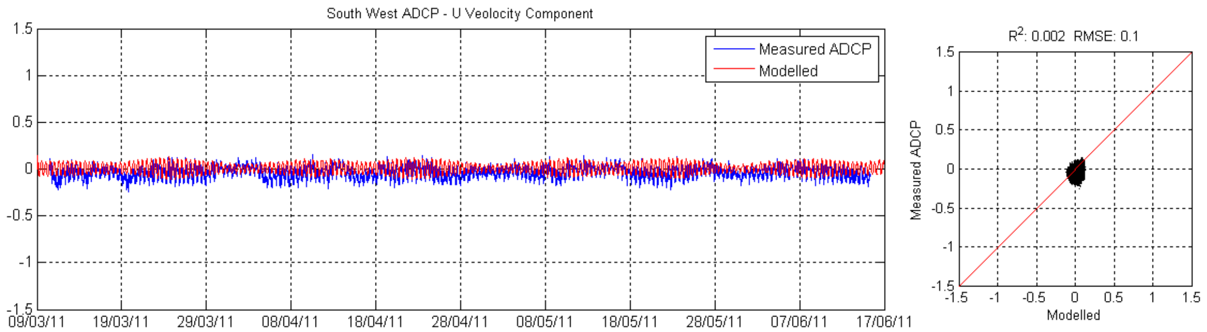
Figure 1-8 Location of ADCP Data Monitoring Locations

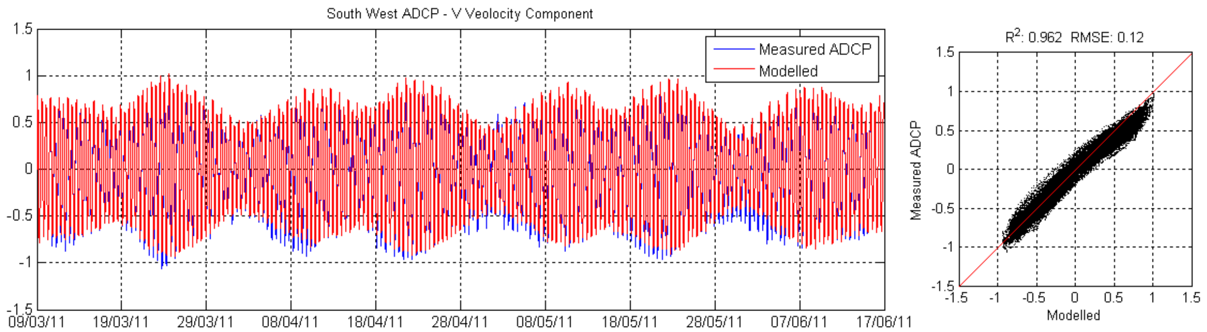
North West ADCP



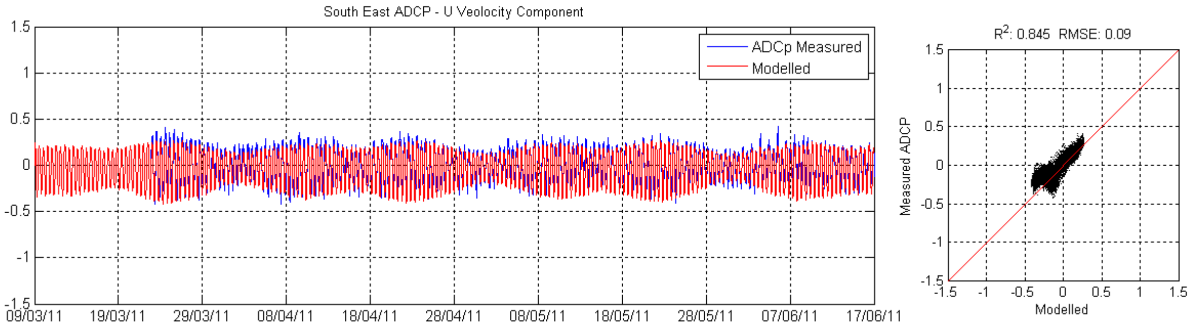


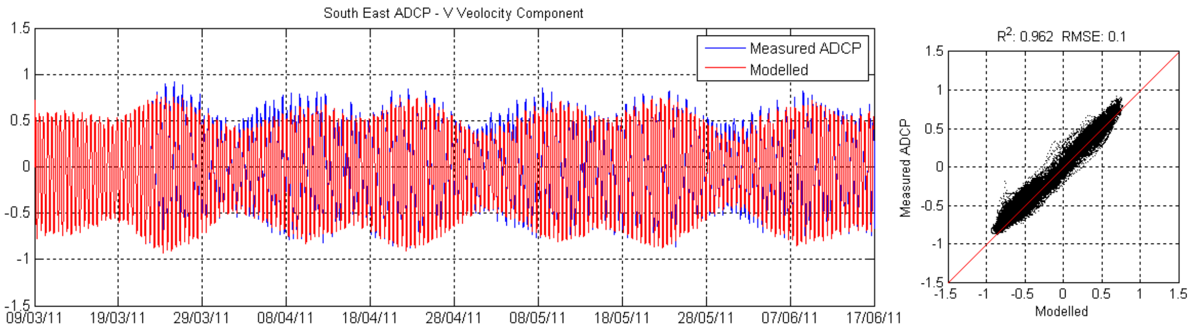
South West ADCP





South East ADCP





1.3.3 ADCP Transects 14th – 15th June 2011

Western Arm Discharge

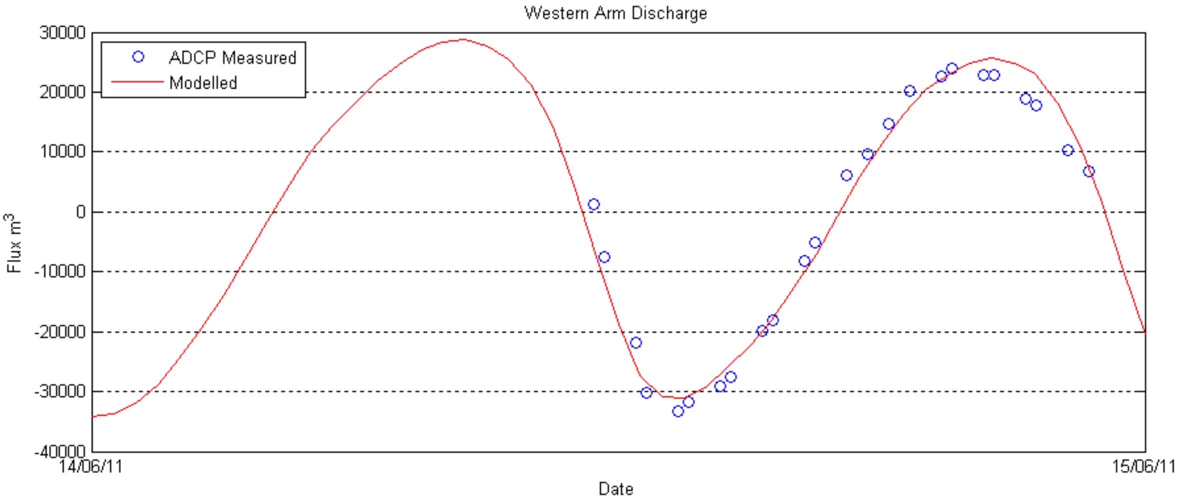


Figure 1-9 Comparison of Measured and Modelled Water Flux through the Western Arm over a Tidal Cycle

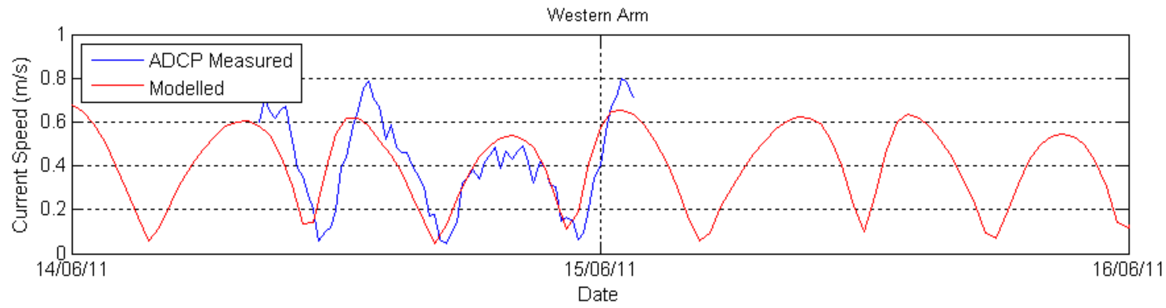


Figure 1-10 Comparison of Measured and Modelled Cross-section Averaged Current Speeds through the Western Arm over Two Tidal Cycles

Eastern Arm Discharge

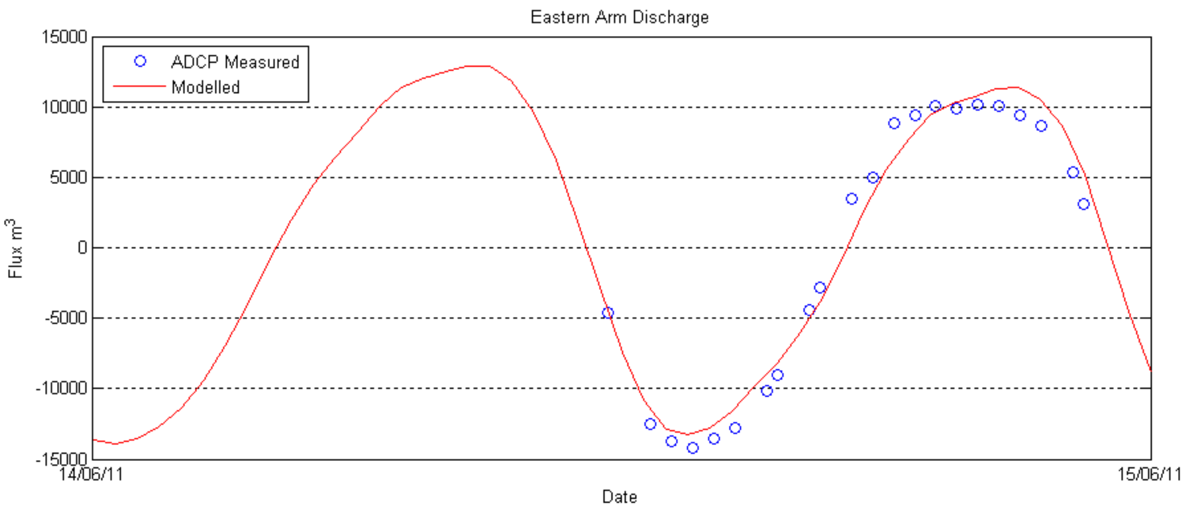


Figure 1-11 Comparison of Measured and Modelled Water Flux through the Western Arm over One Tidal Cycle

