

Seaspray Flood Study – Data Review (R01)



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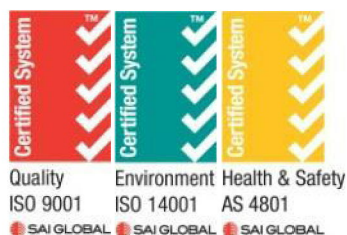


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1. INTRODUCTION

Water Technology was commissioned by the West Gippsland CMA to undertake the Seaspray Flood Investigation. This project includes detailed hydrological and hydraulic modelling of Merriman Creek, taking into account the influences of ocean generated flooding and potential inflows from Lake Reeve, flood mapping of the Seaspray township and will provide recommendations for flood mitigation actions and works.

As part of the initial scoping work, the data required for modelling and mapping was collated and reviewed. This report documents the data review findings and identification of gaps in the data. It also outlines the proposed hydrological and hydraulic modelling scope and methodology.

2. STUDY AREA

Seaspray is a coastal town with 320 permanent residents which rises to more than 1,000 during summer. The township lies at the western end of the Ninety Mile Beach west of the Gippsland Lakes and approximately 200 km east of Melbourne. As detailed in the Urban Design Framework developed for the township by Wellington Shire Council (Wellington Shire Council, 2007), Seaspray is an important access point to the 90 Mile Beach Area and the extensive areas of small lot subdivision that extend easterly for 28km to Golden Beach and Paradise Beach.

The town is located at the downstream end of the Merriman Creek catchment, with the creek flowing along the western boundary of the township, before it passes through the 90 Mile Beach coastal dunes system and enters the ocean. The study area and significant watercourses or flood related influences are shown in Figure 2-1.



Figure 2-1 Seaspray Study Area

3. FLOOD RELATED STUDIES

Following extensive flooding in Seaspray in May and then June 1978, the State Rivers and Water Supply Commission published a flood report for the floods. The report included mapping of approximately 18 surveyed flood levels from the May 1978 flood and 27 flood levels from the June 1978 flood, as well as aerial photography of the June 1978 flood.

The Seaspray Flood Study was then commissioned by the State Rivers and Water Supply Commission in 1980 (Camp Scott Furphy, 1980). A flood frequency analysis was undertaken with limited gauge data and a 1% AEP peak discharge of 330 m³/s was adopted, equal to the June 1978 peak discharge. Flood modelling and damage assessment was undertaken and a flood mitigation scheme recommended, which included the following works and measures:

- Proclamation of flood-prone lands and drainage courses and incorporation into local planning scheme.
- Construction of an approved flood mitigation scheme including levees along Merriman Creek and Lake Reeve Floodway, excavation of the Lake Reeve Floodway and construction of a regulating structure between the Lake Reeve Floodway and Merriman Creek.
- Operation and maintenance arrangements including the clearing of Merriman Creek mouth and operation of the Lake Reeve Floodway.
- Improved flood warning arrangements.

The construction of the scheme was completed in 1987. A levee audit in 1996 (Findlay Irrigation Design Services and BM Consulting Civil Engineers, 1996) found the levees in generally good condition, however in many cases the levee crest was below the design level by up to 0.3 m, and in one area there was a gap in the levees that had allowed floodwaters into the protected area in the 1993 flood.

A flood report was also published for the 1993 flood (Hydrotechnology, 1995), which contained comprehensive details of flood extents, heights and impacts in Seaspray. The flood review of 2007 (EGCMA and WGCMA, 2010) contained a brief reference to the coastal erosion that occurred in Seaspray.

A minor investigation was undertaken for the Duke Energy track crossing of the Lake Reeve floodway in 2002 (ID&A 2002). A Coastal Hazard and Vulnerability Assessment and Flood Study was undertaken for the Seaspray Caravan Park in 2010 (Cardno Lawson Treloar, 2010), which found the existing levee system only protected land up to the 70 year ARI flood event, primarily due to a revision of the 1% AEP flow estimate to more than double the 1980 estimate (679 m³/s). The study also assessed flooding under a range of storm surge and sea level rise conditions, finding that vulnerability to coastal flooding was low.

The recently completed Gippsland Lakes/90 Mile Beach Local Coastal Hazard Assessment Project (Water Technology, 2014a) assessed the sensitivity of Seaspray to flooding from the Gippsland Lakes under present and future sea level rise conditions based on hydrodynamic modelling. The work found that the floodplains around Seaspray only became inundated during the 10% AEP flood event under the +0.8 m sea level rise scenario. Flood levels did not overtop the existing levees. Coincident flooding from Merriman Creek was not considered. However, coastally driven inundation of the township due to overwash of the dune barrier system was also assessed.

4. HYDROLOGICAL DATA

4.1 Rainfall Data

The average annual rainfall at Seaspray is 620 mm. A steep rainfall gradient exists over the catchment with average annual rainfall reaching 1500 mm in the headwaters. At the catchment centroid the average annual rainfall is around 670 mm.

Numerous daily rainfall sites are in operation in and around the catchment. Key stations, including current stations and stations operating over the 1978, 1993 and 1995 floods, are listed in Table 4-1.

Table 4-1 Daily rainfall stations around Merriman Creek catchment

Gauge No.	Location	Period	Years	Distance from catchment centroid
85006 226017	Le Roy (Taylors Rd Quarry)	2000-2009	9	34
85009 226815	Traralgon Epa	2000-2014	14	34
85017	Callignee South	1932-1985	54	26
85033	Giffard	1906-2014	108	21
85071	Rosedale	1878-2005	124	20
85072	East Sale Airport	1943-2014	72	30
85073	Seaspray (Burong)	1989-2014	112	25
85076	Stradbroke West (Inglenook)	1891-1982	91	6
85101	Tarra Valley	1952-1990	10	34
85105	Hazelwood North	1939-1990	37	35
85148	Woodside (Lake View)	1952-1997	46	23
85152	Won Wron Prison	1967-2005	38	27
85160	Darriman (Tarralangi)	1952-2014	62	13
85170	Traralgon L.V.W. & S.B.	1967-1990	23	33
85236	Callignee North	1956-2014	57	28
85281	Traralgon Creek At Koornalla	2000-2014	14	31
85298	East Sale Comparison	1996-2005	9	31
85299	Koornalla Traralgon Ck Rd	2000-2014	14	31

Pluviograph (sub-daily rainfall) stations in and around the Merriman Creek catchment are listed in Table 4-2. The 1993 and 1995 events were captured at East Sale Airport and Calignee North pluviographs. The 1978 events were captured at all listed stations.

Table 4-2 Pluviograph stations around Merriman Creek catchment

Gauge No.	Location	Period	Years	Distance from catchment centroid (km)
85072	East Sale Airport	1953-2011	58	30
85236 226817	Calignee North	1961-2013 1999-2014	51 15	28

Gauge No.	Location	Period	Years	Distance from catchment centroid (km)
85170	Traralgon LVW & SB	1961-1979	18	33
85265	Macks Creek	1975-1978	3.4	26
85264	Novacs	1968-1978	6.8	24
85007 226818	Balook	1999-2014	15	33
585047 227239	Stradbroke West	2006-2014	8	10
585186 226814	Mount Tassie	1998-2014	16	30

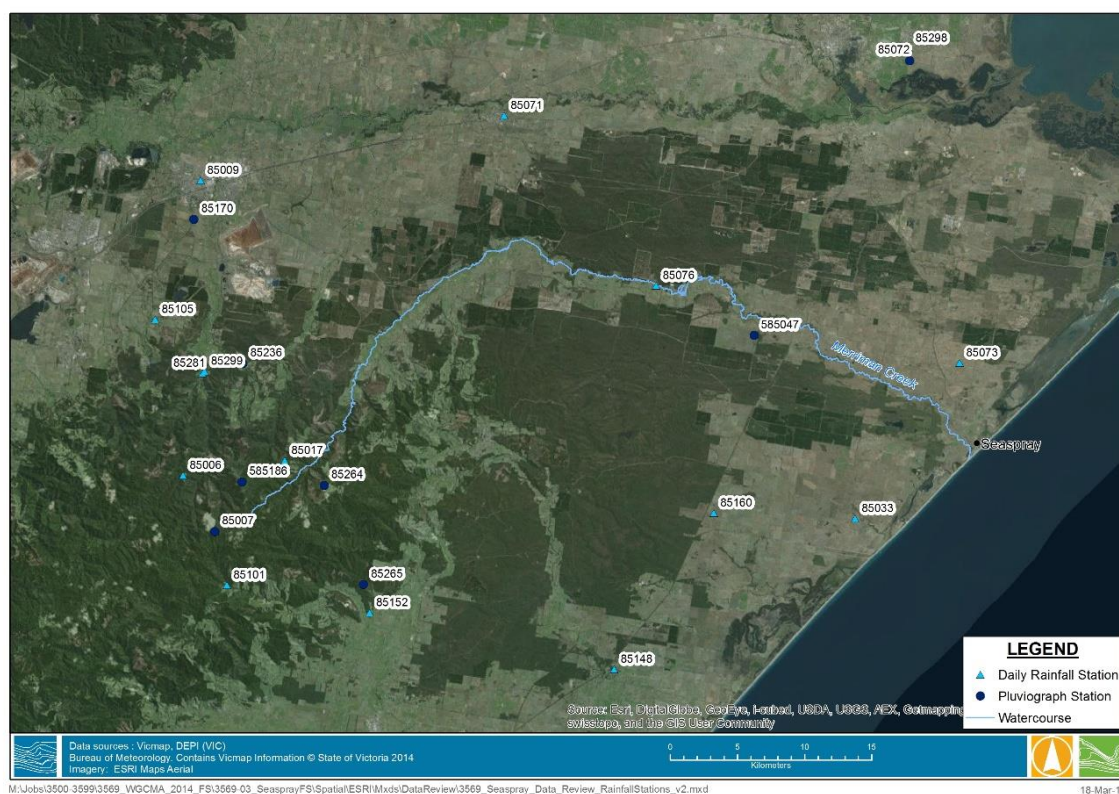


Figure 4-1 Rainfall Stations in and around the Merriman Creek Catchment

4.2 Streamflow Data

Four streamflow gauges operate in the catchment (Table 4-3). The Prospect Road gauge is located close to the town of Seaspray and captures most of the catchment. The Stradbroke West gauge is located upstream and captures just under half of the catchment. The ungauged Monkey Creek joins Merriman Creek between the Stradbroke West and Prospect Road gauges. The Calignee South gauge is located in the headwaters of the catchment, capturing around 7% of the catchment.

Table 4-3 Streamflow gauges in Merriman Creek catchment

Gauge No.	Location	Period	Years	Catchment Area (km ²)
227240	Merriman Creek @ Prospect Road Seaspray	1983-2014	31	529
227001	Merriman Creek @ Seaspray (old Prospect Road site)	1966-1971	5	525
227239	Merriman Creek @ Stradbroke West	1983-2014	31	256
227205	Merriman Creek @ Calignee South	1946-1952 and 1965-2014	54	36
227242	Merriman Creek @ Seaspray Township	1984-2014	31	

The rating curve for the Prospect Road gauges (227240) is shown in Figure 4-2. The rating curve is considered to have only limited reliability up to 3.5 m (8170 ML/d, 95 m³/s), as shown by the scatter in the figure.

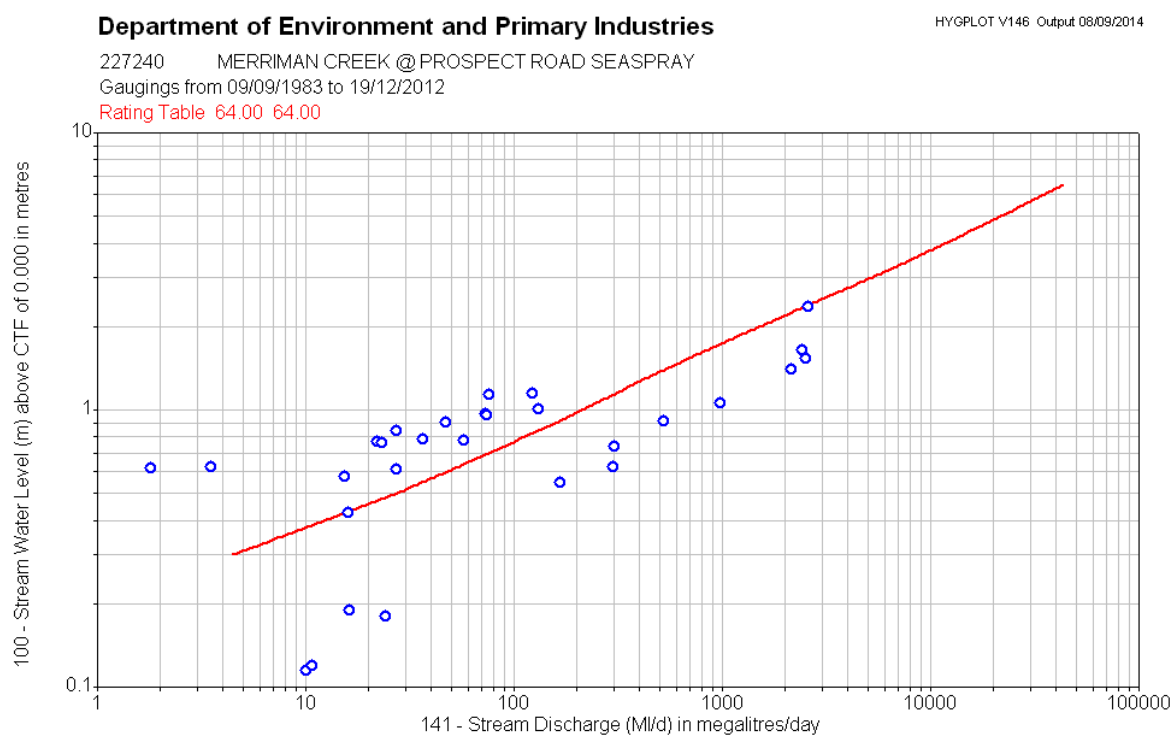


Figure 4-2 Rating Curve for 227240 Merriman Creek @ Prospect Road

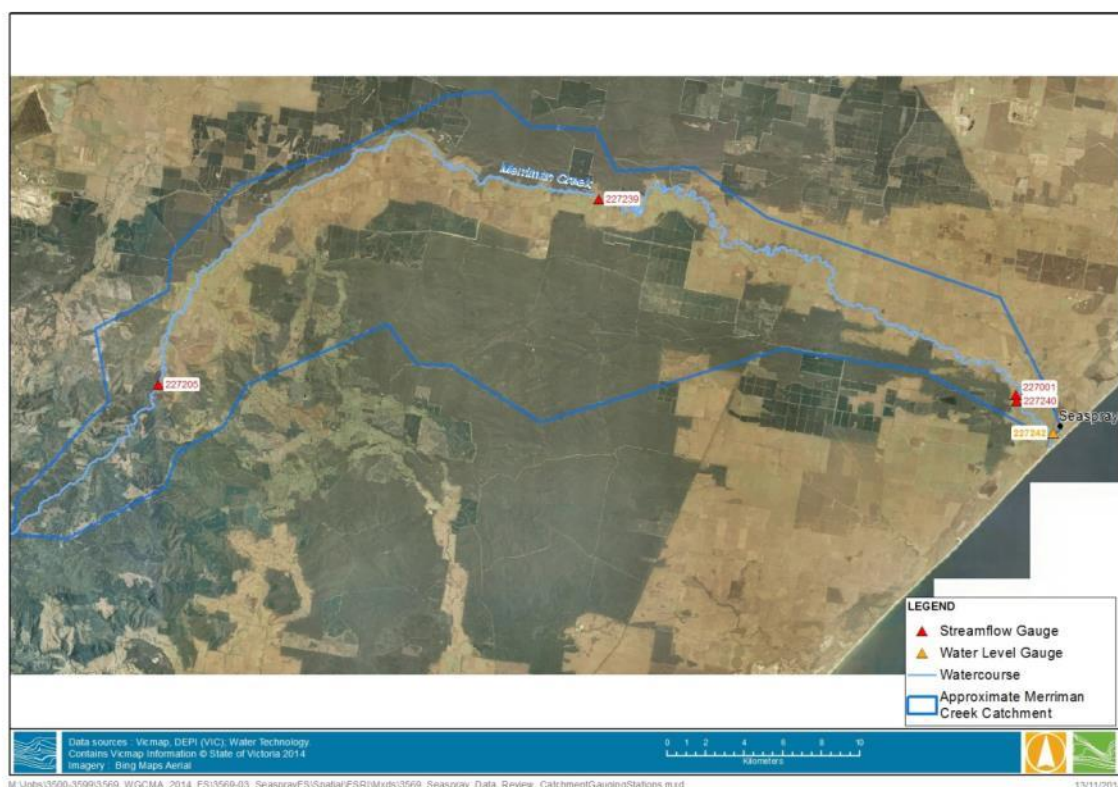


Figure 4-3 Merriman Creek Catchment and Gauging Stations

The Stradbroke West gauge has more high-flow gaugings and less scatter, and might therefore be considered more reliable in high flow estimation. It is considered reliable up to 6.4 m (6790 ML/d, 79 m³/s).

Both the Stradbroke West gauge and Prospect Road gauge ratings are discussed in detail in the accompanying Hydrology Report (Report 2).

4.3 Water Level Data

A water level gauge operates in the Merriman Creek lagoon at Seaspray (Table 4-4). The gauge record spans from 1984-2014 but is incomplete, with 1986-1989 and 1993-1995 missing and a patchy record from 2011-2014. None of the major floods are recorded at this gauge.

Table 4-4 Water Level Gauge in Seaspray

Gauge No.	Location	Period	Years
227242	Merriman Creek @ Seaspray Township	1984-2014 (incomplete)	~20

5. FLOOD RECORDS

5.1 Historic Flood Information

Significant historic flood events have been compiled from available sources and are listed in Table 5-1. The largest flood on record at the Prospect Road gauge, just upstream of Seaspray, was the October 1995 flood, however very little information is available for that event. More information is available for the 1993 event, including digitised flood extent, surveyed flood levels, floor levels and numerous flood photographs, collated for Hydrotechnology's report on the 1993 Gippsland flood. A large amount of data is also available for the 1978 floods which were the largest on record when they occurred, but were later exceeded in 1993 and 1995.

Table 5-1 Historic Flood Events

Event	Description	Data available
1880	Whole area flooded with fresh water	Gippsland Times letter
1895	Salt water flooding of whole area	Gippsland Times letter
1900	Town flooded by salt water	Gippsland Times article
March 1908	Salt water flooding of camping reserve	Gippsland Times article
September 1916		
1930	Two salt water flooding events in the first half of 1930	Gippsland Times article
April 1932	Creek overflowed and swamped a large portion of Seaspray	Gippsland Times article
December 1934	"Flood of unusual magnitude"	Gippsland Times article
April 1935	Every house in north end of town surrounded by water	
July 1936	Flood covering both ends of township	Gippsland Times article
May 1944	Mouth blocked	
July 1949	Mouth blocked	
February 1951	Reported as the worst flood in 60 years. 75% of homes in Seaspray were flooded	Gippsland Times articles
June 1952	Reported as a "record flood" Overflow from Merriman Creek and Blind Creek, with Monkey Creek also in flood Mouth opened readily as floodwaters rose. 75% of homes in Seaspray were flooded	Discharge estimated at 194 m ³ /s in 1980 Flood Study (Camp Scott Furphy 1980) One flood level (VFD) Gippsland Times articles
January 1962	Tidal flooding	
February		Gauged streamflow at 227001 (peak

Event	Description	Data available
1971		discharge 326 m ³ /s)
May 1975	Tidal flooding	
May 1978	Overflow from Merriman Creek and Blind Creek 12 out of 55 permanently occupied residential properties were flooded at or above floor level 9 out of 42 seasonally occupied residences were flooded at or above floor level	Digitised flood extent (VFD) Approx. 18 surveyed flood levels (Camp Scott Furphy 1980)
June 1978	42 out of 55 permanently occupied residential properties were flooded at or above floor level 38 out of 42 seasonally occupied residences were flooded at or above floor level Complete inundation of over 180 built-on allotments Only 15 to 20 allotments out of 320 were not subject to any inundation	Peak discharge estimated at 330 m ³ /s in 1980 Flood Study (Camp Scott Furphy 1980) Digitised flood extent (VFD) Approx. 27 surveyed flood levels
April 1990		Digitised flood extent (VFD)
September 1993	1.5 times larger than the previous largest flood and previous 1% AEP estimate. Land affected by flooding from Blind Creek and overtopping/outflanking of levees. 44 houses flooded above floor level Around 140 built-on allotments affected by flooding in total	Peak discharge estimated at 450 m ³ /s Digitised flood extent (VFD) Approx. 12 surveyed flood levels CFA logs and transcripts 44 surveyed floor levels Numerous flood photographs (Hydrotechnology 1995)
October 1995	Second-largest flood on record.	Flood peak 370 m ³ /s
2007	Coastal erosion of dunes	
June 2012		Digitised flood extent (VFD) Lake Reeve western end only.

Historic flood data recorded in the Victorian Flood Database (VFD) is summarised in Figure 5-1, Figure 5-2 and Figure 5-3 below. Note that flood heights for the May 1978 flood are not recorded in the VFD but are available in the 1978 Flood Report (SRWSC 1978). Some inconsistencies have been noted between the VFD data and the original data recorded in the 1978 and 1993.



Figure 5-1 May 1978 flood information in VFD



Figure 5-2 June 1978 flood information in VFD



Figure 5-3 1993 flood information in VFD

5.2 Imagery

Non-flood aerial imagery is available for April 1965 (B&W), July 1974 (Colour), 1976 (B&W), 1977 (B&W), May 1984 (Colour), January 1992 (Colour), and 2010 (Colour orthophoto).

Flood aerial photography is available for the May 1978 event in Colour and Colour Infrared (CIR) and the June 1978 event in CIR only. An example is shown in Figure 5-4 below. Aerial photography is also available for the aftermath of the 1993 event (28 September 1993) in the Hydrotechnology (1995) flood review.



Figure 5-4 **CIR Imagery of May 1978 event**

6. PHYSICAL FEATURES

6.1 Topographic and Physical Survey

6.1.1 LiDAR Datasets

Aerial LiDAR (Light Detection and Ranging) survey is available for the Seaspray area from two difference sources:

- Vicmap Elevation Coastal 1m DEM (referred to as the 'Coastal DEM')
- 2009-10 Victorian State Wide Rivers LiDAR Project - West Gippsland CMA (referred to as the 'Rivers DEM')

The entrance and lagoon arrangements are different in the two datasets as they were captured at different times. The lagoon water level and entrance berm are lower in Coastal DEM.

The extents of the two datasets are shown in Figure 6-1. A comparison of the two datasets where they overlap (Rivers DEM minus Coastal DEM) is shown in Figure 6-2. A slight vertical offset is discernible between the two datasets in flat floodplain areas. In these areas, the Rivers DEM is around 0.1 m lower than the Coastal DEM. A comparison between the surveyed levee heights from the Urban Levees Review (SKM, 2014) showed that both LiDAR datasets were 0.05 m higher on average than the levee survey, with a standard deviation of 0.13-0.15 m. A survey transect in a flat area would be valuable to further verify the LiDAR datasets and assist in the selection of datasets for modelling. Key metadata for the two datasets is given in Table 6-1.

As the Coastal DEM is rated to a higher accuracy, covers a greater extent and has lower lagoon levels, it is envisaged that it will be used as the primary dataset for modelling.

A coarse (10 m resolution) DEM is available from VicMap for the whole Merrimans Creek catchment for catchment delineation purposes.

Table 6-1 Key metadata for LiDAR datasets

Dataset	Source	Date of Capture	Vertical accuracy (1 sigma)	Resolution
Vicmap Elevation Coastal 1m DEM (Coastal DEM)	LiDAR	Sep 2007-Sep 2009	0.10 m	1 m
2009-10 Victorian State Wide Rivers LiDAR Project - West Gippsland CMA (Rivers DEM)	LiDAR	Feb-Nov 2010	0.20 m	1 m

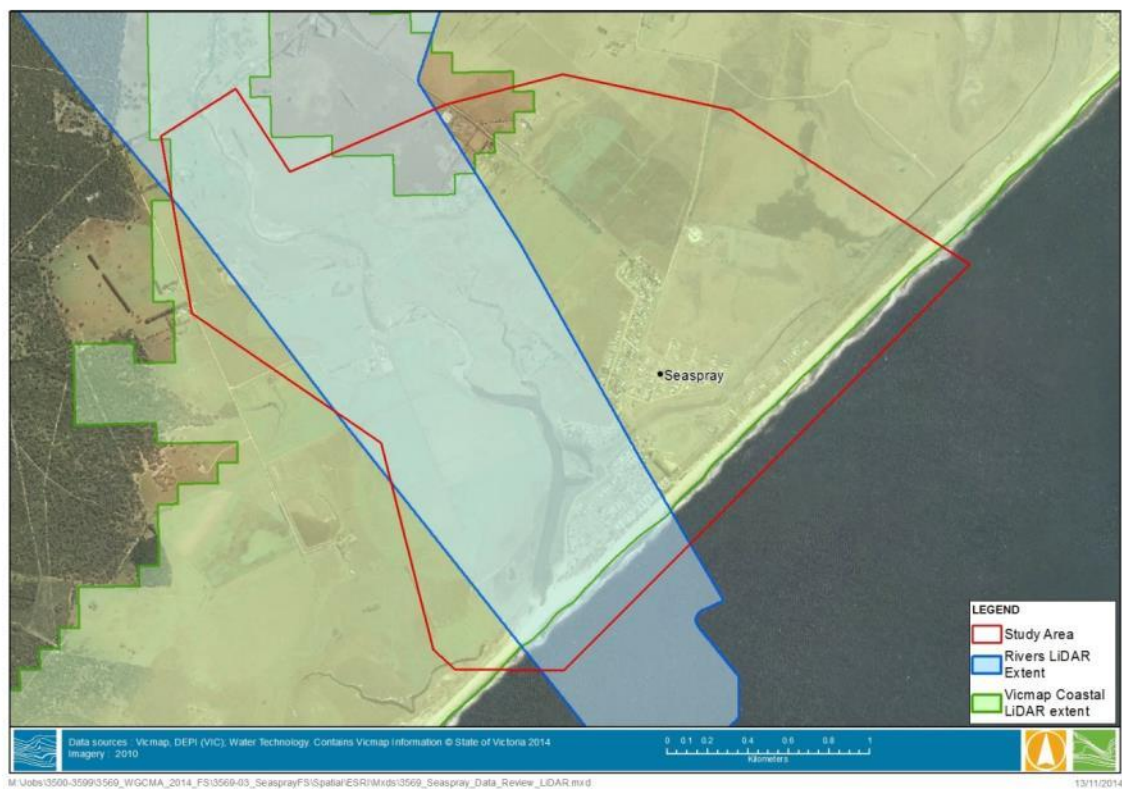


Figure 6-1 DEM extents



Figure 6-2 LiDAR comparison - Rivers LiDAR DEM minus Coastal DEM

6.1.2 Levee Survey

The Urban Levees Review (SKM, 2014) identified the following four levees within Seaspray, which were subsequently surveyed along with a preliminary condition assessment:

- Levee A – situated along the west bank of Merriman Creek between the floodway and Ninety Mile Beach.
- Levee B and C – situated along the north bank of the floodway (to the east of Merriman Creek) between Merriman Creek and the end of Catton Street, where it runs parallel to Centre Road to Main Road (Seaspray Road) and then along Main Road to the junction with Tip Road.
- Levee D – situated along the south bank of the floodway (to the east of Merriman Creek) between Merriman Creek and the most easterly house along Shoreline Drive.

An additional section of levee (Levee E), which is an extension of Levee C, was not surveyed. Each of the surveyed levees is shown in Figure 6-3. The levees labelled 2000 Levee are from the VFD.

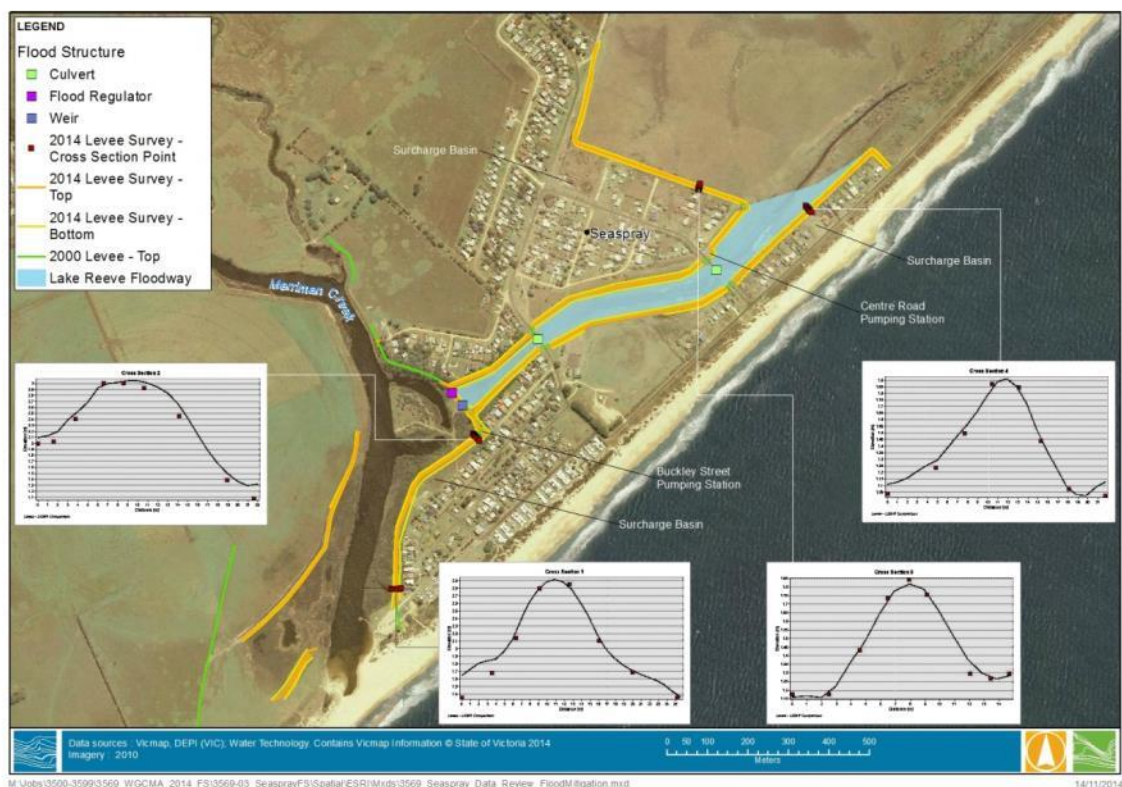


Figure 6-3 Location of Levees and Flood Mitigation Structures within and around Seaspray

6.1.3 Road Survey

West Gippsland CMA provided a road survey that was undertaken for a sub-division near the new caravan park in the Seaspray township. The survey extended around a path joining Futcher Street, Main Road, Centre Road, and Shoreline Drive. The levels from the survey were compared to, and found to agree with, the Coastal DEM. The average difference (survey – Coastal DEM) was -0.04 m with a standard deviation of 0.03 m.

6.2 Drainage Information

GIS layers of local pits and pipes were provided by Wellington Shire (Figure 6-4). from Wellington Shire provided advice on the reliability of the pipe layer and additional/altered pipes. The town of Seaspray is divided into two separate drainage areas. The area east of Futcher Road drains to a pump station on Centre Road on the northern side of the floodway, which pumps stormwater out to the floodway. The area west of Futcher Road drains to a pump station on Irving Street, which pumps to Merriman Creek. Each pump station has two pumps which have a maximum capacity of 170 L/s each. During a high rainfall event both pumps can be operated together giving a maximum discharge rate from each pump station of 340 L/s.

The caravan park has been constructed recently on the site of the former oval and has significant internal drainage infrastructure, including grated pits and pipes running along internal roads, and two retarding basins.



Figure 6-4 Drainage GIS layers (Wellington Shire Council) and additional/alterd pipes as advised by

6.3 Bridges and Culverts

Bridge and culvert dimensions were surveyed by Water Technology on 26 November 2014. The locations of the surveyed culverts are shown in Figure 6-5 and key details are given in Table 6-2.

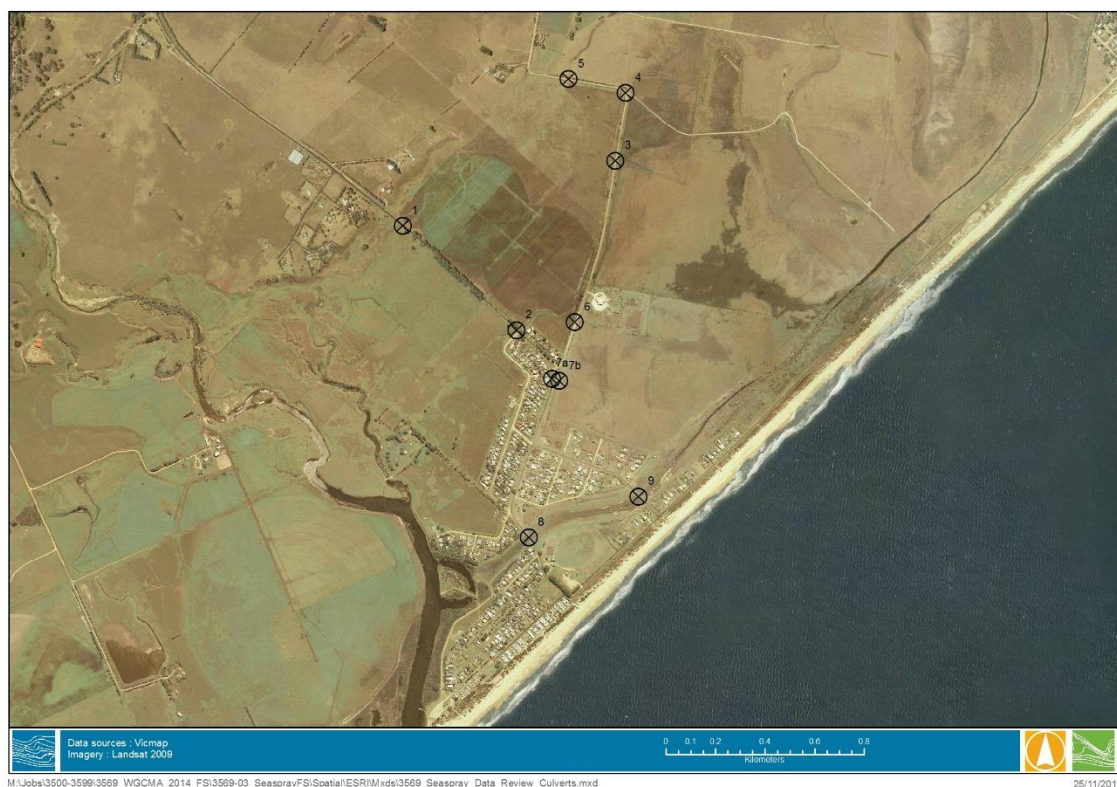


Figure 6-5 **Surveyed bridges and culverts**

Table 6-2 **Key details for surveyed bridges and culverts**

Culvert ref.	Type	No.	Dimensions	Notes
1	Angled RCBC	5	2000 x 1500 mm	Debris grates on upstream side.
2	RCP	3	450 mm	Heavily overgrown with kikuyu
3	culvert not found			Heavily overgrown with Phragmites
4	RCP	1	300 mm	
5	RCP	3	450 mm	
6	RCP	2	300 mm	
7a	RCBC	1	450 x 150 mm	Upstream end cracked, appears fully blocked, Downstream end eroded.
7b	RCP	1	450 mm	Flap gate on downstream side, blocked by soil to one third of its height.
8	Clear span		2 x 8.2 m span, total span 17 m	1 x central rectangular pier 600 mm wide, sitting on square 300 mm piers
9	RCBC	4	1200 x 750 mm	Standing water in culverts to approx. halfway

RCP = reinforced concrete pipe

RCBC =reinforced concrete box culverts

6.4 Estuary Entrance Conditions

The coastal morphology in the vicinity of the township of Seaspray was examined in detail in the recently completed Gippsland Lakes/90 Mile Beach Local Coastal Hazard Assessment Project (Water Technology, 2014a).

Seaspray is located behind the Outer Barrier of the Gippsland Lakes coastal barrier system. The outer barrier extends continuously from the sand islands of eastern Corner Inlet to Red Bluff east of Lakes Entrance. It is the youngest of the coastal barriers and the morphology and present dynamics are highly variable.

Merriman Creek exits to the ocean through this barrier at Seaspray. The Merriman Creek estuary is defined as an ICOLL (intermittently closed and open lake or lagoon) whereby under low creek flows and/or on-shore wave conditions the entrance channel is small or closed off from the ocean by the sand berm. When the entrance is closed, the water level in the estuary will be elevated and can influence flood extents.

Available aerial imagery (Section 5.2) indicates that the location and form of the entrance channel varies over time.

Estuary conditions have also been influenced by the flood mitigation scheme, described in Section 8.1, which included the remodelling of Merriman Creek including the cut-off of the meander bend upstream of the entrance (now known as The Island).

An artificial entrance channel has at times been constructed through the sand berm to allow the water level in the estuary to be lowered prior to a flood event in Merriman Creek. This work is undertaken by Wellington Shire Council; however no formal entrance channel opening procedure exists.

Based on the available gauged water levels (Gauge 227242) the water level in the estuary varies between 1 m AHD to around 2 m AHD, Figure 6-6.

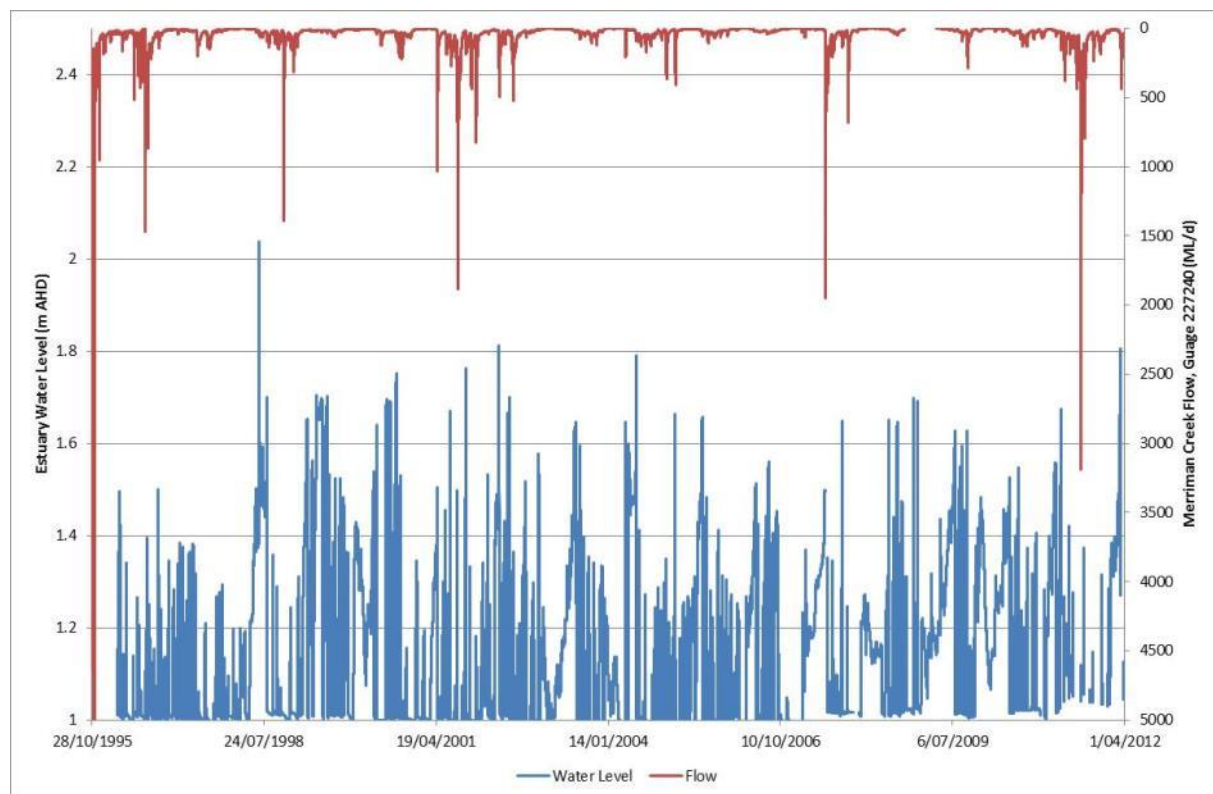


Figure 6-6 Estuary Water Level and Merriman Creek Flows

7. METOCEAN CONDITIONS

7.1 Wind Climate

Along the coastline at Seaspray, winds are predominantly westerly, with minor southerly and easterly components. As the broad orientation of the Outer Barrier is to the NE, the majority of the strongest and most persistent winds blow offshore, however, there is some variation along the study area. At Seaspray the shoreline is aligned to the NE (45°) such that only winds from the south to east have appreciable onshore components.

7.2 Wave Climate

Waves and the variability associated with their height, period and direction comprise the principle source of energy for mobilising the sediments along the shoreline at Seaspray, which influences the conditions at the Merriman Creek entrance.

The wave climate of the East Gippsland coast is largely sheltered from direct exposure to the highly energetic wave climate of the Southern Ocean by the Tasmanian landmass, although waves from this source do refract around the eastern side of Tasmania. Larger waves are therefore principally generated within eastern Bass Strait by south westerly to southerly winds and in the South Tasman Sea by east to south-easterly winds. For the recent coastal hazard study (Water Technology, 2014a), the wave climate along Ninety Mile Beach was simulated over a 63 year period (1948 to 2011).

The percentage exceedance of significant wave heights from the 63 year wave simulation found that significant wave heights are generally between 0.5 and 1.0 m. Significant wave heights exceed 2.0 m less than approximately 1% of the time at Seaspray.

The mean wave directions were found to be principally from the south and south-east with minor contribution from south-westerly and easterly waves. Towards the south-west at Seaspray, the slightly greater easterly aspect of the coastline results in a slightly greater percentage of waves arriving from the east compared to the more south-east facing coastline at Lakes Entrance.

7.2.1 Storm Wave Climate

An analysis of the storm wave climate along the Gippsland Coast (Water Technology, 2014a) found that storm wave events are not distributed evenly over the historical record but display significant clustering on an approximate decadal scale. Relationships between storm intensity and frequencies on the east coast of Australia have been identified as potentially being modulated by the Interdecadal Pacific Oscillation (IPO) and El Niño-Southern Oscillation (ENSO) by You & Lord (2008) and Shand et al. (2010).

Based on the clustering of storm wave events, periods of heightened wave storminess were interpreted as 1950-1956; 1964-1967; 1974-1978; 1984-1987; 1995-2001; and 2007-2011. The study then analysed the frequency of the large storm events to provide a statistical estimate of storm wave recurrence interval.

7.3 Coastal Water Levels

Ocean water level variations are primarily caused by a combination of the inverse barometric pressure affect, coastally trapped waves and astronomical tides.

7.3.1 Astronomical Tides

The gravitational interactions associated with the sun and moon on the earth's oceans generate regular water level variations commonly referred to as the tide. The Eastern Bass Strait coastline

experiences a micro-tidal climate with the tidal range increasing slightly towards the south-west of the study area due to resonance of the tide within Bass Strait. Table 7-1 displays the main astronomical tidal planes at Lakes Entrance and Seaspray relative to Lowest Astronomical Tide (LAT).

Table 7-1 Astronomical Tidal Planes for Lakes Entrance (Outer) and Seaspray (ref. to LAT)

	HAT (m)	MHHW (m)	MLHW (m)	MSL (m)	MHLW (m)	MLLW (m)
Lakes Entrance (Outer)	1.4	1.2	0.8	0.8	0.7	0.3
Seaspray ¹	1.6	1.4	1.0	0.9	0.8	0.2

¹ Approximated from tidal constituents at McGauran's Beach (Hinwood J. &, 1986)

7.3.2 Storm Surges

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

Estimated recurrence intervals of peak storm surge and storm tide levels for the open coast at Seaspray for relevant sea level rise scenarios have been developed by the CSIRO and are displayed in Table 7-2.