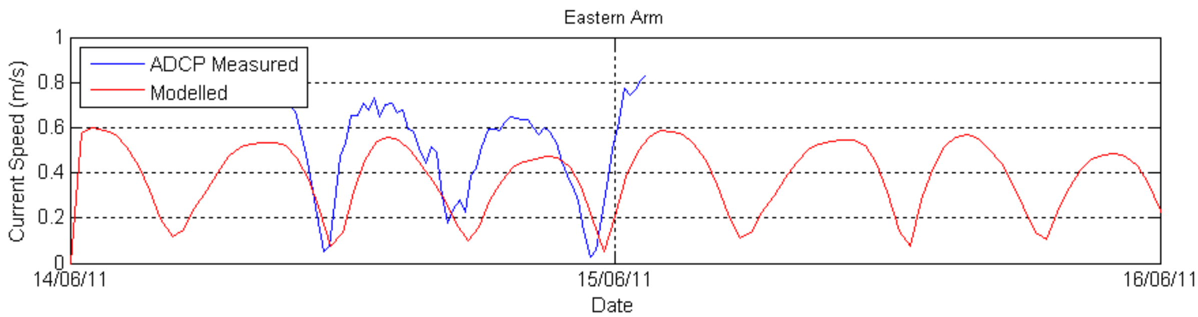


Figure 1-12 Comparison of Measured and Modelled Cross-section Averaged Current Speeds through the Eastern Arm over Two Tidal Cycles

1.3.4 2003 Simulation

The hydrodynamic model was also calibrated over one full year, which included a range of forcing phenomena such as storm surge and wind set up. This was achieved through comparisons of decomposed gauged and modelled water levels at Stony Point. The model was shown to successfully reproduce the astronomical tidal component of water levels to an r^2 value of 0.982 and the storm surge component to an r^2 value of 0.780 as shown in Figure 1-13. These results demonstrated the models ability to simulate the propagation of non-tidal water level variations from Bass Strait, into Western Port and water level variations due to wind forcings.

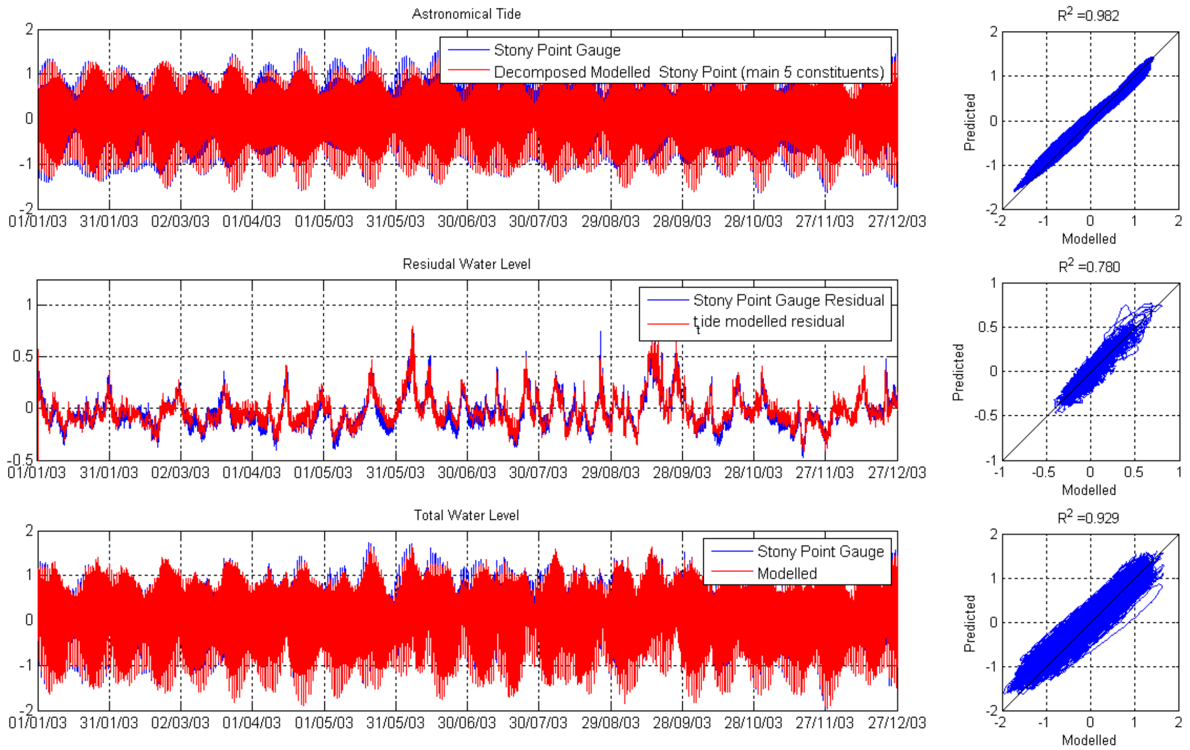


Figure 1-13 2003 Calibration Simulation – Stony Point Tide Gauge vs. Modelled Water Levels

APPENDIX C CATCHMENT HYDROLOGY

CATCHMENT HYDROLOGY

1. FLOOD FREQUENCY ANALYSIS

Annual series for the six gauged catchments are given in Table 1-1 to Table 1-6. The annual maximum flows were sourced from:

maximum flows were sourced from:

- The "Blue Book", the Rural Water Commission of Victoria's *Victorian Surface Water Information to 1987* - referred to as "BB" in tables
- The Victorian Water Resources Data Warehouse (www.vicwaterdata.net) – referred to as "Warehouse" in tables
- Melbourne Water 6 minute instantaneous flow data – referred to as "MW" in tables

For the Lang Lang catchment, two gauges in the lower catchment were available. A regression relationship was developed between daily maximum flows at the two gauges (Figure 1-1) and used to fill in a gap in the record for 228209 Lang Lang @ Hamiltons Bridge.

Table 1-1 Annual Series for 228213 Bunyip @ Iona

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1962	69	BB (incomplete year)	1988	45	MW
1963	40	BB	1989	74	MW
1964	40	BB	1990	166	MW
1965	31	BB	1991	52	MW
1966	43	BB	1992	52	MW
1967	7	BB	1993	113	MW
1968	35	BB	1994	33	MW
1969	48	BB	1995	64	MW
1970	64	BB	1996	145	MW
1971	240	MW	1997	9	MW
1972	22	MW	1998	24	MW
1973	36	MW	1999	30	MW
1974	87	MW	2000	26	MW
1975	55	MW	2001	25	MW
1976	51	MW	2002	6	MW
1977	65	MW	2003	37	MW
1978	64	MW	2004	114	MW
1979	46	MW	2005	52	MW
1980	28	MW	2006	5	MW
1981	71	MW	2007	2	MW
1982	14	MW	2008	10	MW
1983	52	MW	2009	44	MW
1984	145	MW	2010	39	MW
1985	82	MW	2011	218	MW
1986	47	MW	2012	5	MW

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1987	64	MW			

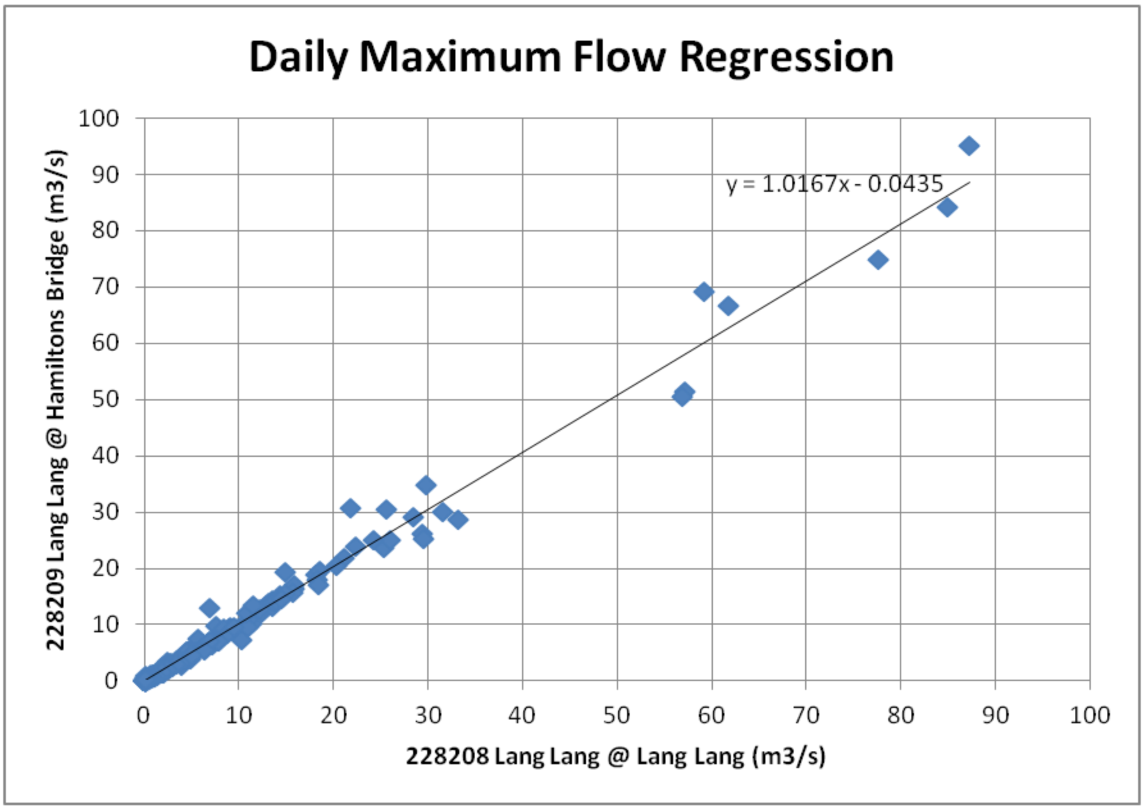


Figure 1-1 **Regression between daily maximum instantaneous flow at 228209 Lang Lang @ Hamiltons Bridge and 228208 Lang Lang @ Lang Lang, Feb 1980 to Dec 1981.**

Table 1-2 **Annual Series for 228209 Lang Lang @ Hamiltons Bridge**

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1960	73	BB (incomplete year)	1992	44	MW
1961	37	BB	1993	48	MW
1962	58	BB	1994	45	MW
1963	21	BB	1995	40	MW
1964	53	BB (incomplete year)	1996	297	MW
1975	71	Regression from 228208 (70 m^3/s)	1997	16	MW
1976	57	Regression from 228208 (56 m^3/s)	1998	16	MW
1977	77	Regression from 228208 (76 m^3/s)	1999	16	MW
1978	40	Regression from 228208 (39 m^3/s)	2000	27	MW
1979	27	Regression from 228208 (27 m^3/s)	2001	38	MW

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1980	87	MW	2002	16	MW
1981	49	MW	2003	24	MW
1982	15	MW	2004	57	MW
1983	67	MW	2005	22	MW
1984	63	MW	2006	8	MW
1985	57	MW	2007	3	MW
1986	44	MW	2008	22	MW
1987	31	MW	2009	12	MW
1988	95	MW	2010	32	MW
1989	68	MW	2011	59	MW
1990	127	MW	2012	147	MW
1991	90	MW			

Table 1-3 Annual Series for 227231 Bass @ Glen Forbes South

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1973	32	Warehouse	1993	54	Warehouse
1974	50	Warehouse	1994	41	Warehouse
1975	47	Warehouse	1995	43	Warehouse
1976	50	Warehouse	1996	69	Warehouse
1977	64	Warehouse	1997	15	Warehouse
1978	38	Warehouse	1998	11	Warehouse
1979	30	Warehouse	1999	21	Warehouse
1980	75	Warehouse	2000	25	Warehouse
1981	45	Warehouse	2001	39	Warehouse
1982	20	Warehouse	2002	22	Warehouse
1983	55	Warehouse	2003	26	Warehouse
1984	70	Warehouse	2004	46	Warehouse
1985	52	Warehouse	2005	37	Warehouse
1986	44	Warehouse	2006	4	Warehouse
1987	37	Warehouse	2007	20	Warehouse
1988	52	Warehouse	2008	30	Warehouse
1989	48	Warehouse	2009	34	Warehouse
1990	66	Warehouse	2010	27	Warehouse
1991	59	Warehouse	2011	51	Warehouse
1992	37	Warehouse	2012	65	Warehouse

Table 1-4 Annual Series for 228225 Yallock @ Cora Lynn

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1966	3	BB	1990	70	MW
1967	0	BB	1991	51	MW
1968	8	BB (incomplete year)	1992	36	MW
1969	24	BB	1993	91	MW
1970	35	BB	1994	28	MW
1971	152	BB (incomplete year)	1995	1	MW
1972	0	BB	1996	75	MW
1973	0	BB	1997	0	MW
1974	57	BB	1998	0	MW
1975	19	BB (incomplete year)	1999	0	MW
1976	23	BB	2000	0	MW
1978	33	BB (incomplete year)	2001	1	MW
1979	23	BB	2002	0	MW
1980	19	BB	2003	8	MW
1981	35	BB	2004	25	MW
1982	6	MW	2005	13	MW
1983	31	MW	2006	0	MW
1984	115	MW	2007	2	MW
1985	42	MW	2008	2	MW
1986	22	MW	2009	7	MW
1987	32	MW	2010	9	MW
1988	28	MW	2011	99	MW
1989	31	MW	2012	53	MW

Table 1-5 Annual Series for 228228 Cardinia @ Cardinia

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1974	75	MW	1994	3	MW
1975	16	MW	1995	8	MW
1976	6	MW	1996	34	MW
1977	13	MW	1997	2	MW
1978	26	MW	1998	2	MW
1979	6	MW	1999	4	MW
1980	13	MW	2000	6	MW
1981	14	MW	2001	3	MW
1982	3	MW	2002	2	MW
1983	26	MW	2003	4	MW
1984	46	MW	2004	27	MW
1985	7	MW	2005	28	MW
1986	12	MW	2006	2	MW
1987	32	MW	2007	6	MW

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1988	5	MW	2008	2	MW
1989	26	MW	2009	5	MW
1990	41	MW	2010	17	MW
1991	14	MW	2011	58	MW
1992	13	MW	2012	27	MW
1993	23	MW			

Table 1-6 Annual Series for 228217 Toomuc @ Pakenham

Year	Annual Maximum Flow (m3/s)	Source	Year	Annual Maximum Flow (m3/s)	Source
1964	10	BB (incomplete year)	1989	20	MW
1965	9	BB (incomplete year)	1990	31	MW
1966	6	BB	1991	12	MW
1967	1	BB	1992	16	MW
1968	4	BB	1993	16	MW
1969	5	BB	1994	5	MW
1970	14	BB	1995	16	MW
1971	24	BB	1996	27	MW
1972	5	BB	1997	1	MW
1973	7	BB (incomplete year)	1998	8	MW
1974	19	BB	1999	9	MW
1975	10	Warehouse	2000	10	MW
1976	7	Warehouse	2001	7	MW
1977	-	No data	2002	0	MW
1978	13	MW	2003	4	MW
1979	4	MW	2004	26	MW
1980	7	MW	2005	31	MW
1981	6	MW	2006	1	MW
1982	1	MW	2007	8	MW
1983	13	MW	2008	2	MW
1984	34	MW	2009	6	MW
1985	20	MW	2010	13	MW
1986	8	MW	2011	45	MW
1987	21	MW	2012	15	MW
1988	14	MW			

The flood frequency analysis was undertaken using the hydrologic software program FLIKE, which adopts a Bayesian inference approach to parameter fitting. FLIKE also has the flexibility of fitting five different probability distributions to select the most appropriate distribution for a particular flood series.

The results of the flood frequency analysis are given in Table 1-7. All five available distributions were fitted to each annual series, and the distribution that gave the best fit was selected. For Yallock

Creek eleven years were deemed to be “non-flood” years and those points were excluded from the analysis. . The e-water CRC Flood Frequency Analysis spreadsheet was used for the Yallock Creek analysis instead of FLIKE, which does not handle excluded low flows well. Graphs of the fitted distributions are given in Figure 1-2 to Figure 1-7.

Table 1-7 Flood Frequency Analysis Results

Catchment	Gauge	Years of Record	Distribution	Q10 (m^3/s)	90% limits (m^3/s)	
Bunyip	228213 Bunyip @ Iona	51	Generalised Extreme Value	122	97	163
Lang Lang	228209 Lang Lang @ Hamiltons Bridge	43	Generalised Extreme Value	109	85	150
Bass	227231 Bass @ Glen Forbes South	40	Generalised Extreme Value	65	59	72
Yallock	228225 Yallock @ Cora Lynn	46	Log Pearson III (with 11 low flows excluded)	76	53	108
Cardinia	228228 Cardinia @ Cardinia	39	Log Normal	41	29	64
Toomuc	228217 Toomuc @ Pakenham	48	Log Pearson III	27	22	34

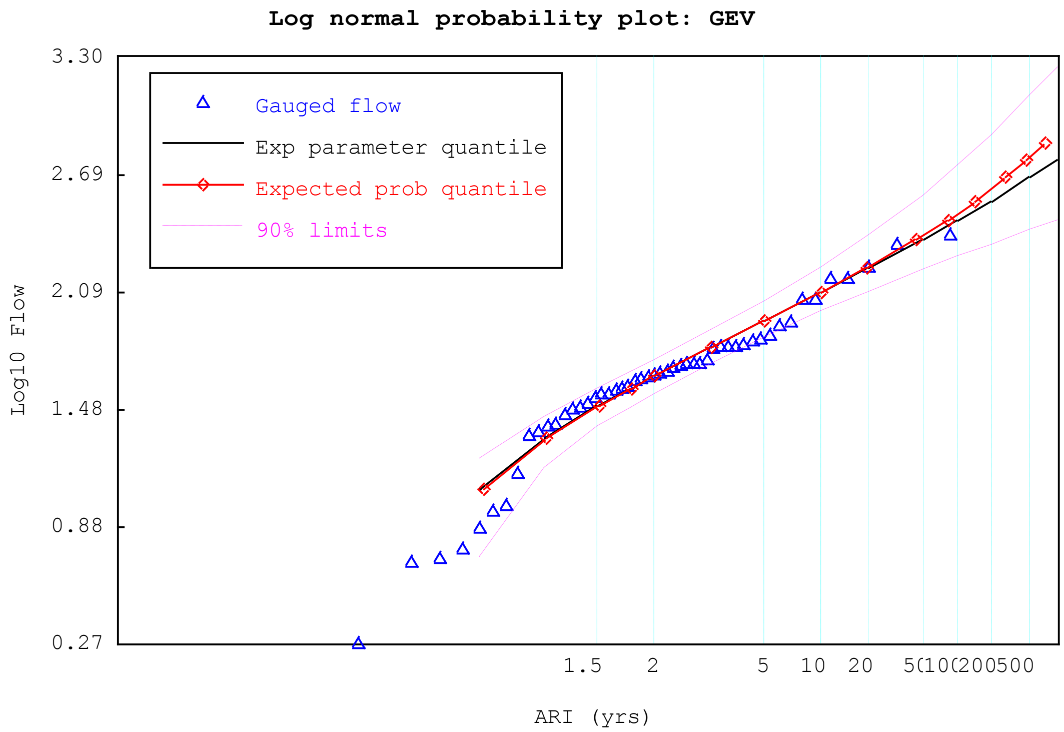


Figure 1-2 Fitted Generalised Extreme Value distribution for annual series at 228213 Bunyip @ Iona

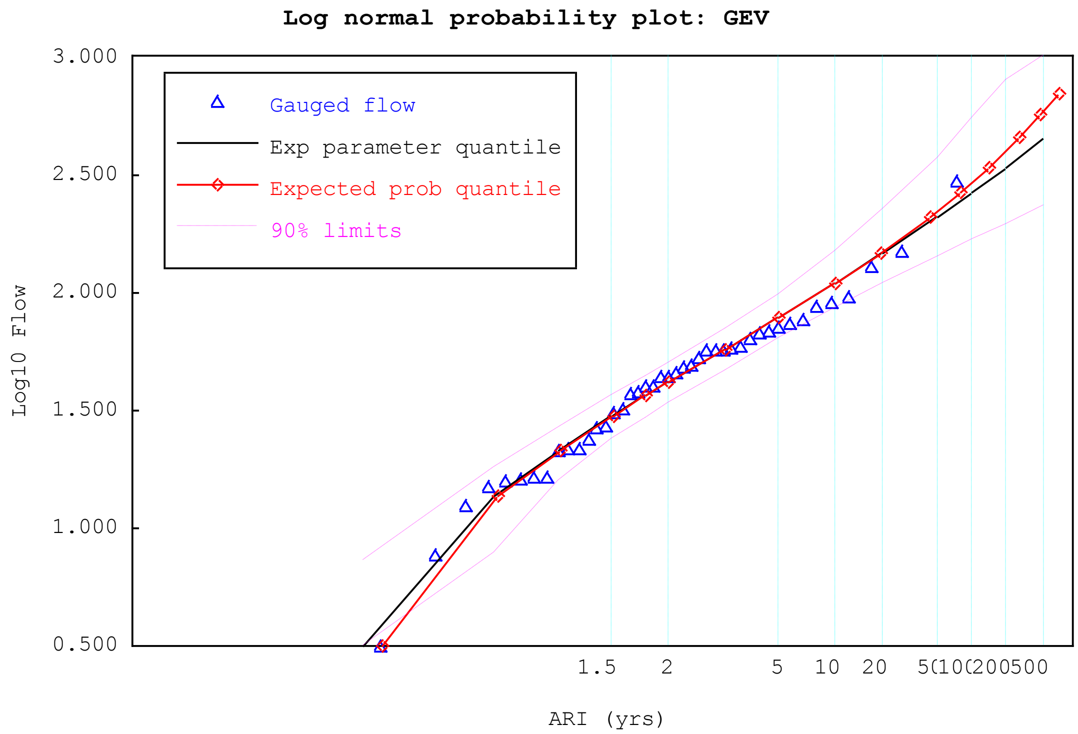


Figure 1-3 **Fitted Generalised Extreme Value distribution for annual series at 228209 Lang Lang @ Hamiltons Bridge**