Table 7-2 Estimated Storm Surge and Storm Tide Recurrence Intervals for Open Coast at Seaspray, based on the IPCC 2007 A1FI Scenario 2, from McInnes, et al. (2009)

Period (years)	Storm Tide (m)			
	Current Climate	+0.15 m SLR	+0.47 m SLR	+0.82 m SLR
10	1.22 ±0.12	1.42	1.87	2.33
20	1.32 ±0.12	1.54	1.98	2.44
50	1.43 ±0.12	1.66	2.09	2.53
100	1.50 ±0.14	1.73	2.18	2.64

The synoptic weather systems responsible for significant storm surge events on the East Gippsland coast were investigated by the CSIRO (2005). A synoptic typing technique was utilised that generated three generic weather systems giving rise to storm surges on the East Gippsland Coast. The three generic weather systems were termed Cold Fronts, Tasman Lows and East Coast Lows.

The study determined that from a population of storm surges greater than 0.4 m at Lakes Entrance, 77% were associated with cold fronts at various stages of progression across Bass Strait. Only around 23% of significant storm surges were identified as being generated from either Tasman Low or East Coast Low weather systems.

8. FLOOD MITIGATION AND WARNING

8.1 Flood Mitigation Scheme

The Seaspray flood mitigation scheme was constructed in response to the 1978 flooding following the 1980 flood study. The flood mitigation scheme was originally intended to protect residential land and allow extra time to get the entrance open. The scheme consists of:

- Levees along Merriman Creek and surrounding developed areas
- Excavation of the Lake Reeve Floodway
- Regulating structure and spillway between Merriman Creek and the Lake Reeve Floodway
- Remodelling of Merriman Creek including cut-off of the meander (now The Island)

Figure 6-3 shows the location of the levees and floodway structures.

Wellington Shire Council is responsible for the operation of the floodway. The opening of the entrance is considered the first priority when a flood alert is issued. If the mouth is not open then it must be mechanically excavated. At times the mouth has been difficult to get open and has tended to reclose due to wave conditions. If the mouth cannot be opened then flows exceeding $45 \text{ m}^3/\text{s}$ are likely to overtop the levees.

The regulator structure can be lowered to allow flood flows to flow towards Lake Reeve to allow additional time to get the entrance open. The capacity of the floodway is $15 \text{ m}^3/\text{s}$. In recent years the regulator has been kept closed in all events, with the aim to allow flood levels to build up in the lagoon to help drive entrance scouring.

A low flow bypass pipe connects Merriman Creek with the Lake Reeve end of the floodway, but is not currently operable due to corrosion.

The flood mitigation scheme refers to a fixed spillway at 2.4 m to Masons Creek to the south west of the lagoon. However there is no evidence of this spillway on the ground or in the DEM. The ground surface has an overflow level of approximately 2.3 m to Masons Creek. The scheme also refers to filling of the land west of the lagoon to compensate for increased flood levels, however the land appears to have been protected with a levee with crest height approximately 1.9 m to 2.5 m AHD rather than fill.

The construction of the scheme was completed in 1987. Some additional works were undertaken after the 1996 audit (Findlay Irrigation Design Services and BM Consulting Civil Engineers 1996) including the levee extension up to Blind Creek and the Eastern Prior Stream culvert upgrade. The levees were surveyed for the Urban Levee Review project (SKM, 2014).

8.2 Flood Warning Arrangements

Bureau of Meteorology gauges at Prospect Road and Stradbroke West were installed following the 1980 flood study for flood warning purposes. Flood alert flow levels at the Prospect Road gauge are listed in the Mitigation Scheme Operating Procedure but it is unknown whether these have been formally implemented by the Bureau of Meteorology. No flood class levels have been defined for either gauge.

The Mitigation Scheme Operating Procedure also includes recommendations for opening of the estuary entrance when flood conditions are expected in Merriman Creek, based on the upstream gauge flows.

9. SITE INSPECTION

A site inspection was undertaken by representatives from Water Technology, West Gippsland CMA and Wellington Shire Council on 16 October 2014. Key locations visited are shown in Table 9-1.

Table 9-1 Key locations visited by project staff, 16 October 2014

Location/Notes	Photo
Centre Road Pump Station All stormwater drains to 2 pump stations. Some stormwater from the northern levee area drains to the southern area via pipes under the floodway.	
Southern Pump Station	
Water Level Gauge Lagoon quite high, 1.3 m on gauge	

Location/Notes	Photo
Regulator - All boards were in. Boards come up to top of concrete walkway. (Wellington Shire Council) advised that current practice is to leave all boards in at all times, which deviates from the operating procedure. The rationale behind this is that the priority during a flood is to get the entrance open, and if the floodway is open it can create conditions where the entrance is less likely to scour out properly when opened. The regulator would only be opened if water was required to top up Lake Reeve. In a typical flood it can be assumed to remain closed.	
Low flow gate at Regulator (closed) Small regulators on each side of the regulator are intended to allow low flow through to Lake Reeve. The floodway side should be kept open but is currently closed and cannot be operated. The lagoon side was also closed. The low flow pipe runs to an outlet at the Centre Road crossing of the floodway.	
Spillway beside regulator Next to the regulator is a wide spillway at a lower level than the top of the boards. The weir is constructed of rock gabions or rock covered with mesh)	

Location/Notes	Photo
Merriman Creek Lagoon - Entrance closed - Large lagoon on ocean side of dunes, stretching toward surf club - Phragmites along fringes of lagoon, some swamp paperbark around The Island	
Floodway (looking west from Futcher Road) High grass in Floodway	
Floodway (looking west from Centre Road)	

Location/Notes	Photo
Levee north of The Island - Levees have recently been surveyed by SKM. A section north of The Island was missed. - All levees appeared in good condition	
Levee between lagoon car park and regulator	

Local historical knowledge was provided on site by John Inglis of Wellington Shire Council, including the following remarks:

- The west regulator/levee (west of Merriman Creek) was never finished. Some flow would head down to Lake Denison under high flood conditions.
- The Prospect Road gauge was moved earlier this year to a more stable location. Previously it had to be re-rated many times due to channel change. The Stradbroke West gauge is in a stable location and provides good flood warning (~24 hours).
- In 1993-1995 Blind Creek was blocked with an unauthorised levee. The levee was rectified after the 1995 flood.
- Usually the entrance is easy to open, but in 1995 and 1998 wave conditions made it hard to get the entrance open. The entrance usually opens up towards the surf club due to the prevailing wind direction
- After the 1995 flood a new culvert was installed on Eastern Prior Stream.

10. MODEL DEVELOPMENT AND SCHEMATISATION

10.1 Hydrologic Modelling Approach

Hydrologic modelling will be undertaken to quantify the catchment generated flows in Merriman Creek at the hydraulic model boundaries as well as stormwater inflows within the hydraulic model extent.

10.1.1 Model Build / Development

A RORB hydrological model will be developed of the entire Merriman Creek catchment to the creek entrance, which includes the hydraulic model extent. ESRI's ArcHydro will be used to delineate the catchment, with a minimum of 3 to 5 sub-areas schematised upstream of any required streamflow hydrograph. Print locations will be included at all locations where flow is required such as at tributaries, streamflow gauges and model boundaries.

10.1.2 Model Calibration

The model will be calibrated to the 1993, 1995, and 2011 flood events for which gauged information is available. Local daily rainfall gauges will be used to construct a spatial pattern of rainfall for the historic events. Local sub-daily rainfall gauges will be used as temporal patterns for the historic storm event. Model parameters kc , m , IL and CL will then be developed for each event by calibrating to the available streamflow gauge. The kc and m will most likely be consistent across events with the losses varied to represent different antecedent conditions.

The design events 0.5%, 1%, 2%, 5% and 10% AEP will be simulated for a range of storm durations.

10.1.3 Hydrological Design Modelling

A flood frequency analysis will be undertaken for the Prospect Road gauge to determine peak flow and volume for the design annual exceedance probabilities specified in the brief.

At this stage it is proposed that the current IFD curves will be utilised for design rainfalls. Presently, the revised curves are not recommended to be used in practice until the revised AR&R manual is finalised. The revised IFD curves will be modelled, however, as part of a sensitivity analysis.

RORB model design parameters (kc , m , IL , CL) will be determined. The kc and m values adopted for design modelling will most likely be the same as that adopted for verification, given the calibration approach will utilise design estimates, but will be compared to nearby model values as well as values from Australian Rainfall and Runoff and the RORB kc prediction equations.

A number of estimation techniques will be tested for design losses, with the flows at Seaspray compared for the various methods:

- Hill *et. al.*
- Australian Rainfall and Runoff
- Other models from nearby catchments
- Losses from verification historic events

Further verification of design flows will be undertaken using regional prediction equations and methods such as the Probabilistic Rational Method. However it must be noted that the recently released draft version of the updated Australian Rainfall and Runoff – A Guide to Flood Estimation, recommends that the Probabilistic Rational Method is no longer used, and goes on to provide an alternative approach. Both methods will be used to provide verification for the RORB design flow estimates.

The design events will consider a range of durations with a focus on critical durations for Merriman Creek at Seaspray. All critical durations will be simulated in the hydraulic model and results enveloped for the final design event mapping.

The final adopted design model parameters will be utilised to produce hydrographs at all hydraulic model boundaries for the 0.5%, 1%, 2%, 5%, and 10%, AEP flood events.

The probable maximum flood (PMF) discharge will be estimated using the rapid assessment method developed by Nathan *et. al.* (1994).

The impacts of climate change will also be tested by increasing the rainfall intensity by an agreed amount in line with available literature, most likely an increase of 32% as advised in the report *Climate Change in Australia* (CSIRO, 2005).

The impacts of bushfires will be tested by adjusting the fraction impervious of the Merriman Creek sub-catchments to reflect an agreed severity of bushfire. For example Blackham *et al* (2012) provides values of equivalent percentage impervious for different levels of burn severity, based on BAER (2009).

10.1.4 Stormwater

Stormwater modelling will use ARR87 IFD and temporal pattern information for input to the direct rainfall model. The ARR87 information will be compared to any updated information available for the ARR update, and if there are significant differences a sensitivity test may be justified.

No calibration data is available for stormwater flooding.

Loss values will be adopted to be consistent with those applied in the RORB model for design flood estimation.

10.2 Hydraulic Modelling Approach

10.2.1 Overview

Water Technology has previously developed both a MIKE21 Flexible Mesh and MIKE21 Fixed Grid model of Seaspray to assess ocean inundation as a result of a breach of the coastal barrier and overflows from Lake Reeve as a result of catchment flooding, Figure 10-1 and Figure 10-2.

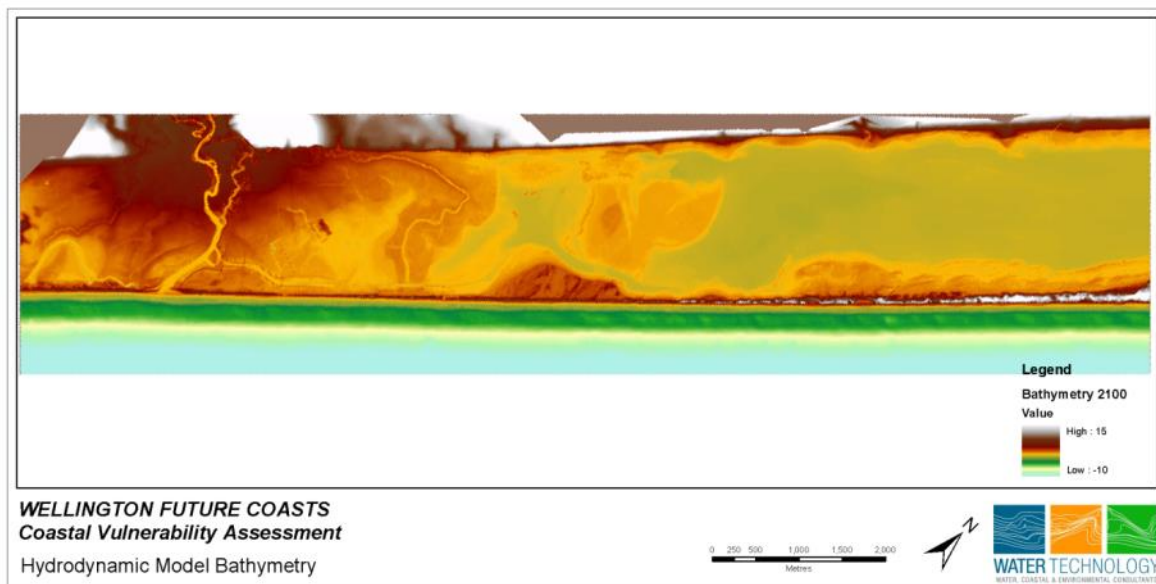


Figure 10-1 Model Extent of Existing MIKE21 Fixed Grid Model

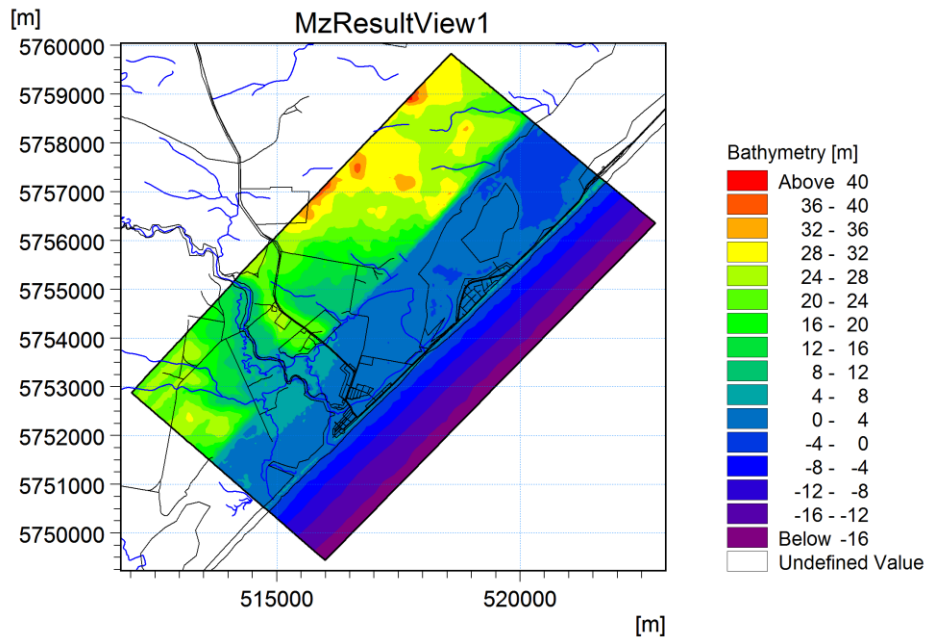


Figure 10-2 Model Extent of Existing MIKE21 Flexible Mesh Model

Both models cover essentially the same study area extent, from approximately the 10m offshore isobath to the higher elevation areas associated with the marginal bluff inland (a former marine cliff), and in the longshore direction from just north of Lake Dennison to Lake Reeve.

The fixed grid model has a 5m grid resolution, with levee features specifically incorporated, while the flexible mesh model has a variable mesh resolution across the model domain, with 10-20m in the township. The flexible mesh model resolution is currently too coarse for detailed flood modelling, however the advantage of the flexible mesh model is that it can be increasingly refined to accurately represent small topographic features in areas of interest without having to alter the size of the grid elements in other locations. This allows greater detail with less impact on model run times. Neither of the models currently includes pipes or the regulating structure. Both types of model can include rainfall on grid as well as the more standard point source inflows.

After reviewing both models, we propose to refine and update the flexible mesh model for this study. This is due to the ability of the flexible mesh model to be refined in areas of interest while minimising impacts on model run times.

The model will predominantly be a 2D model with hydraulic structures such as culverts, bridges and pipes modelled in 1D. The selection of the 2D model extent is a compromise between geographical coverage and model grid resolution.

Preliminary testing during this review has indicated that in the township areas and other locations of specific interest a fine mesh element size can be used (3-5m). In other areas the mesh resolution will be increased up to 10-12m depending on the location and the requirements to resolve specific topographic features. Mesh resolutions will be refined during the model development to optimise model run times.

The model's inflow boundaries will vary depending on the flood mechanism(s) being assessed. For instance, for a catchment generated flood event the model's upstream boundaries will use flow hydrographs generated from the RORB model for Merriman Creek and local tributaries. The model's downstream boundary will use a time series ocean boundary condition.

Hydraulic roughness parameters will be adopted using industry standard values that represent the creek and floodplain vegetated condition, and will be adjusted as required to calibrate the models.

Sensitivity analysis will be performed on hydraulic roughness to assess the model's sensitivity to the adopted parameters.

A 2D roughness map will be developed including appropriate roughness values for residential/commercial/industrial properties, roads and car parks, waterways, public open space, dense vegetation and sparse vegetation.

10.2.2 Ocean Water Levels

The Australian Rainfall and Runoff Revision Project 18 is currently investigating the interaction between coastal processes and severe weather events. As detailed in the Project 18 Stage 2 report, where the level of dependence between extreme rainfall and storm surge has not been determined it recommends assessing whether accounting for dependence would result in significant change to the flood level for the area of interest. We therefore propose to do this through modelling the following events for an open entrance condition:

- 1% AEP catchment generated flood event with a 1% AEP storm tide (peak storm tide elevation based on McInnes et al., 2009 under present mean sea level, Table 7-2)
- 1% AEP catchment generated flood event with an astronomical tide only,
- 1% AEP storm tide (peak storm tide elevation based on McInnes et al., 2009 under present mean sea level, Table 7-2) with no catchment flood event (assumed baseflow condition).

These three simulations are slightly different to those proposed in the original proposal and reflect the latest guidance from Project 18, which recommends testing the level of dependence between catchment flows and storm tide in this way (). Depending on the outcomes of these simulations we may undertake additional sensitivity test to select an appropriate downstream boundary condition for the design event simulations.

The offshore water level boundary will be generated using the following approach:

- Apply a cosine wave with an amplitude equal to that of the 1% AEP storm surge at Seaspray (McInnes et al., 2009),
- Add to this the predicted astronomical tidal water levels for Seaspray (Table 7-1),
- Analyse the resultant time series and select a suitable period where the peak water level is equal to the 1% AEP storm tide level for Seaspray (Table 7-2).
- Apply the selected time series of the model boundary, with further adjustments as required to ensure the correct storm tide elevation at Seaspray.

The process is then repeated for the estimated 10% AEP storm surge and storm tide peak values. This is the same approach as used to develop a storm tide boundary condition for the Corner Inlet Storm Tide Assessment (Water Technology, 2014b) and for previous storm tide assessments at Seaspray (Water Technology, 2010 and 2014a).

Based on the outcomes of this assessment and discussions with WGCMA, suitable ocean water level conditions will be adopted for the remaining design flood events.

A sensitivity analysis will be undertaken for the 10% catchment generated flood event with a 1% storm tide including +0.8m sea level rise.

10.2.3 Lake Reeve Water Levels

To assess flooding impacts from Lake Reeve, boundary conditions will be extracted from the Gippsland Lakes hydrodynamic model, used in the Coastal Hazard Assessment (Water Technology, 2014a) and applied as an inflow boundary to the local Seaspray model.

The previous modelling of Gippsland Lakes including Lake Reeve was undertaken for the estimated 10% AEP catchment flood under present, +0.2, +0.4 and +0.8 m sea level rise conditions and 1% AEP catchment flood under present and a +0.2 m sea-level rise condition. A key result of this analysis was that the floodplains around Seaspray only became inundated during the 10% AEP flood event under the +0.8 m sea level rise scenario. However, the flood levels did not reach a height sufficient to overtop the levees surrounding Seaspray.

For this study, it is proposed that boundary conditions for the 10% AEP flood with +0.8 m of sea level rise will be extracted from the Gippsland Lakes model and used as input to the local refined Seaspray flood model. This will allow more detailed assessment of the impacts of this event on the township and surrounds.

10.2.4 Entrance Conditions

Merriman Creek estuary is an ICOLL (intermittently closed and open lake or lagoon) whereby under low creek flows and/or on-shore wave conditions the entrance channel is small or closed off from the ocean. When the entrance is closed the water level in the estuary will be elevated and can influence flood extents, particularly for more frequent events. It is understood that the entrance is mechanically opened during flood events, although in some events it tends to be resistant to opening. In large floods it is likely the entrance will open naturally.

Under low creek flows and/or strong on-shore wave conditions, the entrance channel of Merrimans Creek is small or closed off from the ocean. Under such conditions the water level in the creek at Seaspray increases until the flows overtop the sandy berm at the entrance or the creek flows are sufficient to erode an entrance channel through the berm. In order to understand the opening and closing regime of the Merriman Creek entrance, the water level gauge at Seaspray will be analysed along with creek inflow record to ascertain under what river flow condition the entrance is open, closed or perched. An initial review of the water level information indicates that when closed the estuary water level increases up to around 1.6 m AHD, after which an opening is initiated. The conditions at the ocean entrance of Merriman Creek are likely only important for more frequent flood events as during rare events (i.e. 1 % AEP flood) an entrance will scour rapidly to the size required to convey the flood through the entrance berm.

For a given design flow, the likely resultant entrance channel dimensions will be estimated. Sediment size information is available for Seaspray from the Local Coastal Hazard Assessment and this will allow calculation of the critical velocity for erosion of the entrance channel. Typical outlet dimensions are also visible in flood aerial photography of the 1978 and 1993 events.

The MIKE FM sand transport module will be run coupled to the flood model for the 10% AEP event to dynamically simulate the effect of flood flows on the entrance channel. This will provide an indication of the rate of scour and changes in flood levels as a result of the entrance channel forming.

Based on the results of the analysis an appropriate entrance channel configuration will be adopted for each design flood event. The choice of ocean water level conditions for the design events will be determined based on the outcome of the catchment and ocean flooding sensitivity assessment (described below). It is anticipated that the downstream ocean boundary condition for the catchment flood design scenarios will be based on a typical tidal condition, and sensitivity tests will be conducted for +0.2m, +0.5m and +0.8m sea level rise.

10.2.5 Stormwater Modelling

Stormwater modelling will be undertaken using a direct rainfall on grid approach. For this type of hydraulic modelling, the design rainfall estimates are applied directly to the hydraulic model surface. Land planning layers, cadastral and aerial imagery will be used to identify unique fraction impervious and loss characteristics for each land parcel in the study area. A representation of Wellington Shire Councils pipe and pit network will be included in the model as 1D elements.

11. DATA GAPS

The following data gaps were identified:

- Merriman Creek bathymetry
- Caravan Park topography and drainage information

11.1 Merriman Creek and Lagoon Bathymetry

LiDAR data does not capture the bathymetry of rivers and waterbodies below the water surface. In the case of the Merriman Creek lagoon, it is fortunate that the Vicmap Elevation Coastal LiDAR dataset was captured when the water level was relatively low (0.8 m AHD), however there remains some uncertainty around the bathymetry of the lagoon below this water level. Since the lagoon is a backwater area, the bathymetry is not of critical importance, however some information on typical bathymetry near the entrance would be valuable.

Following completion of the draft preliminary report additional bathymetric survey of the lower sections of the lagoon was undertaken to fill this data gap. This is discussed in the Hydraulics Report (R03).

11.2 Caravan Park Topography and Drainage

Since the collection of the various LiDAR datasets the Seaspray caravan park has been relocated to an area that was formally the football oval, as shown in Figure 11-1. The ground surface levels in this area have been raised and significant drainage infrastructure has been installed. Information was requested from the Seaspray Foreshore Committee for any “as-built” survey or drainage plans, however no information was available. Wellington Shire Council provided design information including ground levels and drainage layout and this information has been adopted for the study.



Figure 11-1 **Location of Seaspray Caravan Park**

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Seaspray Flood Study – Hydrology (R02)



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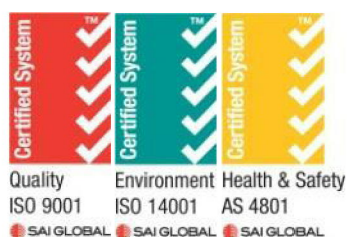


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1. INTRODUCTION

Water Technology was commissioned by the West Gippsland CMA to undertake the Seaspray Flood Investigation. This project includes detailed hydrological and hydraulic modelling of Merriman Creek, taking into account the influences of ocean generated flooding and potential inflows from Lake Reeve, flood mapping of the Seaspray township and will provide recommendations for flood mitigation actions and works.

This report details the hydrological analysis undertaken for the project, and should be read in conjunction with the previous Data Review Report (Report 1).

2. STUDY AREA

Seaspray is a coastal town with 320 permanent residents which rises to more than 1,000 during summer. The township lies at the western end of the Ninety Mile Beach west of the Gippsland Lakes and approximately 200 km east of Melbourne. As detailed in the Urban Design Framework developed for the township by Wellington Shire Council (Wellington Shire Council, 2007), Seaspray is an important access point to the 90 Mile Beach Area and the extensive areas of small lot subdivision that extend easterly for 28km to Golden Beach and Paradise Beach.

The town is located at the downstream end of the Merriman Creek catchment, with the creek flowing along the western boundary of the township, before it passes through the 90 Mile Beach coastal dune system and enters the ocean. The study area is shown in Figure 2-1.



Figure 2-1 Seaspray Study Area

3. HYDROLOGY

3.1 Overview

The hydrological analysis for Seaspray consisted of a review of the hydrological context of the study area followed by flood frequency analysis and hydrologic modelling using RORB. The RORB model was calibrated on three gauged events and parameter sets for design were validated against flood frequency analysis and regional flow estimates. Due to uncertainty in gauged peak flow estimates, a review was undertaken of the rating curve for the Prospect Road gauge (227240) and changes to the rating curve were adopted for the 1993 flood event with a subsequent reduction of the peak flow estimate for that event.

3.2 Catchment Description

The Merriman Creek catchment has an area of 500 km² extending 50 km westward from the estuary mouth at Seaspray to a maximum elevation of 750 m AHD at Mount Tassie (Figure 3-1). The catchment consists of two main streams, Merriman Creek and Monkey Creek, whose confluence is just upstream of Giffard Road. A third minor catchment feeds a wetland swamp with significant storage east of Giffard Road. This wetland is unlikely to overflow into the Merriman Creek catchment in large events (up to at least the 2% AEP), with no outflow occurring in the RORB model (which has the storage explicitly included) for either the 1993 or 1995 calibration events.

The main stream length from the catchment divide to the mouth is 107 km with an equal area slope of 2.0 m/km.

The average annual rainfall at Seaspray is 620 mm. A steep rainfall gradient exists over the catchment with average annual rainfall reaching 1,500 mm in the headwaters, while at the catchment centroid the average annual rainfall is around 670 mm. Effectively the lower catchment is within the rain shadow of the Strzelecki Ranges.

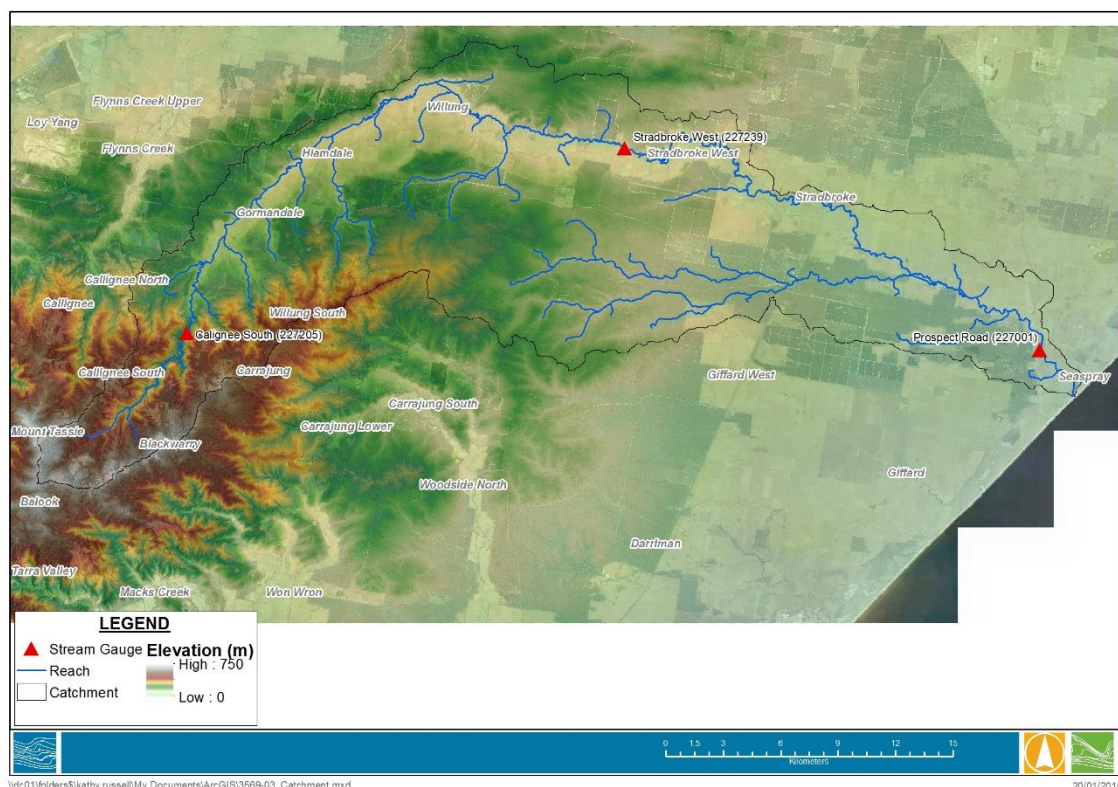


Figure 3-1 Merriman Creek Catchment

3.3 Design Rainfall Estimates

Design rainfalls were calculated for the 10%, 5%, 2%, 1% and 0.5% AEP events using the Intensity-Frequency-Duration analysis from AR&R (1987). The IFD parameters (Table 3-1) were obtained from the Bureau of Meteorology's IFD program website (www.bom.gov.au/ifd) for the catchment centroid.

Table 3-1 IFD parameters

2yr (50% AEP) 1hr rainfall intensity (mm/hr)	19.49
2yr (50% AEP) 12hr rainfall intensity (mm/hr)	4.42
2yr (50% AEP) 72hr rainfall intensity (mm/hr)	1.19
50yr (2% AEP) 1hr rainfall intensity (mm/hr)	40.02
50yr (2% AEP) 12hr rainfall intensity (mm/hr)	7.45
50yr (2% AEP) 72hr rainfall intensity (mm/hr)	2.49
Skew	0.36
F2	4.22
F50	15.22

Comparisons were made with the new IFDs developed for Project 1 of the Australian Rainfall and Runoff Revision Project (<http://www.bom.gov.au/water/designRainfalls/revised-ifd/>). On average the new IFD parameters were around 9% higher than ARR87 (for 1%-10% AEP events), with some AEP/Duration values up to 20% higher.

The 1987 IFD parameters were adopted to be consistent with existing methods for determining areal reduction factors, design losses and hydrologic parameter estimates, which were developed with the older IFD information.

Unfiltered temporal patterns from AR&R (1987) for Zone 1 were applied for the design events, as recommended in Hill et al. (1998) for use with their design losses. Rainfall depths were applied uniformly across the catchment.

Areal reduction factors from Siriwardena and Weinmann (1996) were applied to the point design rainfall estimates. These areal reduction factors are recommended for use in Victoria instead of the original AR&R values (Hill et al. 1998). These have now been recommended for adoption in the ARR Project 2 Stage 2 report (Engineers Australia, 2012).

3.4 Gauge Review

Three stream gauges exist within the Seaspray Catchment along Merriman Creek. Each of these gauges record instantaneous flow data over various time periods as listed below. Prospect Road Seaspray is the closest gauge to the Seaspray Flood Study mapping area, and the design flows determined at this location provide the design event inflows to the hydraulic model. Therefore the Prospect Road Gauge was the focus of the Flood Frequency Analysis (FFA) and for calibration of the hydrologic rainfall-runoff model. The Stradbroke West Gauge is located centrally within the catchment and was used in combination with the Prospect Road gauge to calibrate the hydrologic model. The Calignee South Gauge, as shown in Figure 3-1, is located in the upper reaches of the catchment and was not used specifically within the hydrologic assessment.

Table 3-2 Streamflow Gauges in Merriman Creek catchment

Gauge No.	Location	Period	Years	Catchment Area (km ²)
227240	Merriman Creek @ Prospect Road Seaspray	1983-2014	31	529
227001	Merriman Creek @ Seaspray (old Prospect Road site)	1966-1971	5	525
227239	Merriman Creek @ Stradbroke West	1983-2014	31	256
227205	Merriman Creek @ Calignee South	1946-1952 and 1965-2014	54	36

3.4.1 Merriman Creek at Prospect Road

The Merriman Creek at Prospect Road gauge (227240) has 31 years of instantaneous flow data. There were also 5 years of data from 1966-1971 at a previous gauging station (227001, referred to as Merriman Creek at Seaspray) located near the current Prospect Road gauge. Little is known about the reliability of the rating at the old gauge site, although numerous physical gauging measurements were undertaken for flows up to around 1,470 ML/d. The rating for the current gauge is officially considered reliable up to flows of 8,210 ML/d, however the small number of physical gauging measurements show considerable scatter across the whole range, particularly for low flows. Therefore flow estimates are likely to have a high degree of uncertainty.

Plotting of the published water levels and flows from the gauge clearly show a number of rating curves were used over the 32 year period (Figure 3-4). This is because the creek at this location has a sand bed which has been subject to significant instability over the record period. A changing water level – discharge relationship could be expected as the sand bed accretes and erodes.

The long term water level record (Figure 3-5) was also reviewed and it was found that following the 1993 event, gauged levels were elevated, even during low flow periods, for a number of years. The low flow water level, after suddenly increasing in 1993, slowly settled back to current levels over a period of around ten years. This is indicative of sand accretion in the channel after the 1993 event. The 1995 rating curve had a lower predicted flow for a given water level than the rating curve used in 1993, such that the recorded 1995 peak flow was lower than the 1993 peak, even though the water level was higher.

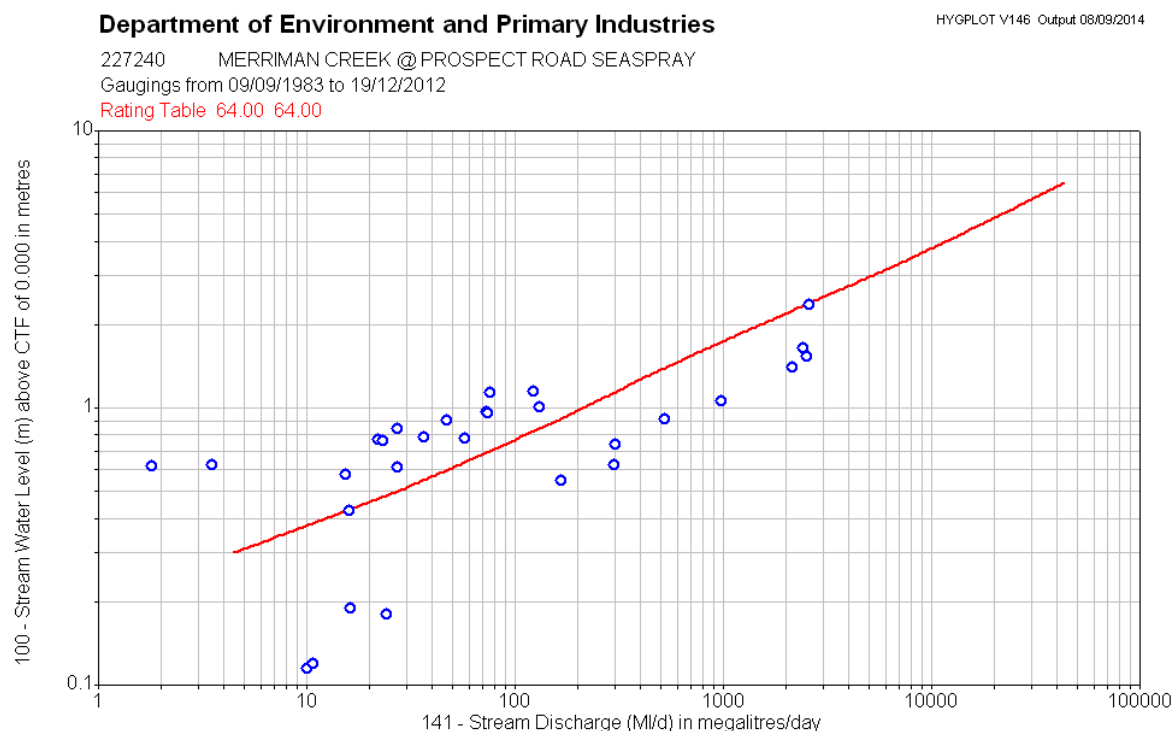


Figure 3-2 Rating Curve for 227240 Merriman Creek @ Prospect Road

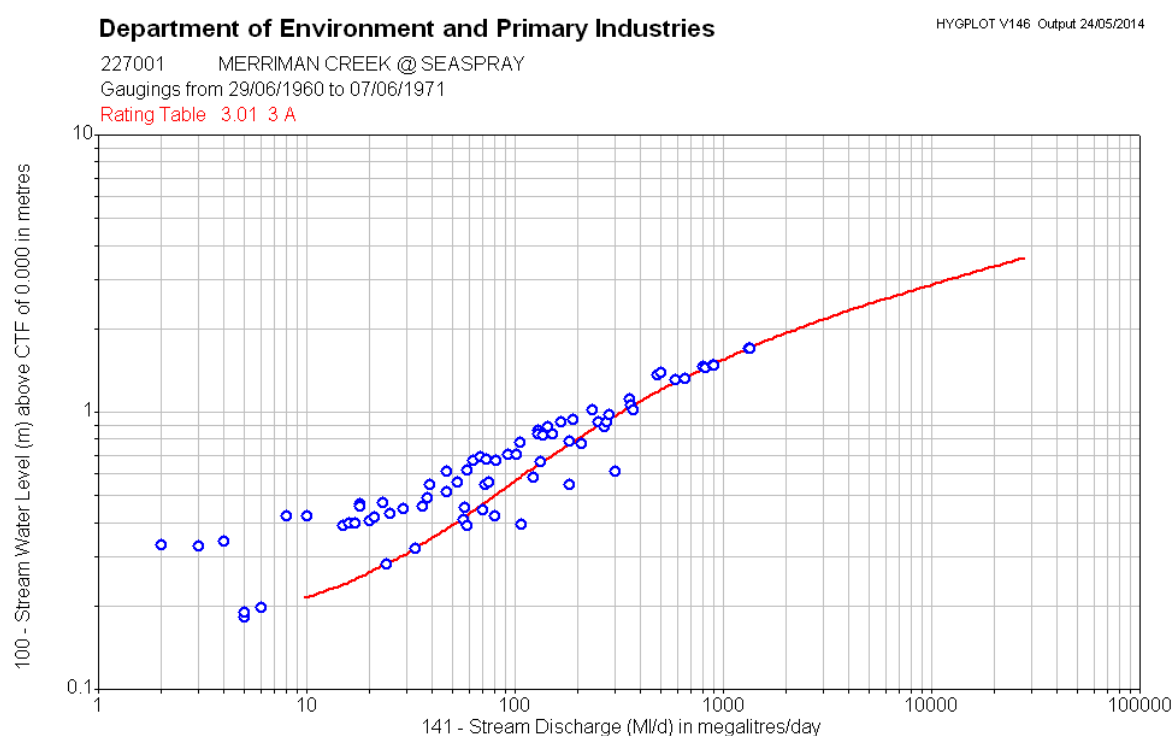


Figure 3-3 Rating Curve for 227001 Merriman Creek @ Seaspray

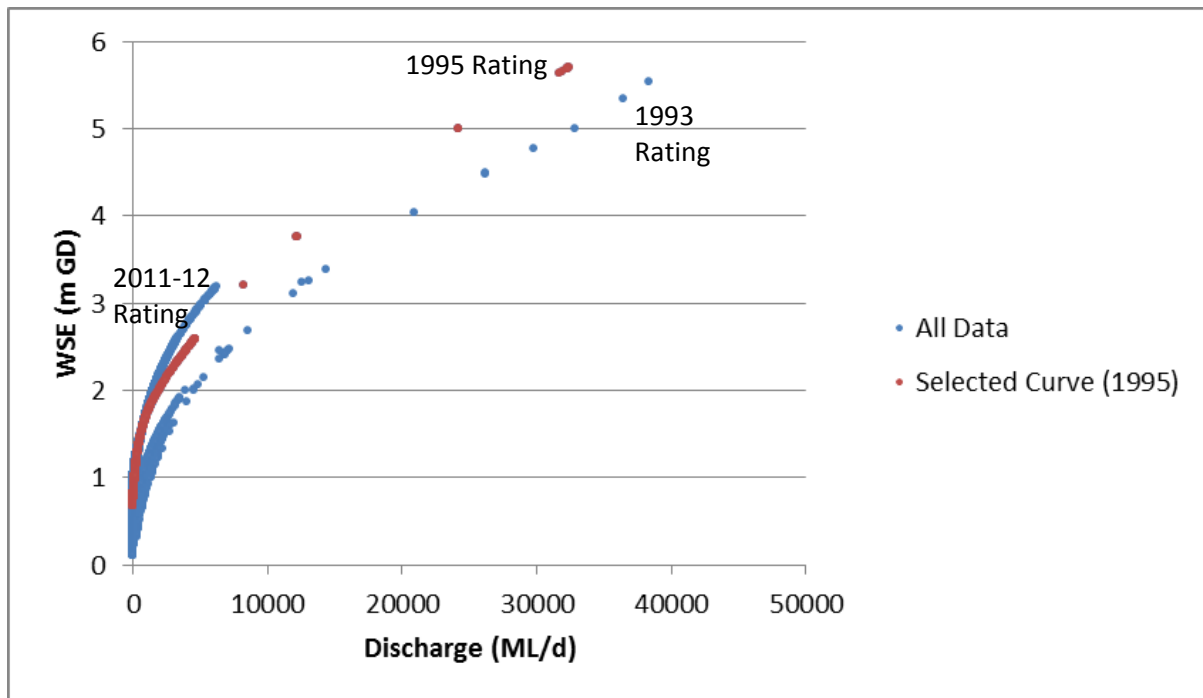


Figure 3-4 Water Level vs. Discharge Plot for Prospect Road gauge (227240) showing variation in rating curves across period of record

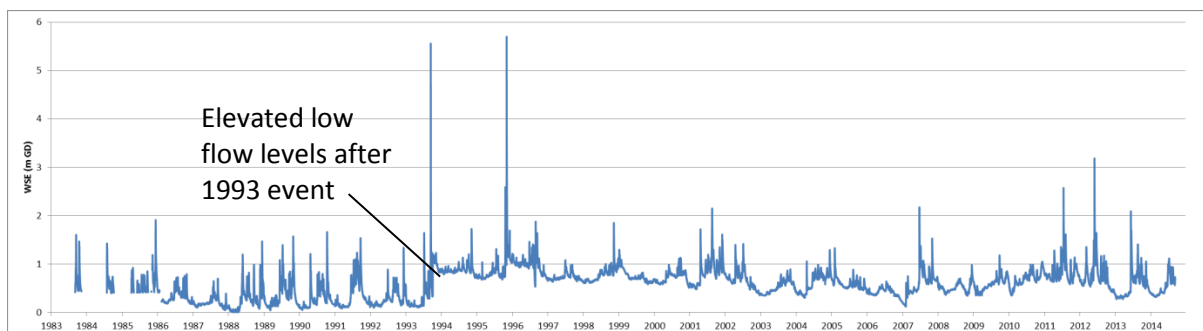


Figure 3-5 Merriman Creek at Prospect Road Gauge Levels

Two dimensional hydraulic modelling of the gauge reach was undertaken to verify the appropriateness of the rating curves for the calibration events. The 1995 event was run with the published flows and successfully reproduced the gauged water levels (Figure 3-6). Initial modelling of the 1993 event in the hydraulic model was not capable of reproducing the gauged levels with the gauged flows, with reasonable hydraulic parameters. The modelled water level over-predicted the measured peak water level by almost half a metre when published flows were used (Figure 3-7). A test was done with the use of the 1995 rating curve (i.e. reduced flows) to estimate a peak flow for the 1993 event, for which the model was able to reproduce the peak water levels. It is therefore recommended that due to likely changes that occurred in the physical characteristics of the gauge site during the 1993 event that the 1993 rating curve be applied before the peak, transitioning to the 1995 rating curve at and after the peak. The revised peak flow for 1993 using this approach is 30,240 ML/d, compared to the previous estimate of 38,360 ML/d.