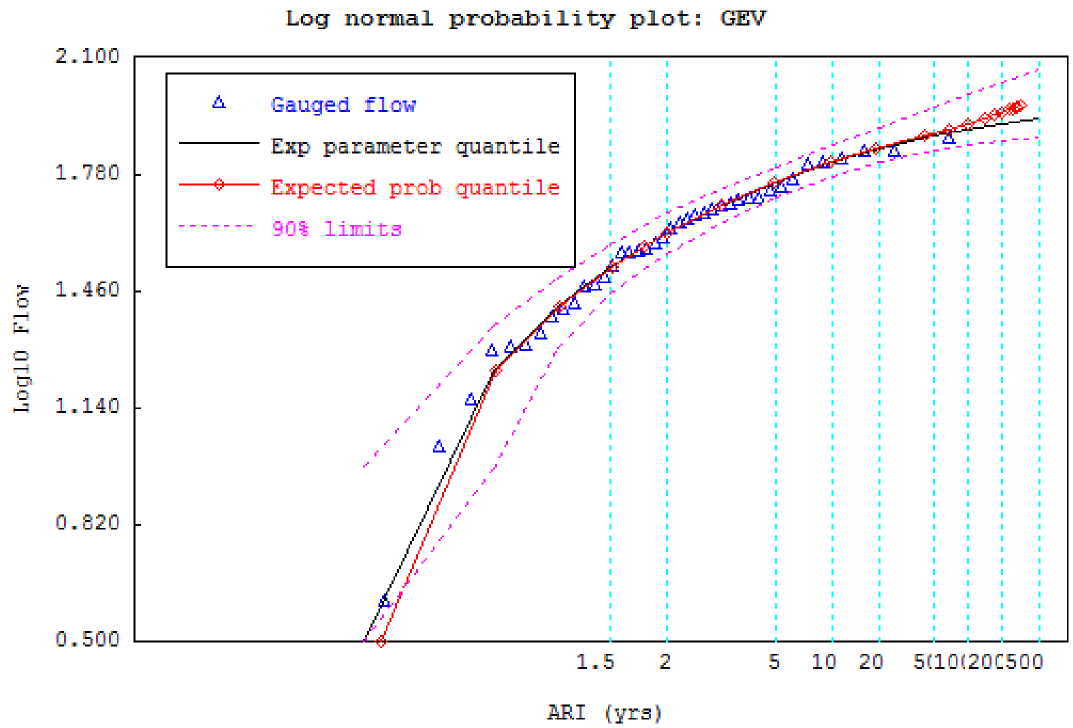


Figure 1-4 **Fitted Generalised Extreme Value distribution for annual series at 227231 Bass @ Glen Forbes South**

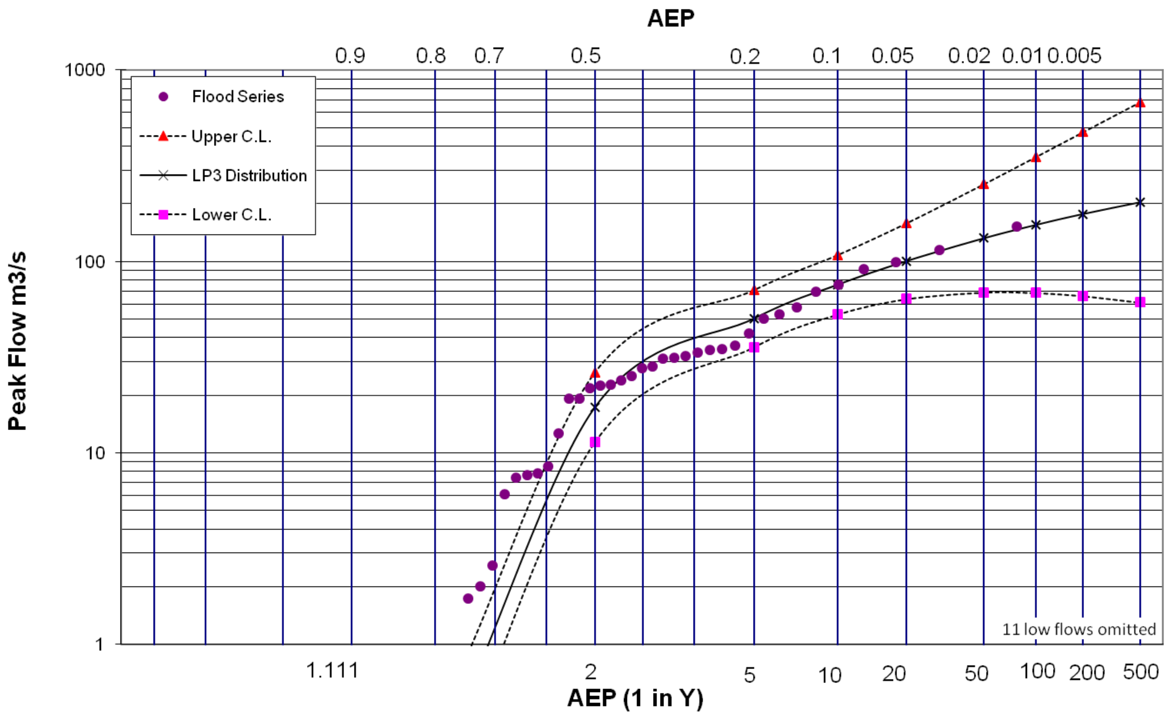


Figure 1-5 **Fitted Log Pearson III distribution for annual series at 228225 Yallock @ Cora Lynn with 11 low flows excluded**

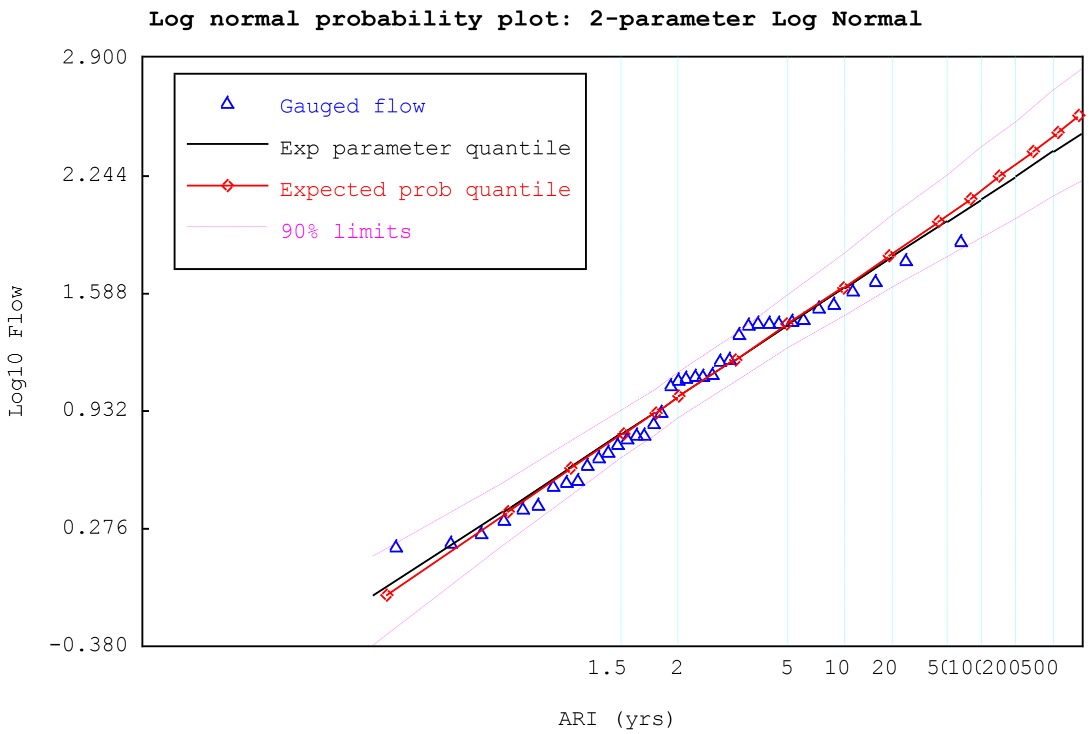


Figure 1-6 **Fitted Log Normal distribution for annual series at 228228 Cardinia @ Cardinia**

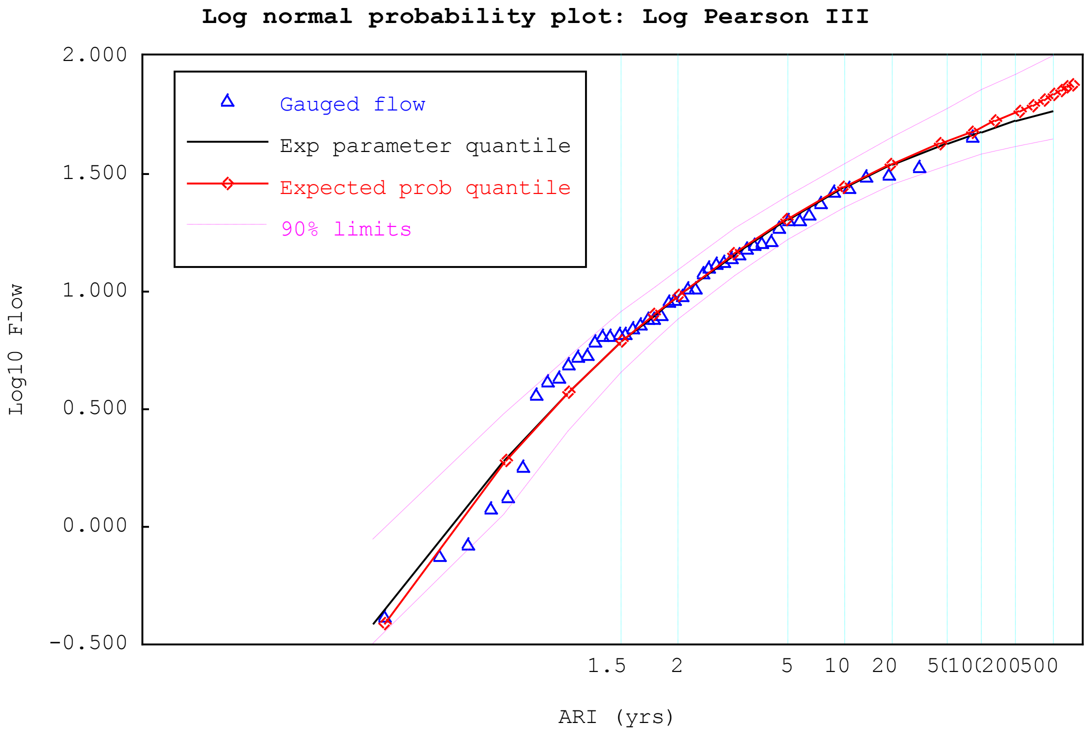


Figure 1-7 Fitted Log Pearson III distribution for annual series at 228217 Toomuc @ Pakenham

2. RATIONAL METHOD

The Probabilistic Rational Method was used to provide estimates of 10% AEP peak flow for each of the catchments. The new C10 values and frequency factors from Rahman et al (2009) were adopted, and Adams (1987) method for computing time of concentration was used. The resulting peak flow estimates are given in Table 2-1.

Table 2-1 Rational Method 10% AEP Peak Flow Estimates

Catchment	Area	Time of concentration	c10	I 10	Q 10
	Km^2	hr		mm/hr	m3/s
Bunyip	979	10.4	0.13	5.9	208
Lang Lang	429	7.6	0.13	7.2	111
Bass	281	6.5	0.13	8.0	81
Yallock	266	6.4	0.13	8.1	77
Cardinia	176	5.4	0.13	8.9	57
Toomuc and Deep	143	5.0	0.13	9.4	48
Tooradin Inlet	59	3.6	0.13	11.6	25

3. RORB MODELLING

Hydrographs for each of the catchment were calculated using RORB models provided by Melbourne Water or from previous work by Water Technology. The RORB models were not calibrated but parameters were set using Melbourne Water standard methods. The resulting peak flows were compared to the flood frequency analysis and rational method, and if necessary, the parameters were adjusted to reconcile the RORB peak flow.

3.1 Bunyip

A RORB model of the Yallock drain system and the Bunyip/Tarago system upstream of Cora Lynn was provided by Melbourne Water. However the model did not include the catchment of the Kooweerup Flood Protection District. The model was extended to include these areas.

Eight additional subareas were added to the model, with a total additional area of 244 km^2. All additional subareas were assigned an impervious fraction of 0.05 to represent the rural nature of the additional area. The d_{av} of the model was reduced from 34.39 km in the original model to 31.37 in the extended model. The kc value from the original model (68 adopted for this study – see Section 3.4) was scaled down by the ratio of the d_{av} values, to 62. The initial loss was increased to 15 mm, as discussed in Section 3.4. The Runoff Coefficient was lowered to 0.35 as recommended by Melbourne Water for modelling of 10% AEP flows. The other parameters and settings from the original model were adopted:

- IFD parameters for Koo Wee Rup
- Flitered temporal patterns
- Uniform areal patterns
- ARR87 Bk II (Figs 1.6 and 1.7) Areal Reduction Factors
- Constant losses

The structure of the extended part of the model is shown in Figure 3-1.

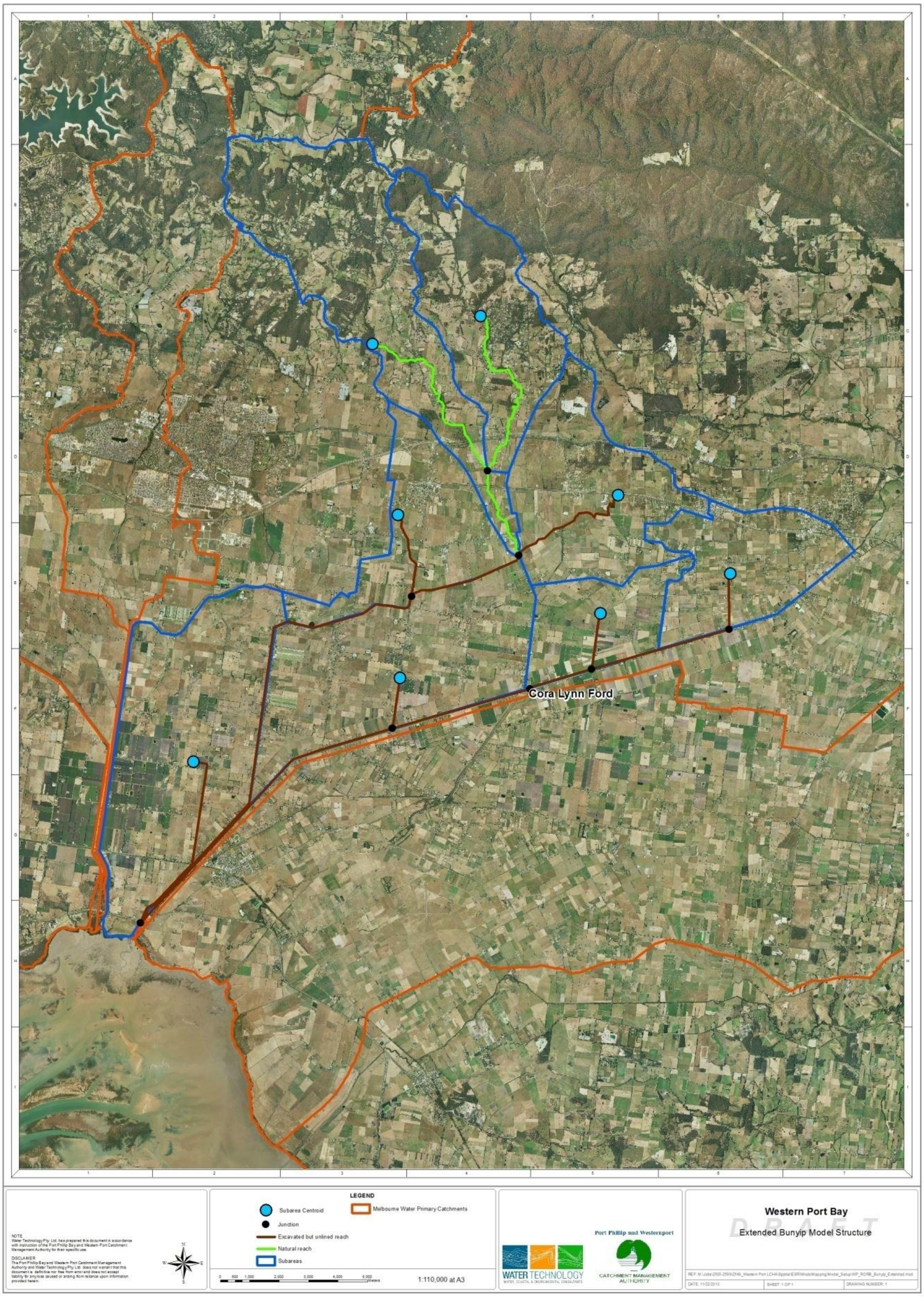


Figure 3-1 Extended Bunyip RORB Model Structure

The resulting peak flows for Bunyip drain at Iona (Table 3-1) were consistent (within the 90% confidence limits) with the flood frequency analysis. The peak flow at the Bunyip outfall was 50% lower than the rational method estimate, however the rational method did not account for the transfer of flow to the Yallock catchment at the Cora Lynn ford. The hydrograph at the outfall is given in Figure 3-5.

Table 3-1 RORB model 10% AEP Design Flows for Bunyip River

Bunyip @ Iona	
kc	62
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	158
Critical Storm (hrs)	48
Bunyip outfall	
kc	62
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	103
Critical Storm (hrs)	48

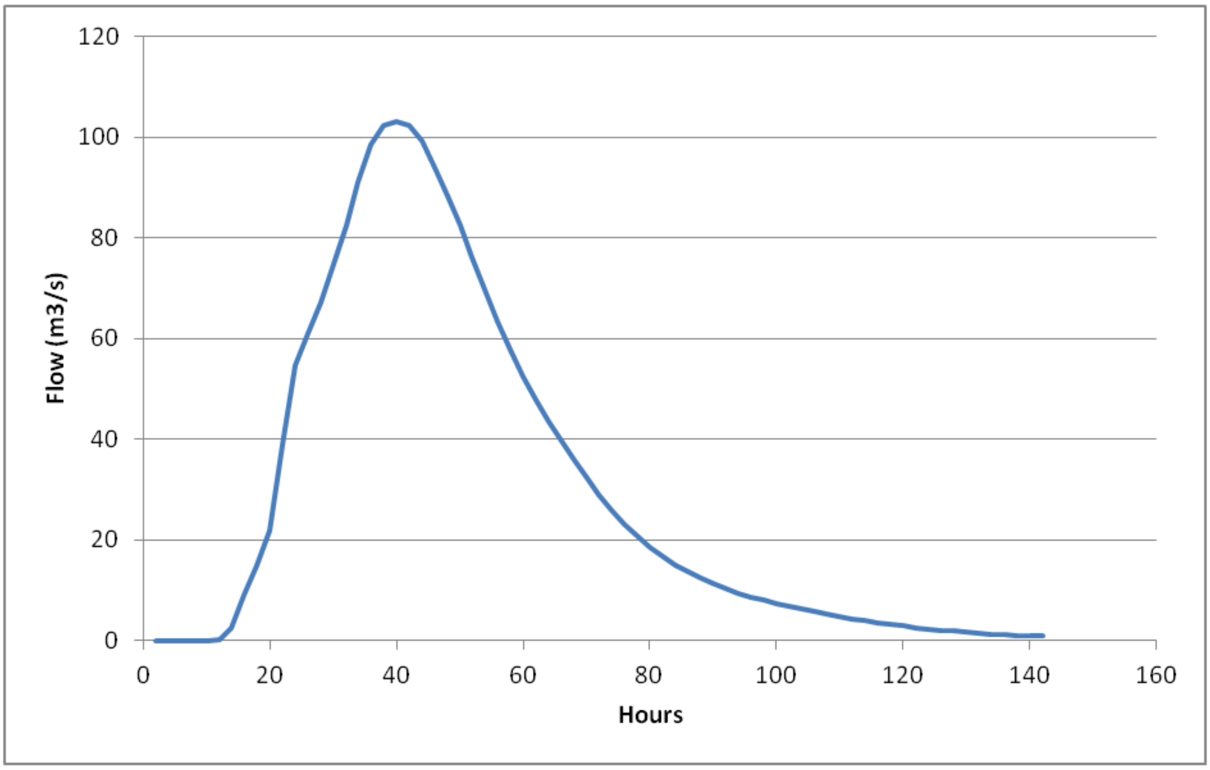


Figure 3-2 Bunyip River 10% AEP Hydrograph

3.2 Lang Lang River

The model “Lang60.catg” provided by Melbourne Water was used. Parameters were not provided by Melbourne Water. Kc was set using the Melbourne Water regional equation $kc = 1.53A^{0.55}$, and m = 0.8 was adopted. An initial loss of 15 mm and Runoff Coefficient of 0.35 (10% AEP) were adopted. The following settings were also applied:

- IFD parameters obtained from the Bureau of Meteorology for the catchment centroid
- Flitered temporal patterns
- Uniform areal patterns
- Siriwardena and Weinmann Areal Reduction Factors
- Constant losses

The resulting flows at the Hamiltons Bridge gauge and the outfall are given in Table 3-2. The calculated 10% AEP flow at Hamiltons Bridge is consistent with the flood frequency analysis estimate. The flow at the outfall is 77% higher than the rational method estimate. The peak flow at the outlet was accepted as the model displayed a good fit to the flood frequency analysis and a reasonable fit to the rational method. The hydrograph is shown in Figure 3-3.