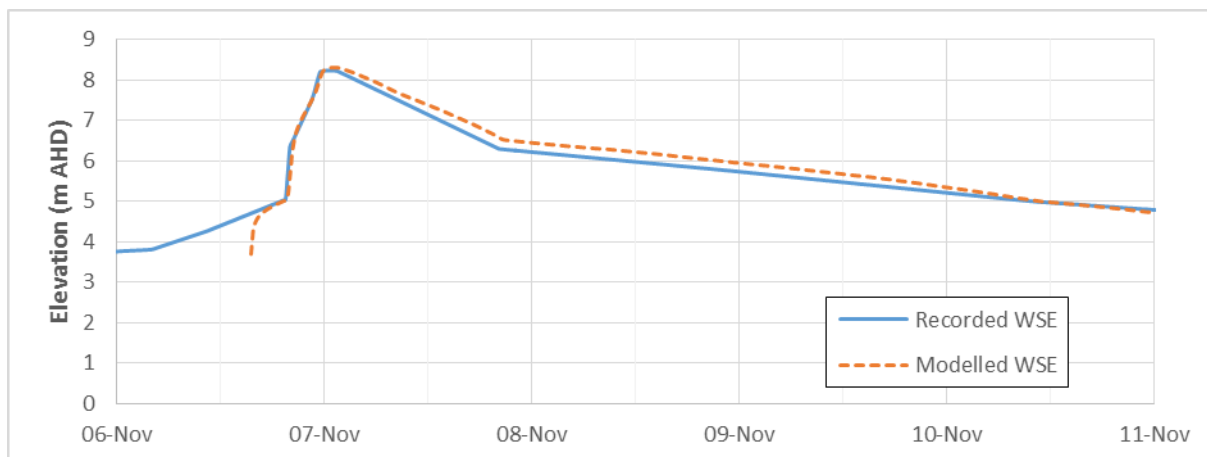


Figure 3-6 1995 Calibration Period Recorded and Modelled Water Surface Elevation at Gauge

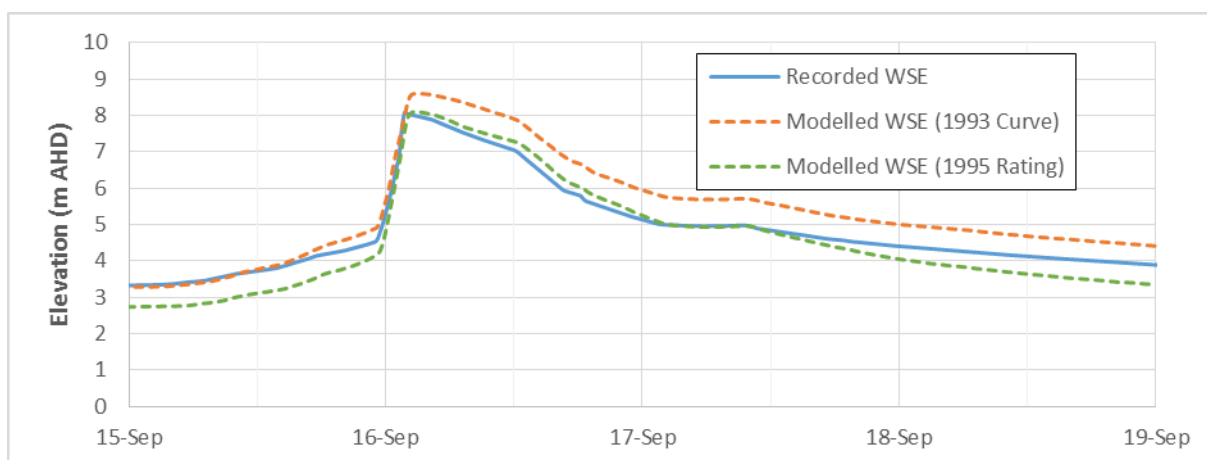


Figure 3-7 1993 Calibration Period Recorded and Modelled Water Surface Elevation at Gauge

3.4.2 Merriman Creek at Stradbroke West

The Merriman Creek at Stradbroke West gauge (227239) has 31 years of flow data from 1983-2014. The rating curve is based on a small number of high flow gauging measurements with very little scatter (Figure 3-8), and is therefore expected to be more reliable for estimating high flows than the Prospect Road gauge.

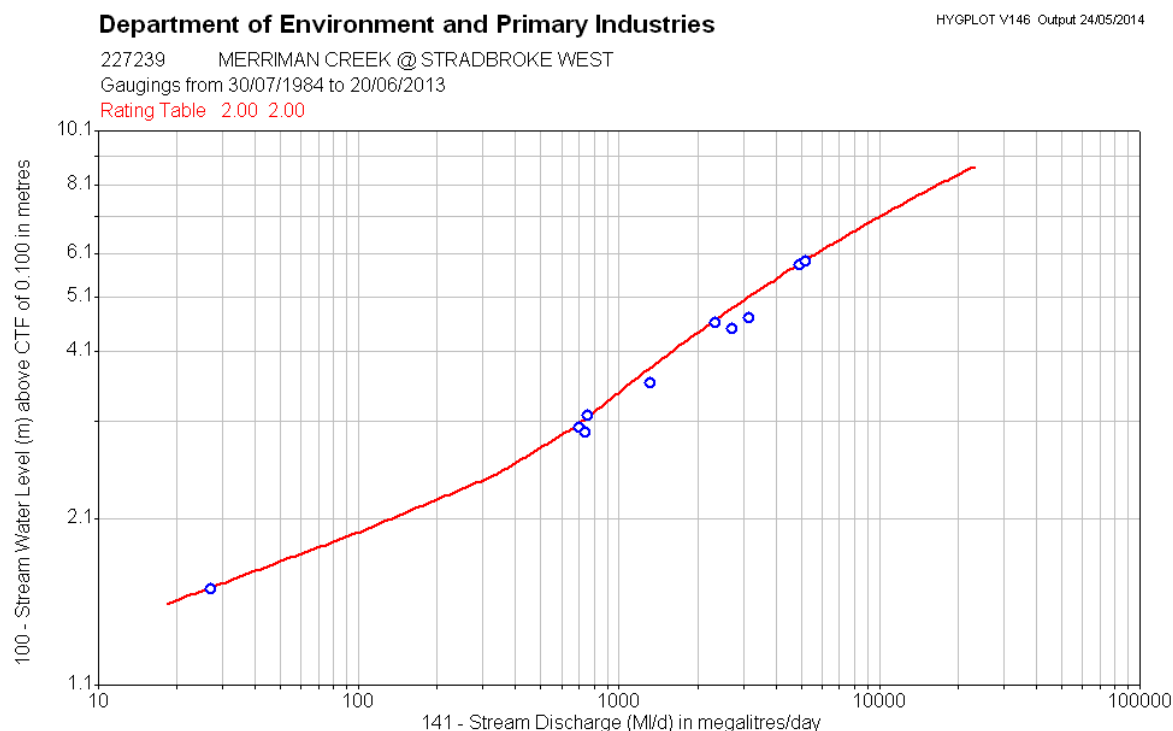


Figure 3-8 Rating Curve for 227239 Merriman Creek at Stradbroke West Gauge

3.5 Flood Frequency Analysis

3.5.1 Overview

Flood frequency analysis (FFA) allows the estimation of peak flows of selected Annual Exceedance Probability (AEP) events based on a statistical analysis. FFA was undertaken for Merriman Creek at the Prospect Road gauge (no. 227240) in order to provide peak flow estimates at Seaspray for verification of the RORB hydrologic model results.

The FFA was undertaken on the annual flow series from the gauge records using the analysis program FLIKE, as recommended in the updated ARR guidelines. FLIKE uses a Bayesian approach to parameter fitting (either the Global Probabilistic or Quasi-Newton fitting algorithms) to the records in order to assess the return period of different magnitude flows. There are a number of probability distributions which can be used to undertake an FFA, including the Log Pearson III, Log-Normal, Generalised Pareto, Generalised Extreme Value and Gumbel distributions and a selection of these are applied in the analysis, with the 'best fit' distribution adopted in the final assessment.

The flow series analysis assumed that the flow year was aligned with the calendar year which is appropriate given the typically wet winter and dry summer climate of the region.

3.5.2 Merriman Creek at Prospect Road

Annual maximum flows were extracted from the gauge record for Merriman Creek at Prospect Road gauge (227240) and are listed in Table 3-3. The 1993 peak flow was adjusted down from its gauged peak of 38,330 ML/d to 30,270 ML/d as per the rating curve review detailed in Section 3.4.

Table 3-3 Summary of annual peak flows for Merriman Creek at Prospect Road

Year	Max Flow (ML/d)	Year	Max Flow (ML/d)	Year	Max Flow (ML/d)
1952	16,760*	1989	2,150	2003	120
1966	800	1990	2,470	2004	360
1967	1,210	1991	2,030	2005	410
1968	1,410	1992	1,420	2006	40
1969	13,480	1993	30,270**	2007	1,950
1970	11,360	1994	1,050	2008	50
1971	28,150	1995	32,410	2009	290
1978	28,510*	1996	1,470	2010	130
1983	2,730	1997	200	2011	3,210
1984	2,080	1998	1,390	2012	6,190
1985	3,480	1999	310	2013	1,740
1986	370	2000	220	2014	290
1987	280	2001	1,880		
1988	1,810	2002	520		

*Estimated in 1980 flood study (Camp Scott Furphy 1980)

**Reduced from 38,330 ML/d gauged peak to 30,270 ML/d as per gauge review

Historic floods are known to have occurred in 1880, 1951, 1952, and 1978. These are floods that are known to have occurred within the catchment, but which have not been captured in the gauged record. The known information about these floods is shown in Table 3-4. The floods with known magnitude have been included in the annual series with the remaining years accounted for as peaks under and over a threshold as shown in Table 3-5. The 1880 flood is assumed to have been larger than the 1952 flood although this is not certain.

Table 3-4 Summary of historic flooding in Seaspray

Year	Comment
1880	Larger than 1951 flood
1951	Reported as the worst flood in 60 years
1952	Larger than 1951 flood. Unknown whether larger than 1880 flood. Discharge estimated at 16,760 ML/d (Camp Scott Furphy 1980)
1978	Reported in 1980 as the largest flood on record. Unknown whether larger than 1880 flood. Discharge estimated at 28,510 ML/d (Camp Scott Furphy 1980)

Table 3-5 Treatment of historic flood peak threshold

Threshold (ML/d)	Peaks above threshold	Peaks below threshold
16,760	1 (1880 flood. 1952 and 1978 peaks included explicitly in annual series)	94

The FFA was undertaken with (Figure 3-10) and without (Figure 3-9) historic information for comparison. The inclusion of historic events (1952 – 1983) produced a much narrower 90% confidence limit and a significantly flatter plot, but with a similar 10% AEP flow estimate. The FFA with historic information results in a 1% AEP flow estimate of 41,130 ML/d (476 m³/s) with 5-95% confidence limits of 23,070 to 90,200 ML/d (267 to 1,044 m³/s).

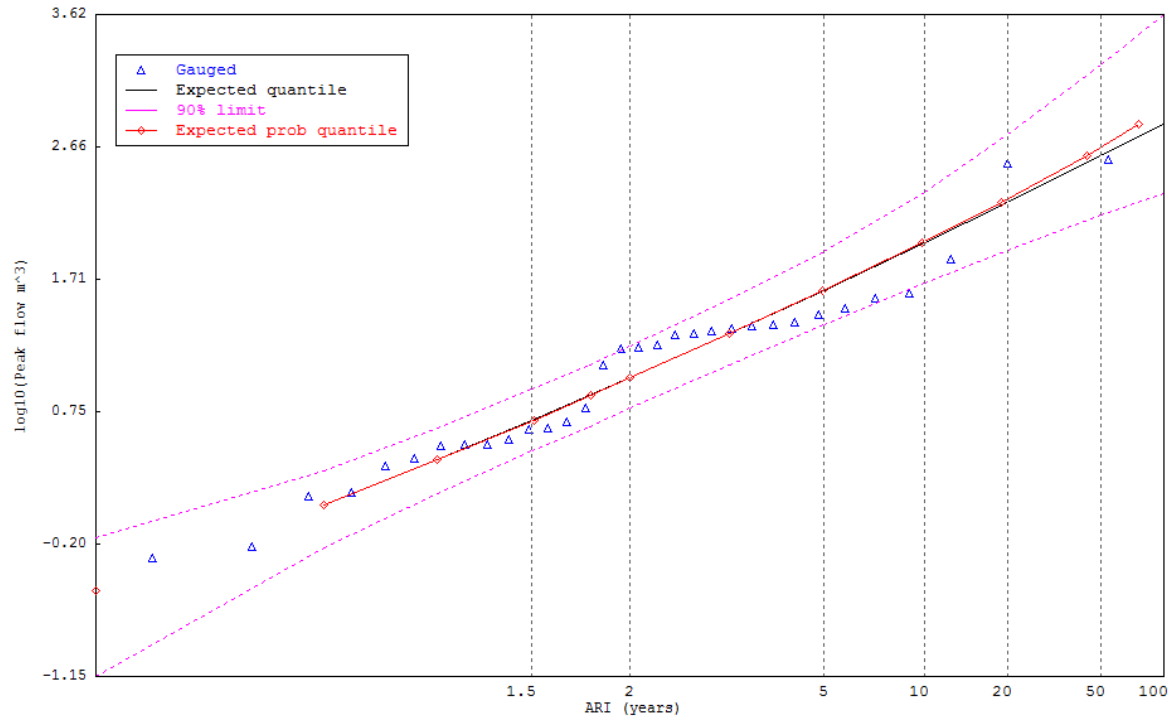


Figure 3-9 Log Pearson III distribution fitted to annual series for Merriman Creek at Prospect Road (gauge data only)

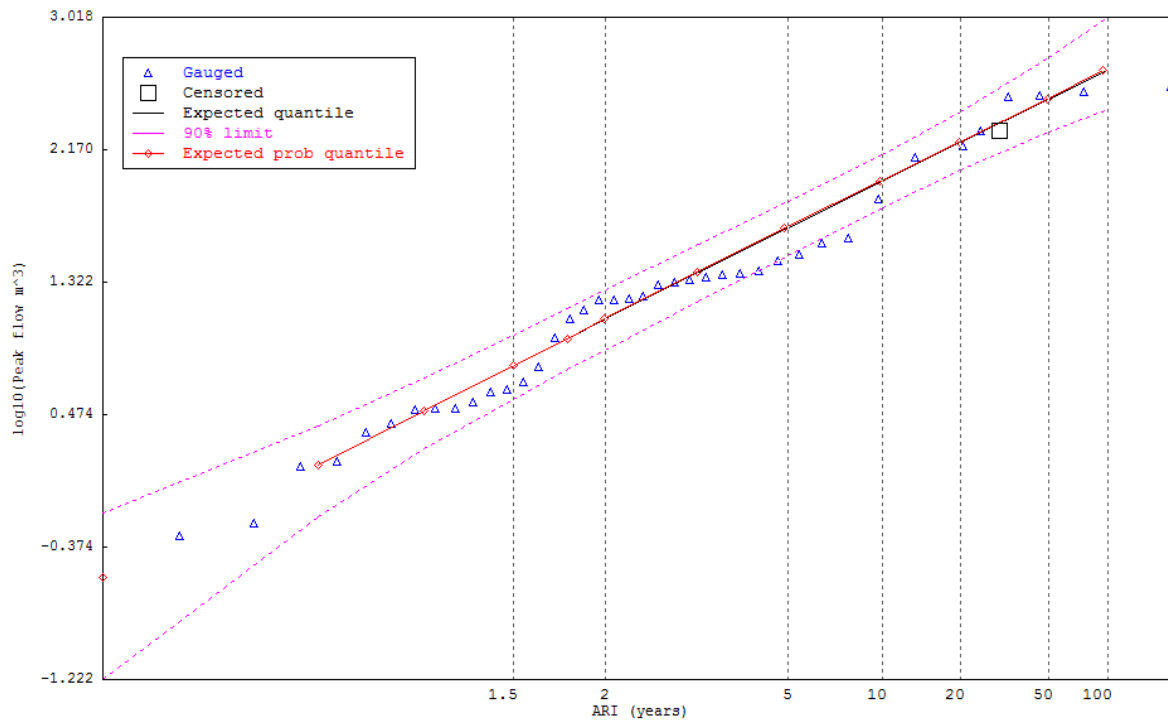


Figure 3-10 Log Pearson III distribution fitted to annual series for Merriman Creek at Prospect Road (with historic information included)

Table 3-6 FFA Peak Flow Estimates for Merriman Creek at Prospect Road

AEP	Peak Flow Estimate (ML/d)		Peak Flow Estimate (ML/d)	
	Log Pearson III (1983 – 2014)	5-95% Confidence Limits	Log Pearson III (with historic information)	5-95% Confidence Limits
10%	8,040	(4,150 - 18,840)	8,120	(5,440 - 11,840)
5%	15,720	(7,080 - 48,470)	14,260	(9,420 - 22,380)
2%	34,210	(12,700 - 15,290)	26,960	(16,500 - 50,110)
1%	58,150	(18,230 - 35,600)	41,130	(23,070 - 90,200)
0.5%	95,390	(25,140 - 78,110)	60,570	(30,150 - 160,190)

3.5.3 Comparison to Regional and Previous Estimates

Estimates of peak flows using the Australian Regional Flood Frequency (ARFF) method and from previous studies by Cardno (2010) and Camp Scott Furphy (1980) are shown in Table 3-7.

The Cardno (2010) estimate aligns closely to that of the FFA without historic data (i.e. only 1983 – 2014), while the Camp Scott Furphy (1980) estimate is much flatter (as it does not include the 1993 or 1995 floods) and aligns closely with the ARFF peak flow. The FFA estimate with historic information included lies between the two.

The current FFA with historic information included is broadly consistent with previous work, however it provides more credible estimates than the previous analyses, as it includes much more gauged data than the 1980 analysis, as well as historical information on flooding prior to gauge records (which was not included in the 2010 analysis).

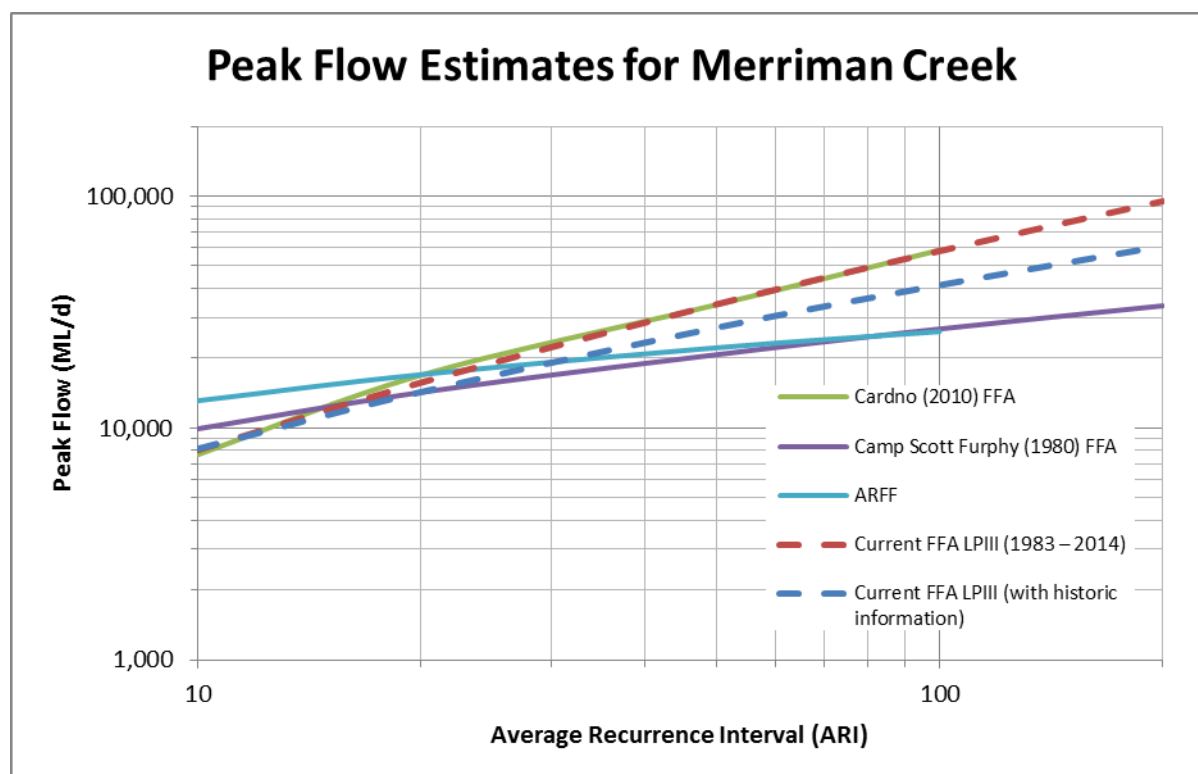


Figure 3-11 Comparison of Peak Flow Estimate Methods

Table 3-7 ARFF Flow Estimates and previous Flood Frequency Estimates for Prospect Road gauge

AEP	Peak Design Flow (ML/d) ARFF	Peak Design Flow (ML/d) (Cardno, 2010)	Peak Design Flow (ML/d) (Camp Scott Furphy, 1980)
10%	13,130	7,690	9,940
5%	17,020	16,930	14,260
2%	22,200	34,040	20,740
1%	26,090	58,580	26,780
0.2%	-	-	33,700

3.6 Hydrologic Modelling

3.6.1 Overview

A hydrological model of the catchment was developed for the purpose of extracting design flows to be used as boundary conditions in the Seaspray hydraulic model. The rainfall-runoff program, RORB (Version 6) was used for this study.

RORB is a non-linear rainfall runoff and streamflow routing model which is used for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas, connected by a series of conceptual reach storages. Design storm rainfall is input to the centroid of each subarea. Specified losses are then deducted, and the excess routed through the reach network.

There are two streamflow gauges within the catchment which were used to calibrate the RORB model. The Merriman Creek gauge at Prospect Road is located only 9 km from Seaspray and has 31 years of flow records from 1983-2014, covering the 1993, 1995 and 2012 floods and other smaller floods. As noted in Section 3.4 the accuracy of the rating curve for the Prospect Road gauge is likely to be relatively low, particularly for low flows. The Merriman Creek gauge at Stradbroke West has the same record period, and a more reliable rating curve. Parameter selection was based on calibration to both gauges and comparison to accepted regional methods, and the design flows were validated against the flood frequency analysis. The approach for the selection of routing and loss parameters is outlined in the following sections.

3.6.2 RORB Model Development

The ArchHydro tool was used to determine the initial delineation of catchments, sub-catchments and drainage paths of Merriman Creek for input to the hydrologic model (Figure 3-12). The catchment was split into 49 sub-catchments with areas ranging from 5.5 to 17.5 km². A RORB model was then constructed of the catchment using miRORB (MapInfo RORB tools).

The RORB model was at a resolution adequate to resolve the main drainage paths and sub-catchments, and to provide smooth hydrographs for the hydraulic model.



Figure 3-12 RORB model structure

3.6.3 Fraction Impervious

The fraction impervious (FI) was included in the hydrologic model by applying a representative FI for each planning zone, as listed in Table 3-8. The planning zones used for designation of FIs are shown in Figure 3-13. The values chosen to represent each planning zone were adopted from Melbourne Water's MUSIC guidelines (2010).

Table 3-8 Fraction Impervious Designation for Planning Zones

Zone Code	Zone Description	Fraction Impervious (FI)
B1Z – B5Z	Business (1 – 5) Zone	0.9
FZ	Farming Zone	0.1
FZ	Farming Zone - Forestry*	0.1
IN1Z – IN2Z	Industrial (1 – 2) Zone	0.9
LDRZ	Low Density Residential Zone	0.2
MUZ	Mixed Use Zone	0.7
PCRZ	Public Conservation and Resource Zone	0
PPRZ	Public Park and Recreation Zone	0.1
PUZ1	Public Use Zone - Service and Utility	0.05
PUZ2	Public Use Zone - Education	0.7
PUZ3	Public Use Zone - Health and Community	0.7
PUZ4	Public Use Zone - Transport	0.7
PUZ5	Public Use Zone - Cemetery/Crematorium	0.6
PUZ6	Public Use Zone - Local Government	0.7
PUZ7	Public Use Zone - Other Public Use	0.6
R1Z	Residential 1 Zone	0.45
R2Z	Residential 2 Zone	0.6
RCZ, RCZ1, RCZ2	Rural Conservation Zone	0
RDZ1	Road Zone - Category 1	0.7
RDZ2	Road Zone - Category 2	0.6
RLZ, RLZ2, RLZ4, RLZ6	Rural Living Zone	0.2
SUZ1, SUZ2, SUZ10	Special Use Zone (1, 2, 10)	0.6
TZ	Township Zone	0.55

*Farming Zone was split into farming and forestry areas to allow sensitivity to imperviousness caused by bushfire to be tested.

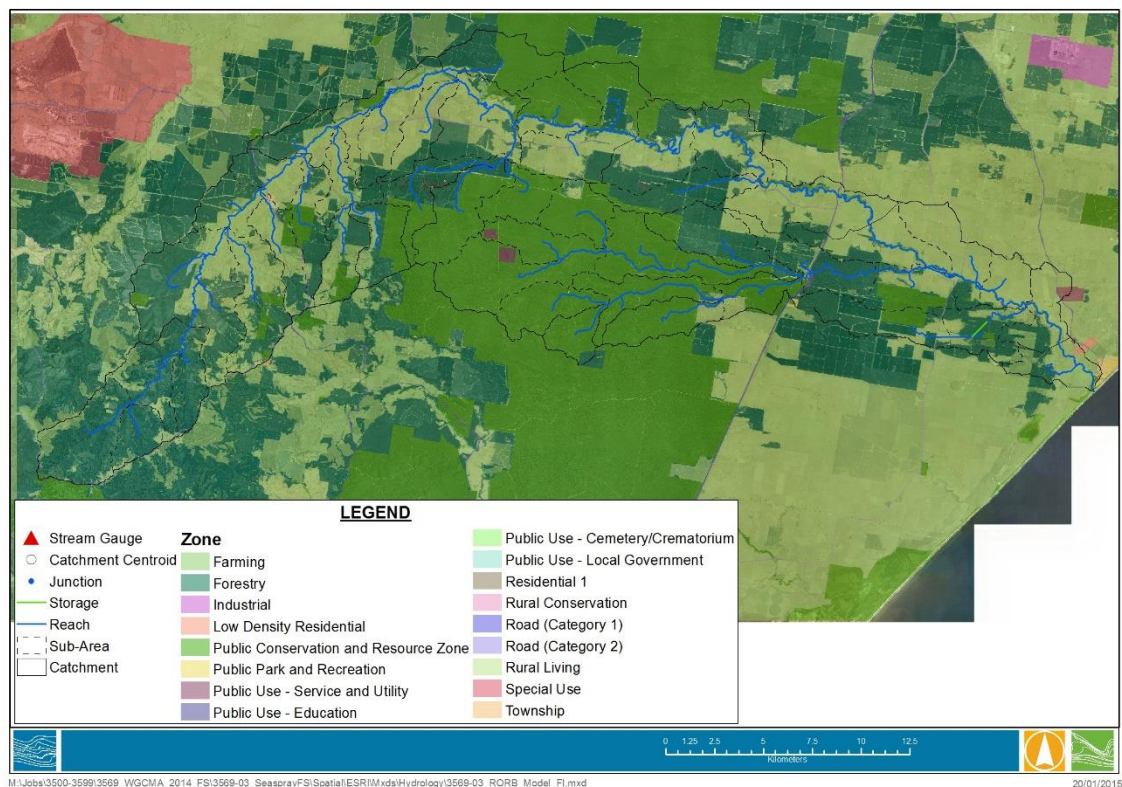


Figure 3-13 Planning Overlay used for Fraction Impervious

3.6.4 Calibration Events

Three recorded events were used for the calibration of the RORB hydrologic model; namely the September 1993, November 1995, and June 2012 floods. Each of these events was represented with a unique temporal and aerial rainfall pattern generated from local rainfall gauge records. The outflow hydrographs from the RORB model were then compared to stream gauges at two locations; Prospect Road, and Stradbroke West. A pluviograph was installed at Stradbroke West in 2006 and has been incorporated into the June 2012 calibration event.

The 1993 and 1995 events were large events with AEP between 1% and 2%, while the 2012 was a moderate event with AEP of around 10%. The 2012 event exhibited quite different behaviour to the other two, with a longer lag between the Stradbroke West and Prospect Road gauges and an attenuation rather than an increase in flow between the two gauges.

Figure 3-14, Figure 3-16, and Figure 3-18 illustrate the aerial rainfall patterns used in each of the calibration runs. Figure 3-15, Figure 3-17, and Figure 3-19 show the rainfall hyetographs and recorded flows at Prospect Road and Stradbroke West. The 1993 Prospect Road flow has been adjusted as detailed in Section 3.4.

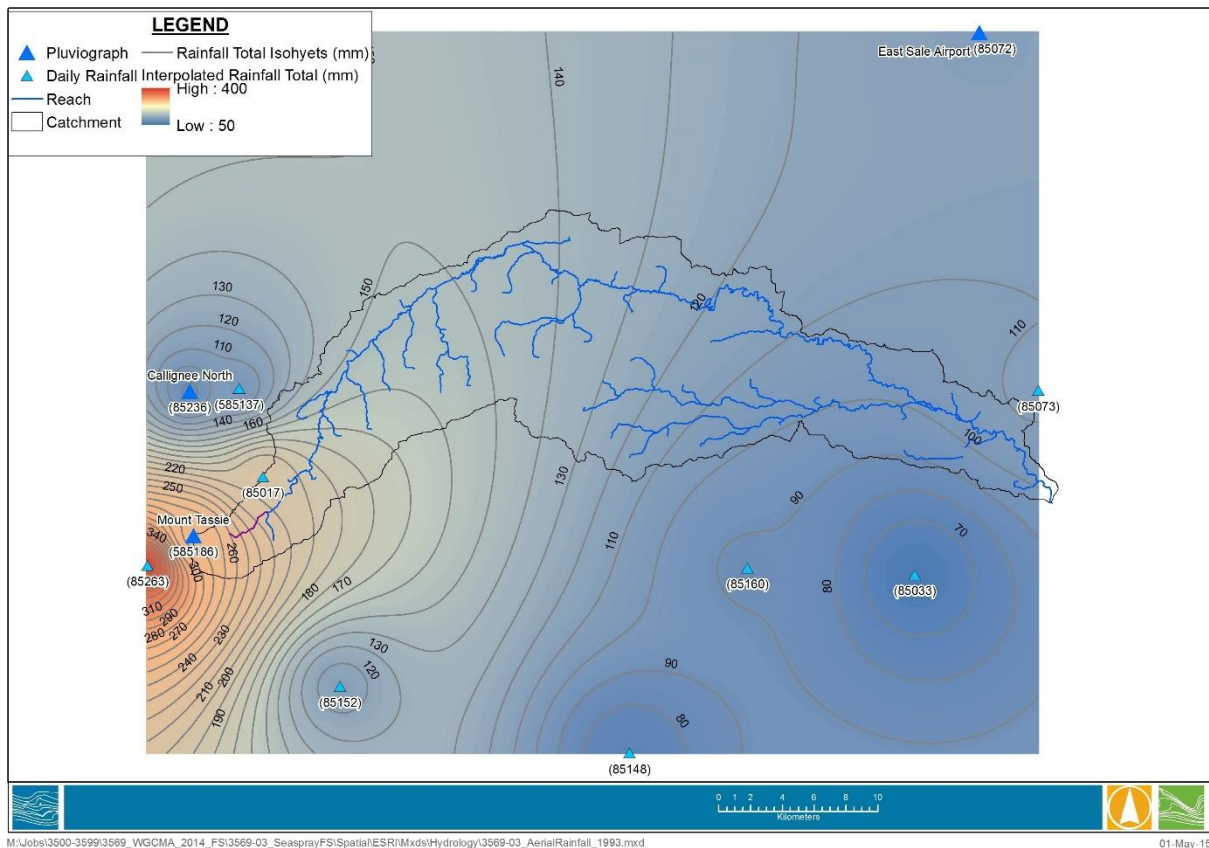


Figure 3-14 Total rainfall across the catchment for the September 1993 calibration event

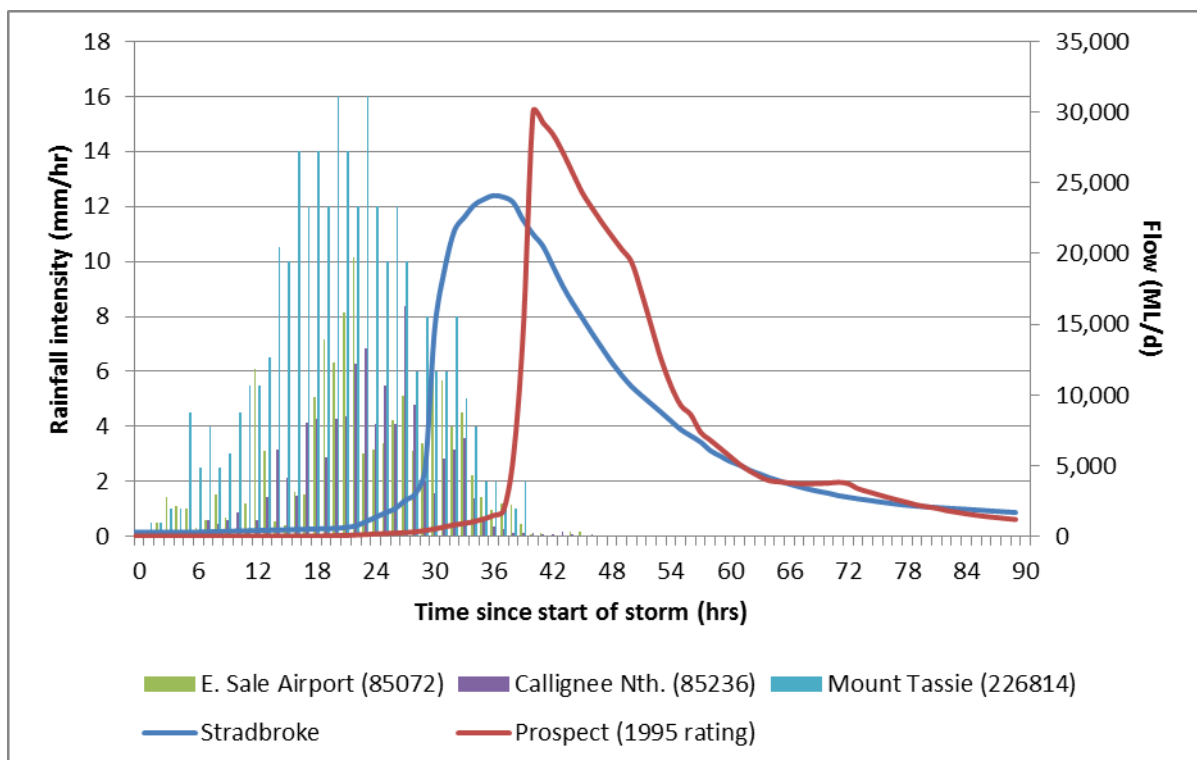


Figure 3-15 Calibration event hyetographs and instantaneous flow hydrographs for the September 1993 calibration event

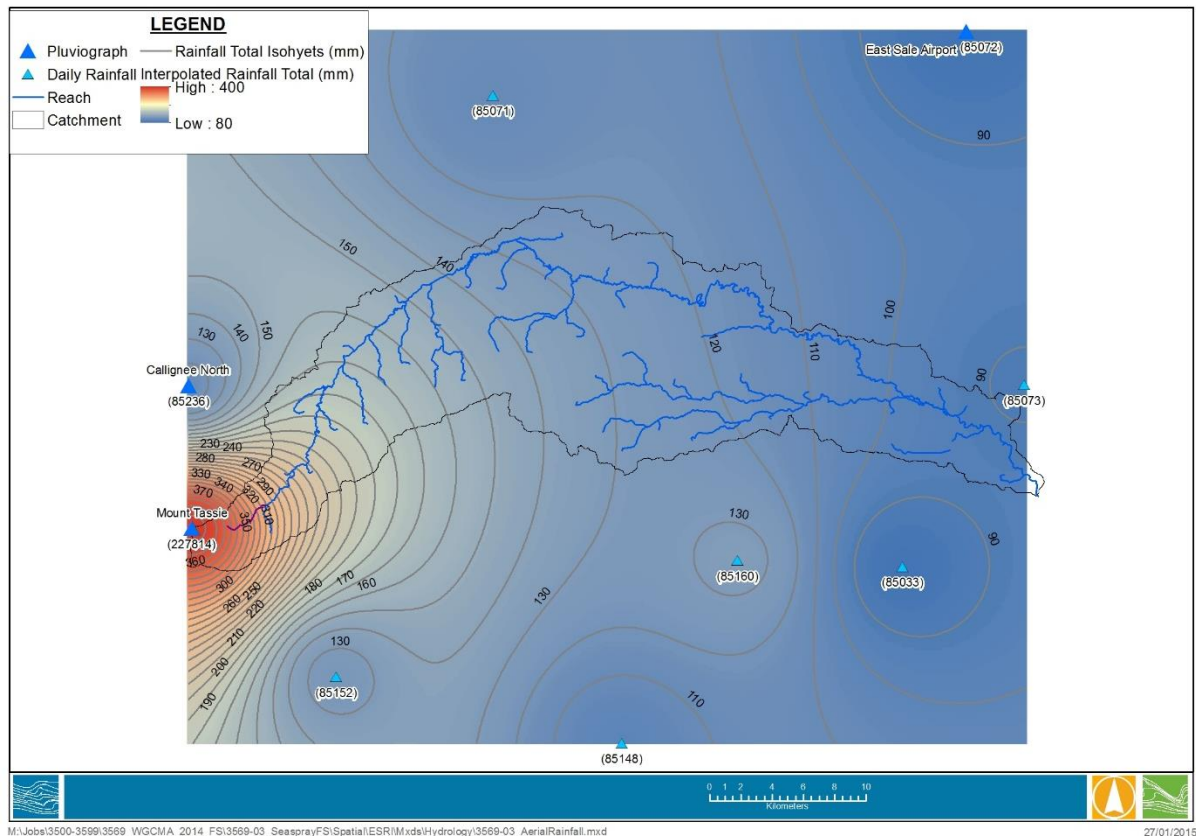


Figure 3-16 Total rainfall across the catchment for the November 1995 calibration event

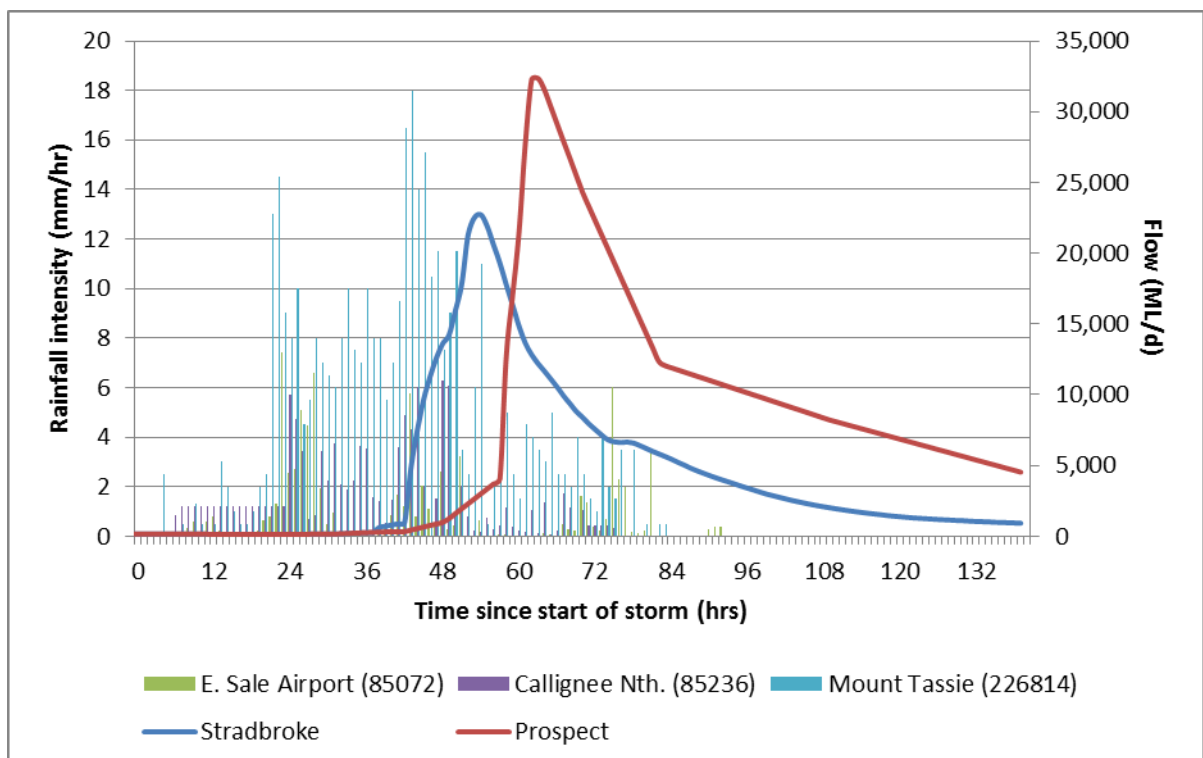


Figure 3-17 Calibration event hyetographs and instantaneous flow hydrographs for the November 1995 calibration event

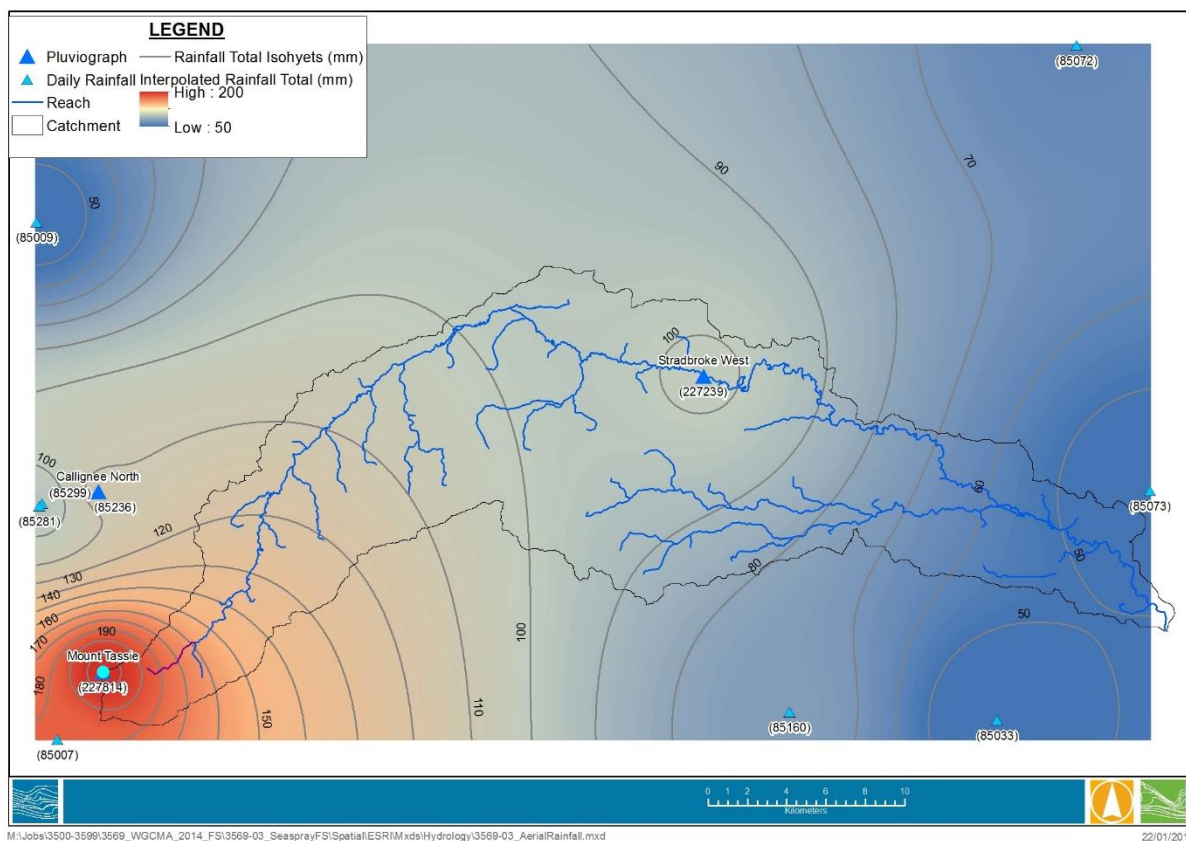


Figure 3-18 Total rainfall across the catchment for the June 2012 calibration event

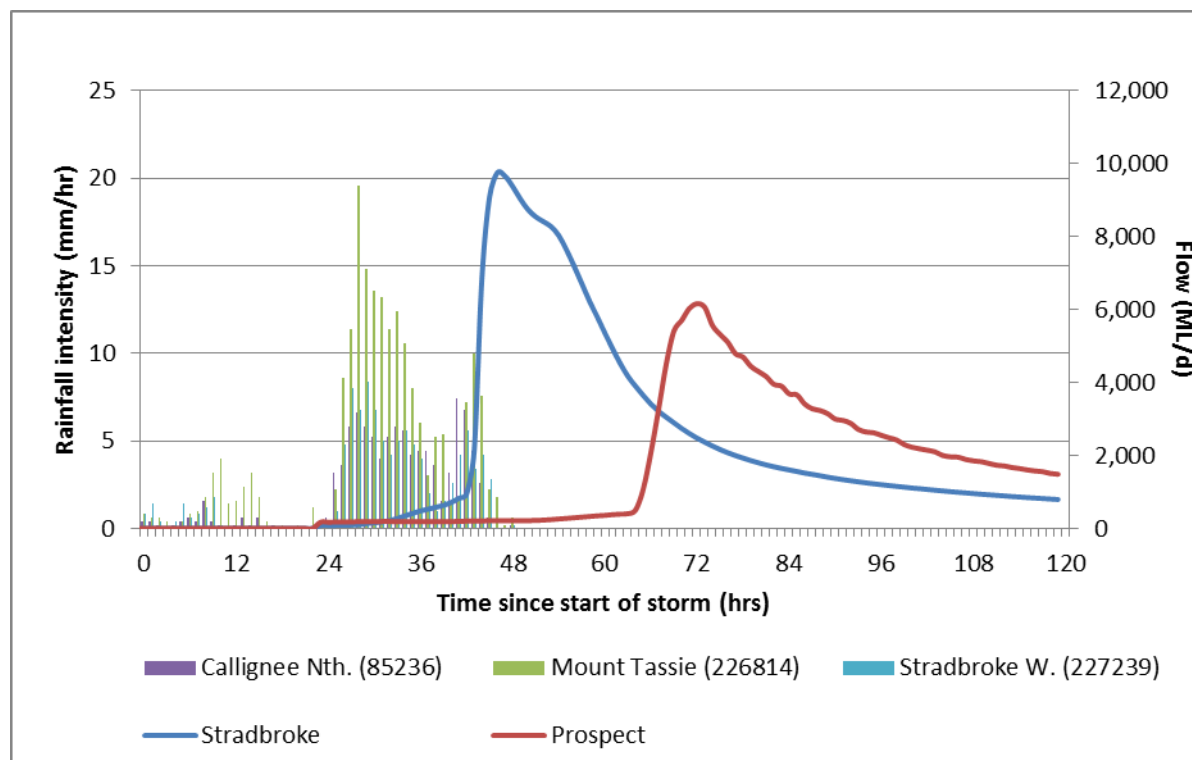


Figure 3-19 Calibration event hyetographs and instantaneous flow hydrographs for the June 2012 calibration event

3.6.5 Calibration of RORB Parameters

The RORB model was run using the initial loss/continuing loss model. The RORB storage and loss parameters – k_c , m , initial loss (IL) and continuing loss (CL) – were fitted for each calibration event to give the best fit to the observed hydrographs. Priority was given to fitting the hydrograph peak and shape, with the volume and timing fitted as a lower priority. The routing parameter k_c was set using a combination of calibration and regional equations. The exponent m was set to 0.8 as per RORB default.