Table 3-2 RORB model 10% AEP Design Flows for Lang Lang River

Lang Lang River at Hamiltons Bridge	
kc	42.9
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	112
Critical Storm (hrs)	24
Lang Lang River Outfall	
kc	42.9
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	196
Critical Storm (hrs)	24

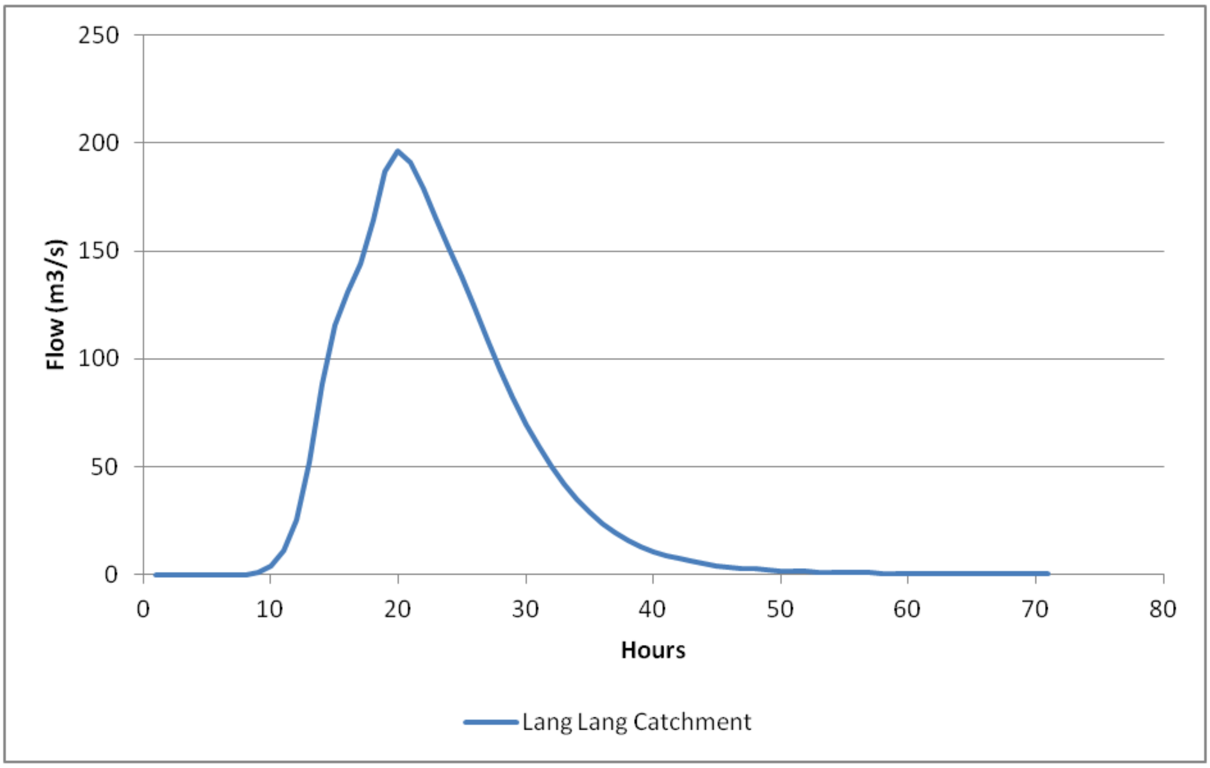


Figure 3-3 Lang Lang River 10% AEP Hydrograph

3.3 Bass River

The model "Bass River Catchment File.cat" provided by Melbourne Water was used. Parameters, settings and IFD data were provided with the model. Kc was set to 17.93 using the ARR method. The parameters m = 0.8, IL = 15 mm and RoC = 0.60 (1% AEP) were also provided. The runoff coefficient was lowered to 0.35 as recommended by Melbourne Water for modelling of 10% AEP flows. The following settings were also applied:

- IFD parameters for Loch provided in .cat file
- Flitered temporal patterns
- Uniform areal patterns
- ARR87 Bk II (Figs 1.6 and 1.7) Areal Reduction Factors
- Constant losses

Upon initially running the model, the resulting peak flow at Glen Forbes South was much higher than the flood frequency analysis estimate, and was outside the 90% confidence limits. Kc was adjusted to reconcile the peak flow to the flood frequency analysis. Kc was set to 54.97, based on the regional equation for Pearse et al. (2002) for Victoria. The resulting peak flows (Table 3-3) agreed well with the flood frequency analysis and the rational method. The hydrograph is shown in Figure 3-4.

Table 3-3 RORB model 10% AEP Design Flows for Bass River

Bass River at Glen Forbes South	
kc	54.97
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	67
Critical Storm (hrs)	36
Bass River Outfall	
kc	54.97
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	63
Critical Storm (hrs)	48

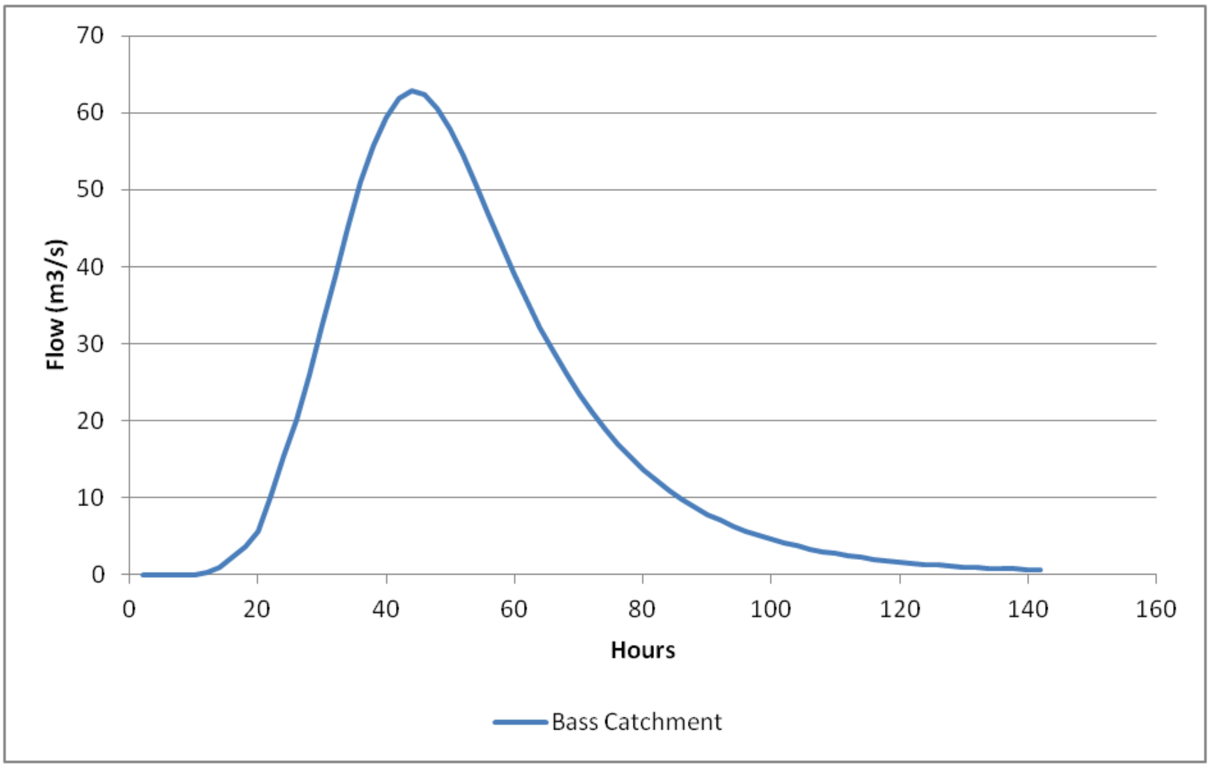


Figure 3-4 Bass River 10% AEP Hydrograph

3.4 Yallock

A RORB model of the Yallock drain system and the Bunyip/Tarago system upstream of Cora Lynn ("RORB_OMahoney.cat") was provided by Melbourne Water. The Runoff Coefficient was lowered to 0.35 as recommended by Melbourne Water for modelling of 10% AEP flows. The following parameters and settings were also adopted:

- IFD parameters for Koo Wee Rup
- Flitered temporal patterns
- Uniform areal patterns
- ARR87 Bk II (Figs 1.6 and 1.7) Areal Reduction Factors
- Constant losses

The parameter set provided by Melbourne Water produced peak flows in the Bunyip drain at Iona that were outside the 90% confidence limits of the flood frequency analysis. To reconcile the peak flows, the initial loss was raised from 10 m mot 15 mm, and the kc value was increased slightly from 66.72 to 68.

The resulting peak flows for Bunyip drain at Iona and Yallock drain at Cora Lynn (Table 3-4) were consistent (within the 90% confidence limits) with the flood frequency analysis. The peak flow at the Yallock outfall was 104% higher than the rational method estimate, however the rational method did not account for the transfer of flow from the Bunyip catchment at the Cora Lynn ford, which makes a significant contribution to the Yallock system under high flows. The hydrograph at the outfall is given in Figure 3-5.

Table 3-4 RORB model 10% AEP Design Flows for Yallock drain

Bunyip @ Iona	
kc	68
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	158
Critical Storm (hrs)	48
Yallock @ Cora Lynn	
kc	68
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	82
Critical Storm (hrs)	36
Yallock Outfall	
kc	68
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	145
Critical Storm (hrs)	36

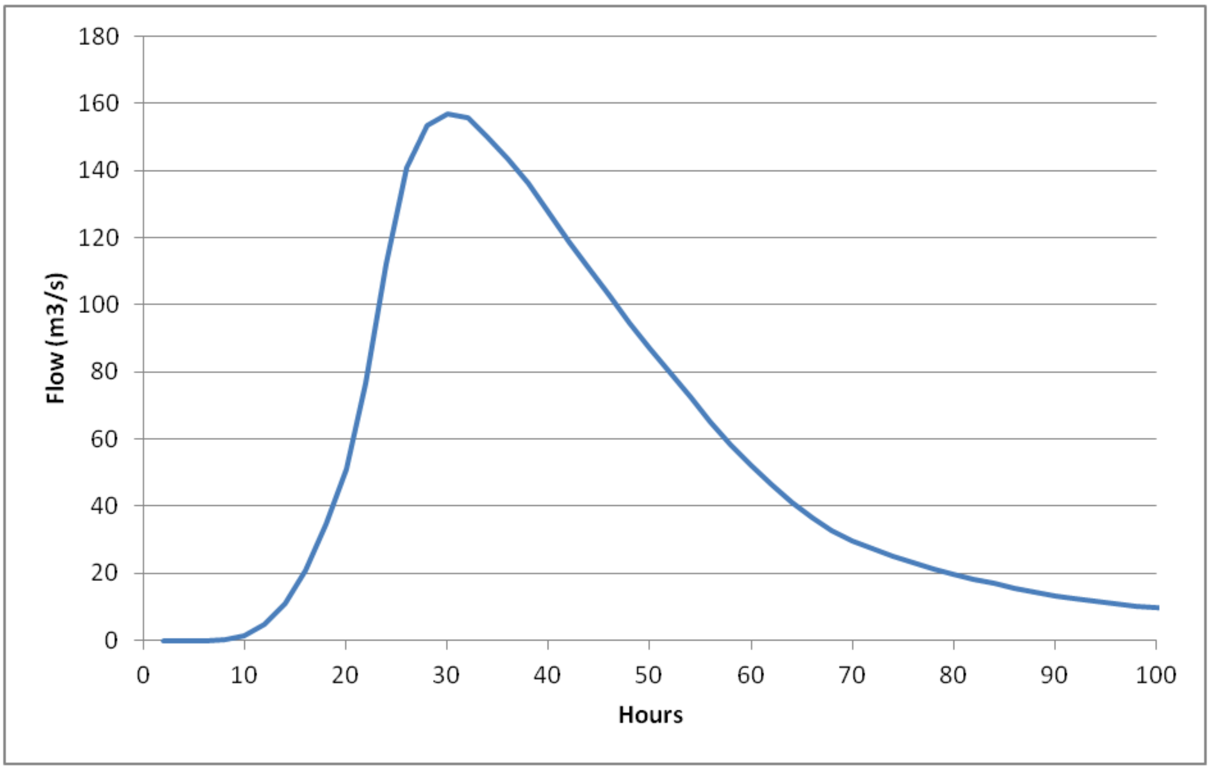


Figure 3-5 Yallock Creek 10% AEP Hydrograph

3.5 Cardinia

The model "CARD_ORD_2010.CAT" provided by Melbourne Water was used. Parameters and settings provided with the model were adopted:

- IFD parameters for Pakenham
- Flitered temporal patterns
- Uniform areal patterns
- ARR87 Bk II (Figs 1.6 and 1.7) Areal Reduction Factors
- Constant losses

The resulting peak flows (Table 3-5) were consistent with the flood frequency analysis and the rational method. The hydrograph is shown in Figure 3-6.

Table 3-5 RORB model 10% AEP Design Flows for Cardinia Creek

Cardinia Creek	
kc	20.2
m	0.8
IL (mm)	10
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	47
Critical Storm (hrs)	9

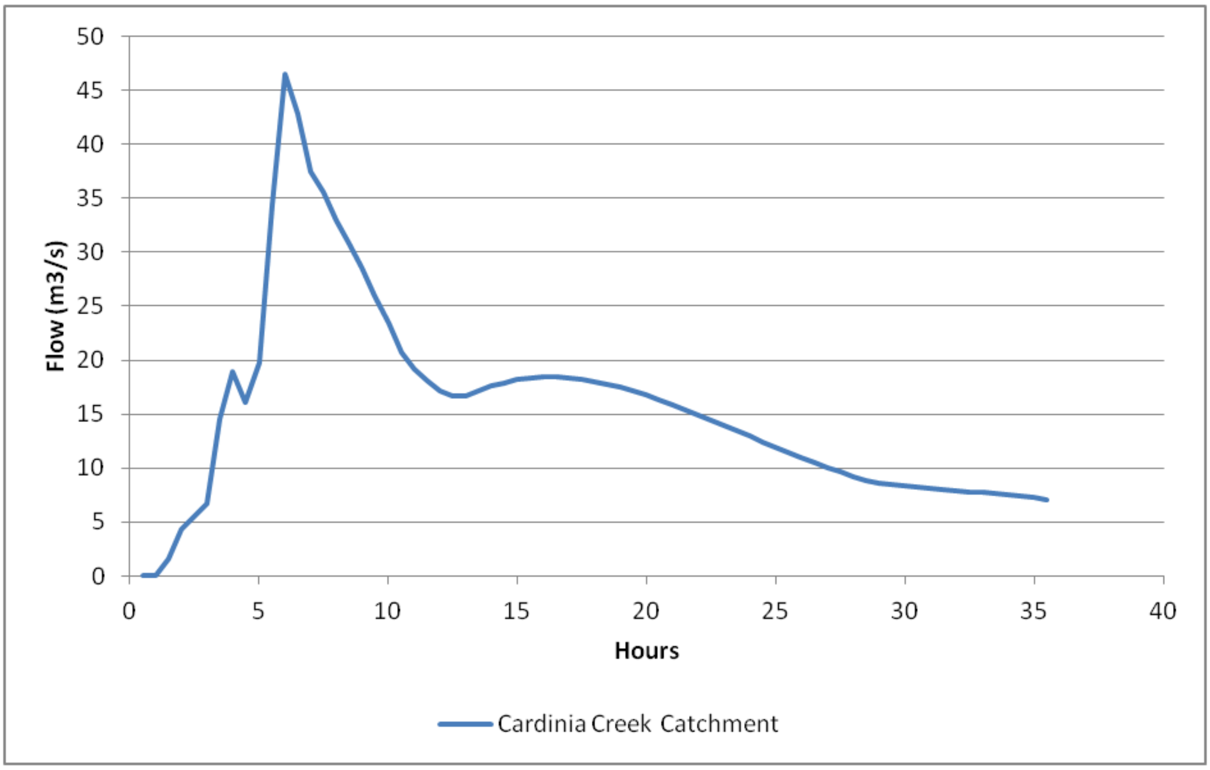


Figure 3-6 Cardinia Creek 10% AEP Hydrograph

3.6 Deep and Toomuc

The model "EXISTING.CAT" provided by Melbourne Water was used. Parameters and settings provided with the model were adopted. The Runoff Coefficient was lowered to 0.35 as recommended by Melbourne Water for modelling of 10% AEP flows. The following parameters and settings were also adopted:

- IFD parameters for Pakenham
- Flitered temporal patterns
- Uniform areal patterns
- ARR87 Bk II (Figs 1.6 and 1.7) Areal Reduction Factors
- Constant losses

The resulting peak flows at Pakenham (Table 3-6) was at the lower end of the 90% confidence limits of the flood frequency analysis and the peak flow at the outfall was approximately 60% higher than the rational method. The peak flows given by the RORB model are considered a good compromise between matching the FFA and rational method estimates. The hydrograph at the outfall is shown in Figure 3-7.

Table 3-6 RORB model 10% AEP Design Flows for Deep and Toomuc Creek

Toomuc Creek at Pakenham	
kc	23.2
m	0.8
IL (mm)	10
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	22

Critical Storm (hrs)	12
Combined Deep/Toomuc Creek	
kc	23.2
m	0.8
IL (mm)	10
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	76
Critical Storm (hrs)	36

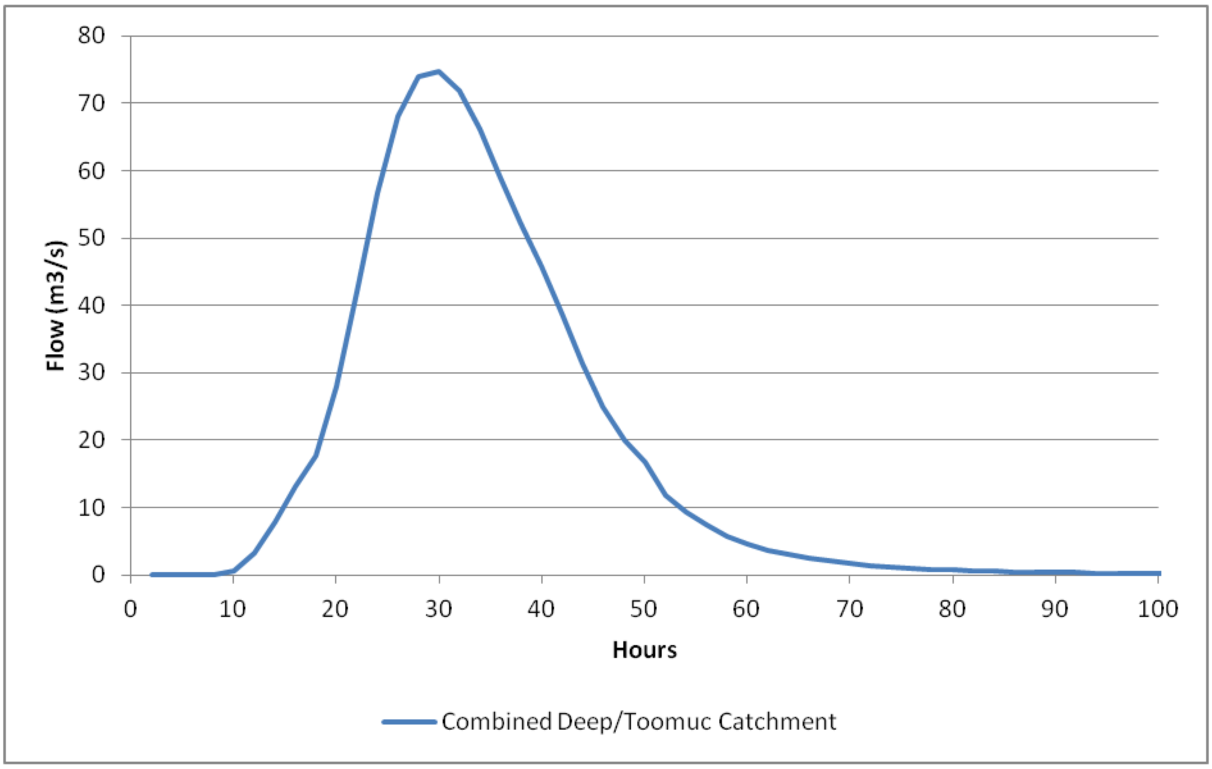


Figure 3-7 Deep/Toomuc Creek 10% AEP Hydrograph

3.7 Tooradin Inlet

The RORB model developed for the Tooradin Drainage Study (Water Technology 2009) was adopted for the Tooradin Inlet and Muddy Gates catchment. The model was rerun for an AEP of 10%, adopting the kc, m and initial loss values from the Tooradin Drainage Study, and a runoff coefficient of 0.35 as recommended by Melbourne Water. The following parameters and settings were also adopted:

- IFD parameters for Tooradin
- Flitered temporal patterns
- Uniform areal patterns
- Siriwardena and Weinmann Areal Reduction Factors
- Constant losses

The resulting combined peak flow (Table 3-7) agreed well with the rational method. The hydrographs for Tooradin Inlet, Muddy Gates Drain and the combined catchments are shown in Figure 3-8.