A number of regional equations for the calculation of k_c that are recommended in ARR87 and the RORB manual were used to guide the selection of this parameter (Table 3-9).

Table 3-9 Regional equations for the calculation of k_c

Source	k_c
Aust wide Yu (1989) (Pearse et al 2002)	53
Vic MAR>800 mm - Eq 3.21 ARR87 (Book V)	42
Vic MAR<800 mm - Eq 3.22 ARR87 (Book V)	28
Aust wide Dyer (1994) (Pearse et al 2002)	62
Victoria data (Pearse et al, 2002)	69
Default RORB	49

The final calibrated model parameters for each event are summarised in Table 3-10. The fit of the RORB modelled hydrographs to the gauged hydrographs at the Stradbroke West and Prospect Road gauges for each calibration is shown in Figure 3-20, Figure 3-21, and Figure 3-22.

Table 3-10 RORB Calibration Parameters

Event	k_c	m	IL (mm)	CL (mm/hr)
1993	40	0.8	70	2.3
1995	28	0.8	40	1.0
2012	50	0.8	50	3.8

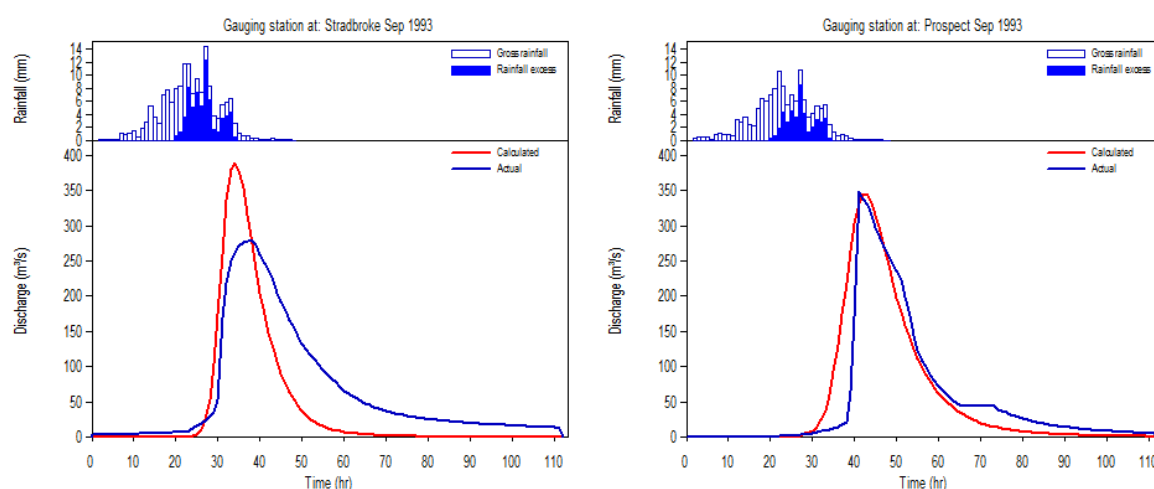


Figure 3-20 Calibration plots for September 1993

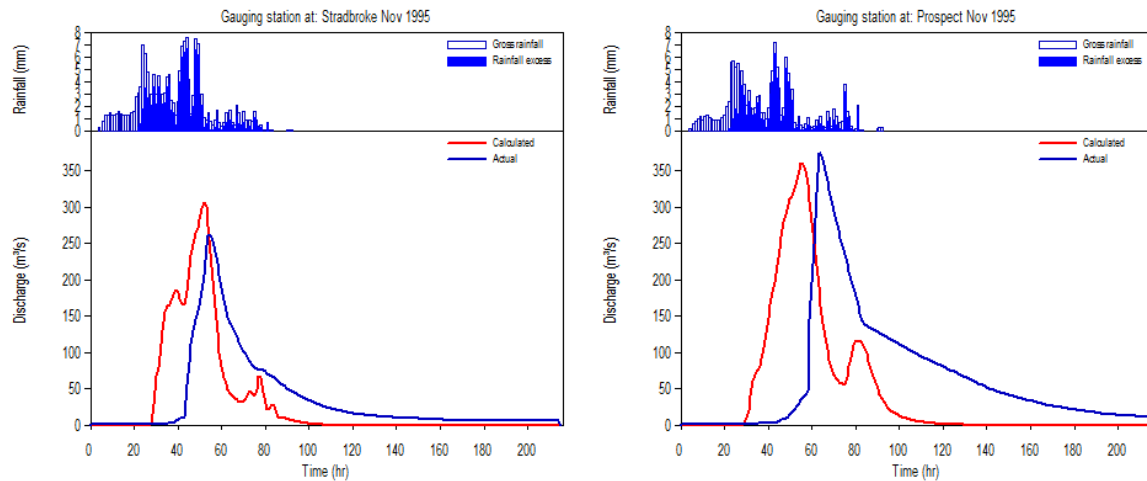


Figure 3-21 Calibration plots for November 1995

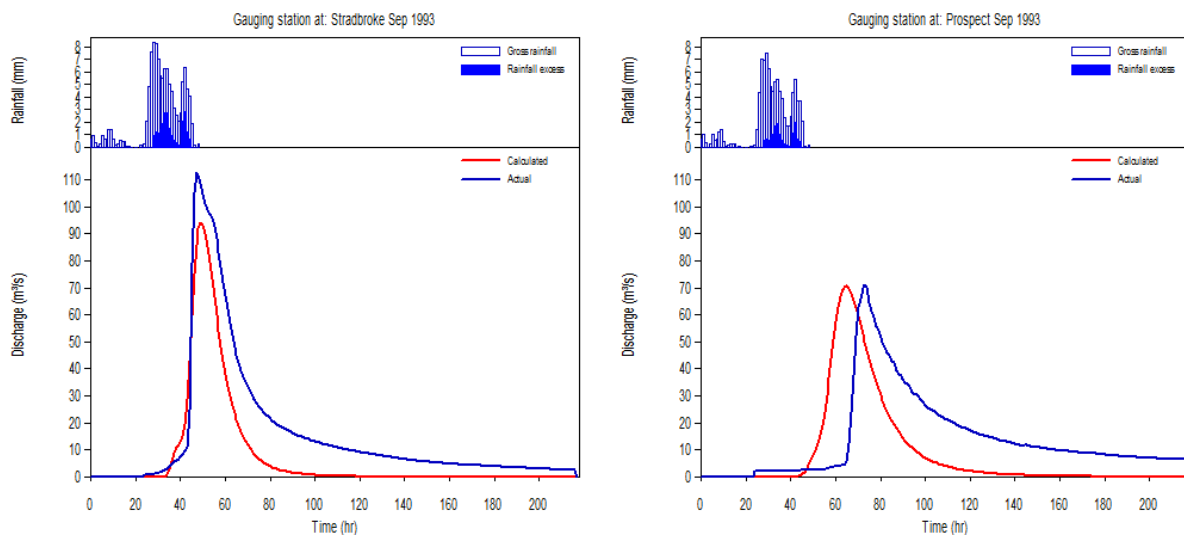


Figure 3-22 Calibration plots for June 2012

The shape and volume of the fitted hydrographs at Prospect Road agrees well with gauged data. The 1993 event was particularly well-matched in shape, timing and volume. The volume at Stradbroke West was a little low while the peak was too high, indicating that the routing parameters applied for the whole model may not be appropriate for prediction at Stradbroke west. This could be improved by use of an interstation area at the Stradbroke West gauge, which would allow RORB to use separate routing parameters upstream and downstream of that point. This issue is common in RORB modelling; however the aim of this model is to provide a reasonable flow estimate as a boundary to our hydraulic model, therefore matching the closest gauge to the study area (Prospect Road) is the main focus of calibration. The mismatch may also indicate that the spatial distribution of rainfall across the catchment was not well represented. The spatial variability across the gauges in and around the catchment is extremely high; therefore it is reasonable that there may have been additional variation within the catchment that was not picked up by the rain gauge network.

In two of the events (1995 and 2012) the RORB model predicted the peak of the hydrograph approximately ten hours early at the Prospect Road Gauge. One possible reason for significant variation in the timing of the hydrograph peak is poor capture of the actual temporal patterns within the catchment by the pluviographs, only two of which were located in the catchment. There were no pluviographs in the lower catchment for the 1995 event. For both, the timing at Stradbroke West was well matched.

The k_c values fitted for the calibration events are within the range of those provided by regional equations in Table 3-9. The initial losses tend to be higher than design values and continuing losses tend to be on the low side for the 1993 and 1995 events but reasonable for the 2012 event.

3.6.6 Design Loss Estimates

Design losses were estimated by the design loss prediction equations developed by Hill et al (1998) and (Hill et al. 2014). The losses recommended in ARR87 consistently overestimated peak flows. The Hill et al (1998) losses, in combination with the new areal reduction factors from Siriwardena and Weinmann (1996), produced peaks that were more consistent with the results of flood frequency analysis.

The initial loss is calculated by first calculating the storm initial loss using Equation 1, then the burst initial loss (Equation 2). The burst initial loss varies with storm duration and accounts for the embedded nature of AR&R design rainfalls (Hill et al 1998). The continuing loss is estimated using Equation 3.

Storm initial loss:

$$IL_s = -25.8BFI + 33.8 \quad (1)$$

Burst initial loss:

$$IL_b = IL_s \left\{ 1 - \frac{1}{1 + 142 \frac{\sqrt{\text{duration}}}{MAR}} \right\} \quad (2)$$

Continuing loss:

$$CL = 7.97BFI + 0.00659PET - 6.00 \quad (3)$$

The ARR Project 6 Stage 3 report (Hill et al. 2014) contains new recommended initial and continuing loss equations, as shown below. No recommendation has been made for the conversion of storm initial loss to burst initial loss.

Storm initial loss:

$$IL_s = 16.7 + 0.141P_{24h}^{2\%} - 0.291MedianAPI \quad (4)$$

Continuing loss:

$$CL = 3.0 \text{ mm/hr} \quad (5)$$

The values of the input parameters to the previous equations are as follows:

- Median API 34 mm
- BFI 0.52
- PET 1,007 mm
- MAR 670 mm
- $P_{24h}^{2\%}$ 144 mm

The design loss estimates produced by both methods are given in Table 3-11. The design initial losses tend to be lower than those calibrated in RORB, while the continuing losses tend to be higher. The Project 6 losses (Hill et al. 2014) are the most consistent with calibration out of the two sets.

Table 3-11 Design loss parameter estimates

Source	IL_s (mm)	IL_b (mm)	CL (mm/hr)
Hill et al (1996)	20	8-13 (for 9-72 hour events)	4.8
Project 6 Stage 3 Report (Hill et al. 2014)	27	-	3

3.6.7 Design Flood Validation

Various k_c and loss values were compared to the FFA at Prospect Road (including historical events) to determine the best fit for the design hydrology. The ARR Project 6 Stage 3 Report (Hill et al. 2014) did not find evidence of variation of loss values with AEP, a finding which is consistent with a range of previous studies. Therefore, rather than varying losses to fit the design flows exactly to the FFA, the validation process was aimed at selecting a single set of loss parameters which provided a reasonable fit across the whole range. This approach was justified given the FFA was based on a dataset with 38 years of historical gauged data, at a gauge with known rating curve uncertainties, reducing the confidence in the design flood estimates at high flows (for events larger than 2% AEP).

Initial loss values ranging from 13 mm to 70 mm were tested with a preference for values closer to the lower end of the calibration results (40 mm) and higher end of the design loss equation estimates (27 mm). The k_c values tested ranged from 30 to 50 and a value of 30 was found to produce the hydrograph most consistent with measured data. Continuing losses ranged from 1 to 10 mm/hr with a preference to values within the range used for calibration (1.0 – 3.8 mm/hr). Selected results are shown in Figure 3-23 below.

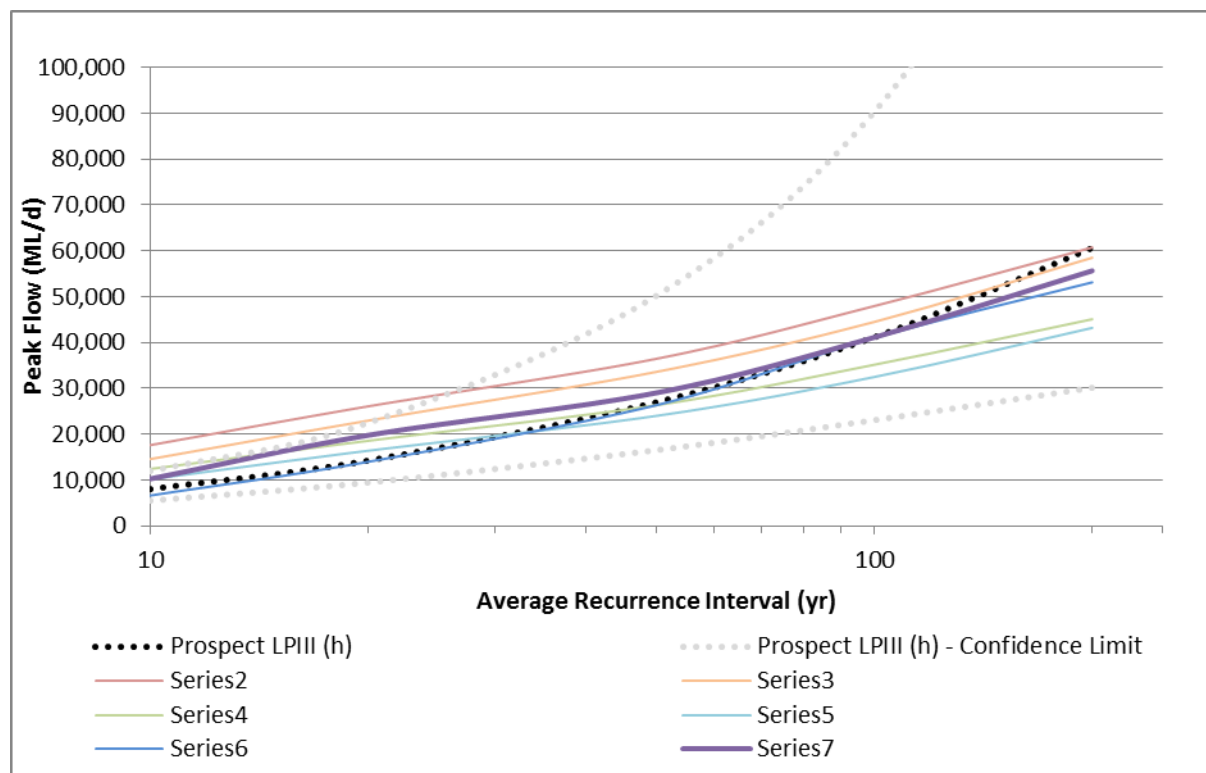


Figure 3-23 RORB peak flow results for various k_c and IL values

The shape of the hydrographs for selected parameter sets was compared to the 1993 gauged hydrograph to check that the selected k_c value matched produced a hydrograph shape consistent with actual events (Figure 3-24). The comparison shows that a k_c of 30 provides a better fit to the hydrograph shape than a k_c of 40, regardless of the loss values.

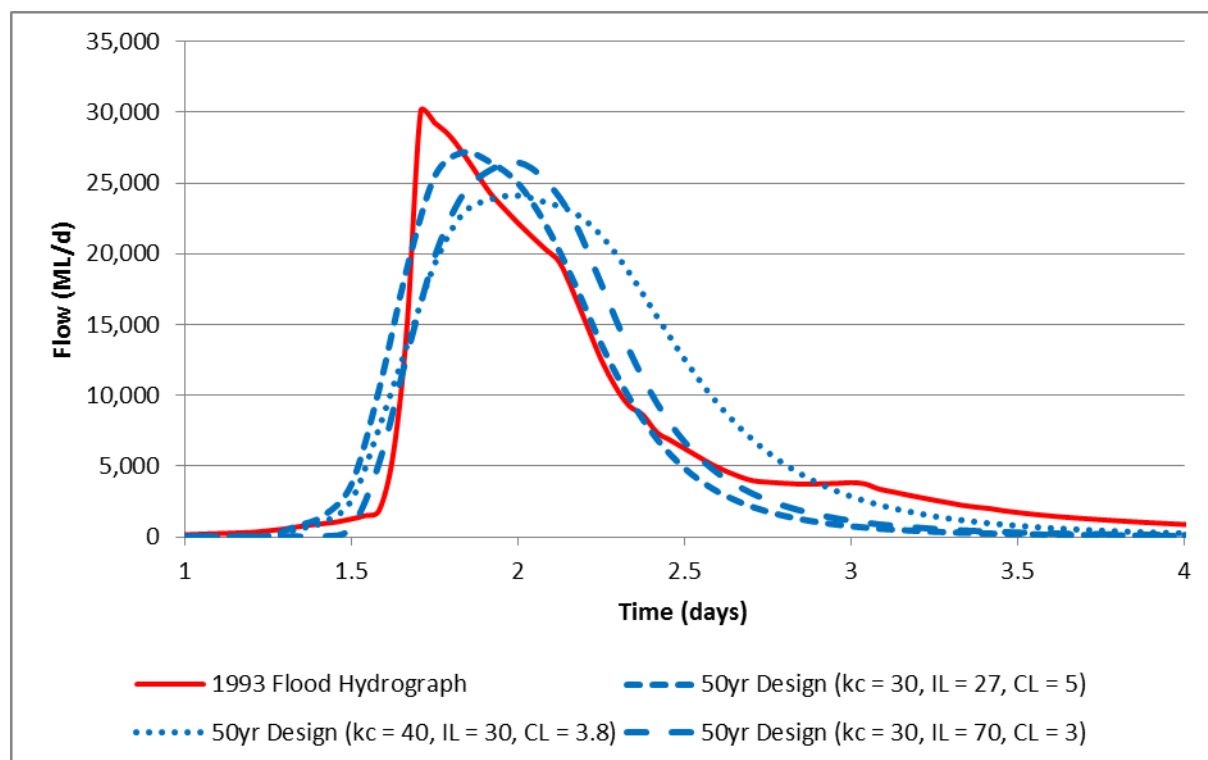


Figure 3-24 Comparison of shape of design hydrographs for selected parameter sets

The parameter set with k_c of 30, initial loss of 50 mm and continuing loss of 3.8 mm/hr was selected as the most reasonable fit to the FFA. These parameter values provide a reasonable fit to both high and low frequency events. The estimates are above, but within the confidence limits of the FFA for the 10%, 5%, and 2% AEP events, and are approximately equal to the FFA for the 1% AEP. The 0.5% AEP peak flow is around 8% lower than the FFA, but well within its confidence limits.

3.7 Design Flows and Hydrographs

The routing and loss parameters summarised in Table 3-12 were found to produce a reasonable fit to the FFA while also lying within the range of calibration values and were thus adopted for design event modelling. The resulting hydrographs are shown in Figure 3-25. Note that for this project the RORB model design event flows were given equal confidence with the flood frequency analysis due to the issues with the gauge data, rating curve and small period of record. Therefore the design flows have adopted the RORB results and have not adjusted the flows to the flood frequency analysis.

Table 3-12 Recommended design parameters

k_c	m	IL (mm)	CL (mm/hr)
30	0.8	50	3.8

Table 3-13 Recommended design peak flows

AEP	RORB Design Flow (ML/d)	Critical Duration
10%	10,280	48hr
5%	19,790	48hr
2%	29,030	36hr
1%	41,130	48hr
0.5%	55,640	48hr

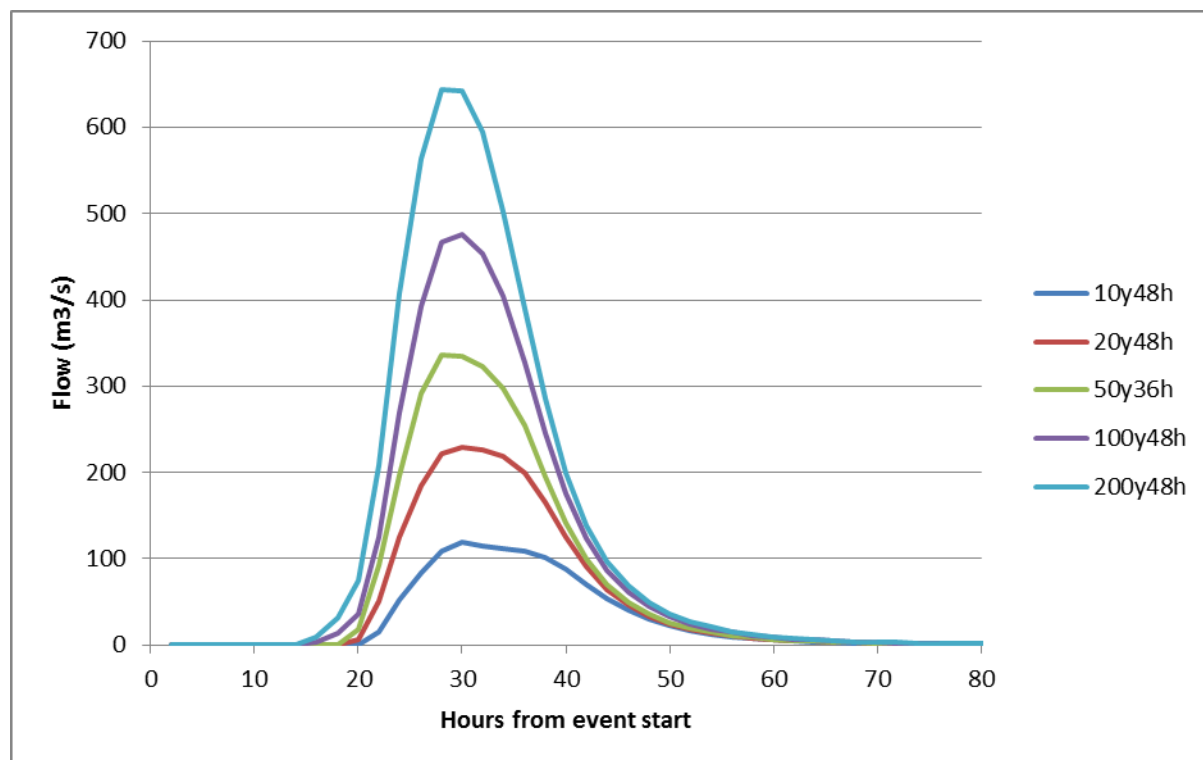


Figure 3-25 Merriman Creek at Seaspray design flood hydrographs

3.8 Sensitivity Analysis

3.8.1 Climate Change

The impacts of climate change were tested by increasing the rainfall intensity by 5% per degree of warming, in line with latest guidance from Australian Rainfall and Runoff (Engineers Australia 2014). A scenario of 2°C of warming (i.e. 10% increase in rainfall intensity) was adopted for this sensitivity test. This is consistent with 'Climate Change in Australia Projections' (CSIRO 2015) report which suggests for an intermediate climate scenario a temperature increase of between 1.1 °C to 2.0 °C is likely for the Southern Slopes region, which includes Merriman Creek catchment.

The 10% increase in rainfall intensity was applied to the burst rainfall depth in the RORB storm files for each of the design events for the critical duration (36 or 48 hr). The results of the climate change sensitivity test at the catchment outlet are summarised in Table 3-14 and shown in Figure 3-26. Due to the hydrologic response of the catchment, the proportional increases in catchment flows are significantly larger than the applied percentage increase in rainfall. This is most notable in the more frequent events where the 10% AEP design flood event produces a 46% increase in flow. As the

magnitude of the design event increases, the percentage increases in flow are reduced to a value more closely aligned to the percentage increase in rainfall.

Table 3-14 Comparison of peak flows for increases in rainfall intensity due to climate change

AEP	Current Conditions		Scenario of 2°C of Warming		Increase in Flow (%)
	Burst Rainfall Depth (mm)	RORB Design Flow (ML/d)	Burst Rainfall Depth (mm)	RORB Design Flow (ML/d)	
10%	107.5	10,250	118.3	14,970	46
5%	126.6	20,040	139.3	26,010	30
2%	135.0	28,390	148.5	35,700	26
1%	175.1	41,080	192.6	52,490	28
0.5%	198.3	55,720	218.1	66,330	19

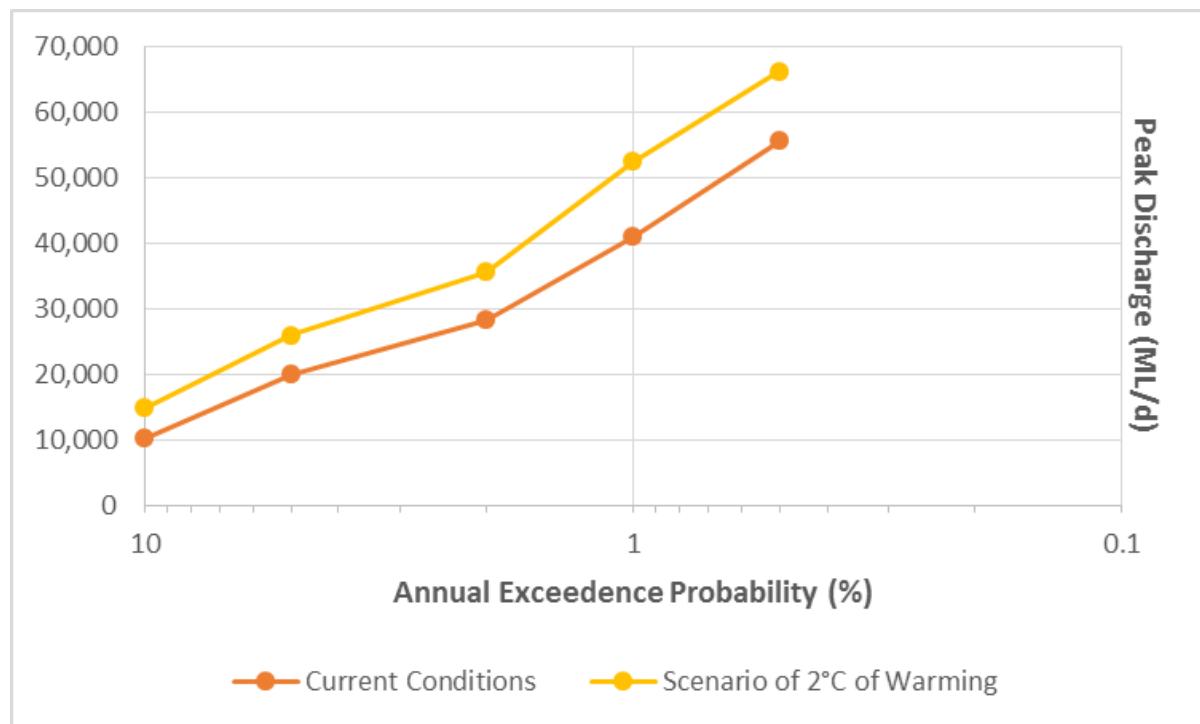


Figure 3-26 Climate change sensitivity comparison for Merriman Creek at Seaspray

3.8.2 Bushfire

The impacts of bushfires were tested by adjusting the fraction impervious values of the Merriman Creek sub-catchments to reflect an agreed severity of bushfire. For example, Blackham et al (2012) provides values of equivalent percentage impervious for different levels of burn severity, based on BAER (2009) (Table 3-15).

Table 3-15 Equivalent percentage impervious for different levels of burn severity

Burn severity	Equivalent percentage impervious
Unburned	0.1
Low	0.3
Moderate	0.7
High	0.9

The impervious fraction for all Farming Zone (Forestry), and Public Conservation and Resource Zone areas across the catchment have been increased to 0.3, 0.7, and 0.9, to represent low, moderate and high severity burns across the catchment. The resulting fraction impervious for each of the sub-catchments was recalculated and the RORB model was rerun for a range of design events.

As expected, the increase of impervious fraction due to bushfire had a significant impact on the peak flows generated from the catchment at the catchment outlet.

Table 3-16 Comparison of peak flows for increases in impervious fraction due to bushfire

AEP	Peak Catchment Discharge (ML/d)						
	Unburned Catchment	Low Intensity Bushfire	Increase in Flow (%)	Moderate Intensity Bushfire	Increase in Flow (%)	High Intensity Bushfire	Increase in Flow (%)
10 %	10,280	17,880	74%	29,810	190%	36,720	257%
5 %	20,040	26,520	32%	39,660	98%	47,000	135%
2 %	28,430	37,930	33%	52,790	86%	61,260	115%
1 %	41,130	50,280	22%	66,360	61%	74,820	82%
0.5 %	55,730	64,110	15%	81,390	46%	89,600	61%

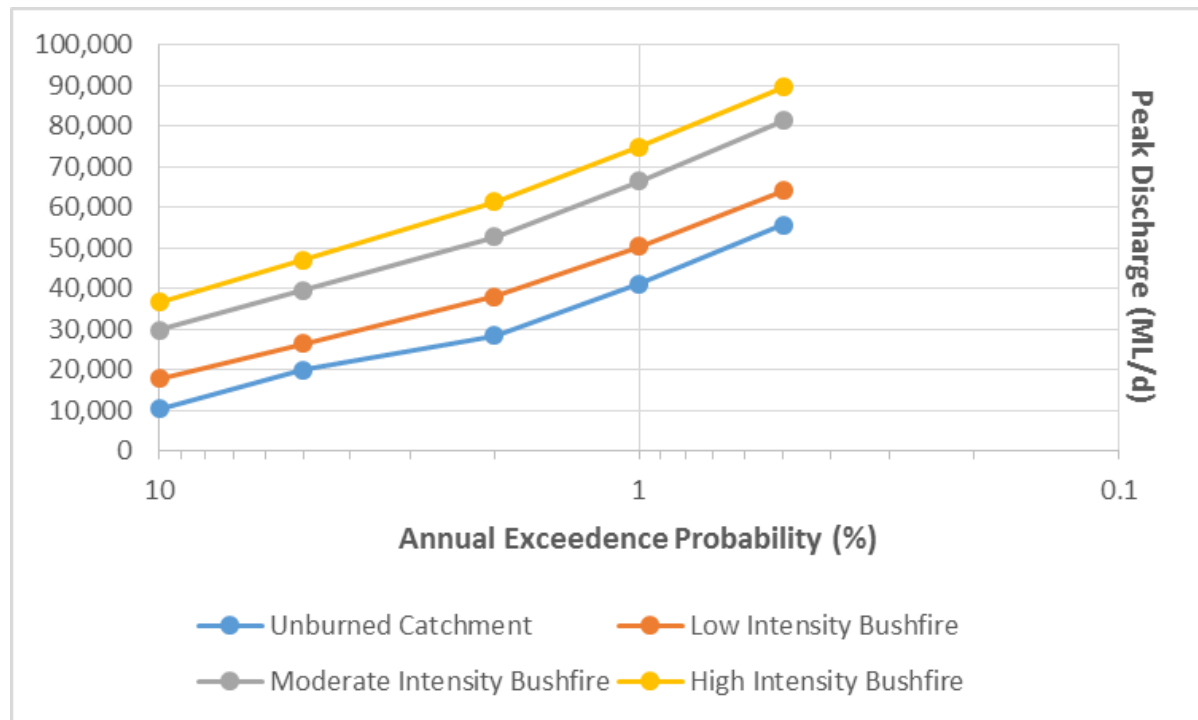


Figure 3-27 Bushfire sensitivity comparison for Merriman Creek at Seaspray

3.9 PMF Estimation

The Probable Maximum Flood (PMF) is the flow generated from the theoretical maximum precipitation for a given duration under current climate conditions. A PMF Estimate for Merriman Creek at Seaspray (at the lagoon mouth) was prepared using the Quick Method of Nathan et al. (1994). This method applies a set of empirical equations to compute a triangular PMF hydrograph. The equations are:

$$Q_p = 129.1 A^{0.616}$$

$$V = 497.7 A^{0.984}$$

$$T_p = 0.0001062 A^{-1.057} V^{1.446}$$

$$T_r = V / (1.8 Q_p)$$

Where:

Q_p is peak flow (m^3/s)

A is catchment area (km^2)

V is hydrograph volume (ML)

T_p is time to peak of hydrograph (h)

And T_r is base length of hydrograph (h)

The equations are applicable to southeast Australian catchments from 1 to 10,000 km^2 that do not have large lakes or storages. As the Merriman Creek catchment contains a reasonably large swamp storage (around 35 ha in area increasing to around 90 ha when full), a correction to the catchment area is required before the equations are applied. The catchment area used was 484 km^2 (the total catchment area minus the swamp catchment). The resulting PMF peak flow, volume, time to peak and hydrograph length are:

A	484 km ²
Q _p	502,680 ML/d (5,818 m ³ /s)
V	218,200 ML
T _p	8.1 h
T _r	20.8 h

The resulting triangle hydrograph is shown in Figure 3-28.

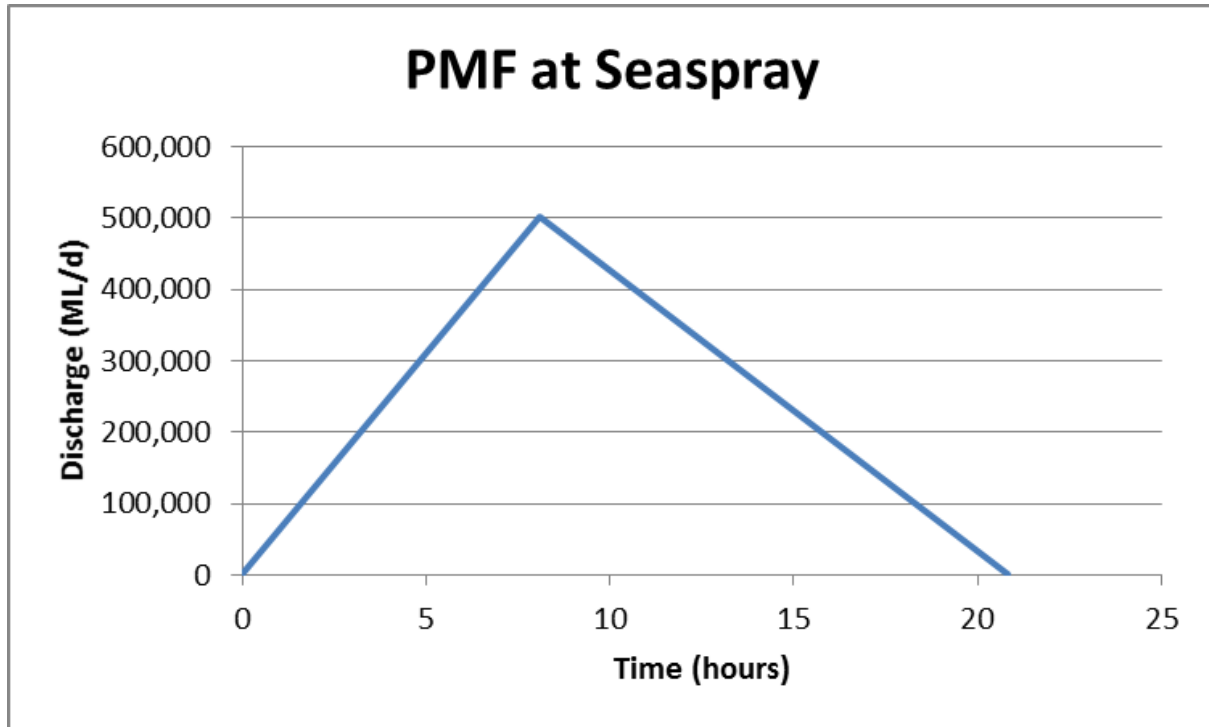


Figure 3-28 PMF hydrograph for Merriman Creek at Seaspray

4. CONCLUSION

Through a process of flood frequency analysis, hydrologic modelling and application of various regional methods, a set of design flood hydrographs have been developed for Merriman Creek at Seaspray. While the hydrological analysis has been based on robust methods and extensive cross-validation, there are limitations due to issues with the gauge at Prospect Road and difficulties in calibration of the RORB model. However, validation of the design peak flows against flood frequency analysis and comparison of the design hydrographs against gauged hydrographs gives confidence that they are representative of actual flood risk and appropriate for design.

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APPENDIX A RATING CURVE CHECK WITH HYDRAULIC MODEL

The hydraulic model developed in this project was used to verify the appropriateness of the rating curves for the 1993 and 1995 calibration events.

Boundary

A discharge boundary a little over one kilometre north of Prospect Road was used to apply flows to the hydraulic model. Flows were calculated from gauged water levels and the rating curves from 1993 and 1995.

Resolution

In the vicinity of the gauge, a 5 metre quadrilateral mesh was used in order to sufficiently represent the topography in the area and the modelled flood extent. Upstream and downstream of this area the main river channel was made up of quadrilateral elements 3 to 6 metres in width and with varying lengths. To the sides of the main river channel across the floodplain triangular elements increased in size to approximately 2,000 m².

Roughness

A roughness map was used in this early version of the model with the vegetated river channel upstream of Prospect Road modelled with a Manning's n roughness of 0.100 and the surrounding area was given a Manning's n roughness of 0.045. The roughness map was refined and modified for the calibration and validation simulations.

Results

Water levels were extracted from the model at the gauge site and compared directly to the gauge record, and the flood extents around the gauge were compared with those reported in the flood summary report (HydroTechnology 1995).

Modelling of the 1995 event produced a close fit of water levels with the recorded levels at the gauge, as shown in **Error! Reference source not found.** below. Both the rising and falling limbs, as well as the peak, produced a close match. It is therefore considered that the 1995 rating curve was appropriate for the 1995 event.

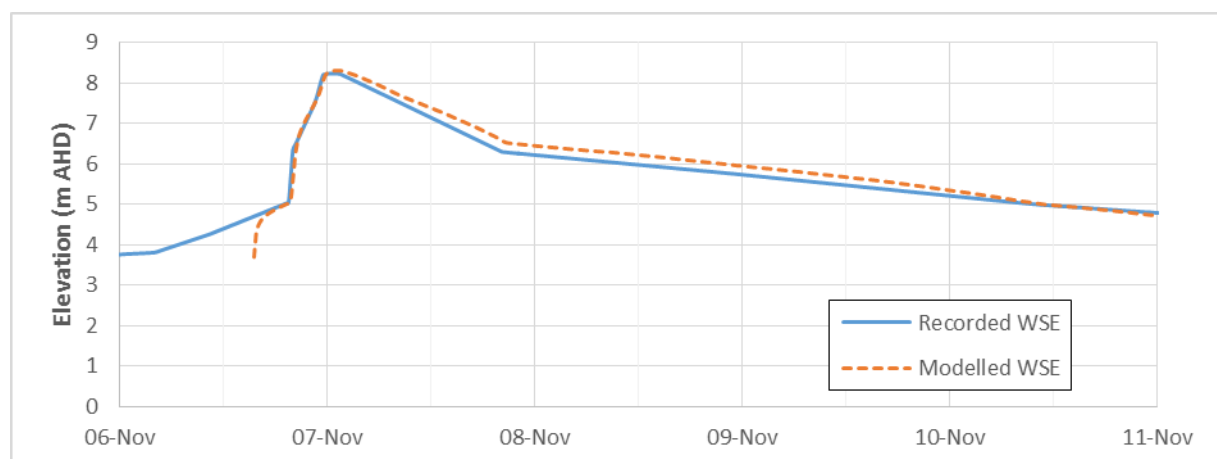


Figure A-1 1995 Calibration Period Recorded and Modelled WSE at Gauge

Initial modelling of the 1993 event with the 1993 rating curve produced a water level at the gauge that matched in the rising limb, overshoot the peak, then maintained an elevated position on the falling limb. Adjustment of the flow rates with the application of the 1995 rating curve produced a

curve that under-predicted the antecedent water level but had a close fit for the rise, peak, and retreat of the water. A comparison of these two simulations and the recorded water level at the gauge are illustrated in Figure A-2 below.