Table 3-7 RORB model 10% AEP Design Flows for Tooradin Inlet

Muddy Gates	
kc	14.85
m	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	9.5
Critical Storm (hrs)	24
Tooradin Inlet	
Kc	9.03
M	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	19.5
Critical Storm (hrs)	12
East Catchment	
Kc	2.39
M	0.8
IL (mm)	15
RoC (10 year ARI)	0.35
RORB Peak Q_{10}	2.6
Critical Storm (hrs)	9
Combined Tooradin Inlet Catchment	
RORB Peak Q_{10}	29.3
Critical Storm (hrs)	30

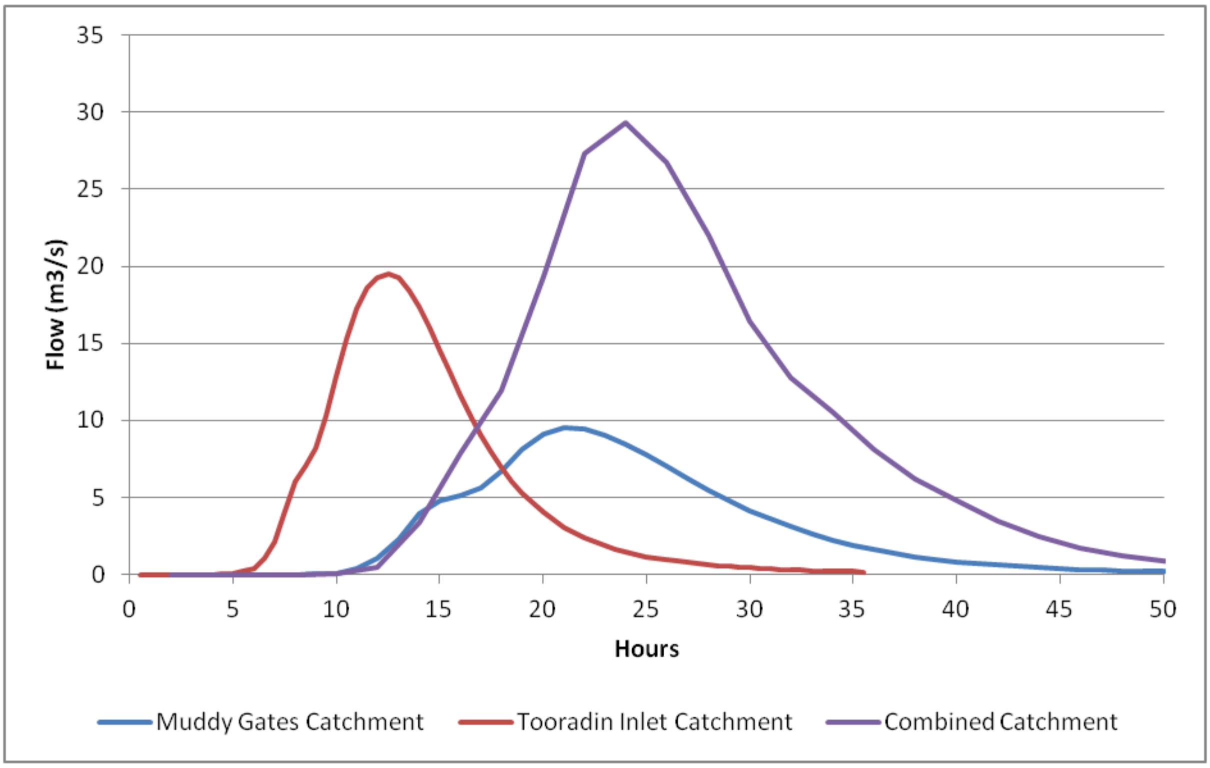


Figure 3-8 Tooradin Inlet 10% AEP 30 hour Hydrographs

4. EVALUATION OF UNCERTAINTY

4.1 Climate Change Impacts

Although mean annual rainfall is likely to decline due to climate change, extreme precipitation is likely to increase in intensity. Abbs and Rafter (2008) used a high-resolution regional atmospheric model to predict the likely changes in extreme rainfall intensity in the Western Port catchment in 2030 and 2070. Their research found that:

- An increase in the magnitude of future extreme rainfall events is expected across most of the Western Port catchment.
- The largest increases occur for the short durations events
- By 2030 an short duration (2 hour) extreme rainfall is expected to increase while long duration (72 hour) rainfall is expected to decrease. Mid-duration (24 hour) rainfall is expected to increase in the southern parts of the catchment and decrease in the northern part of the catchment.
- By 2070 an increase in extreme rainfall is expected across all durations.

The average fractional change in extreme rainfall is shown in Figure 4-1. The extreme rainfall in this analysis was defined as the 10 most extreme events of each duration, corresponding to return periods of 1 in 40 to 1 in 4 years.

The report presents specific results for the likely change in extreme rainfall intensity for the Bunyip and Lang Lang catchments. Flooding in these catchments is driven by long-duration (24-48 hour) events. The median predicted changes in the Bunyip and Lang Lang catchments for 24 and 72 hour durations for 2030 and 2070 are given in Table 4-1. By 2070, long-duration extreme rainfall intensity is expected to increase by 13% in the Bunyip catchment and 17-20% in the Lang Lang catchment.

Table 4-1 Average percentage change in extreme rainfall intensity for Bunyip and Lang Lang catchments (Abbs and Rafter 2008)

Duration (hrs)	Catchment	2030 Median % Change	2070 Median % Change
24	Bunyip	-4	13
	Lang Lang	3	17
72	Bunyip	-16	13
	Lang Lang	-9	20

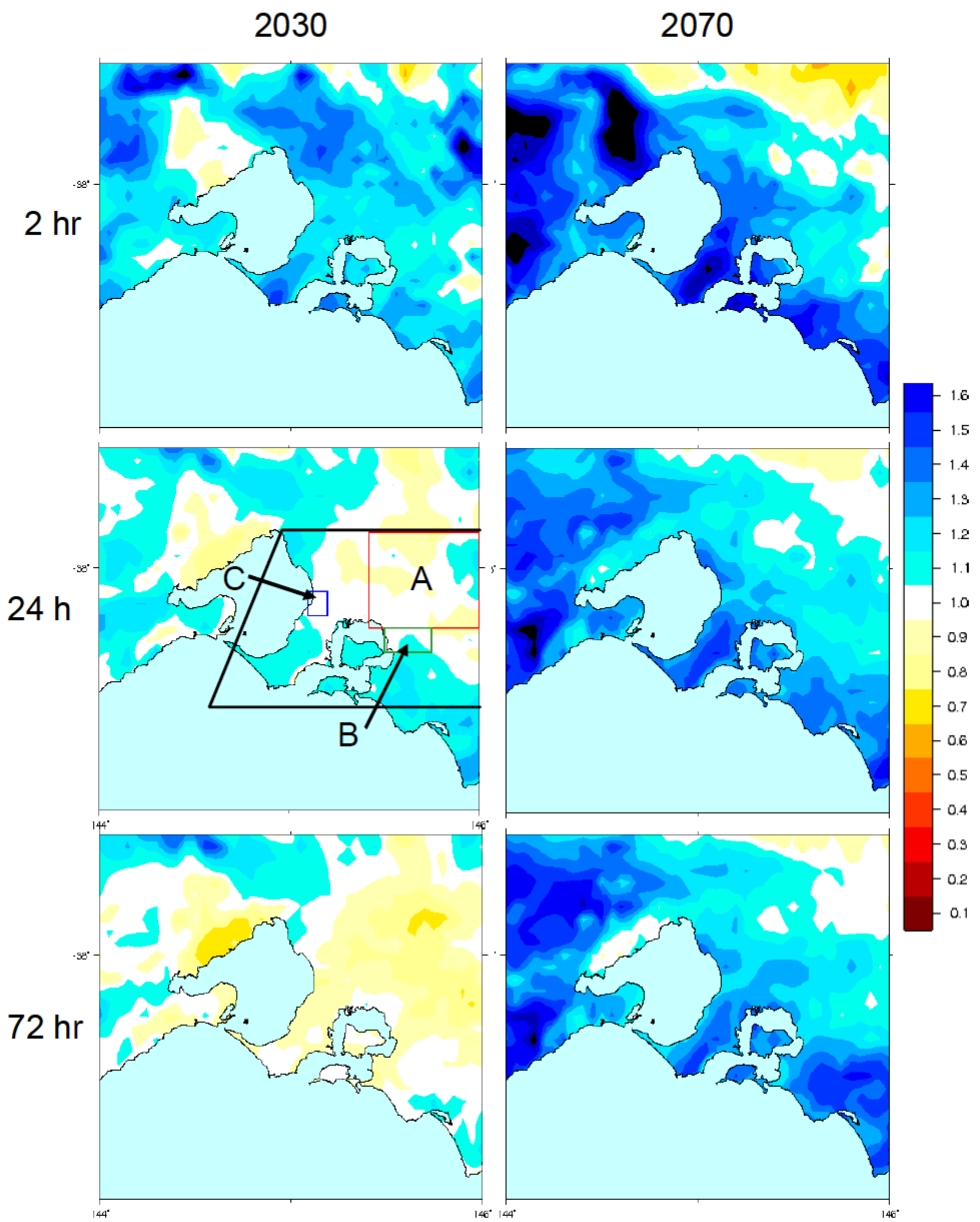


Figure 4-1 **Average fractional change in accumulated rainfall for 2030 and 2070 for extreme rainfall events of 2, 24 and 72 hours (Abbs and Rafter 2008)**

5. SUMMARY

A summary of the RORB peak flows for each catchment compared to the flood frequency analysis and rational method is given in Table 5-1. The locations of the inflows are shown in Figure 5-1.

Table 5-1 Summary of RORB 10 year ARI peak flows compared to flood frequency analysis and rational method estimates

Catchment	Location	FFA	Rational	RORB
Bunyip	Iona	122	-	155
	Outfall	-	208**	103*
Lang Lang	Hamiltons Bridge	109	-	112
	Outfall	-	111	196*
Bass	Glen Forbes South	65	-	67
	Outfall	-	81	63*
Yallock	Cora Lynn	76	-	82
	Outfall	-	77**	145*
Cardinia	Outfall	41	57	47*
Toomuc and Deep	Pakenham	27	-	22
	Outfall	-	48	76*
Tooradin Inlet	Tooradin Inlet Drains	-	-	19.5*
	Muddy Gates Drain	-	-	9.5*
	Combined Outfall	-	25	29

** Adopted for input to hydrodynamic model*

*** Rational method does not account for Bunyip overflow to Yallock at Cora Lynn ford*

Figure 5-1 Location of Catchment Inflows

6. REFERENCES

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