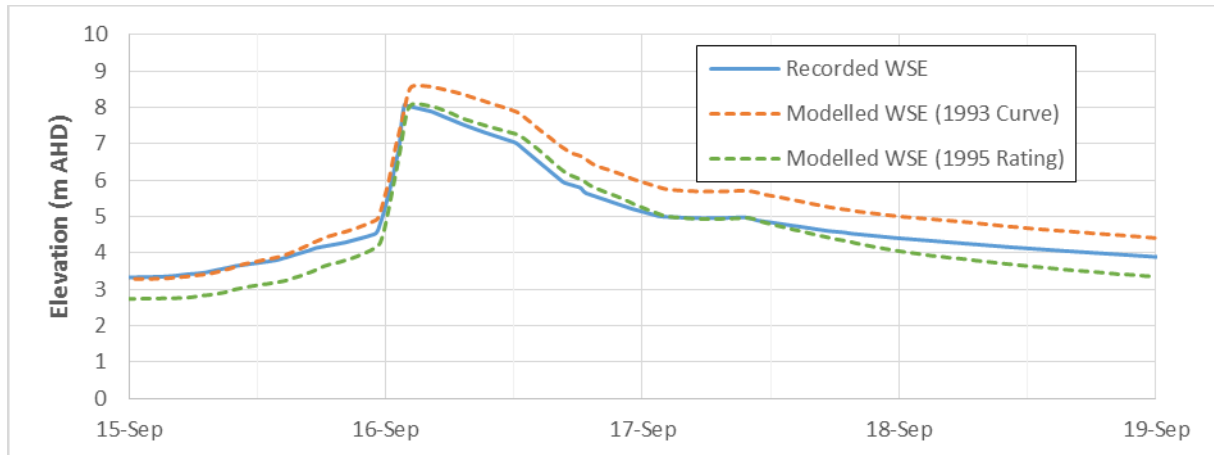


Figure A-2 1993 Calibration Period Recorded and Modelled WSE at Gauge

Seaspray Flood Study – Hydraulics (R03)



March 2016

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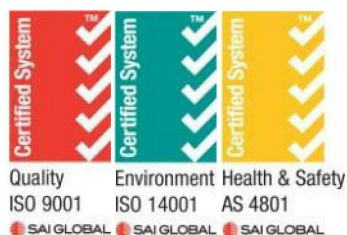


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1. INTRODUCTION

Water Technology was commissioned by the West Gippsland CMA to undertake the Seaspray Flood Investigation. This project includes hydrologic analysis of the Merriman Creek catchment and detailed hydraulic modelling of the Merriman Creek estuary, ocean conditions, Lake Reeve levels, and stormwater run-off, all of which contribute to the flood risk within Seaspray. The key outcomes of this investigation will be to provide flood mapping of Seaspray for a series of design storm events, and to present recommendations for flood mitigation actions and works.

This report details the hydraulic modelling undertaken for the project, and should be read in conjunction with the previous Data Review Report (R01) and Hydrology Report (R02).

1.1 Available Information

All data received and sent regarding the project has been logged by Water Technology in a Project Data Management Record detailing information about the data. The hydraulic models discussed within this report have been developed based on the previously reported information assembled for the Data Review Report and the Hydrology Report.

2. STUDY AREA

Seaspray is a coastal town with 320 permanent residents which rises to more than 1,000 during summer. The township lies at the western end of the Ninety Mile Beach west of the Gippsland Lakes and approximately 200 km east of Melbourne. As detailed in the Urban Design Framework developed for the township by Wellington Shire Council (Wellington Shire Council, 2007), Seaspray is an important access point to 90 Mile Beach, and the areas of small lot subdivision that extend easterly for 28 km to Golden Beach and Paradise Beach.

The town is located on the estuarine floodplain of the Merriman Creek catchment. Merriman Creek flows along the western boundary of Seaspray before it passes through the 90 Mile Beach coastal dune system and enters Bass Strait. As part of the flood mitigation scheme completed in 1987, an engineered floodway running through the middle of the town, has the ability to distribute water from the Merriman Creek estuary to Lake Reeve in the east.

The hydraulic model area includes Merriman Creek downstream of Prospect Road, the lagoon entrance with Bass Strait, and the floodway extending all the way to the western end of Lake Reeve as shown in Figure 2-1.

In this study area, flooding can occur as a result of a range of potential mechanisms, including;

- Overbank flows from Merriman Creek,
- Coastal inundation due to elevated ocean levels,
- Flooding from the Gippsland Lakes via Lake Reeve,
- Localised stormwater flooding.

Each of these flooding mechanisms and their interactions have been investigated as part of the hydraulic assessment and detailed within this report.



Figure 2-1 Seaspray Hydraulic Model Area

3. MODEL DEVELOPMENT AND SCHEMATISATION

3.1 Overview of the Modelling Approach

A 2-dimensional (2D) flexible mesh hydraulic model was developed for the township of Seaspray and its surrounding floodplains using the industry standard software MIKE21FM (Mike by DHI). Adopting a flexible mesh modelling approach allowed the hydraulic model to incorporate greater detail in areas of importance, whilst maintaining computational efficiency through a larger element size in less sensitive regions of the modelled area. This allows features within the broader floodplain and the township areas to be resolved in varying detail in the same model whilst maintaining manageable run times.

The MIKE21FM model was used to investigate overbank flooding from Merriman Creek, coastal inundation and the influence of flooding from Lake Reeve. These events were considered separately and in combination with one another.

The MIKE21FM model ensures model accuracy as well as computational efficiency, and is well suited for the riverine and coastal flooding mechanisms. For the stormwater flooding mechanism, a separate 1D-2D TUFLOW rainfall-on-grid model was established. TUFLOW was chosen due to the ease of incorporating pipes and pits within the model. The localised TUFLOW model incorporated inflow boundaries extracted from the larger MIKE21FM model with the addition of rainfall-on-grid excess across the TUFLOW model area. Stormwater drainage assets were included within the model, incorporating key pipes, pits and pumps.

3.2 Model Schematization

3.2.1 Mesh Extent and Resolution

A detailed representation of the ground surface is required to appropriately capture the topographic detail within the hydraulic model. A flexible mesh approach provides high resolution in areas requiring enhanced detail such as the waterway channels, whilst allowing lower resolution in less hydraulically sensitive areas. This includes the western end of Lake Reeve and the coastal boundary. Spatially varying the resolution allows the model run times to be optimised whilst maintaining element sizes needed to adequately meet the study requirements.

As a result, the mesh comprises of triangular and rectangular elements of varying size. The adopted mesh for this study is illustrated in Figure 3-1. The model extends from Prospect Road (Owens Lane) in the north to approximately 1.5 km offshore of the Merriman Creek lagoon entrance.

The mesh resolution within the town is approximately 9 m² with quadrilateral elements aligned in most areas with the roads and properties. Quadrilateral elements of high resolution were also used to delineate channels and levees. The resolution of the mesh reduces across the floodplain and towards the offshore boundary. Along the tidal boundary the mesh resolution is reduced to approximately 30,000 m². Triangular elements are used to migrate between quadrilateral sections and areas with different resolutions.

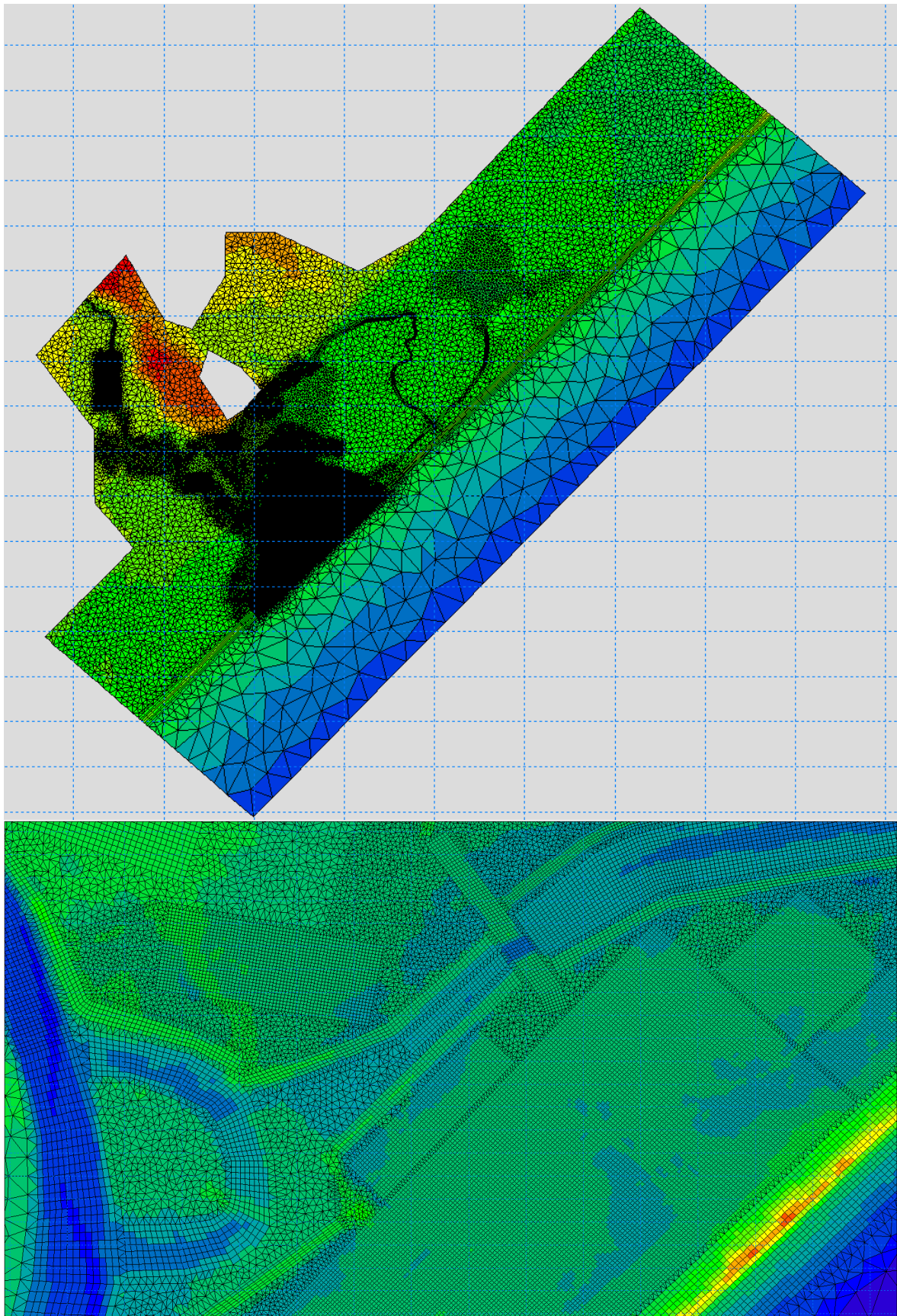


Figure 3-1 Hydrodynamic Model Domain and Mesh

3.2.2 Topography

The model topography was developed from four data sources as shown in Figure 3-2. The VicMap Coastal DEM (green) was used where available, with the exception of the main river channel where surveyed bathymetric levels collected for this project were used (red). Spot height data was available for the new caravan park and this was used to update the ground levels across the site as the ground surface was reshaped in this area during construction which occurred subsequent to the Coastal DEM data capture. VicMap 20m bathymetry (blue) was used seaward of the shore line and Rivers DEM (pink) was used in a small area of the upstream section of the model where the coastal DEM did not have coverage. Modifications were made directly to the mesh for the various entrance conditions representation. Further discussion on the quality and availability of LiDAR is provided in the Data Report (R01).

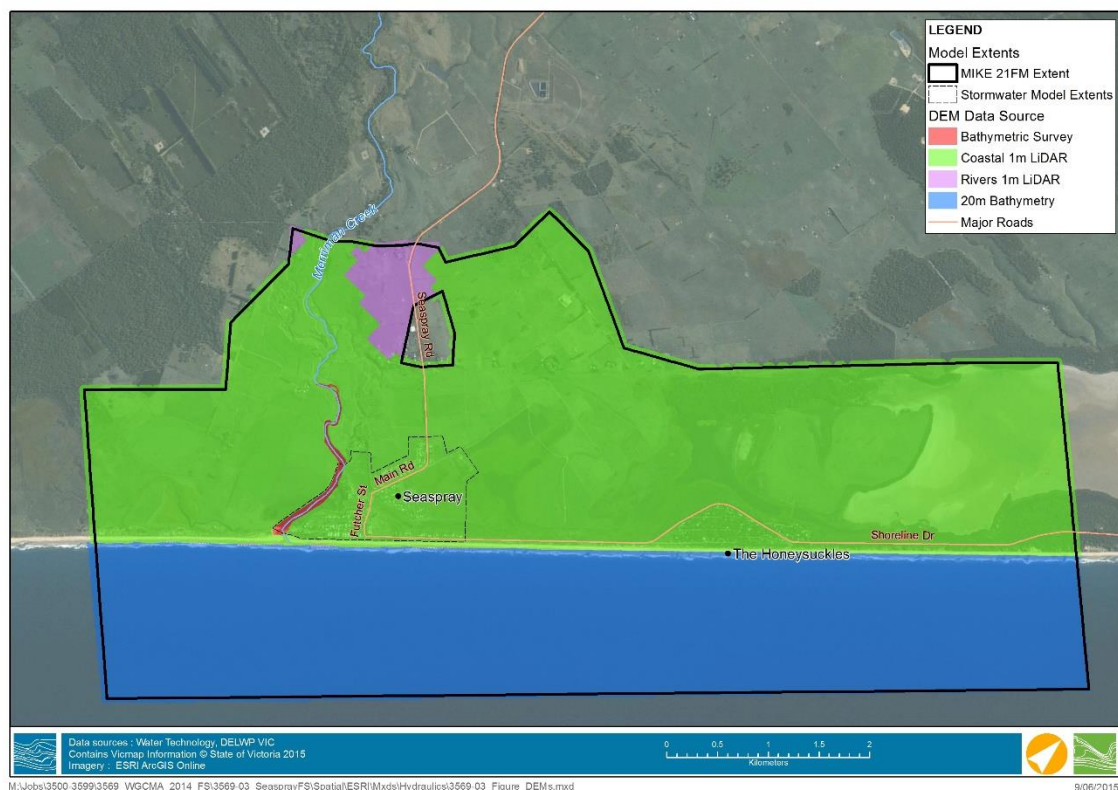


Figure 3-2 Extent of DEM Data Sources Used in Model

3.2.3 Merriman Creek Entrance

The entrance to Merriman Creek is intermittently opened by catchment flows, with a berm rebuilding during periods of low flow. The entrance condition is a key control on flood behaviour in the estuary and has therefore been a major consideration during the modelling process and schematisation. Sensitivity testing has included modelling different berm elevations including fully open and fully closed scenarios. In addition, a dynamic bathymetry and sediment transport model was also used to assist in understanding the potential and nature of scour likely to occur during a catchment flood event.

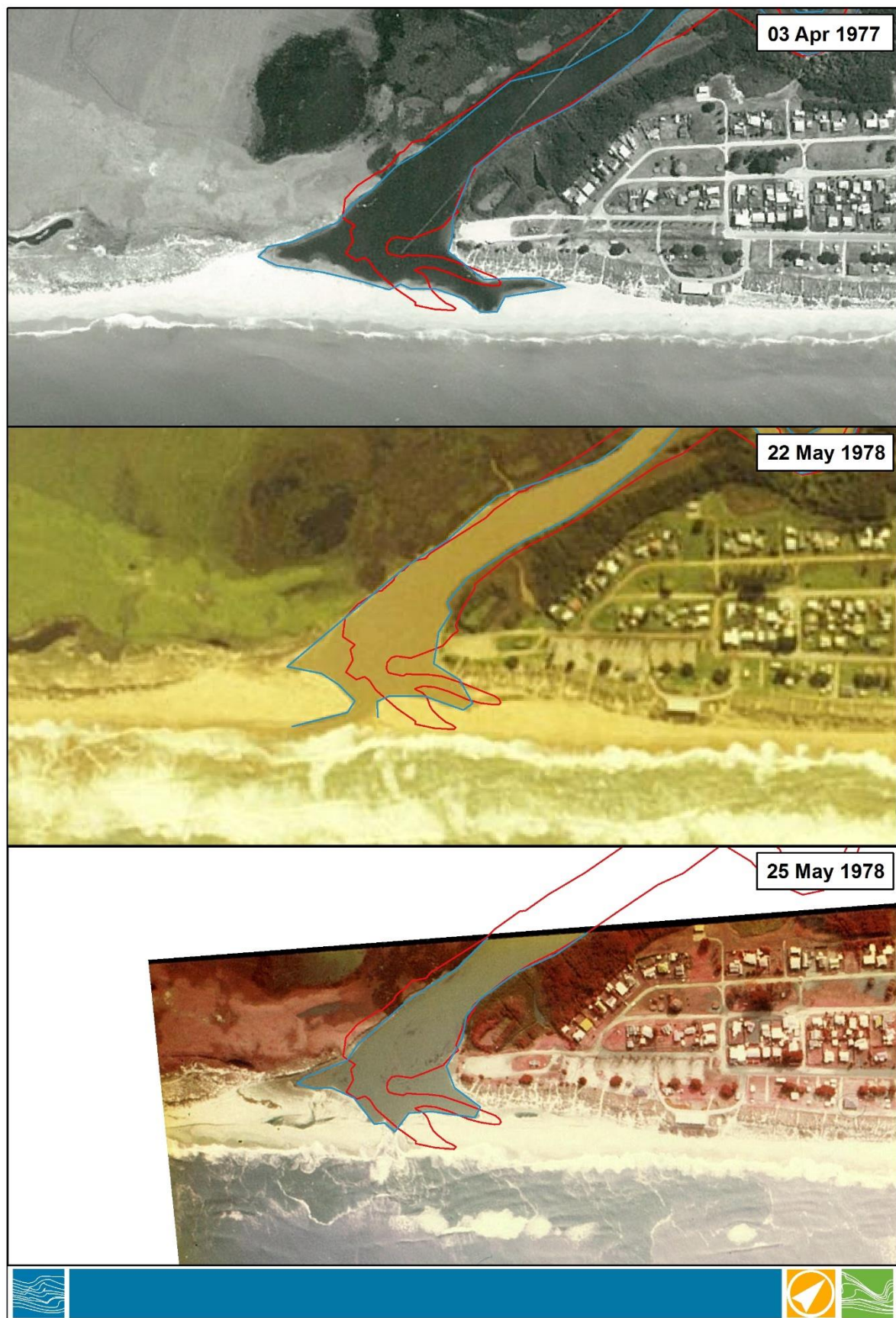
A selection of aerial photographs has been collated showing various open and closed configurations of the entrance between 1965 and 2011. Each of the photographs are overlaid by a red polygon showing the most recent March 2011 condition of the entrance to allow a comparison to the range of historical conditions. Throughout the historical photographs, the lagoon can be seen migrating

east and west behind the accumulated sand berm. However, the photographs also suggest that when a break-out flow occurs, the berm is breached either directly straight out into the sea, or to the west, in alignment with the lower reach of Merriman Creek.



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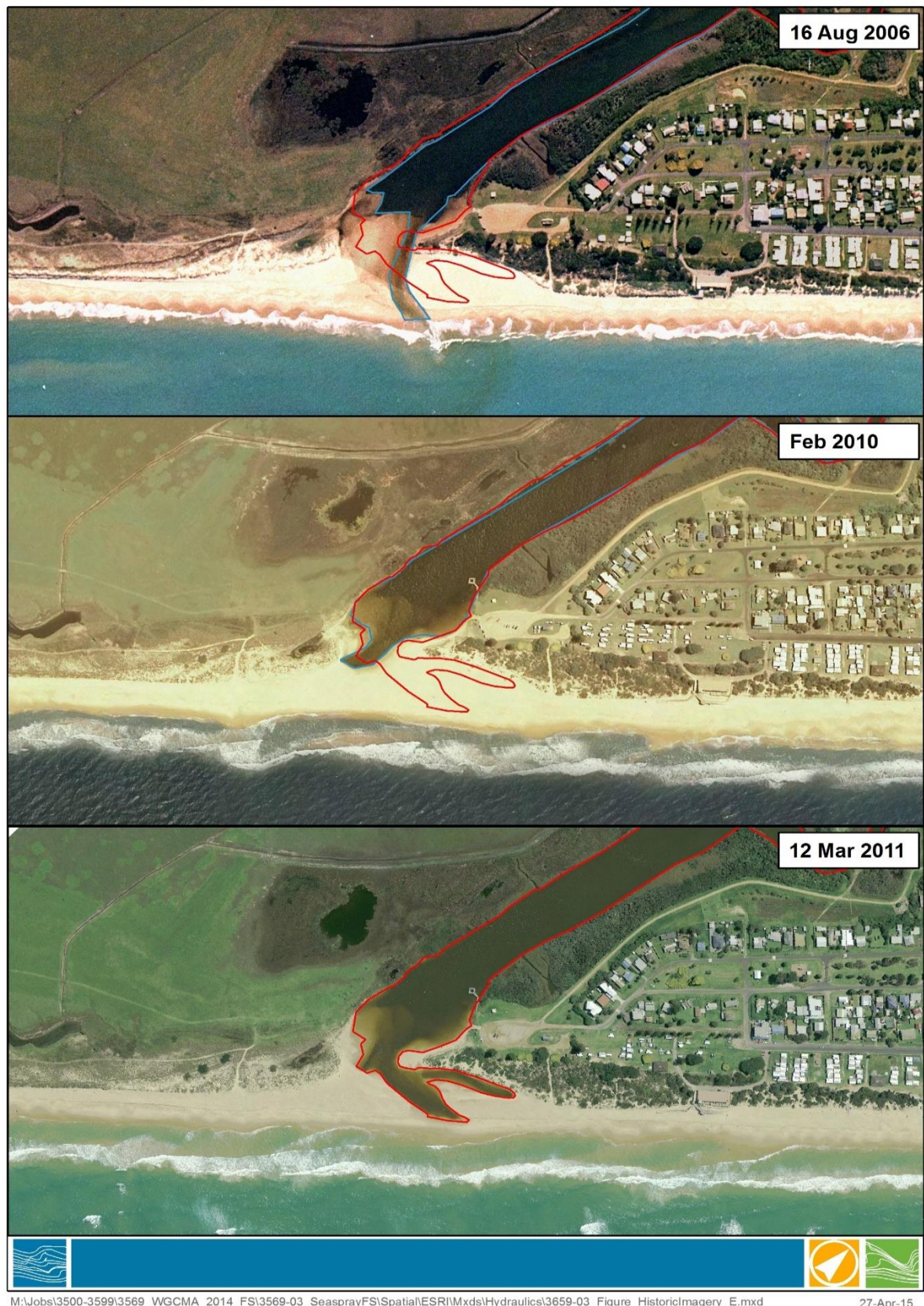


Figure 3-3 Historic Entrance Conditions of Merriman Creek

3.3 Model Boundaries

There are two water level boundaries and one flow boundary controlling the exchange of water into and out of the hydraulic model. These boundaries are identified in Figure 3-4.

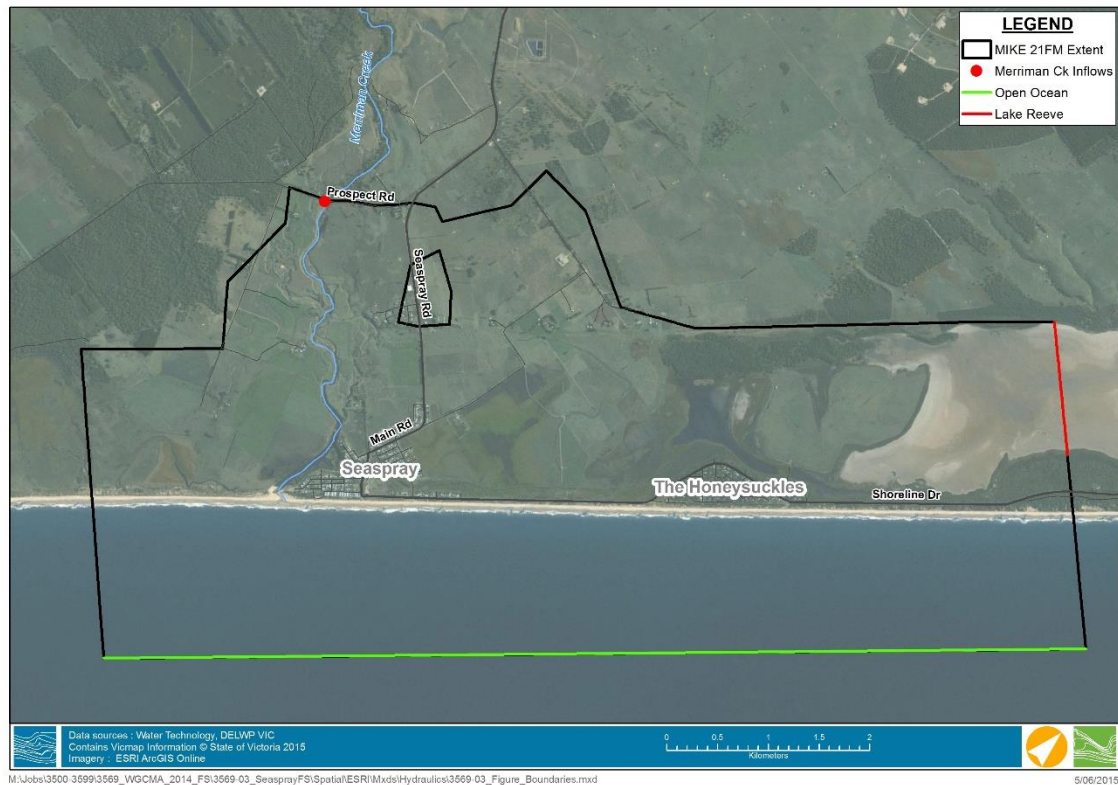


Figure 3-4 Model Boundaries

3.3.1 Merriman Creek Inflow

Merriman Creek inflows for the design events and sensitivity testing were provided from the hydrologic model as discussed in the Hydrology Report (R02).

Calibration flows for the 1993 historical event were extracted from the Prospect Road Gauge recorded heights with the revised rating curve to determine flows, as discussed in the Hydrology Report (R02).

These flows were applied directly to the hydraulic model at the location of the Prospect Road Gauge. When undertaking assessment of coastal inundation or other scenarios that did not include catchment derived flooding, a base flow of $1 \text{ m}^3/\text{s}$ (86.4 ML/d) was applied. This mean daily flow was determined through analysis of the gauge record at the Prospect Road gauge.

3.3.2 Lake Reeve

A steady-state water level boundary has been applied to represent tailwater conditions in Lake Reeve. A mean level of 0.0 m AHD has been applied as a standard level representative of the western end of the Gippsland Lakes. Further sensitivity of the model to downstream levels in discussed in Section 6.6.

3.3.3 Ocean Boundary

An open ocean boundary was defined along the coastal extents of the model. This was used to apply the astronomical tide and storm surge. The astronomical tide was predicted from the five main tidal constituents (M2, S2, K1, O1, and N2) derived by Hinwood (1986) and is shown in Figure 3-5. These constituents represent semi-diurnal, diurnal and long period tidal oscillations. This approach has been adopted in preference to the Australian National Tides Tables due to the proximity of tidal observation locations within the Hinwood (1986) study. As part of the study by Hinwood (1986) a tidal observation station was situated at McGaurans Beach, 10km to the south-west of Seaspray; a more appropriate reference point than locations available within the Australian National Tide Tables.

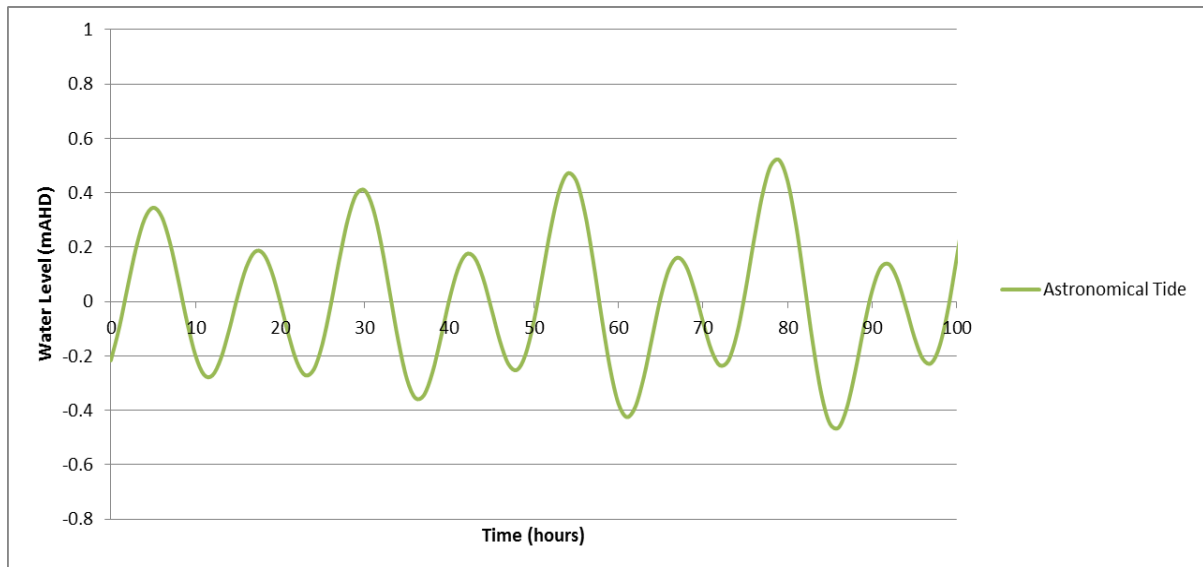


Figure 3-5 Astronomical Tide Time Series

Table 3-1 and Table 3-2 present the peak magnitude of the storm tide and storm surge respectively (McInnes et al. 2009) over a range of Annual Exceedance Probabilities for the current mean sea level and climate condition and for three future scenarios incorporating predicted sea level rise. These scenarios include the IPCC (2007) high scenario for mean sea level in combination with the equivalent high annual average wind speed change averaged over Bass Strait from CSIRO and Australian Bureau of Meteorology (2007).

A dynamic storm tide boundary was generated for this project using the following approach:

- A cosine wave with an amplitude equal to the 1% AEP storm surge at Seaspray;
- The addition of astronomical tide levels for Seaspray to produce a peak storm tide equal to the magnitude of 1% storm tide level for Seaspray as reported by McInnes et al. (2009).

Figure 3-6 illustrates the combined effect of the storm surge and the astronomical tide to produce a storm tide.

Table 3-1 Estimated Peak Storm Tide for Open Coast at Seaspray, from McInnes, et al. (2009)

Annual Exceedance Probability	Current Climate	Sea Level Rise Scenarios		
		+0.15 m	+0.47 m	+0.82 m
10%	1.22 ±0.12	1.42	1.87	2.33
5%	1.32 ±0.12	1.54	1.98	2.44
2%	1.43 ±0.12	1.66	2.09	2.53
1%	1.50 ±0.14	1.73	2.18	2.64

Table 3-2 Estimated Storm Surge for Open Coast at Seaspray, from McInnes, et al. (2009)

Annual Exceedance Probability	Current Climate	Sea Level Rise Scenarios		
		+0.15 m	+0.47 m	+0.82 m
10%	0.83 ±0.06	1.04	1.51	1.96
5%	0.86 ±0.07	1.08	1.55	2.01
2%	0.90 ±0.08	1.13	1.61	2.07
1%	0.93 ±0.10	1.15	1.64	2.10

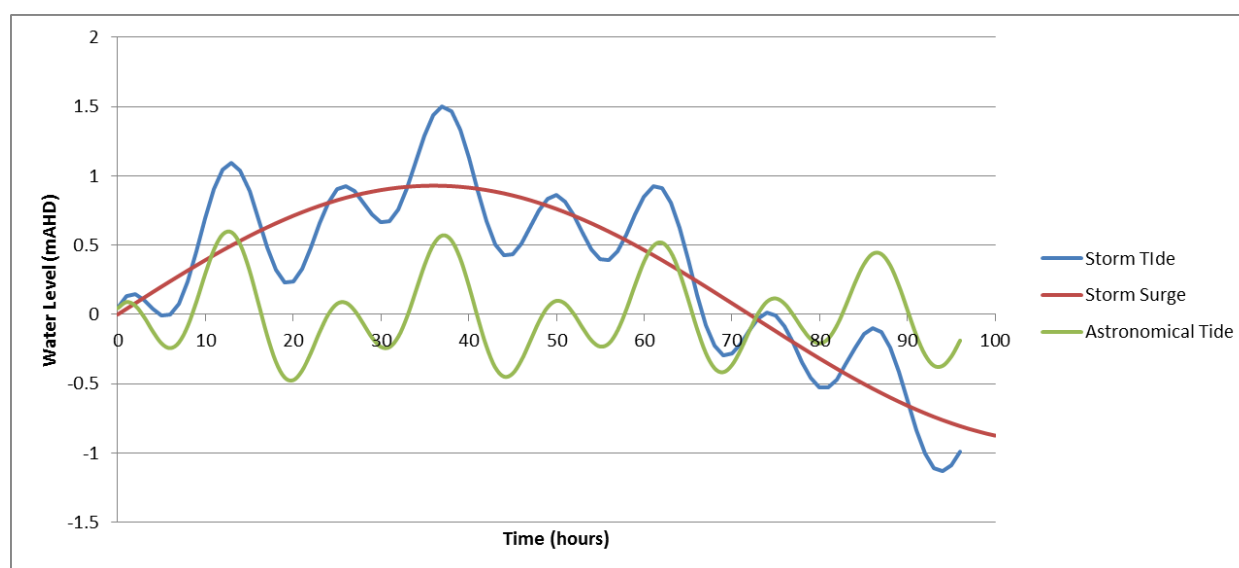


Figure 3-6 1% AEP Storm Tide under current mean sea level and climate

3.3.4 Stormwater Model Boundaries

The localised stormwater hydraulic model has five boundaries as listed below and illustrated in Figure 3-7.

- Flow vs. time boundary on Merriman Creek allowing catchment inflows, extracted from larger MIKE21FM model.
- Flow vs. time boundary on floodplain allowing overland flows from Blind Creek, extracted from larger MIKE21FM model.
- Water level boundary at the entrance controlling tail water level with tides.
- Free flow boundary on east boundary with a slope of 1% towards Lake Reeve
- Rainfall excess applied directly to every grid cell, with rainfall based on Intensity-Frequency-Duration design estimates.

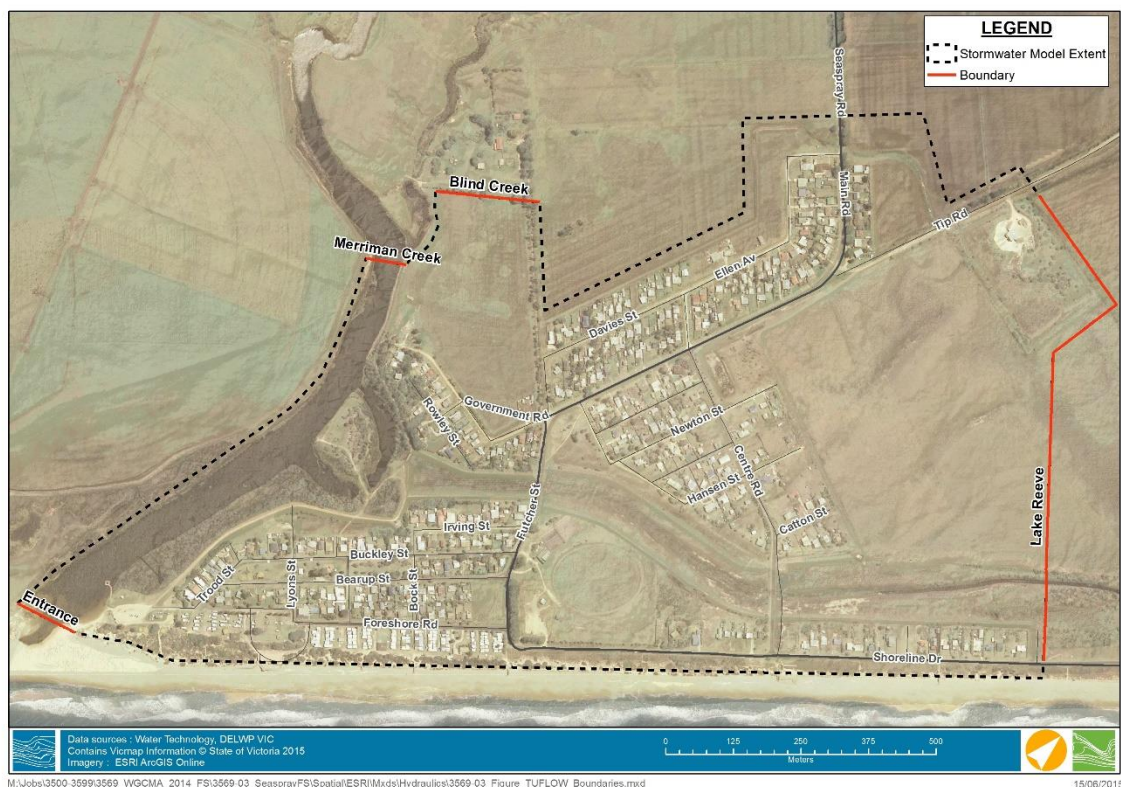


Figure 3-7 Stormwater Model Boundaries

3.4 Surface Roughness

Surface roughness has important effects on flood velocities, flow paths, flood depths, and extents. Roughness was included in the model as spatially varying Manning's roughness values determined by land use mapping (Vicmap, DELWP 2015), aerial imagery (Vicmap, DELWP 2010) and observed catchment conditions. The values were adjusted during model calibration and the final adopted roughness values are listed below in Table 3-3 and displayed in Figure 3-8.

Table 3-3 Roughness Categories

Description	Manning's n	Manning's M
Open Water	0.03	33
Beach	0.04	25
Vegetated Dune	0.04	25
River Mouth	0.015	67
River (Lower)	0.04	25
River (Upper)	0.063	16
Riparian Vegetation	0.059	17
Swamp Vegetation	0.125	8
Lake Reeve	0.05	20
Bush	0.10	10
Grass	0.04	25
Grassed Channel	0.04	25
Channel to Lake Reeve	0.10	10
Road	0.025	40
Residential	0.20	5
The Honeysuckles	0.10	10
Around Island	0.08	12.5

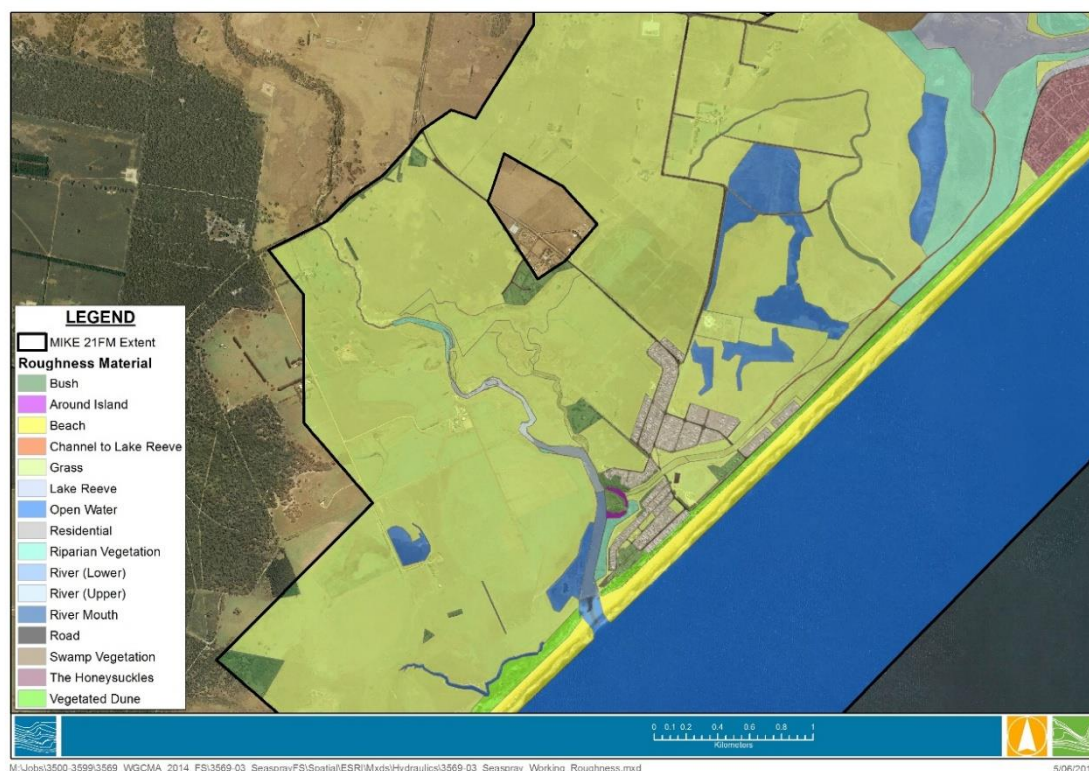


Figure 3-8 Roughness Map

3.5 Key Structures

There are several key hydraulic structures within the Seaspray township, predominantly associated with the previous mitigation works (HydroTechnology 1995). These structures (levees, bridges, culverts and the regulator) play an important role in flood events ranging from small frequent events through to large flood events.

3.5.1 Levees

Flood mitigation levees were included in the 2D model as additional structures. These allow the high points along the crest of the levee to be enforced in the model. This method ensures that the correct crest elevations are incorporated into the model independent of the mesh resolution.

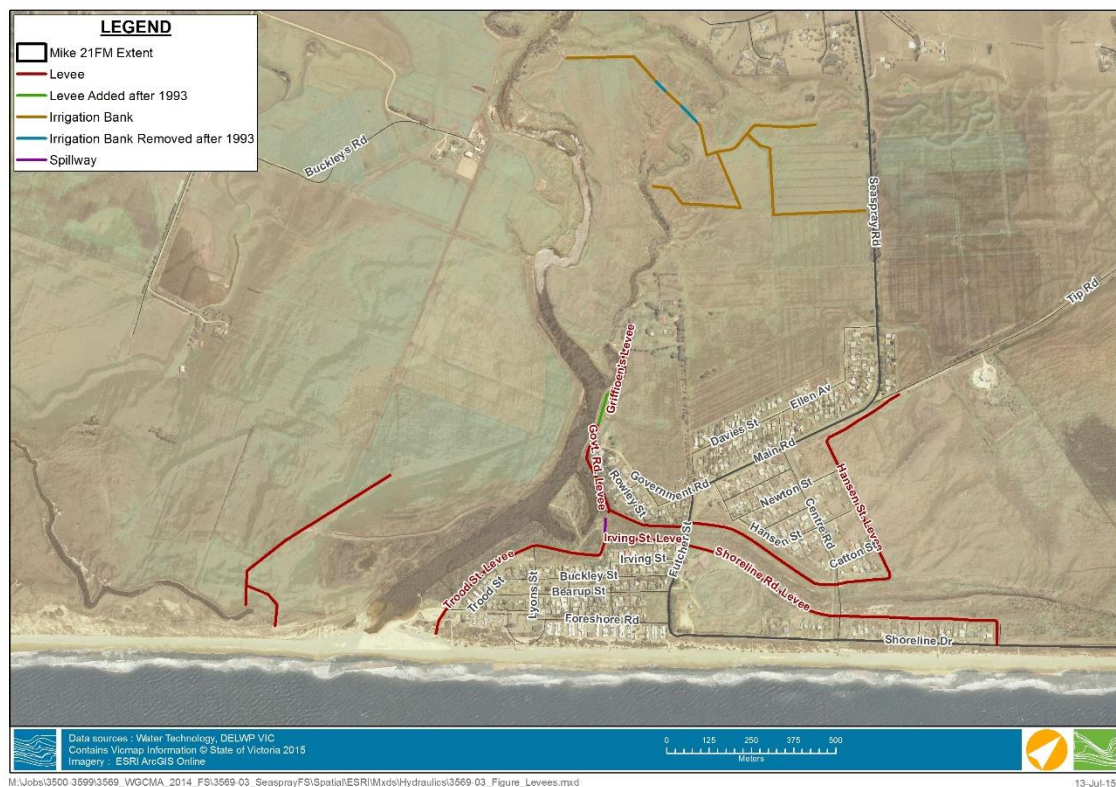


Figure 3-9 Levees and Irrigation Banks

For the 1993 calibration event, an additional levee was included in the model at the northern end of Blind Creek to represent the local landowner's irrigation bank which blocked flows to the south (HydroTechnology 1995). This flow path was opened after the 1993 flood event on the recommendations of the flood summary report (HydroTechnology 1995). This structure has therefore not been included in the sensitivity and design flood assessments.

3.5.2 Bridges and Culverts

The main bridges and culverts were included in the model as structures defined by their dimensions. The dimensions for the bridges and culverts were determined during a site visit as listed in the Data Review Report (R01).



Figure 3-10 Surveved Bridges and Culverts

For the 1993 calibration event, the Eastern Prior culvert under Seaspray Road (Number 1 in Figure 3-10) was a single 450 mm reinforced concrete pipe. After the flood, this was replaced with five 2000 x 1500 mm reinforced concrete box culverts. The modelling of the 1993 event therefore applied a 450 mm pipe culvert, while the sensitivity and design runs applied the new box culvert dimensions.

3.5.3 Regulator and Adjacent Weir

The regulator and adjacent weir at the western end of the Lake Reeve Floodway were included in the model. The minimum level of the weir is 2.2 m AHD and this level was enforced within the model. The regulator structure was implemented using the same methodology with a single crest elevation specified representing the top of the uppermost board at any given time. For the 1993 calibration event, the control structure elevation was set to a level of 1.7 m AHD as all boards had been removed from the structure prior to the flood event (Hydrotechnology, 1995). It is the current operating practice to retain all boards within the structure, raising the overall elevation to 2.7 m AHD. This level, representing the highest setting of the regulator, preferentially directs flow toward the lagoon entrance and away from the Lake Reeve Floodway. The impact and control of the regulator structure has been investigated within a sensitivity analysis.

3.5.4 Pit and Pipe Network (Stormwater Model)

For the stormwater model of Seaspray, the urban stormwater system was included in the model as a 1D network dynamically linked to the 2D domain. Seaspray is divided into two separate drainage areas. The area east of Futcher Street drains to a pump station on Centre Road on the northern side of the floodway, which pumps stormwater out to the floodway. The area west of Futcher Street drains to a pump station on Irving Street, which pumps directly to Merriman Creek. Each pump station has two pumps which have a maximum capacity of 170 L/s each. During a high rainfall event both pumps can be operated together giving a maximum discharge rate from each pump station of 340 L/s.

Details of the caravan park infrastructure including pit and pipe layout, and detention basin locations was provided by Wellington Shire Council. This data also contained a number of spot heights of the finished surface which was used to modify the DEM in this area.

Some modifications were made to the data provided to ensure that invert levels were defined at all junctions. Changes have been noted in the original MapInfo table database. Initially only pipes of diameter equal to, or greater than 300 mm were included in the hydraulic model. After preliminary modelling it was determined that the smaller pipes are important in Seaspray as they drain large areas of the township including Foreshore Road, and Futcher Street with connections to the Caravan Park system, and were thus subsequently included.

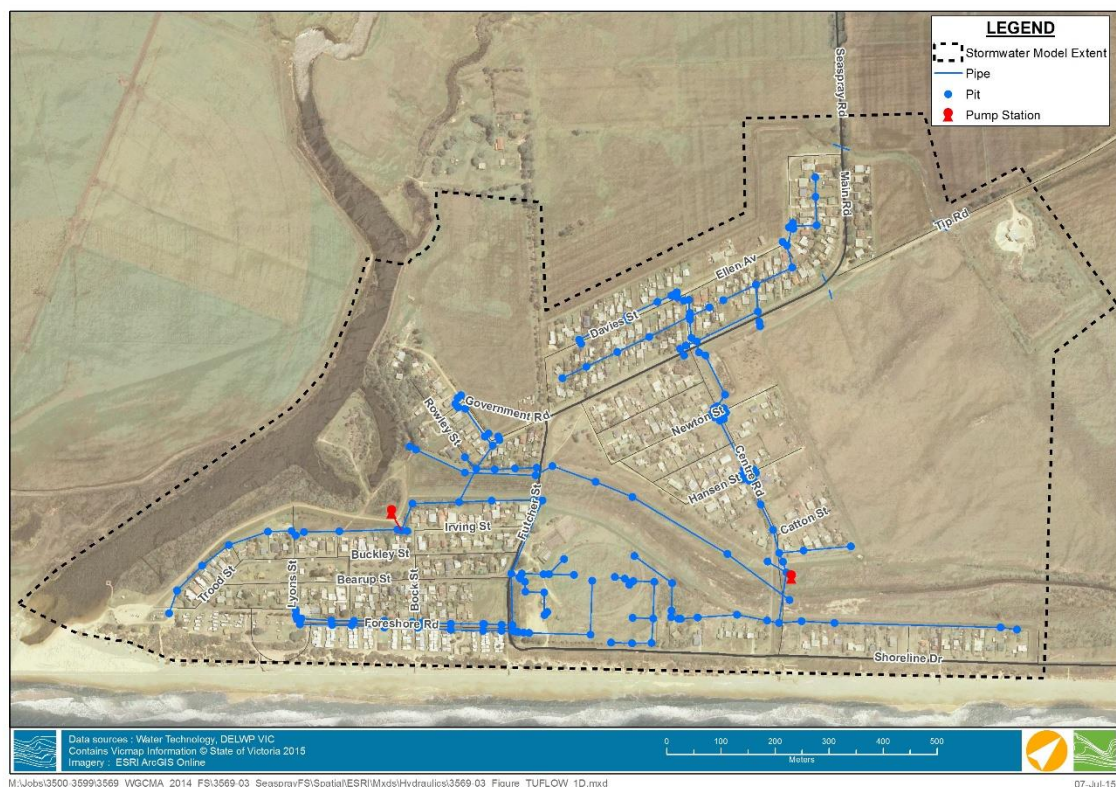


Figure 3-11 Seaspray Pit and Pipe Network

3.6 Fraction Impervious (Stormwater Model)

For the stormwater modelling simulations, rainfall excess was applied directly to the model grid in addition to flow boundaries. To calculate the rainfall excess, initial and continuing losses were subtracted from the design rainfall, varying for each material type identified. The losses adopted for each material are listed in Table 3-4.

Table 3-4 Adopted Losses

Description	Initial Loss (mm)	Continuing Loss (mm.hr ⁻¹)
Open Water	0	0
Beach	50	4
Vegetated Dune	50	4
River	0	0
Riparian Vegetation	50	4
Bush	50	4
Grass	20	2
Grassed Channel	20	2
Channel to Lake Reeve	20	2
Road	0	0
Residential	10	1
Around Island	0	0
Blind Creek	20	2

5. HYDRAULIC MODEL CALIBRATION

5.1 Calibration Approach

The hydraulic model parameters have primarily been refined through calibration against observed flood levels of the September 1993 flood event. The completion of the Flood Mitigation Scheme in 1987 significantly altered the flood behaviour through Seaspray. The introduction of flood defence levees and the construction of the Lake Reeve Floodway as part of this scheme, removed the ability to achieve any relevant calibration to flood events prior to the scheme's implementation. Additionally, the more recent changes to flood levees and culvert upgrades following the 1993 flood event have shown a positive influence in reducing flood impacts within Seaspray. As such, more recent flood events have resulted in limited impact on properties and infrastructure, and has led to insufficient data with which model comparisons can be based.

The calibration of the model centred on the determination of floodplain and river channel hydraulic roughness values (Manning's n values) to achieve a reasonable agreement between observed and modelled flood levels. Initial Manning's n values were assigned to various land uses based on previous modelling experience and within Australian Rainfall and Runoff guidelines. Where necessary to achieve a better agreement between modelled and observed levels, these initial Manning's n values were further refined.

Other factors found to significantly impact the water levels and flood extents in the model were the resolution of features such as levees and the performance of structures including bridges, culverts and the floodway regulator. These factors were also tested during the calibration and adjusted where necessary.

5.1.1 Available Observed Flood Data and Event Selection

Given the change in catchment conditions following the construction of the flood mitigation scheme in 1987, and the scarcity of data in more recent events, the September 1993 flood event was the only event appropriate for model calibration. Flood level data was available in the form of digitized flood extents derived from aerial imagery, spot heights from the Victorian Flood Database (VFD), and anecdotal reports.

In September 1993 an intense low off the east coast generated south-easterly winds blowing across eastern Victoria resulting in heavy rainfall. The catchment of Merriman Creek received the highest precipitation on record and as the flow reached the coast, it caused extensive flooding of the township of Seaspray (HydroTechnology, 1995). The Flood Summary Report prepared by HydroTechnology (1995) provides details of the 1993 flood including inundation extents, spot flood levels, and impacts in Seaspray. A hydraulic model was simulated for the 5 day period from 15 September 1993 to 20 September 1993.



Figure 5-1 1993 Flood Information in VFD

5.2 Calibration Event Conditions

5.2.1 Boundary Conditions

The water levels recorded at Prospect Road Gauge were used in conjunction with the 1995 rating curve to prepare Merriman Creek inflows to the model at Prospect Road (refer to the Hydrology Report (R03) for more discussion on the gauge data and rating curves).

The September 1993 astronomical tide was replicated through the application of the Hinwood (1986) data at McGauran's Beach. It is important to note that the peak flood occurred around low tide, resulting in the tidal boundary having little impact on flooding through the town.

Given the lack of available data, a steady-state water level of 0 m AHD was applied to represent Lake Reeve water levels.

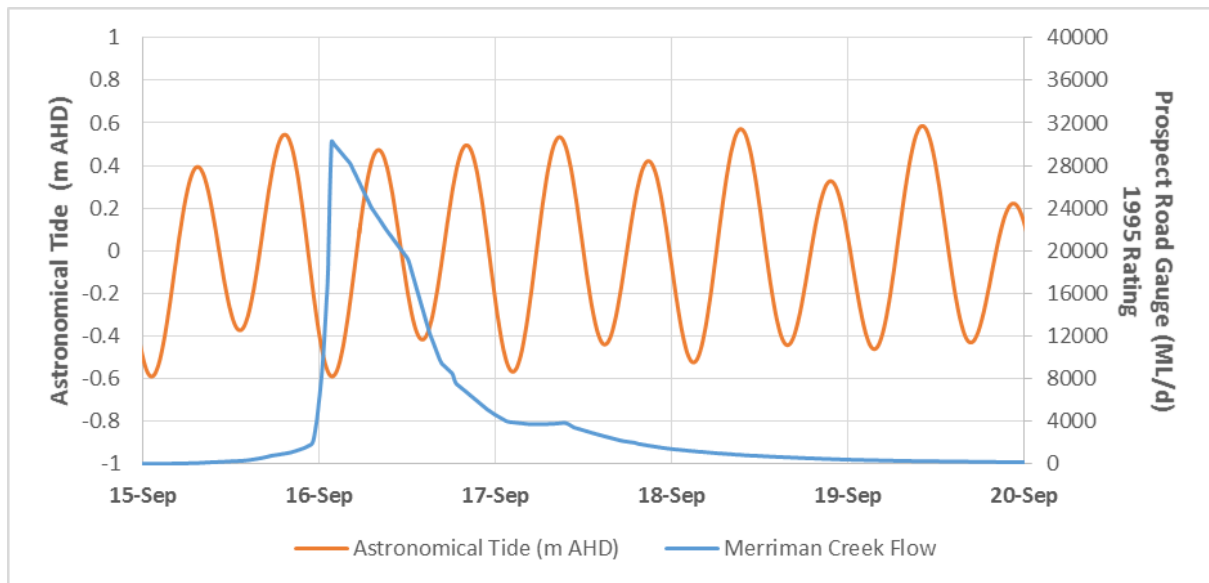


Figure 5-2 Catchment Inflows and Astronomical Tide for September 1993 Calibration

5.2.2 Entrance Condition

The condition of the sand berm is not known preceding the 1993 flood event. However, through anecdotal evidence it is recorded that the entrance of Merriman Creek was open prior to the start of the event (HydroTechnology 1995). To best represent this within the hydraulic model, the 2D model topography generated from the LiDAR was modified to represent an open entrance condition. Based on the historical photographs provided in Section 3.2.3, the model replicated a typical open entrance condition with the entrance lowered to a level of 0 m AHD and a width of approximately 60 m.

5.2.3 Levees

The existing stretch of levee immediately upstream of Government Road was removed from the topography as this was not present during the 1993 flood (HydroTechnology 1995). Levels across this area were estimated from the surface immediately to the east grading from 2.5 m AHD at the southern end to 3.0 m AHD at the northern end.

Levees or irrigation banks which have been removed since the event around the junction of Eastern Prior Stream and Blind Creek were included as straight lines with elevations taken from the embankments present on the sides of the channels in the DEM.

5.2.4 Culverts

The culvert under Seaspray Road was a 450 mm diameter pipe at the time of the 1993 floods. After the flood the culvert was upgraded to a set of five 2000 x 1500 mm RCBCs. All other culverts within the model are consistent with existing conditions.

5.2.5 Regulator Structure

The current operating guidelines for the regulator and floodway are centred on a recommended design flow capacity of 15 m³/s. HydroTechnology (1995) noted that prior to the main flood event, all boards within the regulator were removed to encourage an early pulse of water through the floodway. This was intended to increase hydraulic conductivity along the floodway via flattening of vegetation growth that had occurred in the preceding years. Unfortunately as the water levels rose it became too dangerous to manually restore the flood boards within the regulator and the structure was left at its minimum level of 1.7 m AHD for the duration of the flood event. This level has

therefore been applied to the regulator structure within the hydraulic model for the calibration scenario.

5.3 Results

5.3.1 Flood Levels and Extents

The modelled flood extents down Merriman Creek from Prospect Road to the entrance, across Eastern Prior, and up Blind Creek match closely with the extent determined from aerial imagery. The model appears to under predict the flow entering town from the north down Seaspray Road, which may be indicative in part to the effect of rainfall on this area.

The modelled flood levels match well with those recorded in the Victorian Flood Database (VFD) for the 1993 event as illustrated in Figure 5-3. Spot heights were matched to within 0.1 m for the five points through town from the end of Government Road Levee to Fatcher Street Bridge. The point near the entrance was also modelled to within 0.1 m of the VFD level. The point on the northern edge of the breakout around Government Road Levee was 0.4 m lower in the model than the recorded level. It is noted that the recorded level appears to be very high when compared to the point on the southern side of the same breakout. Additionally, there is a spot height off Rowley Street that exceeds a difference of 0.4 m. This appears to be another erroneous recorded height due to an immediately adjacent point recording a flood height 0.2 m lower.

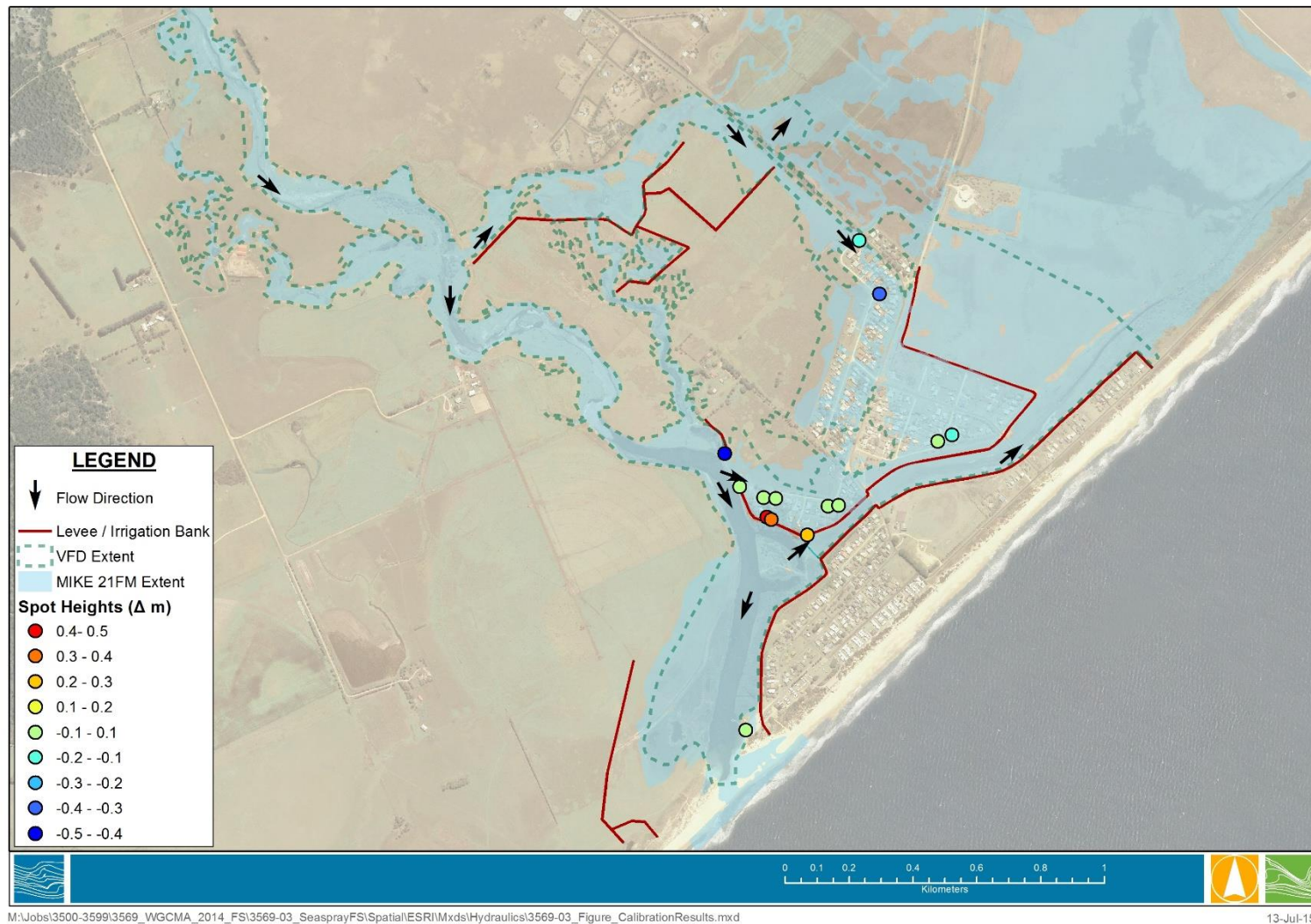


Figure 5-3 Modelled vs. Recorded Flood Extents and Spot Heights

5.3.2 Flood Event Behaviour

The flood pattern matched what was recorded in the flood review (HydroTechnology 1995). The flow directions indicated in that review are consistent with the model results.

Flood waters first entered Eastern Prior Stream when the water level in Merriman Creek reached 4.0 m AHD. This flow then filled up the area behind the irrigation banks to a level of 5.7 m AHD before continuing north-east to Seaspray Road. A small flow continued through the culvert under Seaspray Road, some water overtopped the road, and the rest continued down beside the road to the drainage channel. The water continued down the drainage channel in both directions and an additional flow path continued down Seaspray Road. These flows spread out between Main Road and the drainage channel and connected to flows crossing Futchter Street north of the bridge.

Flows entered Seaspray between what was Government Road Levee and Griffioen's Levee when the water level in Merriman Creek reached 2.6 m AHD. This flow ran down Government Road and Rowley Street before overtopping the levee into the floodway and crossing Futchter Street north of the bridge. The area between Hansen Street Levee and Main Road filled with water until the levee to the north was breached and the flood waters could continue east towards Lake Reeve.

5.3.3 Comparison with Current Topography

Due to the significant impact that alterations to the floodplain had on the flooding mechanisms during the 1993 event the model was rerun using the current topography levee heights, and culverts. This simulation represents the hypothetical flood behaviour if the same storm event was to occur again, and assuming the entrance is open. The resultant flood map is shown in Figure 5-4.

In this simulation the irrigation banks were opened at the top of Blind Creek, the culvert under Seaspray Road was upgraded, and Government Road Levee was extended north to meet Griffioen's Levee. In this simulation the flow in Eastern Prior was directed immediately down Blind Creek with the water level behind the irrigation banks not exceeding 4.4 m AHD. Water now flowed further east along Eastern Prior to Seaspray Road. No flood water entered town in the vicinity of Government Road Levee, and none of the levees were overtopped.

It appears that the removal of the farmer's levee allows water to flow down Blind Creek and there is subsequently limited flow further east through Eastern Prior towards Seaspray Road. The extension of the Government Road levee has also cut off a previous flow path into the town.

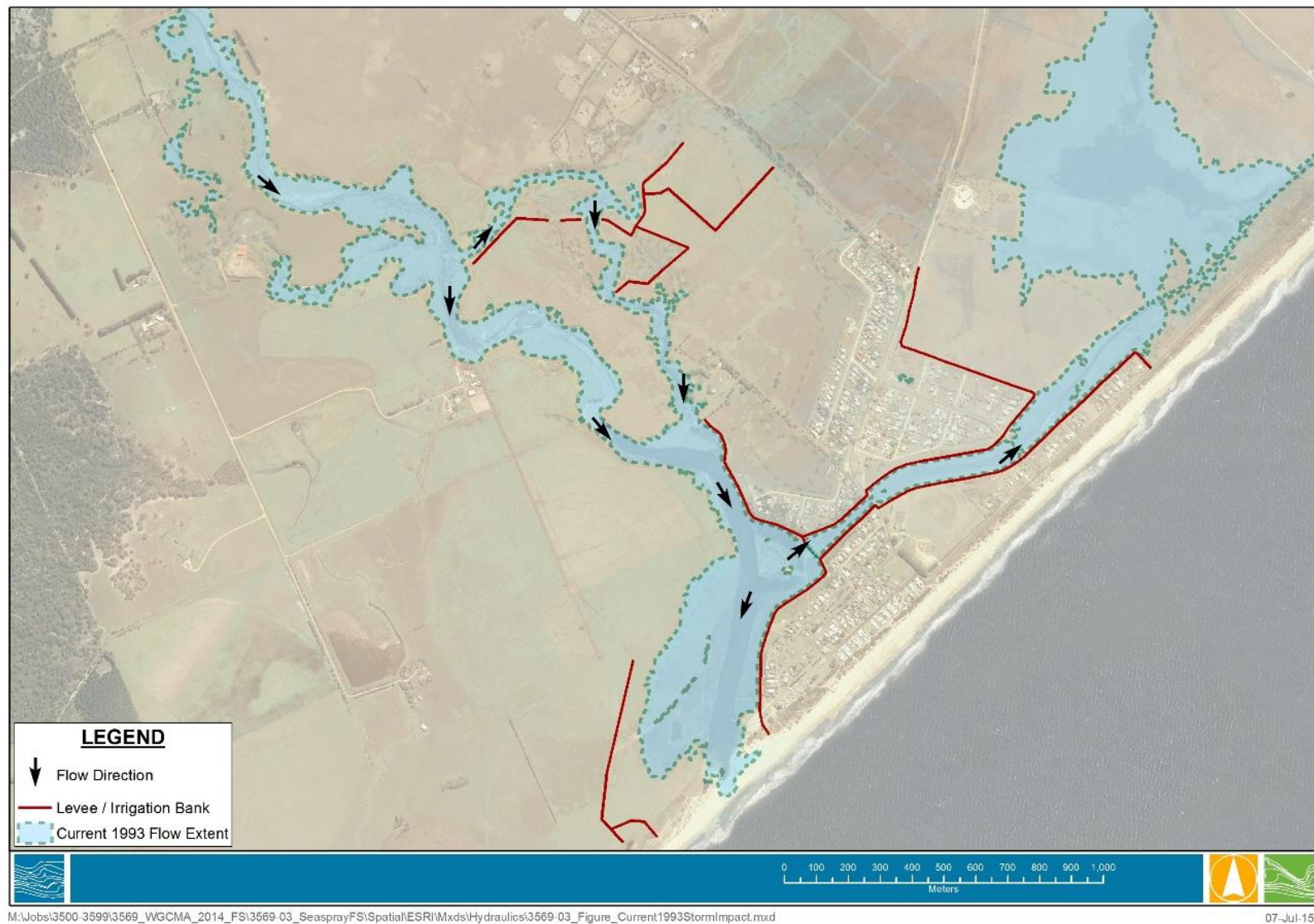


Figure 5-4 Modelled Flood Extents for 1993 Flows with Current Topography

6. SENSITIVITY ANALYSIS

6.1 Overview

There are a range of mechanisms influencing flooding at Seaspray and therefore it is important to understand how each mechanism works and the relative interaction between them. To do this, the following series of conditions were assessed in order to compare flood behaviour associated with the different flood mechanisms and to aid the choice of design flood event parameters:

- Varying roughness as a result of different vegetation being present in the floodway,
- Coincidence of catchment flows and coastal water levels,
- Coincidence of catchment flows and stormwater flooding,
- Entrance opening conditions, including,
 - A 'bathtub' type inundation assessment,
 - A fixed height entrance berm,
 - A variable height entrance berm,
- Operation of the regulator structure, and
- Flooding from Lake Reeve.

The results of these assessments are detailed in the following sections.

6.2 Increased or Decreased Vegetation in the Floodway

The local community has shown considerable interest in the removal of substantial reed growth from within the floodway. For instance, a heavily vegetated channel, as may be the case when there are large areas of tall reeds or dense vegetation present, can result in higher water levels in waterways.

The impact of reeds in the floodway have been assessed through the comparison of simulations with different roughness values representing different vegetative conditions. The floodway with reeds applied a Manning's n roughness coefficient of 0.08, while an n of 0.04 was applied to present conditions without reeds.

The simulation chosen for the comparison had an open entrance condition, with the 1% AEP flow in Merriman's Creek and the astronomical tide applied offshore.

The results showed that the presence of reeds in the floodway slowed the progression of flow through the floodway resulting in higher water levels at the upstream section (between the regulator and Fulcher Street Bridge) and lower levels at the downstream (near the Centre Road Bridge). Water levels immediately downstream of the weir were increased by 0.12 m, levels at Fulcher Street Bridge raised by 0.05 m, and levels at Centre Road Bridge lowered by 0.05 m. Flood levels between the floodway and Main Road increased by 0.09 m.

As a result of this increased gradient in the flood surface, a potential greater risk of overtopping may occur in the western end of the floodway causing greater depths and flood extents on both sides of the floodway. A potential reduction in risk of overtopping may occur in the downstream of the floodway. Figure 6-1 shows the difference between the vegetated and non-vegetated cases with increased water levels shown in red and decreased water levels shown in green.

This analysis represents a conservative approach with regard to the impact of vegetative growth within the floodway. Whilst tall reeds will present a significant obstruction to flow at shallow depths, as flow increases in the floodway the reeds typically respond lying flat against the flow and the hydraulic roughness would be reduced to a value closely aligned to the intended design of the floodway.

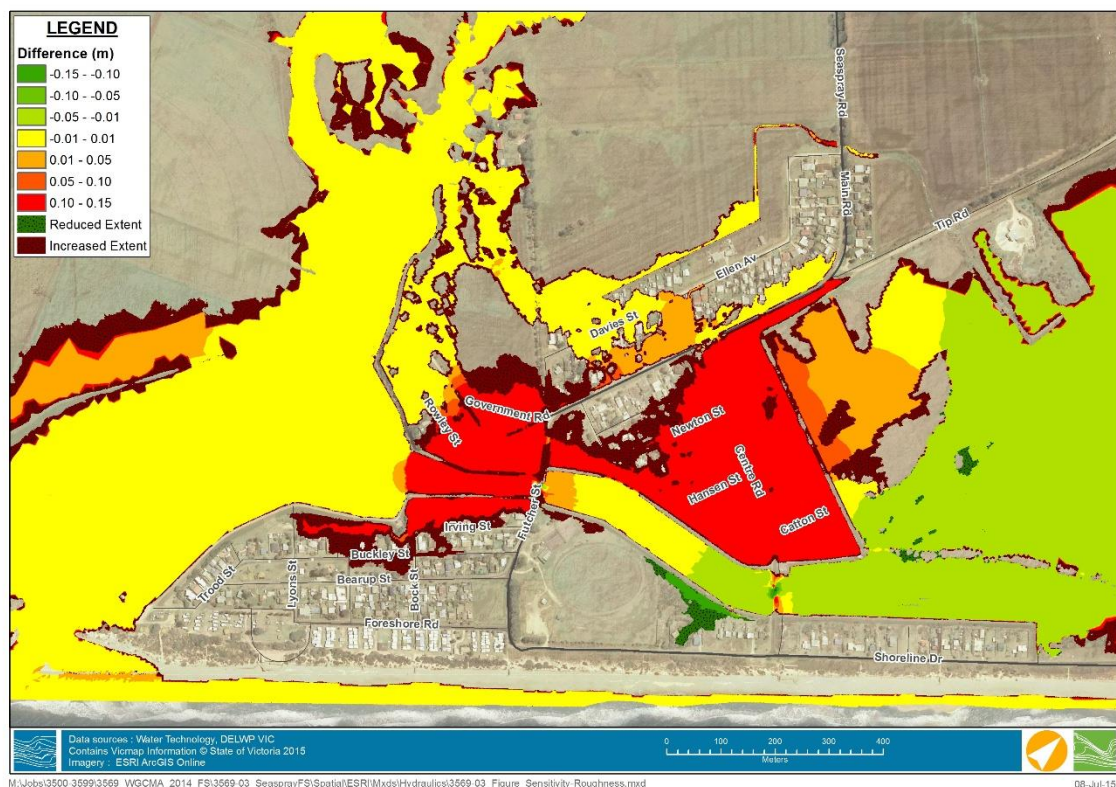


Figure 6-1 Difference plot of water surface elevation with increased roughness

6.3 Flooding Mechanism Interactions

6.3.1 Coincidence of Catchment Flows and Coastal Water Levels

The revision of Australian Rainfall and Runoff (AR&R) is currently investigating the coincidence of fluvial flooding events and coastal water levels in estuarine areas, with results of the revision project detailed in the draft AR&R Project 18 Stage 3 report (AR&R, 2014). Where fluvial flooding and coastal water levels occur at the same or similar time both flood mechanisms could have an effect on water levels in an estuary. Where there is a measureable influence from both mechanism they are considered “dependent”.

For areas where the level of dependence between extreme rainfall and storm surge has not been determined, the Project 18 report recommends an initial screening analysis be undertaken to classify the flooding as either:

- Dominated by rainfall processes;
- Dominated by ocean processes; or
- Influenced by both processes.

A dependence analysis has not previously been undertaken for Seaspray and therefore the initial screening analysis was undertaken using the following modelled scenarios:

- 1% AEP catchment generated flood event with a 1% AEP storm tide (peak storm tide elevation based on McInnes et al. (2009) under present mean sea level).
- 1% AEP catchment generated flood event with an astronomical tide only; and
- 1% AEP storm tide (peak storm tide elevation based on McInnes et al., 2009 under present mean sea level, Table 3-1) with no catchment flood event (assumed baseflow condition).

The results of these scenarios showed the following:

- The 1% AEP storm tide alone with a maximum elevation of 1.5 m AHD (Section 3.3.3) did not cause inundation of any of the Seaspray township, even assuming a fully open entrance condition.
- The 1% AEP storm tide combined with 1% AEP catchment flooding resulted in flood levels that were only 0.05m higher than the 1% AEP catchment only flood event.

It can therefore be concluded that flooding at Seaspray is dominated by rainfall processes. However, for the purposes of the design flood events it is recommended that the 1% AEP storm tide condition be applied to the ocean boundary for all design flood events to ensure an increase in water levels as a result of elevated ocean levels is accounted for. This has therefore been adopted for the design event simulations.

6.3.2 Coincidence of Catchment Flows and Stormwater Flooding

Section 3.7 of the Hydrology Report (R02) established the design flow hydrographs at the Prospect Road gauge. For all assessed catchment flood events ranging from the 10% AEP to the 0.5% AEP, the critical duration was either a 36 hour or 48 hour storm with the peak flow occurring at the Prospect Road gauge approximately 30 hours after the commencement of rainfall within the catchment.

Given the relatively small urban area of Seaspray, rainfall excess contributing to localised stormwater flooding will occur almost immediately after the commencement of an extreme rainfall event. As such, peak flows resulting from stormwater flooding will occur, and move through the stormwater system, well before the peak catchment flows reach Prospect Road.

As such, it is considered extremely unlikely that catchment flooding would occur in Merriman Creek and the Seaspray stormwater system coincidentally, due to the difference in the timing of flows from the contributing catchment areas. However, as the stormwater flows discharge into the floodway system, catchment generated flooding may limit the ability of the stormwater system to effectively drain the excess water from the township.

In order to account for the effect of stormwater flooding in the event of a catchment generated flood, the following approach was adopted for the design flood event simulations:

- Direct rainfall on grid was applied to the stormwater model of Seaspray which provides an indication of the performance of the stormwater system,
- On completion of each stormwater design event simulation, the areas that remain inundated across the township was then applied as an initial condition for the main floodplain model,
- The resultant flood inundation extents therefore incorporate the effects of stormwater retained on the floodplain together with the inundation occurring as a result of catchment flooding.

6.4 Entrance Opening Conditions

The report by Camp Scott Furphy Pty. Ltd. (1980) recommended a sand berm level of between 2.0 m AHD and 2.5 m AHD be maintained at the entrance to Merriman Creek at Seaspray in order to limit flooding as a result of inundation from elevated ocean water levels. There is local concern however, that maintaining the berm at these elevations may increase inundation of the township during catchment flooding events because the entrance may not open quickly enough to release flood flows prior to the levees being overtopped.

In order to investigate the sensitivity of flood levels and inundation extents to the elevations and conditions at the entrance berm the following assessments have been undertaken:

- Bathtub modelling – comparing static water levels to the DEM;
- Modelling catchment flooding with various fixed berm heights;
- Modelling catchment flooding with dynamic scouring of the entrance using the sand transport module; and
- Modelling catchment flooding with a dynamically opening entrance for various initial berm heights.

6.4.1 Bathtub Model

As an indication of the potential flood impacts associated with different berm heights, a “bathtub” type approach applied to the Seaspray model DEM. Bathtub modelling is a relatively simple technique, which only requires two input datasets; a DEM, and a single water surface elevation level. The water surface elevation level is intersected with the DEM, and all elevations below the specified water surface elevation are considered to be areas where potential inundation could occur. This approach has been used in Victoria to predict storm tide inundation along the Victorian coast (Lacey & Mount, 2012).

In this instance, instead of dynamic storm tide inundation, the potential still water inundation as a result of a fixed entrance berm was modelled. The resultant bathtub inundation extents for berm heights of 2.2 m and 2.5 m AHD are shown in Figure 6-2. The inundation extents in the figure show all the locations within the town which lie below 2.2 m and 2.5 m AHD.

If the berm height is maintained above the elevation of the weir adjacent to the regulator structure (2.2 m AHD) it would be possible for water to accumulate behind the berm and for the floodway to become activated during non-flood flows. This would allow water to move east down the floodway potentially overtopping the floodway levees where their crest levels is below 1.8 m AHD.

The limitation of this approach is that the modelled extents only take into account a single water level and do not take into account the dynamics and volume of the flood event. It is unlikely that a flood event in Merriman Creek would be of sufficient volume to inundate all of the areas shown. Additionally, the effect of levees on preventing inundation of certain areas is not explicitly taken into account. For example, if the crest elevations of the levees along Merriman Creek are taken into account, the flood extent to the east of the creek is significantly limited for a water level of 2.2 m AHD, as shown in Figure 6-2 (labelled 2.2 m AHD constrained).



Figure 6-2 Bathtub Inundation Extents at Seaspray for Berm Heights of 2.2 m and 2.5 m AHD

6.4.2 Fixed Berm Height Modelling

In order to better understand the potential impact on inundation extents associated with different elevations of the entrance berm, the 1% AEP flows were run through the model with fixed initial berm heights of 0.0, 1.6, 1.85, and 2.1 m AHD. In each case the initial elevation of the berm was adjusted from the 1.1 m AHD, as shown in the available LIDAR, over the full width of the entrance. These simulations assume that no scouring of the entrance berm takes place over the model simulation period. The inundation extents resulting from each scenario are illustrated in Figure 6-3.

Very little difference is observed in flood extents for the 1% AEP event for the fixed berms of heights of 1.6 m to 2.1 m, with significant inundation occurring on both sides of the floodway. An open entrance condition (initial level of 0.0 m AHD) results in reduced inundation extent, particularly south of the floodway. A summary of the simulations is provided in Table 6-1.

Table 6-1 Summary of Fixed Berm Height Sensitivity Tests

AEP Event	Initial Berm Elevation	Flooding Description
1%	2.1 m	Initial overtopping of the Hansen Street Levee followed by overtopping of levees on both sides of the floodway. Two hours later Government Road Levee overtops, and flow enters town from the north (Blind Creek).
	1.85 m	Similar flood behaviour and extent to 2.10 m.
	1.6 m	Similar flood behaviour and extent to 2.10 m.
	0.0 m	Flow from the north (Blind Creek) and overtopping of Government Road Levee, followed by overtopping of the Hansen Street Levee. The levee on the northern side of the floodway west of Fatcher Street Bridge overtops from the inside, then the levees on the southern side of the floodway are briefly overtopped.
10%	2.1 m	No overtopping of levees or inundation of town.

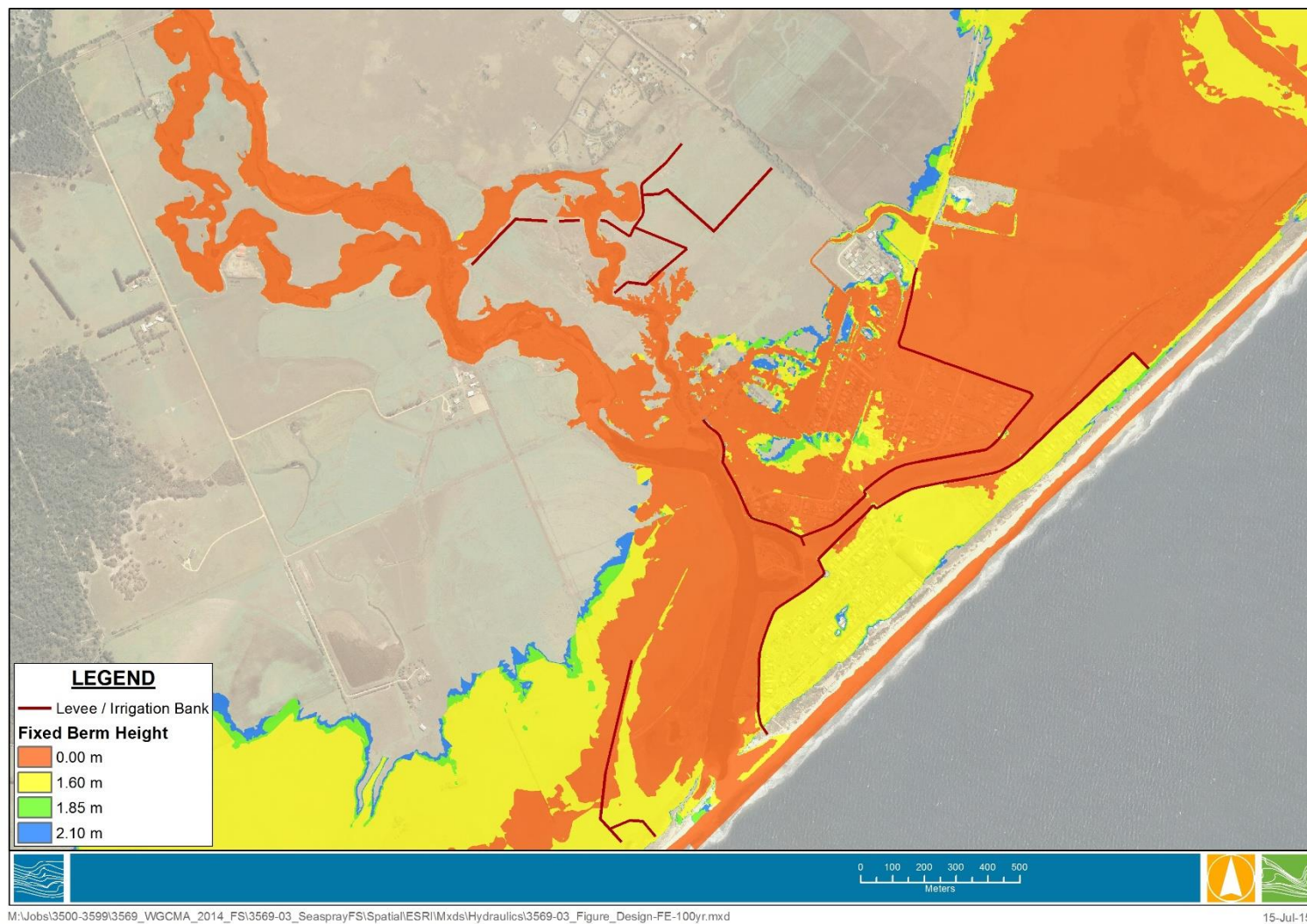


Figure 6-3 Inundation Extents (1% AEP) for Fixed Entrance Berm Heights of 0.0m, 1.6m, 1.85m, and 2.1 m AHD

6.4.3 Entrance Scour

The flow of water across the entrance berms erodes the sandy sediment and scours an outlet channel between Merriman Creek and the ocean. The width and depth of this channel is determined by the flow rate and velocity of this outflow.

In order to understand whether an entrance channel would form as a result of flood flows across the berm the following assessment was undertaken:

- Flow velocities at the berm were extracted from the fixed berm simulations and compared to the minimum velocities required to move sand of a particular particle size;
- A mobile bed sediment transport simulation was undertaken using the sediment transport module within the MIKE software suite. This module explicitly takes into account the size of the sand material in the berm and the erodability of that material.

Critical Velocity

Scour of sand occurs when a critical threshold velocity is reached. There are various methods available for estimating this threshold and for the purposes of this study two commonly methods; the Hjulstrom (1935) and Hancu (1971), were used to determine an appropriate threshold velocity to assess if the sand particle sizes available on the beach and berm at Seaspray would be eroded under flood flow conditions.

Sand samples have previously been collected at Seaspray (Water Technology, 2014) and Table 6-2 details the results of the grain size analysis used to assess the typical sand sizes present at this location on the dune face and the swash (beach) zone. The median grain size is the d_{50} value, whereby 50% of the grains in a sample are larger and 50% are smaller. Based on the results shown in Table 6-2, the d_{50} grain size is between 0.225 mm and 0.425 mm, which is classified as a “fine sand” (ASTM D2487-11). Table 6-2 also indicates that the sand is well sorted, meaning there are only a narrow range of grain sizes present. A narrow range of grain sizes means that the behaviour of the d_{50} size sand will be typical of the majority of sand present on the beach.

Table 6-2 Grain Size Distribution for a Sediment Samples Collected from Seaspray (Water Technology 2014)

Grain Size, d (mm)	Percent of Sample Smaller than (%)	
	Dune Face	Swash Zone
19.0	100	100
9.5	100	100
4.75	100	99
2.36	100	99
1.18	97	90
0.60	85	66
0.425	78	49
0.300	58	25
0.150	1	1
0.075	1	1

Based on the methods of Hjulstrom (1935) and Hancu (1971), the critical velocity for movement (and hence scour) of sand with a $d_{50} = 0.225$ to 0.445 mm, is between 0.3 m/s to 0.7m/s, depending on the water depth.

To confirm whether sand on the berm at Seaspray would therefore be eroded during a flood event, a time series of velocity was extracted from the model results at the berm for a series of fixed berm height condition simulations. These results are shown in Figure 6-4. In all three simulations the velocities are significantly higher than the threshold velocities required to initiate scouring for over 18 hours, and hence sand from the berm would erode and form an entrance channel.

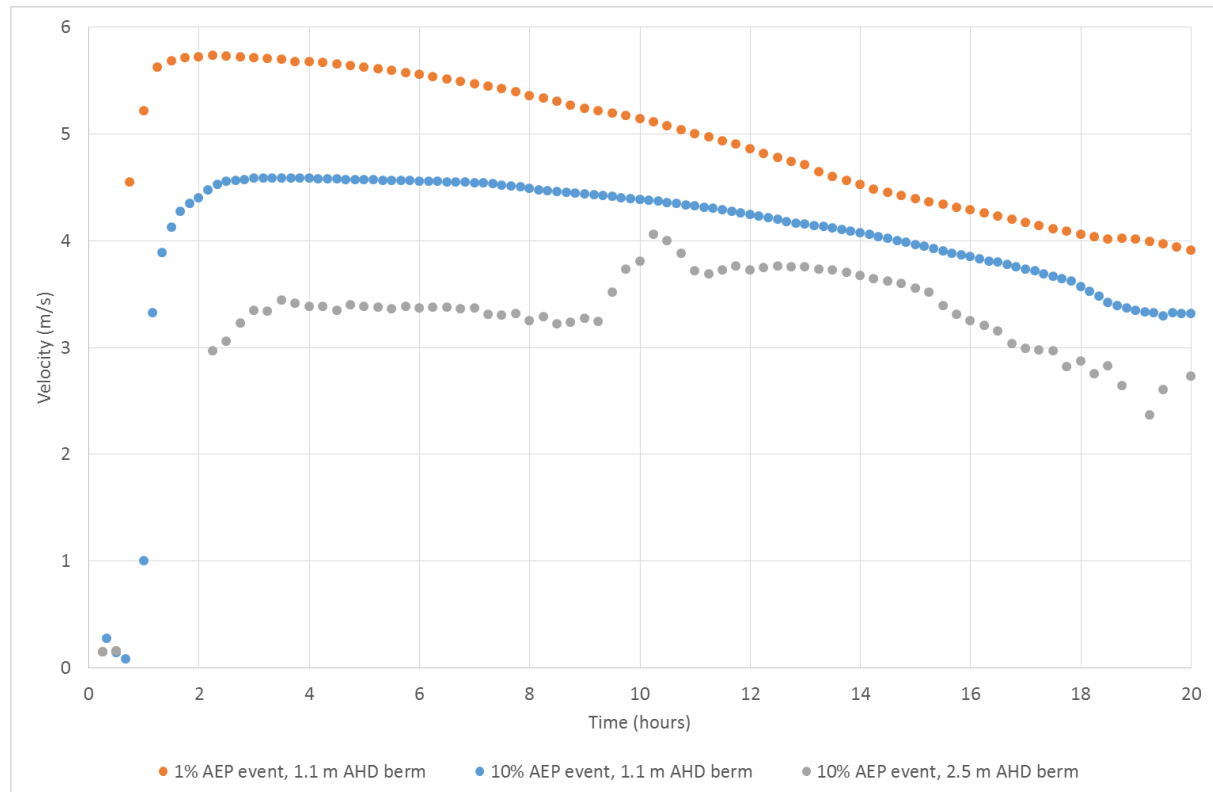


Figure 6-4 Modelled velocities across the entrance berm

Although these results clearly show that an entrance channel will form when there is a flood flow in Merriman Creek which overtops the berm, they do not indicate how wide or deep the channel may be.

Therefore a simulation was undertaken using the sediment transport module of the MIKE21FM software to assess what size channel may form during a 10% AEP catchment generated flood event. The entrance berm was set at an initial height of 2.5 m AHD and this was allowed to dynamically adjust within the model over time, based on the modelled flow velocities over the berm.

Figure 6-5 shows the topography at the entrance berm before and after the simulation, where the channel that has formed has a width of approximately 30 m and its depth has reached the minimum elevation of 0.0 m AHD during the simulation. This width is consistent with channel widths determined from historic aerial imagery, Figure 3-3. In this model the entrance took approximately 2 hours to scour to an elevation of 0.0 m AHD.

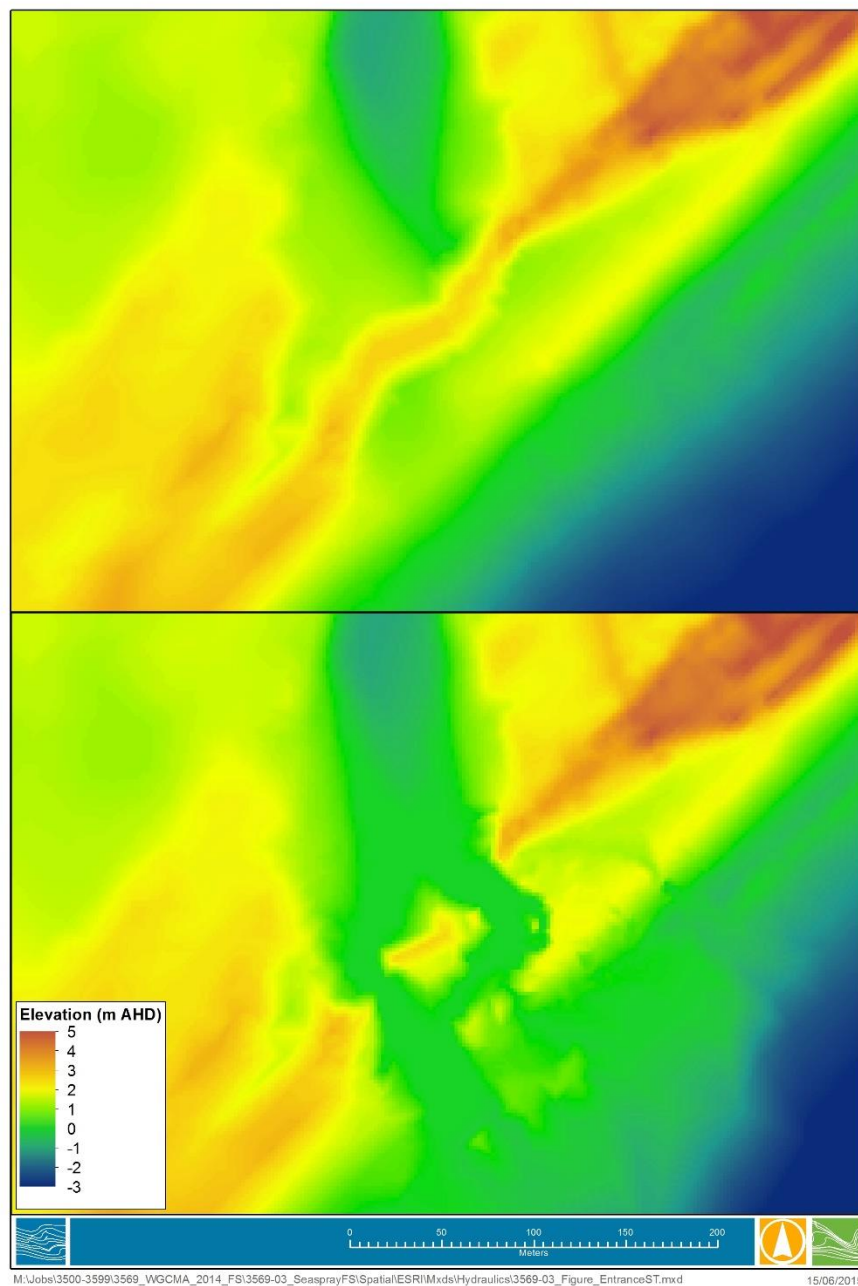


Figure 6-5 Entrance berm within dynamically scouring channel during 10% AEP flood event

6.4.4 Entrance Berm Level Assessment

The results of the initial fixed berm height assessment along with the analysis of flow and scour conditions across the berm indicate that under flood flows an entrance channel will scour through the berm area, allowing connection between the estuary and the sea and resulting in a lowering of the water level in Merriman Creek.

Therefore, in order to develop a suitable berm opening condition for the design flood simulations, a number of scenarios were run with various opening periods and initial berm heights. Three of the scenarios consider the period over which the entrance scours (1, 2, or 3 hours), and the others consider various initial berm heights; 2.10, 1.85, 1.60, 0.00 m AHD. The reason for selecting these levels was as follows:

- An initial height of 2.10 m is 0.10 m below the current minimum elevation of the floodway weir which is set at 2.20 m AHD.
- The peak 1% AEP storm tide elevation under current mean sea level conditions is 1.50 m $\pm$ 0.14m (Table 3-1). A level of 1.60 m AHD is therefore just above the peak storm tide elevation.
- 1.85 m AHD was selected as an intermediate elevation, as a trade-off between limiting the impacts of storm tide inundation and catchment generated flooding.
- The initial level of 0.00 m AHD represents the conditions where the entrance is fully open, as was the case in 1993. In 1993 a small flood prior to the main flood event overtopped the berm and scoured a large entrance channel.

Entrance Scour Initiated at Maximum Berm Height

In each scenario scouring of the entrance channel commences as the water level in the estuary reaches the maximum berm height. The channel then continues to deepen until a level of 0.00 m AHD across the 30 m width of the entrance is achieved over a specified period. Three durations over which the scouring occurs were tested (1, 2 and 3 hours), based on the results of the sediment transport modelling, as described in Section 6.4.3. The results of the assessment are shown in Table 6-3 and illustrated in Figure 6-7. Also included is the result of the fixed berm height modelling scenario for comparison.

The results demonstrates the sensitivity of inundation extents at Seaspray to the initial condition of the entrance. For the initial berm height of 2.10 m the duration of scouring at the entrance has very limited impacts on flood extents and produces very similar flood extents to the open entrance condition.

Table 6-3 Berm Height Sensitivity Tests - Scour at Maximum Berm Height

AEP Event	Initial Berm Elevation	Flooding Description
1%	2.10 m (fixed)	Major flooding on both sides of town. Overtopping of levees and flow from the north (Blind Creek)
	2.10 m (3hr)	The first water to enter town is from the north (Blind Creek). The northern end of the Government Road levee is breached and Hansen Street Levee. The levee on the northern side of the floodway west of Futcher Street is breached from the inside. Levees on southern side of floodway are breached.
	2.10 m (2hr)	Very similar to 2.10 m (3hr)
	2.10 m (1hr)	Very similar to 2.10 m (3hr)
	1.85 m (1hr)	Very similar to 2.10 m (3hr)

AEP Event	Initial Berm Elevation	Flooding Description
	1.60 m (1hr)	Very similar to 2.10 m (3hr)
	0.00 m (fixed)	Flow from the north (Blind Creek) and overtopping of Government Road Levee, followed by overtopping of the Hansen Street Levee. The levee on the northern side of the floodway west of Futcher Street Bridge overtops from the inside, then the levees on the southern side of the floodway are briefly overtopped.

Entrance Scour Initiated at Flood Peak

A further series of scenarios were undertaken to assess the sensitivity of the initial berm height for various AEP events with the scouring of the entrance channel is initiated when the water level inside the estuary peaks, rather than when the maximum berm height is reached. For each of these scenarios the entrance channel was scoured over a 1 hr period to a final bed level of 0.0 m AHD. The results of these scenarios are summarised in Table 6-4 and illustrated in Figure 6-7.

The results show limited difference in flood extents for the various initial berm heights tested under the 1% AEP event conditions, with flooding on both sides of town for all three scenarios tested. The resultant flood extent under the opening entrance condition is similar to the fixed berm height scenarios.

Table 6-4 Berm Height Sensitivity Tests - Scour at Peak Water Level

AEP Event	Initial Berm Elevation	Flooding Description
1%	2.10 m	Initial overtopping of the levee north of the floodway between Futcher Street and Centre Road bridges. Followed by overtopping of levees on both sides of the floodway, Government Road Levee, and flow from the north (Blind Creek). Major flooding on both sides of town. Initial overtopping of the Hansen Street Levee followed by overtopping of levees on both sides of the floodway. Overtopping of levee near the southern pumping station and Trood Street Levee near the carpark. Two hours after the initial overtopping along the floodway, Government Road Levee overtops, and flow enters town from the north (Blind Creek).
	1.85 m	Major flooding on both sides of town. Overtopping of levees and flow from the north (Blind Creek)
	1.60 m	Major flooding on both sides of town. Overtopping of levees and flow from the north (Blind Creek)
	0.0 m	Flow from the north (Blind Creek) and overtopping of Government Road Levee, followed by overtopping of the Hansen Street Levee. The levee on the northern side of the floodway west of Futcher Street Bridge overtops from the inside, then the levees on the southern side of the floodway are briefly overtopped.

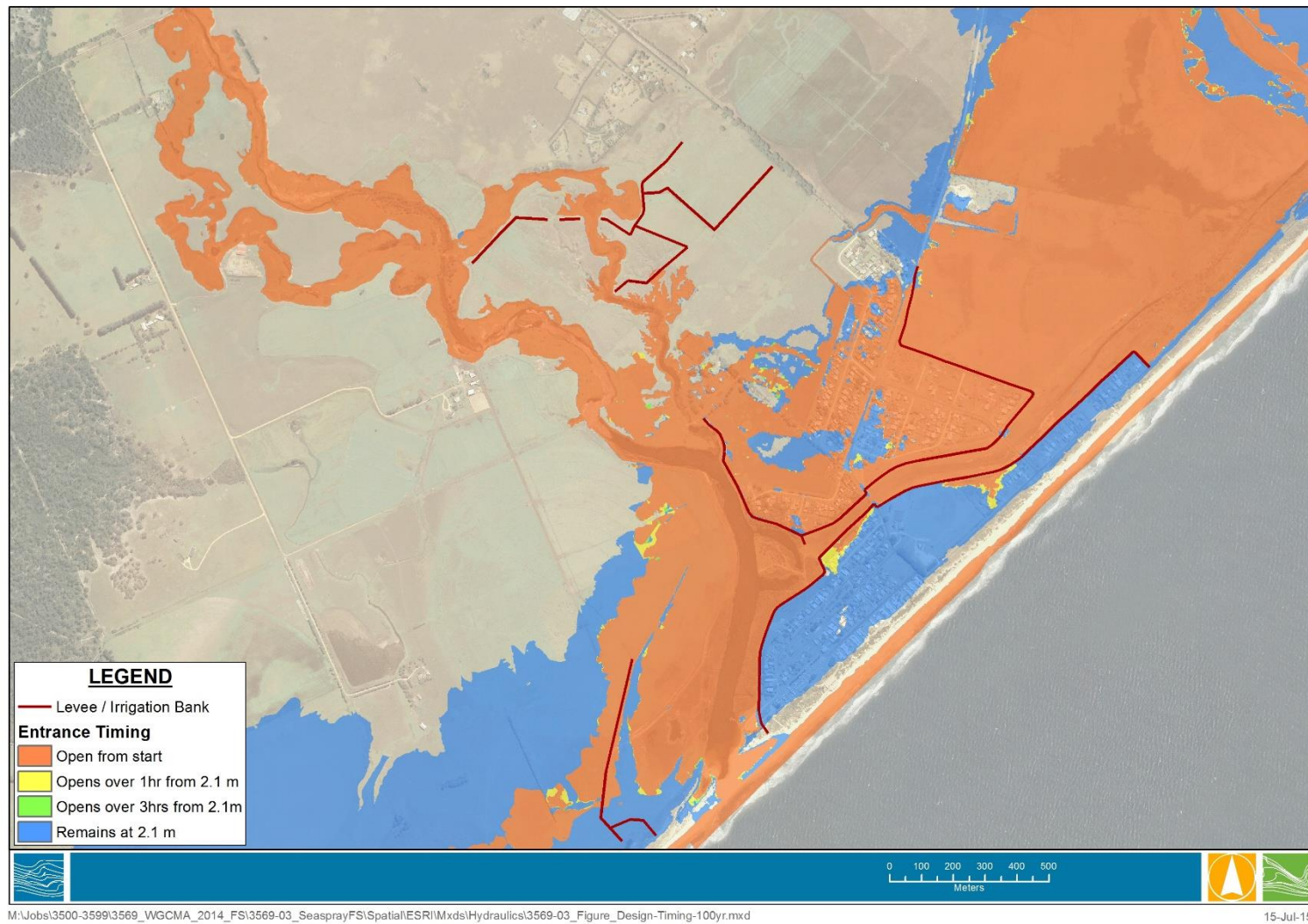


Figure 6-6 Initial Berm Height Sensitivity – scour initiated at maximum berm height (1% AEP event)

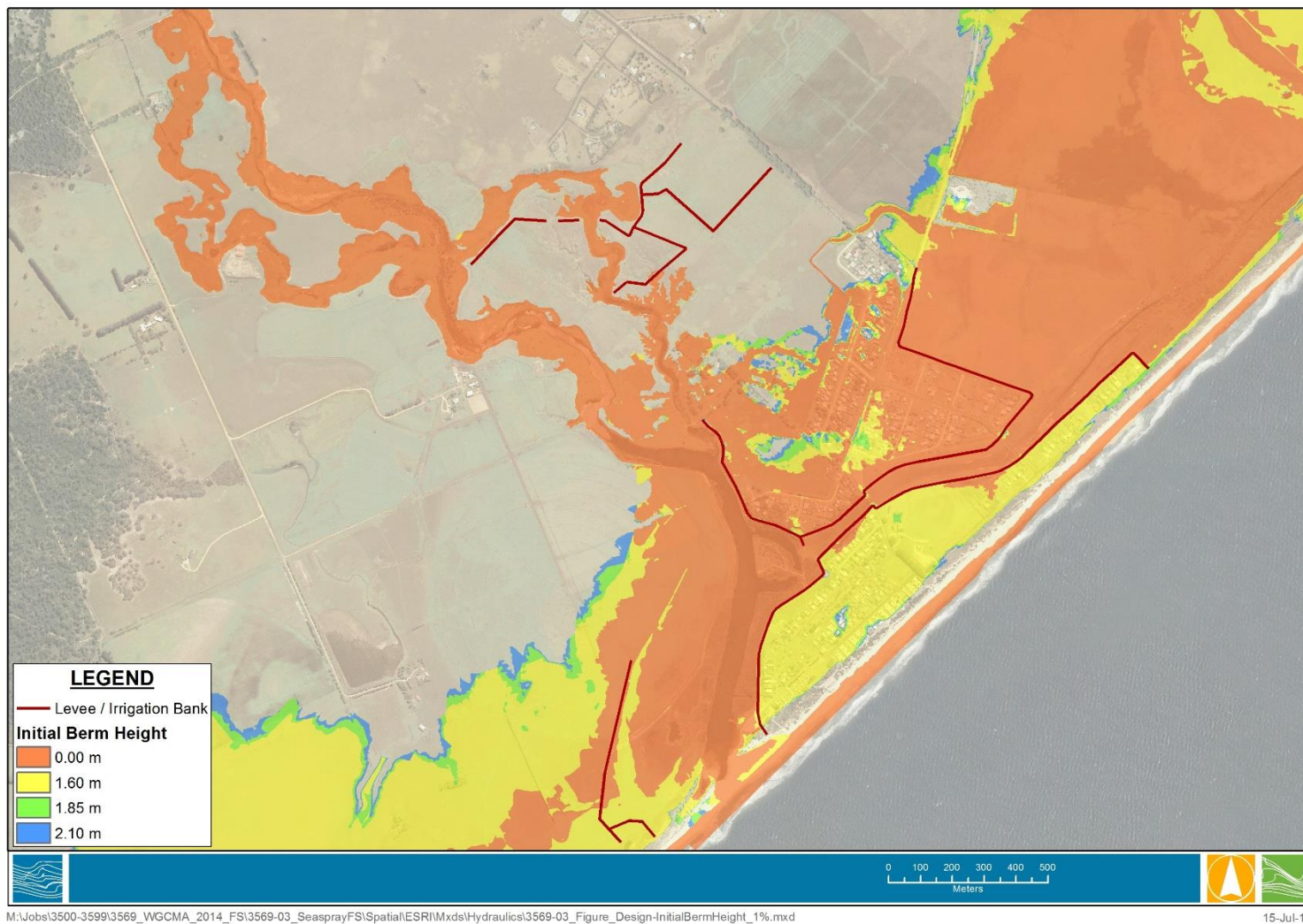


Figure 6-7 Initial Berm Height Sensitivity - scour initiated at peak water level in the estuary (1% AEP event)

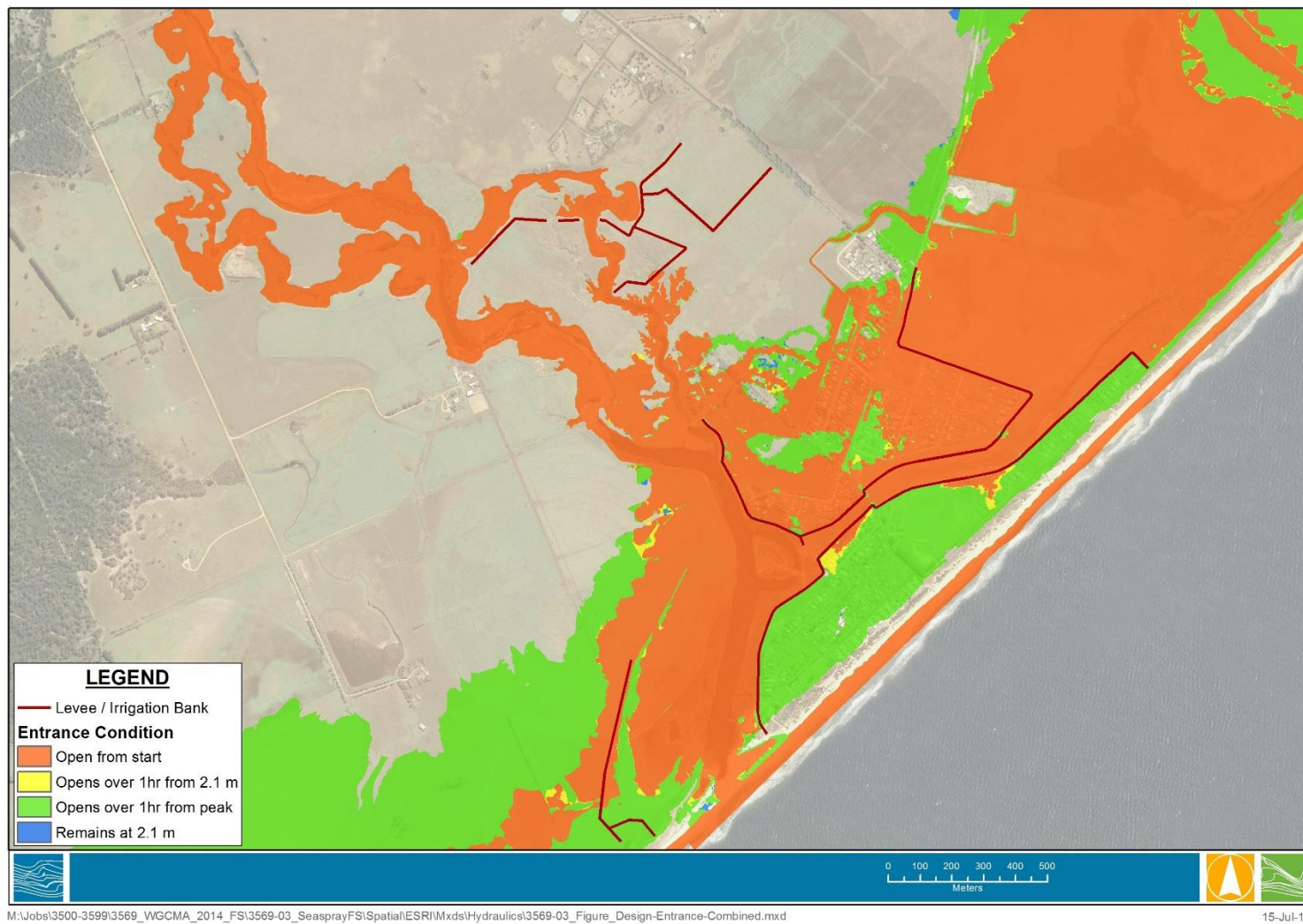


Figure 6-8 Effect of Entrance Opening Conditions (1% AEP, Combined)

6.4.5 Summary of Entrance Opening Conditions

The simulations discussed above demonstrate the importance of an open entrance condition prior to the peak of catchment flows to minimize resulting flooding. It is considered reasonable that the entrance channel would begin to scour from the time the berm is overtopped, rather than when the peak water level occurs in the estuary.

The initial berm height and rate of scouring of the entrance channel appear to have limited impact on the maximum flood extents, and therefore for the design events an initial berm height of 2.10 m AHD was adopted, with the channel opening over a 3 hr period. This represents likely conditions under the present operating strategy for the floodway regulator and represents realistic flood extents throughout the study area.

6.5 Regulator Structure

The regulator structure was operated with a level of 1.70 m AHD for the calibration event based on details of the actual operation during the event. It was subsequently raised to 2.70 m AHD for sensitivity testing in the model to represent the current recommended operating strategy. Raising the height of the structure preferentially directs water towards the entrance berm rather than down the floodway towards Lake Reeve. This increases the rate and potential of scour of the sand berm.

The purpose of the regulating structure is to allow and manage catchment generated flows into the floodway but also prevent storm tide inundation of the floodway channel. Therefore a sensitivity test was undertaken to assess the performance of the structure for the 1% AEP storm tide event.

For the simulation, a mean daily flow condition was applied to inflows from Merriman Creek, the entrance channel was assumed fully open with a minimum elevation of 0.0 m AHD and a width of 30 m, and 1% AEP storm tide was applied to the ocean boundary condition. As mentioned previously, the peak 1% AEP storm tide elevation under current mean sea level conditions is 1.50 m $\pm$ 0.14m (Table 3-1).

The model results showed no overtopping of the regulator structure or the adjacent levees as a result of the assessed 1% AEP storm tide event.

6.6 Flooding from Lake Reeve

The Gippsland Lakes/90 Mile Beach Local Coastal Hazard Assessment Report (Water Technology, 2014) found that the low lying areas around Seaspray would not be inundated from overflows from Lake Reeve to the east during a 1% AEP catchment generated flood event in the Gippsland Lakes under present mean sea level and for a 0.2 m rise in sea level.

Further modelling work was undertaken to test this further assuming the occurrence of a 10% AEP catchment generated flood in the Gippsland Lakes under present mean sea level and for sea level rise increments of +0.2m, +0.4m, and +0.8m. The model results showed that the floodplains around Seaspray only became inundated during the 10% AEP flood event under the +0.8 m sea level rise scenario. The flood levels did not reach a height sufficient to overtop the levees surrounding Seaspray (Water Technology, 2014).

For the present assessment this previous work was reassessed using the more detailed model of Lake Reeve and Seaspray that has been developed. The level in Lake Reeve was set to 1.37 m, based on the 2014 results for the 10% AEP flood with +0.8m sea level rise, and water from the lake was allowed to flow towards Seaspray.

The resultant flood extent is shown in Figure 6-9, with water reaching the levees at the eastern end of the floodway but no overtopping of the levees occurred. It is therefore unlikely that flooding from Lake Reeve is a mechanism for flooding of the township of Seaspray.



Figure 6-9 Flood Extent with water levels at Lake Reeve raised to 1.37 m AHD

7. DESIGN EVENT MODELLING

This section discusses the application of the hydraulic model to simulate and map flood behaviour (extents, depth, velocities) for a range of flood magnitudes.

Design flood modelling for the 10%, 5%, 2%, 1% and 0.5% AEP events and PMF event was completed using the approach discussed below.

The mapping outputs can be used for flood response planning, and land use planning purposes.

7.1 Approach

Based on the results of the calibration and sensitivity testing undertaken, the following approach was adopted for the design storm simulations.

7.1.1 Catchment Flooding

- The 10%, 5%, 2%, 1%, and 0.5% AEP Merriman Creek design flows were applied to the model at the Prospect Road Bridge.
- The lower roughness value was applied through the floodway to represent grassed channel conditions.
- An initial berm height of 2.10 m AHD.
- Within the simulation, the berm height reduces to 0.0 m AHD. The scouring commences when the water level reaches the maximum elevation of the berm and occurs over a 3 hour period. The entrance then remains open for the peak of design flood events as the sand transport modelling has shown rapid scouring of the berm.

- The 1% AEP storm tide has been applied for the tidal boundary with the peak water level coinciding with the peak of catchment flows.

7.1.2 Coastal Flooding

No further modelling of coastal flooding for the current sea level is required as the 1% AEP storm tide elevation has been applied to each of the design simulations.

7.1.3 Stormwater

- Stormwater modelling applied a direct rainfall on grid approach at a 1m grid resolution.
- The drainage network included in 1D all pipes that details were provided for which connect to one of the two pump stations.
- Each pump was assumed to be operating at full capacity
- ARR87 IFD and temporal pattern information were used to develop the design storms.
- Each AEP event was modelled with storm durations ranging from 10 minutes to 48 hours.
- Loss values as detailed in Section 3.6 were adopted.

The approach to incorporating the outputs of the stormwater simulations into the catchment flooding assessment are described further in Section 6.3.2.

7.2 Design Flood Behaviour

Design flood modelling for the 10%, 5%, 2%, 1% and 0.5% AEP events and PMF event has been completed with map outputs of flood levels, depths, velocities and extents provided in the hydraulic modelling deliverables. In addition, high flood hazard, LSIO and FO map outputs have been produced.

The 1% AEP design flood depth and velocity maps are provided in Figure 7-1 and Figure 7-2 respectively. The general flood behaviour is similar to the 1993 flood event with the overbank flow occurring at the junction of Merriman Creek and Blind Creek adjacent to Griffioens Levee. Breakouts from the floodway also occur, leading to flow over Government Road Levee from the floodway.

Localised rainfall causes water to pool over a number of properties, with around 195 houses within the flood extent, many between Main Rd and Catton St. Inundation of a large number of roads occurs, with deep water on Centre Rd, Catton St and Hansen St.

The removal of the earthen levee across the top of Blind Creek allows Blind Creek to function as overflow of Merriman Creek with water no longer carried west along Eastern Prior stream.

A detailed description of the flood behaviour for all events in Seaspray and the associated floodplain is provided in the Assess and Treat Risk Report (R04).



Figure 7-1 Design 1% Flood Depth Map



Figure 7-2 Design 1% AEP Peak Flood Velocity Map

8. CONCLUSIONS

Water Technology believes that the calibration of the Seaspray hydraulic model to available observations is good. Given the thorough sensitivity analysis of key complexities within and affecting the study area the model calibration is deemed fit for purpose and the model suitable for modelling design conditions.

On receiving approval from the West Gippsland CMA of the hydraulic model calibration and the proposed approach to undertaking the design events, Water Technology completed the remainder of the hydraulic modelling of the design flood events. Flood damage assessment and modelling of several mitigation options were then carried out. The results of these can be found in the Seaspray Flood Study – Assess and Treat Risk (R04).

9. REFERENCES

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Seaspray Flood Study – Assess and Treat Risk (R04)



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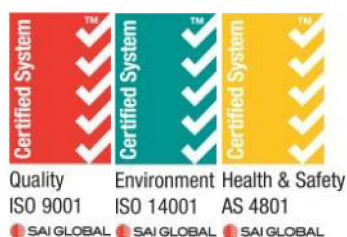
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Cover Photo: Seaspray Flooding 1993.

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1. INTRODUCTION

Following the flood events affecting Seaspray during September 1993, October 1995 and June 2012, Water Technology was commissioned by the West Gippsland CMA to undertake the Seaspray Flood Study. This study included detailed hydrologic and hydraulic modelling of the Merriman Creek estuary, including impacts from ocean conditions, water levels in Lake Reeve and stormwater run-off from Seaspray. A series of flood maps have been prepared for the town of Seaspray, including recommendations for flood mitigation works and review of flood warning and planning controls.

This report details the flood risk and assessment outcomes of the hydraulic analysis undertaken for the project, and should be read in conjunction with the previous reports:

- Data Review Report (R01)
- Hydrological Report (R02)
- Hydraulic Report (R03)

2. STUDY AREA

Seaspray is a coastal town with 320 permanent residents which rises to more than 1,000 during summer. The township lies at the western end of the Ninety Mile Beach west of the Gippsland Lakes and approximately 200 km east of Melbourne. As detailed in the Urban Design Framework developed for the township by Wellington Shire Council (Wellington Shire Council, 2007), Seaspray is an important access point to 90 Mile Beach, and the areas of small lot subdivision that extend easterly for 28 km to Golden Beach and Paradise Beach.

The town is located on the estuarine floodplain of the Merriman Creek catchment. Merriman creek flows along the western boundary of Seaspray before it passes through the 90 Mile Beach coastal dune system and enters Bass Strait. As part of the flood mitigation scheme completed in 1987, an engineered floodway running through the middle of the town, has the ability to distribute water from the Merriman Creek estuary to Lake Reeve in the east.

In this study area, flooding can occur as a result of a range of potential mechanisms, including;

- Overbank flows from Merriman Creek,
- Coastal inundation due to elevated ocean levels,
- Flooding from the Gippsland Lakes via Lake Reeve,
- Localised stormwater flooding.

Each of these flooding mechanisms and their interactions have been investigated as part of the hydraulic assessment and detailed within the Hydraulics Report (R03).



Figure 2-1 Seaspray Flood Study Model Area

3. FLOOD INTELLIGENCE

3.1 Overview

Hydraulic model results from this study has allowed for further flood intelligence to be added to the existing information.

Flood mitigation measures for the township of Seaspray were implemented in 1987 following the 1978 flood event and subsequent flood study. The works included construction of the floodway to Lake Reeve with a regulator structure at Merriman Creek, and the multiple earthen levees along the eastern edge of Merriman Creek and either side of the floodway. These measures have proven successful in limiting flood damage within Seaspray in subsequent flood events in 1995 and 2012. However, during the 1993 flood event flooding did occur in Seaspray. This occurred as a result of flow diversions associated with an unauthorised levee being constructed across the flow path down Blind Creek and limited culvert capacity under Seaspray Road. Modelling of the same event with the levee removed and culverts upgraded indicated that no floodwaters would have entered the township and the levees would not have been overtopped.

Despite the relative success of existing mitigation measures during recent flood events, concern still exists for future flooding of Seaspray given the very low topographical relief of the town. Also, there are currently no flood warning gauge levels for Seaspray despite the presence of stream flow gauges at Prospect Rd and further upstream at Stradbroke West. Recent historic flood events and design flood levels are listed in Table 3-1 with respect to the Prospect Road Gauge.

The gauge at Prospect Road (227240A) has now been relocated to the upstream 227272A Merriman Creek at Gippsland Water Offtake location. The monitoring of level and flow data will in future be based on the new monitoring location. However, it is still intended for both gauges along with high flow measuring to remain in both locations, so that a flow and level relationship can be developed between the two sites. At this stage there have been no significant high flows recorded at the new site to verify the top end of the rating table.

All gauge heights and flows presented in this report are relative to the 227240A gauge location. A period of data collection of high flow events will be required to develop a rating curve relationship between the 227240A and 227272A gauge locations. After a reliable relationship is established, all gauge height references will need to be updated to the new 227272A gauge.

Table 3-1 Prospect Road Stream Flow Gauge Heights and Levels

Flood Event (% AEP)	Gauge Height (m)	Water Surface Level (m AHD)
0.5	NA*	NA*
1	6.36	8.90
1995	5.64	8.18
2	5.57	8.11
1993	5.53	8.07
5	4.82	7.36
10	3.78	6.32
2012	3.18	5.72

* Beyond Gauge Rating Curve

Table 3-2 describes the flood consequences associated with different design events and the associated gauge level at Prospect Road. The table has been set out to be read from the top down to include all flood consequences from the frequent to infrequent flood events.

When using Table 3-2 to identify particular consequences for a given flood event, the reader should read all rows of consequences above the selected magnitude design event.

Table 3-2 Design Flood Events and Associated Flood Consequences

Design Flood Event (% AEP)	Gauge Height at Prospect Road (m)	Flood Consequences
10	3.78	River flooding confined to banks of Merriman Creek and Blind creek and the floodway toward Lake Reeve. There is some flow across the floodplains west of Merriman Creek at the estuary. There is some pooling of water throughout the town due to localised stormwater runoff with minor impacts on houses along Foreshore Rd and Rowley St. Depths of around 200mm on parts of Foreshore Rd and Futchers St. Total Properties Flooded = 56 Properties Flooded above floor level = 0
5	4.82	River flooding confined to banks of Merriman Creek and Blind creek and the floodway toward Lake Reeve. Some flow breakout to the north of the floodway, east of the town, into undeveloped farmland. Pooling of stormwater throughout Seaspray. Depths of around 200mm on Foreshore Rd and parts of Futchers St and Centre Rd. Around 25 houses are within the flood extent. Properties along Rowley St begin to become inundated. Total Properties Flooded = 57 Properties Flooded above floor level = 0
2	5.57	River flooding is mostly confined to banks of Merriman Creek and Blind Creek with some flow breaking the banks but remaining close to the main channels. Larger breakouts occurring to the east of the town, to the north of the floodway onto low lying farmland. Localised rainfall causing stormwater flooding within town. Around 45 houses within the flood extent. Properties along Shoreline Dr and Catton St begin to become impacted. Deeper flooding along Foreshore Rd, Futchers St and Centre Rd. Total Properties Flooded = 58 Properties Flooded above floor level = 0
1	6.36	River flows overtopping some banks of Merriman Creek and Blind Creek adjacent to Griffioens Levee. Breakouts from the floodway also occur, leading to flow over Government Road Levee from the floodway. Deep flows to the east of Seaspray flowing towards Lake Reeve. Overland flow to the west of the Merriman Creek towards the estuary. Localised pooling of stormwater runoff over a number of properties. Around 195 houses within the flood extent, many between Main Rd and Catton St. Inundation of a large number of roads, with deep water on Centre Rd, Catton St and Hansen St.

Design Flood Event (% AEP)	Gauge Height at Prospect Road (m)	Flood Consequences
		Total Properties Flooded = 177 Properties Flooded above floor level = 37
0.5	NA*	Breakout of flows in Merriman Creek north east of Buckleys Rd. Breakouts from Blind Creek and Merriman Creek adjacent to Griffioens Levee. Breakouts to the west of Merriman Creek moving westward. Seaspray Rd inundation, north of the town, resulting in potential isolation of the community. Shoreline Drive inundated between Seaspray and The Honeysuckles, causing potential isolation of the Honeysuckles. All houses within Seaspray township are within the flood extent except for several along Seaspray Rd, north of Tip Rd, and the south of Bearups St between Lyons St and Bock St. Total Properties Flooded = 322 Properties Flooded above floor level = 180

* Beyond Gauge Rating Curve

3.2 Flood Risk

Flood risk is the product of the consequence and the frequency of a flood event occurring. To assess the flood risk within Seaspray, the hydraulic modelling outputs (depth and velocity) can be used to identify both the frequency of flood impacts (in terms of annual exceedance probability) and the consequence of the magnitude of the flood impacts. The method used to delineate high flood hazard is broadly based on the new Australian Rainfall and Runoff Project 10 'Appropriate Safety Criteria for People'. Criterion for delineating the flood overlay considers both vehicle and people safety, and are as follows:

- Depth > 0.3 m
- Velocity > 1.5 m/s
- Depth x velocity > 0.3 m²/s.

If any of the above criteria is exceeded, there area is considered to be High Hazard. Low Hazard areas are locations within the flood extent, but do not exceed any of the above criteria. It is understood that the WGCMA favours this method as the board has approved development guidelines which adopt a depth of 0.30 m as the cut off for safety. The maximum flood hazard for the 1% AEP flood event is shown in Figure 3-1.

When considering the emergency response requirements for Seaspray during a flood event, it is important to consider the areas that are flooded first within the township and not only areas of high consequence (or high hazard). Areas first flooded are often areas that have a higher likelihood of flooding occurring and the flooding will occur at lower flow rates.

Areas that are considered High Hazard in more frequent events have a much higher flood risk. Figure 3-2 identifies areas of High Hazard in the more frequent 5% AEP flood event. High Hazard areas in the 5% AEP include Centre Road between Catton Street and Newton Street, and the intersection of Foreshore Road with Futcher Road. Both areas are impacted by localised stormwater runoff in relatively frequent flood events. Centre Road is of particular high Flood Risk, as it and Futcher Street are the only two access routes across the floodway to the southern region of Seaspray.

High Hazard areas within the 1% AEP flood extent may have a lower flood risk than High Hazard areas within the 5% AEP flood extent, due to the relevant flood mechanism. The 1% AEP flood High

Hazard area is largely due to the overbank breakout flows from Merriman Creek, which is much less likely to occur than stormwater flooding from localised rainfall events with a smaller AEP.



Figure 3-1 High and Low Hazard Areas in the 1% AEP flood event



Figure 3-2 High and Low Hazard Areas in the 5% AEP flood event

Even though High Hazard areas outside of the designated waterways are minimal for the 5% AEP flood event, it is also important to consider Low Hazard areas which are inundated frequently in smaller AEP events and the associated risks of regular nuisance flooding.

3.3 Flood Warning

3.3.1 Flood Event Timing

Two streamflow gauges located upstream of the township (Prospect Road and Stradbroke West) provide valuable information on the approaching streamflow towards Seaspray. This information can be used to provide a warning to an approaching flood to the Seaspray township. The time between peak flows at the two gauges can provide a good indication of the travel time and the available flood warning time, this is outlined in Table 3-3. However, the time between peak flows at the Stradbroke West gauge and Prospect Road gauge is highly variable and can range from 4 - 26 hours depending on the flood magnitude. The 1993 and 1995 flood events were significantly larger than the 2012 event and both have a flood peak travel time less than 10 hours. It appears that the larger the flood event the shorter the travel time.

While Table 3-3 shows the time between peak flows at the streamflow gauges, flood impacts in Seaspray can occur well before the flood peak arrives at the Prospect Road gauge. Localised rainfall can lead to stormwater flooding within Seaspray independently of any catchment flooding in Merriman Creek. High Risk areas, including Centre Road between Catton Street and Newton Street can be inundated from local rainfall, well before the flood peak from the catchment reaches Seaspray. The source of flooding is important to consider for emergency response procedures such as road closures, sandbagging and if required, evacuation.

Table 3-3 Historical Travel Time between Flood Peaks (based on gauge flows)

Historical Event	Time from Peak at Stradbroke West to Peak at Prospect Road
1993	~ 4 Hours
1995	~ 9 Hours
2012	~ 26 Hours

While the streamflow gauges at Stradbroke West and Prospect Road can provide up to 4 hours of warning time of an approaching flood, analysing the rainfall falling throughout the catchment can also provide a rough estimate of the expected flood magnitude. Rainfall totals also provide longer warning time in the range of 24-36 hours once heavy rain has started falling in the catchment. Antecedent conditions throughout the catchment will play a major role in how much of the rainfall fallen in the catchment is converted to runoff. Historical rainfall totals and flood events along with design rainfall totals and design flood magnitudes are shown in Table 3-4. These tables show the large difference between the rainfall totals in the upper and lower catchments that have been recorded in historical flood events.

Across the catchment, the rainfall totals change considerably from the lower area around Seaspray itself having far lower rainfall totals compared to the upper catchment totals around Mt Tassie. Seaspray is effectively in the rainfall shadow of the Strzelecki Ranges. It is useful to note that while using the rainfall totals to estimate a flood event magnitude, there are a number of factors that should be considered including where the rainfall fell within the catchment and the antecedent conditions of the catchment.

Table 3-4 Merriman Creek catchment rainfall and Design Flood Magnitude

Flood Event (% AEP)	Prospect Rd Gauge Height (m)	Rainfall Total (mm)			Rainfall Duration (Hours)
		Seaspray (Burong)	Stradbroke West	Mt Tassie	
1	6.36	134	193	295	48
1995	5.64	89	NA*	394	72
2	5.57	141	149	233	36
1993	5.53	111	NA*	267	40
5	4.82	125	136	210	48
10	3.78	105	113	178	48
2012	3.18	42	101	204	24

*Stradbroke West rainfall gauge was not installed

3.3.2 Flood Warning Levels

There are currently no flood warning gauge levels for Seaspray despite the presence of stream flow gauges at Prospect Rd and further upstream at Stradbroke West. Based on the information presented in Table 3-2 and the historical gauge record in Figure 3-3, recommendations are provided in Table 3-5 to establish flood warning levels at the Prospect Road gauge.

These levels seek to balance the need to ensure that flood warnings are implemented with reasonable consideration of frequency and impacts, and to provide valid warning time to allow preparation for an estimated flood peak. It is important to note that whilst no flood impact in Seaspray occurs due to flooding from Merriman Creek in the 2% AEP flood event, with a gauge height of 5.57m, and that the 1995 flood did not impact Seaspray at a gauge height 5.64m, a lower major flood warning level should provide adequate time to enact flood preparations.

Table 3-5 Proposed Flood Warning Levels

Flood Warning Level	Prospect Rd Height (m)
Minor	2.0
Moderate	3.0
Major	4.5

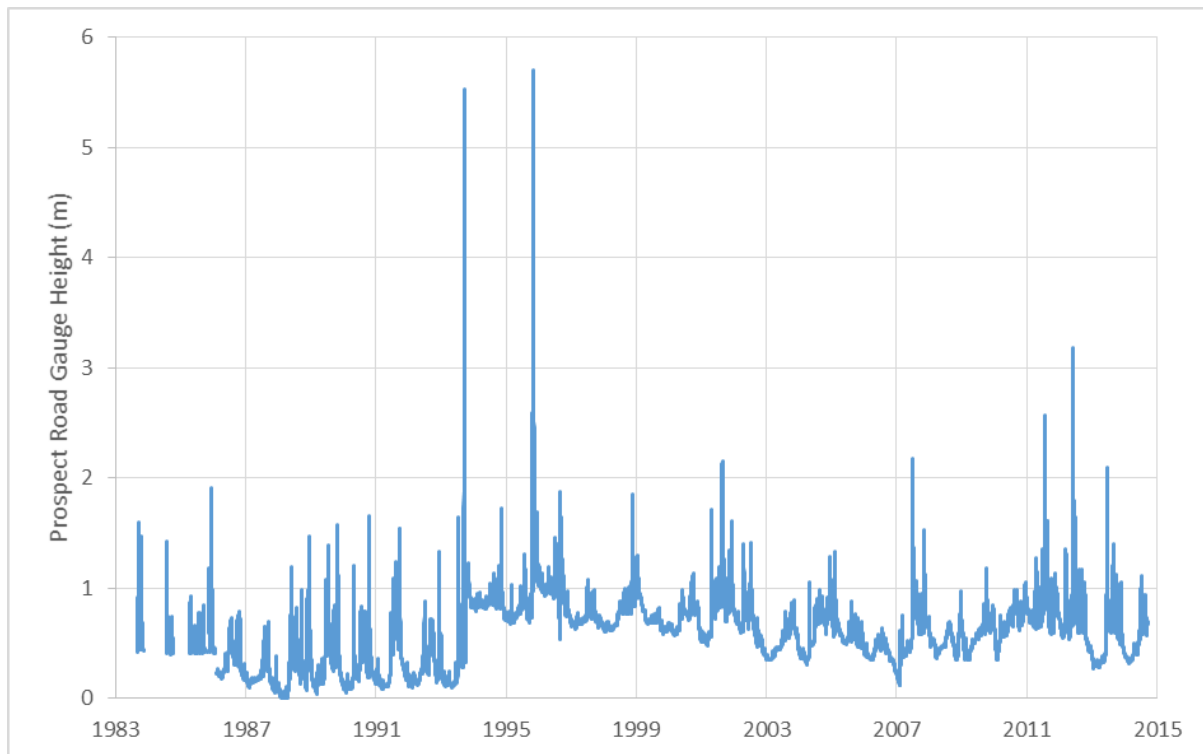


Figure 3-3 Prospect Road Gauge Height – Historical Record

3.4 Flood Management Scheme Operation

The existing operating procedure for of the Seaspray flood mitigation scheme, including the floodway regulator and manual opening of the estuary entrance is detailed in the operating manual maintained by Wellington Shire Council (Wellington Shire Council, 1996).

3.4.1 Regulator Structure

The intention of the regulator structure is to provide flood protection from both riverine and coastal flooding. In practical terms, if the sand berm is closed the regulator should maintain a pool level at 2.0 m AHD to 2.2 m AHD, which will preferentially direct river flows down the floodway to limit flows overtopping the berm and scouring an entrance channel. This therefore would prevent ingress of water from the ocean during extreme storm tide events. However, the analysis of coastal flooding reported in the Hydraulics Report (R03) has concluded that under existing conditions, there is no risk posed to Seaspray from coastal flooding as the 1% AEP storm tide elevation at Seaspray is 1.5 m AHD under existing mean sea level. The crest elevation of the existing levees along Merriman Creek are above this level and would prevent inundation for storm tides, regardless of the operation of the regulator structure.

The existing operating procedure (Wellington Shire Council, 1996) recommends that the regulator structure is to have boards incrementally added manually throughout the duration of a riverine flood event to limit and control flow down the floodway. In practice this has proven dangerous and impractical, as reported in the failed attempts to place flood boards during the 1993 flood event. Maintaining all boards in place within the regulator structure does however force flood water towards the estuary entrance and encourages earlier natural scouring of the entrance berm if the entrance channel is initially closed.

It is therefore recommended that in future all boards remain in place to provide protection against riverine flooding, as there is no flood benefit to removing them during an event.

3.4.2 Entrance Opening

Estuary closure as a result of sedimentation of the entrance and creation of a sand berm is a continued concern for local residents, with pooling of water within the estuary a perceived flood risk if riverine flooding is to occur. Results of sensitivity analysis within the Hydraulics Report (R03) showed that once overtopped, the sand berm will naturally erode within 1-3 hours through natural erosion processes.

However, to address local concern and to manage entrance opening conditions a safe operating height of 2.1 m AHD is proposed as a maximum berm level crest for the sand berm to open naturally without negative flood impacts in Seaspray. The proposed maximum height of 2.1 m AHD allows 100 mm of freeboard before the earthen weir adjacent the regulator structure is overtopped and water is diverted down the floodway. The height of 2.1 m AHD also provides 400 mm of freeboard to the crests of the levees along Merriman Creek. If the entrance sand berm is greater than 2.1 m AHD, manual opening of the entrance is recommended if weather warnings indicate significant rainfall is likely to occur in the catchment. This differs from the existing operating procedure for the structure as detailed in the operating manual. It is therefore recommended that the operating manual be updated to reflect this revised information.

There is currently no accurate measure of the berm height, although the water level behind the berm is recorded by the gauge at Seaspray. A water level gauge board located at the boat ramp or on the berm would allow Council, WGCMA and the local community to have up to date information on the sand levels and provide a trigger for implementing potential artificial openings of the entrance prior to predicted flood events.

3.4.3 Floodway Maintenance

Results of sensitivity analysis conducted for the Hydraulic Report (R03) showed the flood mitigation benefits associated with limiting the build-up of established vegetation or reeds within the floodway. Under a conservative modelling approach, the sensitivity results suggest that water levels could be increased by 100mm – 150mm on either side of the floodway if vegetation roughness (associated with extensive growth of tall reeds) is increased. It is therefore recommended that vegetation in the floodway is maintained as grass only.

3.5 Land Use Planning

The Victoria Planning Provisions (VPPs) contain a number of controls that can be employed to provide guidance for the use and development of land that is affected by inundation from floodwaters. These controls include the Floodway Overlay (FO), the Land Subject to Inundation Overlay (LSIO), the Special Building Overlay (SBO), the Urban Floodway Zone (UFZ) and the Environmental Significance Overlay (ESO).

Section 6(e) of the Planning and Environment Act 1987 enables planning schemes to ‘regulate or prohibit any use or development in hazardous areas, or likely to be hazardous’. As a result, planning schemes contain State planning policy for floodplain management requiring, among other things, that flood risk be considered in the preparation of planning schemes and in land use decisions.

Guidance for applying flood controls to Planning Schemes is available from the Department of Planning and Community Development’s (DPCD) Practice Note on Applying Flood Controls in Planning Schemes.

Planning Schemes can be viewed online at <http://services.land.vic.gov.au/maps/pmo.jsp>. It is recommended that the planning scheme for Seaspray be amended to reflect the flood risk identified by this project.

The method used to delineate the proposed FO is broadly based on the new Australian Rainfall and Runoff Project 10 'Appropriate Safety Criteria for People'. Criterion for delineating the flood overlay considers both vehicle and people safety, and are as follows, based on the 1% AEP flood:

- Depth > 0.3 m
- Velocity > 1.5 m/s
- Depth x velocity > 0.3 m²/s.

It is understood that the WGCMA favours this method has and has approved development guidelines which adopt 0.30 m as the cut off flood level for safety considerations.

The Land Subject to Inundation (LSIO) extent has been taken as the full 1% AEP flood extent minus small disconnected puddles such as farm dams. Figure 3-4 shows the proposed FO and LSIO maps based on the Australian Rainfall and Runoff Revision Project 10 methodology.



Figure 3-4 Draft LSI0 and FO Map for Existing Conditions

4. MITIGATION

Three structural mitigation options as identified in Figure 4-1 were presented to the steering committee and community meeting held in September 2015. The three options are listed below, and are described in further detail in the following sections.

1. **Increase Conveyance of Eastern Prior Stream** – It was proposed to assess whether deepening of Eastern Prior Stream to the invert of the culvert at Seaspray Road would allow water to flow along Eastern Prior Stream and reconnect with Lake Reeve. This aims to reduce the flood flows directly towards Seaspray.
2. **Increase flood storage along Mason Creek** – It was proposed that additional storage of flood waters could be achieved by utilising the natural depression along Mason Creek towards Lake Denison. The aim was to reduce flood levels within the Merriman Creek estuary and consequently overtopping of the adjacent levees.
3. **Increase Existing Flood Levee Function** - The existing network of levees constructed around Seaspray as part of the flood mitigation scheme have ensured that Seaspray was not impacted during the 1995 or 2012 floods. Design modelling of the 1% AEP flood event, does however, show that the existing levee scheme remains vulnerable to overtopping and breakout flows during a 1% AEP event. Raising of levees in specific locations could be undertaken to eliminate this overtopping.

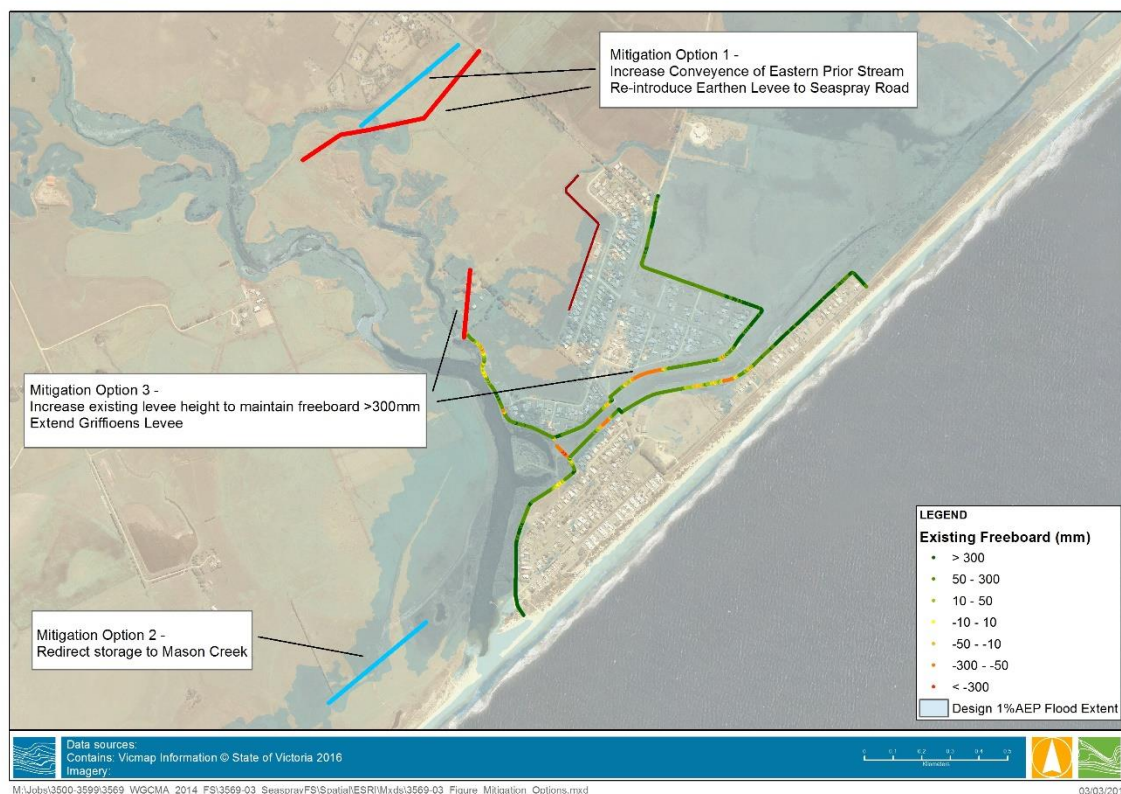


Figure 4-1 Proposed Mitigation Options

At Seaspray the operation of the floodway regulator structure and entrance opening are also important tools for managing flood risk. These aspects were discussed in the preceding section.

4.1 Prefeasibility Assessment

Each mitigation option was assessed against a number of criteria: potential reduction in flood damage, cost of construction, feasibility of construction, and environmental impact. The score for each criterion was based on a ranking system of 1 to 5, with 1 being the worst score and 5 the best. Each criteria score was then weighted according to the weighting shown in Table 4-1. The reduction in flood damage was the most heavily weighted criteria as this is really the main objective for all flood mitigation. Table 4-2 reviews and scores each mitigation option against the four criteria and calculates a total score for each option. The options with the higher scores indicate the more appropriate mitigation solutions for Seaspray. While these options were reviewed and scored individually it is important to consider a combination of options when developing a flood mitigation scheme.

Using the prefeasibility assessment above, the 3 identified mitigation options are listed in order of total weighted score as seen in Table 4-2.

Table 4-1 Ranking score for mitigation criteria

Score	Reduction in Flood Damages	Cost (\$)	Feasibility/Constructability	Environmental Impact
Weighting	2	1	0.5	0.5
5	Major reduction in flood damage	Less than \$ 50,000	Excellent (Ease of construction and/or highly feasible option)	None
4	Moderate reduction in flood damage	\$ 50,000 – \$ 100,000	Good	Minor
3	Minor reduction in flood damage	\$ 100,000 – \$ 500,000	Average	Some
2	No reduction in flood damage	\$ 500,000 – \$ 1,000,000	Below Average	Major
1	Increase in flood damage	Greater than \$ 1,000,000	Poor (No access to site and/or highly unfeasible option)	Extreme

Table 4-2 Mitigation option prefeasibility results

No.	Mitigation Option	Criteria				
		Damage Reduction	Cost	Feasibility/Constructability	Environmental Impact	Score
1	Existing Levee Improvements	5	3	3	5	17
2	Mason Creek Storage	2	2	3	4	9.5
3	Eastern Prior Stream Conveyance	2	1	2	3	7.5

4.2 Structural Mitigation Options Modelled

Following the prefeasibility assessment, preliminary modelling was undertaken to analyse the potential for the three proposed structural mitigation options to have an impact on flood levels and extents and assess the potential costs and benefits for each option. All mitigation modelling has been undertaken with an initial entrance berm height of 2.1 m AHD and ponding of water in the estuary to this level. It is assumed that the entrance channel will form through the berm through natural scouring processes (described in detail in Report 3) over a period of 3 hours once the water level in the estuary exceeds 2.1 m AHD.

The mitigation works were costed based on a number of key references:

- Melbourne Water's standard rates for earthworks and pipe/headwall construction costs.
- Rawlinsons Australian Construction Handbook Rates (Rawlinsons, 2011)
- The United Kingdom Environmental Agency Temporary and Demountable Flood Protection Guide (EA UK, 2011)
- Comparison to cost estimates for similar mitigation works for other flood studies

4.2.1 Increase Conveyance of Eastern Prior Stream

Following the 1993 flood event, the existing culverts on Eastern Prior stream under Seaspray Road were upgraded to a bank of 5 rectangular box culverts with dimensions of 2 m wide and 1.5 m high and an upstream invert level of 4.02 m AHD. At the same time, earthen levees preventing flow from Eastern Prior Stream into Blind Creek during the 1993 flood event were removed. Design modelling of the 1% AEP flood event shows that flood water under existing conditions flows down Blind Creek with no overland flow continuing along Eastern Prior stream to Seaspray Road. The floodwater must pond to approximately 5.5 m AHD for flow to continue along Eastern Prior Stream to Seaspray Road. In comparison to the entrance of Blind Creek is about 3.8 m AHD, which therefore provides a preferential flow path.

Proposed deepening of Eastern Prior stream to the inverts of the culvert at Seaspray Road (4.02 m AHD) allows water to flow along Eastern Prior Stream and reconnect with Lake Reeve, bypassing Seaspray. Model results of this scenario as shown in Figure 4-2 and confirms that the formally dry Eastern Prior stream is now able to re-direct flow around Seaspray. However, the overall impact of flood levels is negligible. Despite the lower bed level of Eastern Prior stream, flood waters still preferentially flow down Blind Creek.

Interestingly, if the earthen levees present during the 1993 flood event are reinstated and extended to Seaspray Road, this would preclude water from flowing down Blind Creek and greatly increase the conveyance of Eastern Prior stream.

Under this scenario, the resulting flood impact map in Figure 4-3 shows the reduction of flows down Blind Creek reduces overbank flows at the confluence of Blind Creek and Merriman Creek adjacent to Griffioens Levee. Water surface levels are therefore reduced in Seaspray, along Merriman Creek and the floodway.

This does however change the flooding mechanisms in and around Seaspray. Flow from Eastern Prior stream now directly enters low lying land to the east of Seaspray and the Eastern Levee is vulnerable to overtopping. Under this mitigation scenario, areas of Seaspray bordering Merriman Creek and the floodway experience slightly reduced flooding, however there is a much greater risk of levee failure on the eastern side of Seaspray. This mitigation option is therefore not considered a preferred option as it shifts the location of levee vulnerability rather than resolving the issue.

4.2.2 Mason Creek Storage

The proposed increase in water storage in the natural depression along Mason Creek towards Lake Denison has been modelled to assess its effectiveness to relieve flood levels within the Merriman Creek Estuary and consequently overbank flow into Seaspray.

The model results showed that improving the hydraulic connection to the natural depression to the west of Merriman Creek did increase the capacity for water storage in this area. However, the additional storage is not sufficient to reduce water levels within Seaspray and there was only a marginal reduction in peak flood levels within the estuary itself. As such there was no tangible benefit from this mitigation option.



Figure 4-2 Flood Impact of Deepening Easter Prior Stream

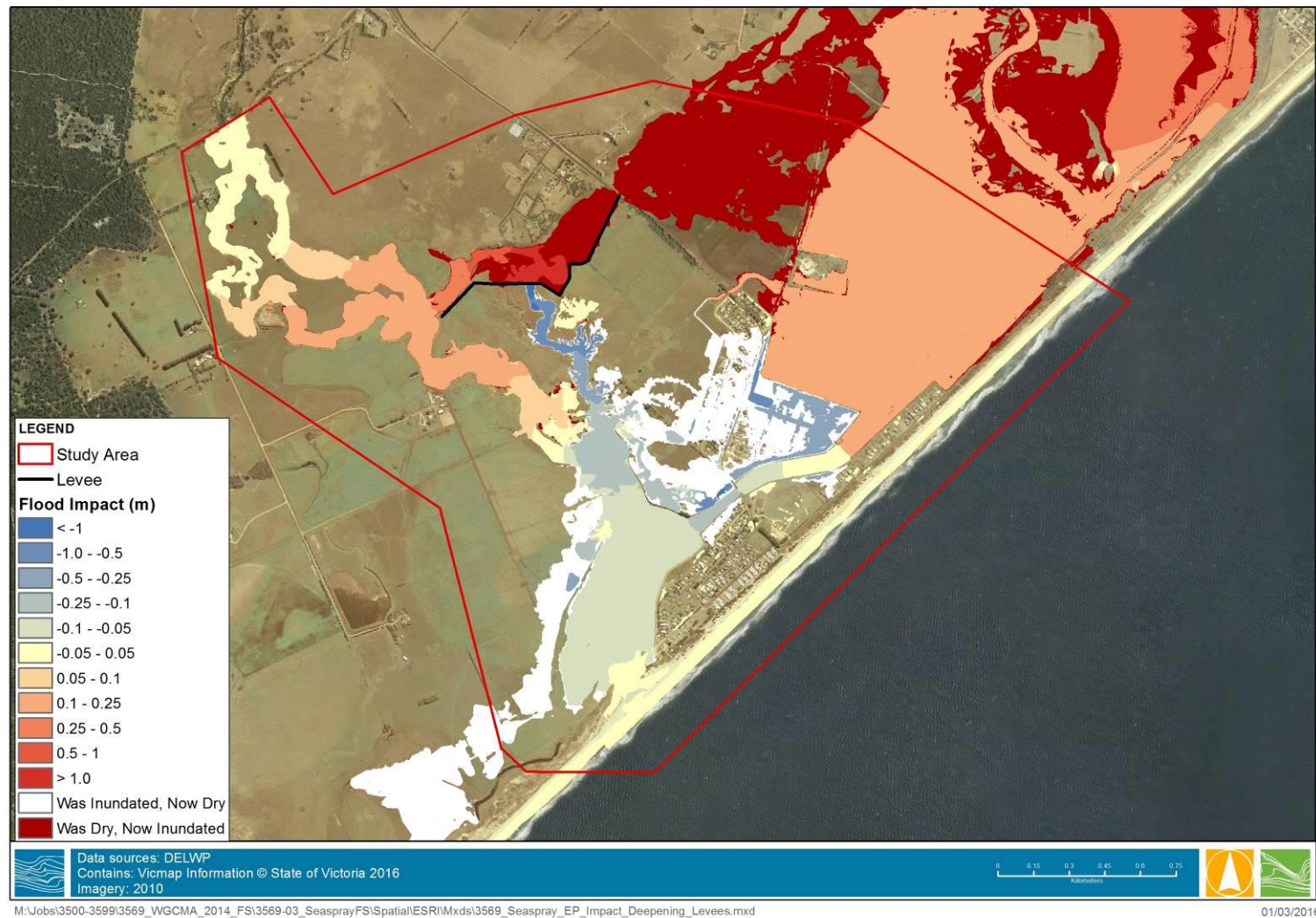


Figure 4-3 Flood Impact of Deepening Easter Prior Stream and reinstating levee across Blind Creek



Figure 4-4 Flood Impact of Increased Water Storage in Mason Creek, towards Lake Denison

4.2.3 Increased Flood Levee Function

The existing network of levees constructed around Seaspray have ensured the town has not been impacted during the 1995 or 2012 floods. Design modelling of the 1% AEP flood event, does however, show that the existing levee scheme remains vulnerable to overtopping and breakout flows during a 1% AEP event.

For this assessment, all existing levees crest levels were elevated well above any potential flood level which has allowed identification of the integrity of the existing flood levees under 1% AEP flood event conditions. Figure 4-5 displays sections of the levee that would be overtopped under existing conditions with a 1% AEP flow if the levees are not raised. Thematic colours represent the existing crest level of each levee with respect to 1% AEP flood level. Whilst much of the levee system shows good capacity with a freeboard of greater than 50mm above modelled flood levels, several sections have crest levels lower than required levels. This assessment allows a targeted approach to raise underperforming sections of the levee and improve the levee network to mitigate flood levels in the 1% AEP flood event.

Figure 4-6 provides a longitudinal profile of Griffioens levee along the eastern edge of Merriman Creek, Government Road levee along the northern side of the floodway, and the eastern levee of Seaspray. The profile confirms that sections of the existing levee, most notably between Futcher Street and Centre Road, are lower than the design 1% AEP water surface level.



Figure 4-5 Vulnerability of existing levees in the 1% AEP flood event

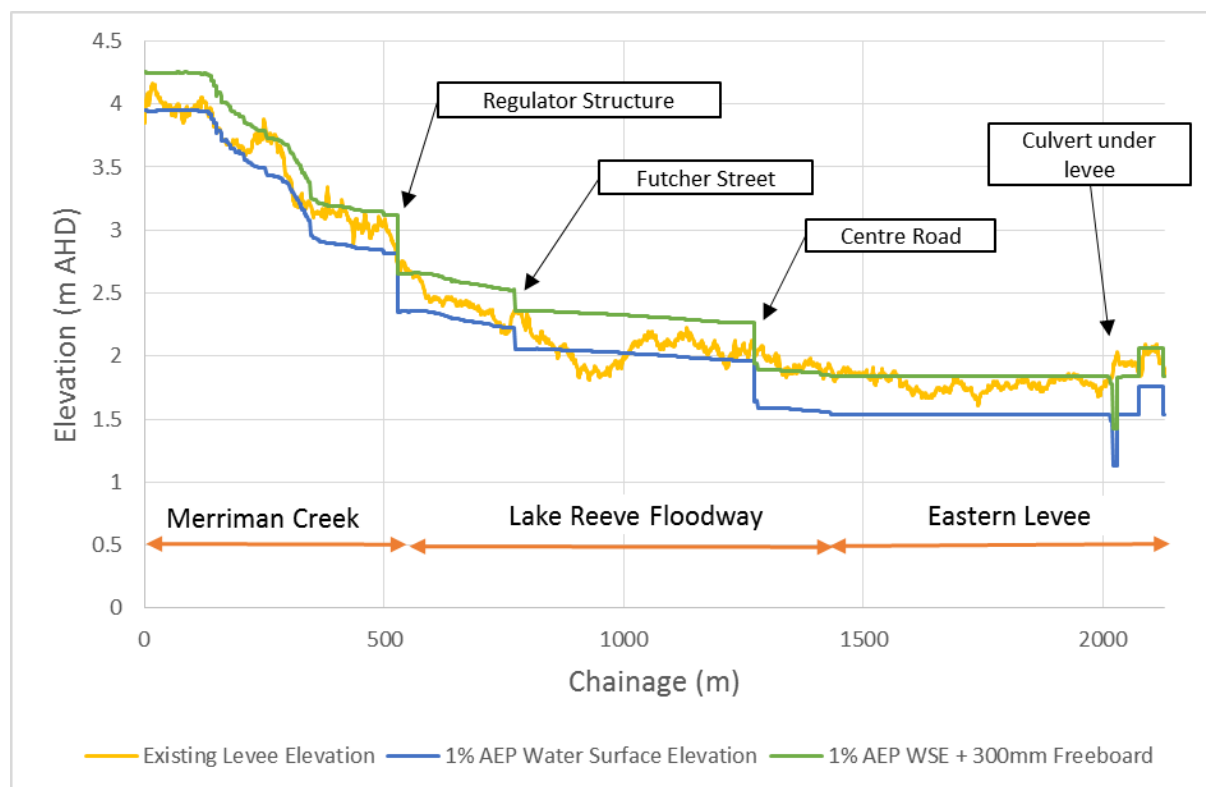


Figure 4-6 Longitudinal Profile of existing Griffioens Levee and Government Road Levee

In addition to the existing levee upgrade, an extension of Griffioens Levee would be required to prevent water from outflanking the existing levee. A conceptual alignment has been assessed, however this is not to be considered a final design layout. The raising of existing levees in required locations, and the addition of the Griffioens Levee extension, has limited flood impacts within Seaspray to those properties affected by localised rainfall and stormwater flooding as showing in Figure 4-7.

Initial cost estimations for an upgraded levee scheme are summarised in Table 4-3 .

Table 4-3 Levee Upgrade and Extension Costs

1. Government Road Levee				Construction Cost	Annual Maintenance
Description	Qty	Unit	Rate		
Construction and Compaction	529	m ³	22.4	\$11,869.16	
Topsoiling	529	m ³	19.1	\$10,086.67	
Grassing	2,193	m ²	1.1	\$2,456.46	
SUB-TOTAL				\$24,412.30	\$732.37
Engineering Fee	1		15%	\$3,661.84	
Administration Fee	1		9%	\$2,197.11	
Contingencies	1		30%	\$9,081.37	
TOTAL				\$39,352.62	\$732.37

2. Southern Levee				Construction Cost	Annual Maintenance
Description	Qty	Unit	Rate		
Construction and Compaction	395	m ³	22.4	\$8,873.38	
Topsoiling	395	m ³	19.1	\$7,540.79	
Grassing	2,651	m ²	1.1	\$2,969.35	
SUB-TOTAL				\$19,383.53	\$581.51
Engineering Fee	1		15%	\$2,907.53	
Administration Fee	1		9%	\$1,744.52	
Contingencies	1		30%	\$7,210.67	
TOTAL				\$31,246.24	\$581.51

3. Griffieons Levee				Construction Cost	Annual Maintenance
Description	Qty	Unit	Rate		
Construction and Compaction	190	m ³	22.4	\$4,256.63	
Topsoiling	190	m ³	19.1	\$3,617.38	
Grassing	271	m ²	1.1	\$303.30	
SUB-TOTAL				\$8,177.32	\$245.32
Engineering Fee	1		15%	\$1,226.60	
Administration Fee	1		9%	\$735.96	
Contingencies	1		30%	\$3,041.96	
TOTAL				\$13,181.83	\$245.32



Figure 4-7 Flood Impact of raising all levees about 1% AEP flood level

5. FLOOD DAMAGES

5.1 Overview

A flood damages assessment was undertaken for the study area under existing conditions. The flood assessment determined the monetary flood damages for design floods (10%, 5%, 2%, 1% and 0.5% AEP events). The flood damage assessment was also undertaken for the final mitigation package.

Water Technology has developed an industry best practice damage assessment methodology that has been utilised for a number of studies in Victoria, combining aspects of the Rapid Appraisal Method, ANUFLOOD, more recent damage curves from the NSW Office of Environment and Heritage and other relevant flood damage literature. The model results for all mapped flood events were processed to calculate the numbers and locations of properties affected. This included properties with buildings inundated above floor, properties with buildings inundated below floor and properties where the building was not impacted but the grounds of the property were. In addition to the flood affected properties, lengths of flood affected roads for each event were also calculated. Note, that rural agricultural damages have not been included in this study as the focus for mitigation is on the township.

5.2 Existing Conditions

A flood damages assessment was undertaken on existing conditions and is shown in Table 5-1. The 1% AEP damage calculated was **\$2,259,374** with 36 residential properties flooded above floor and 1 commercial property also flooded above floor. The average annual damage (AAD), a measure of the flood damage, per year over an extended period was estimated for existing conditions to be **\$46,124**. The AAD is an estimate of the cost of flooding to the community that includes both public and privately owned assets.

Table 5-1 Existing Conditions Flood Damages Summary

EXISTING CONDITIONS					
ARI (years)	200yr	100yr	50yr	20yr	10yr
AEP	0.005	0.01	0.02	0.05	0.1
Residential Buildings Flooded Above Floor	166	36	0	0	0
Commercial Buildings Flooded Above Floor	14	1	0	0	0
Properties Flooded Below Floor	142	177	58	57	56
Total Properties Flooded	322	214	58	57	56
Direct Potential External Damage Cost	\$1,218,495	\$738,020	\$60,090	\$48,791	\$48,733
Direct Potential Residential Damage Cost	\$9,992,403	\$1,997,493	\$0	\$0	\$0
Direct Potential Commercial Damage Cost	\$170,813	\$3,779	\$0	\$0	\$0
Total Direct Potential Damage Cost	\$11,381,711	\$2,739,292	\$60,090	\$48,791	\$48,733
Total Actual Damage Cost (0.8*Potential)	\$9,105,369	\$2,191,434	\$48,072	\$39,033	\$38,986
Infrastructure Damage Cost	\$160,563	\$67,940	\$35,906	\$32,534	\$19,876
Indirect Clean Up Cost					
Indirect Residential Relocation Cost					
Indirect Emergency Response Cost					
Total Indirect Cost	\$0	\$0	\$0	\$0	\$0
Total Cost	\$9,265,932	\$2,259,374	\$83,978	\$71,567	\$58,862

5.3 Levee Improvement Mitigation

Damages were costed for mitigation Option 3, the upgrade of the existing levee mitigation scheme. This option aims to prevent flooding of Seaspray under the 1% AEP design flow from Merriman

Creek. It does not address local stormwater flooding. The results (Table 5-2) showed that the number of residential properties flooded above floor in a 1% AEP event are reduced to 4, and no commercial properties are now flooded above floor level. The flooding of these properties is limited to localised rainfall and stormwater flooding. This reduced the flood damage estimate for a 1% AEP flood event to **\$429,895**. The AAD was also reduced to **\$33,413**.

Table 5-2 Levee Mitigation Flood Damages Summary

MITIGATION CONDITIONS					
ARI (years)	200yr	100yr	50yr	20yr	10yr
AEP	0.005	0.01	0.02	0.05	0.1
Residential Buildings Flooded Above Floor	158	4	0	0	0
Commercial Buildings Flooded Above Floor	1	0	0	0	0
Properties Flooded Below Floor	125	212	58	57	56
Total Properties Flooded	284	216	58	57	56
Direct Potential External Damage Cost	\$462,331	\$324,188	\$60,090	\$48,791	\$48,733
Direct Potential Residential Damage Cost	\$11,480,148	\$168,532	\$0	\$0	\$0
Direct Potential Commercial Damage Cost	\$19,781	\$0	\$0	\$0	\$0
Total Direct Potential Damage Cost	\$11,962,260	\$492,720	\$60,090	\$48,791	\$48,733
Total Actual Damage Cost (0.8*Potential)	\$9,569,808	\$394,176	\$48,072	\$39,033	\$38,986
Infrastructure Damage Cost	\$100,203	\$35,719	\$35,906	\$32,534	\$19,876
Indirect Clean Up Cost					
Indirect Residential Relocation Cost					
Indirect Emergency Response Cost					
Total Indirect Cost	\$0	\$0	\$0	\$0	\$0
Total Cost	\$9,670,011	\$429,895	\$83,978	\$71,567	\$58,862

6. BENEFIT COST ANALYSIS

Table 6-1 Benefit-Cost Analysis of Levee Mitigation Option

Item	Existing Conditions	Levee Mitigation Option
Annual Average Damages	\$46,124	\$33,413
Annual Maintenance Cost*		\$1,559
Annual Cost Saving		\$11,152
Net Present Value (6%)		\$156,821
Capital Cost of Mitigation		\$83,781
Benefit-Cost Ratio		1.87

* It should be noted that any maintenance cost associated with the proposed mitigation works are only in relation to the proposed additional works and do not consider the existing ongoing cost of maintenance to the existing structure.

The levee improvement mitigation option shows a positive benefit to cost ratio due to the relatively limited extent of levee raising required and associated costs compared to the reduction of flood impacted properties in the 1% AEP flood event. The magnitude of the 0.5% AEP flood event is such that raising the existing levees is not able to prevent Seaspray from becoming inundated. Overbank flow from Blind Creek continues to outflank the existing levee arrangement with breakout flows occurring all the way along Blind Creek to Eastern Prior stream.

6.1 Non-Economic Flood Damages

The previous discussion relating to flood damages has concentrated on monetary damages, that is, damages that are easily quantified. In addition to those damages, it is widely recognised that individuals and communities also suffer significant non-monetary damage, i.e. emotional distress, health issues, etc. There has been extensive research undertaken and documented in the scientific literature relating to the individuals and communities response to natural disasters. A recent publication entitled *“Understanding floods: Questions and Answers”* by the Queensland Floods Science Engineering and Technology Panel, when discussing the large social consequences floods have on individuals and communities states:

Floods can also traumatise victims and their families for long periods of time. The loss of loved ones has deep impacts, especially on children. Displacement from one’s home, loss of property and disruption to business and social affairs can cause continuing stress. For some people the psychological impacts can be long lasting.

The *“Disaster Loss Assessment Guidelines”* (EMA, 2002) make the following key points:

- *Intangibles are often found to be more important than tangible losses.*
- *Most research shows that people value the intangible losses from a flooded home—principally loss of memorabilia, stress and resultant ill-health—as at least as great as their tangible dollar losses.*
- *There are no agreed methods for valuing these losses.*

There is no doubt that the intangible non-monetary flood related damage in Seaspray is high. The benefit-cost analysis presented later in this report has not considered this cost. Nor has the perceived reduction in property value as a result of land not currently zoned within the flood overlay being placed within the updated flood overlay if no mitigation option is implemented. The cost benefit analysis has also not taken into account the possibility for land currently zoned within the flood overlay or land subject to inundation which may have the flood risk significantly reduced as a result of the mitigation options discussed. Any decisions made that are based on the benefit-cost ratios need to understand that the true cost of floods in Seaspray is far higher than the economic damages alone. This would have the effect of increasing the benefit cost ratio, improving the argument for approving a mitigation scheme at Seaspray.

7. CONCLUSION AND RECOMMENDATIONS

The hydraulic modelling undertaken for the Seaspray flood study has allowed for locations within Seaspray that pose a high flood risk to be identified. It has been shown that at Seaspray the operation of the floodway regulator structure and entrance opening are important tools for managing flood risk.

The modelling has also identified a number of potential structural mitigation options to reduce flood risk, although only one of the structural mitigation options shows significant benefits to flood risk reduction in the township. The updating of the floodway and entrance channel operating procedure, and the potential targeted increased in flood levee heights, along with the updated planning information aim to treat the existing risk within Seaspray.

It is recommended that the WGCMA and Wellington Shire Council develop an entrance opening protocol for Merriman Creek as part of updating the operating procedure for the floodway regulator structure. This should address the opening of the entrance during flood events and clearly outline responsibilities including safety and timing of any artificial opening works. There is also currently no accurate measure of the berm height, although the water level behind the berm is recorded by the

gauge at Seaspray. A water level gauge board located at the boat ramp or on the berm would allow Council, WGCMA and the local community to have up to date information on the sand levels and provide a trigger for implementing potential artificial openings of the entrance prior to predicted flood events.

To implement the targeted levee raising option at Seaspray detailed design of the levee structures is required which may require geotechnical investigations .

Implementation of the planning scheme amendments will rely upon Wellington Shire Council who have been part of the steering committee throughout the project.

Through the series of steering committee and community meetings, it was identified that accessing information about an approaching flood event was often difficult. Currently the Bureau of Meteorology (BoM) provide flood warnings via the BoM website, while streamflow data is also available through a different section of the website as well as through the DELWP data monitoring website.

It was noted that having a community portal that incorporated several key pieces of information regarding flooding specific to Seaspray in one place may reduce some of the confusion about where this information can be obtained. Allowing community members to get important information will likely raise the resilience of locals to potential flooding issues. This information may include any warnings issued by the BoM, the two streamflow gauges on Merriman Creek and a radar image of the area to show if there is more rain approaching. Regular monitoring of the berm height and height of the water in the estuary could be incorporated. Flood extent maps which identify the houses impacted during a major flood event could also be included within a community portal.

In terms of structural mitigation options, it was found that targeted raising of levees in and around Seaspray has the potential to significant reduce flood damage and costs for the 1% AEP event and have a favourable benefit to cost ratio.

8. REFERENCES

DELWP (2015), *Victorian Water Monitoring web portal*, Department of Environment, Land, Water and Planning, accessed April 29, 2015.

EA (2014). *Australian Rainfall and Runoff Discussion Paper: An Interim Guideline for considering Climate Change in Rainfall and Runoff*. Engineers Australia

HydroTechnology (1995), *Documentation and review of 1993 Victorian Floods - Gippsland Floods September 1993 Volume 1 and 2*

Rawlinsons (2011) Australian Construction Handbook Rates

The United Kingdom Environmental Agency Temporary and Demountable Flood Protection Guide (EA UK, 2011)

Wellington Shire Council (1996). Seaspray Flood Mitigation Scheme – Operating Procedures, Amended October 1996, Wellington Municipal Emergency Management Plan

APPENDIX A – PROPERTIES INUNDATED

Easting	Northing	Floor Level	Ground Level	Building Type	Address	Description	0.5% AEP Depth Above Floor	1% AEP Depth Above Floor	2% AEP Depth Above Floor	5% AEP Depth Above Floor	10% AEP Depth Above Floor
516520.7	5753105	3.465	2.053	RES	43 MAIN RD		-1.253	-1.481	N/A	N/A	N/A
516612.8	5752872	3.34	1.429	RES	10 NEWTON ST		-1.21	-1.508	-1.878	-1.878	-1.878
516507.2	5753380	3.142	2.485	RES	77 MAIN RD		-0.699	-0.699	-0.699	-0.699	-0.699
516546.3	5753336	2.976	2.157	RES	71 MAIN RD		-0.667	N/A	N/A	N/A	N/A
516226.9	5752190	2.973	2.079	RES	44 BUCKLEY ST		-0.572	N/A	N/A	N/A	N/A
516095.8	5752668	3.615	2.708	RES	36-38 GOVERNMENT RD		-0.538	-0.818	N/A	N/A	N/A
516264	5752624	3.261	2.354	RES	27 GOVERNMENT RD		-0.513	-0.803	N/A	N/A	N/A
516600.6	5753335	2.451	1.933	RES	68 MAIN RD		-0.455	N/A	N/A	N/A	N/A
516499	5752473	2.807	1.935	RES	6 BUCKLEY ST	STOOP	-0.452	N/A	N/A	N/A	N/A
516476.3	5752521	2.913	1.976	RES	5 IRVING ST		-0.443	N/A	N/A	N/A	N/A
516563.3	5753217	2.576	1.758	RES	57 MAIN RD		-0.437	-0.67	N/A	N/A	N/A
516781.8	5752955	2.478	1.405	RES	20 HANSEN ST		-0.435	-0.653	N/A	N/A	N/A
516373.7	5752413	2.912	1.797	RES	23 IRVING ST		-0.423	-0.998	N/A	N/A	N/A
516359.9	5752826	3.14	2.597	RES	4 DAVIES ST		-0.418	N/A	N/A	N/A	N/A
516448.5	5753335	2.948	2.409	RES	38 ELLEN AVE		-0.41	N/A	N/A	N/A	N/A
516449.8	5752245	2.709	1.843	RES	23 FORESHORE RD		-0.382	N/A	N/A	N/A	N/A

516304.6	5752263	2.78	1.672	RES	34 BUCKLEY ST	STOOP	-0.378	N/A	N/A	N/A	N/A
516431.4	5753152	2.91	2.114	RES	1-9 ELLEN AVE		-0.362	-0.785	N/A	N/A	N/A
516516.1	5753077	2.605	2.024	RES	39 MAIN RD		-0.361	-0.57	N/A	N/A	N/A
516432.5	5752241	2.658	2.353	RES	25 FORESHORE RD	GRANNY FLAT	-0.326	N/A	N/A	N/A	N/A
517222.3	5752919	2.283	1.781	RES	43 SHORELINE DR		-0.322	N/A	N/A	N/A	N/A
516321.6	5752819	3.097	2.68	RES	3 DAVIES ST		-0.316	N/A	N/A	N/A	N/A
517167	5752868	2.277	1.469	RES	39 SHORELINE DR		-0.303	N/A	N/A	N/A	N/A
516259.2	5752591	3.061	2.325	RES	27 GOVERNMENT RD	GRANNY FLAT	-0.298	-0.564	N/A	N/A	N/A
516547.1	5752336	2.588	1.701	RES	11 FORESHORE RD		-0.294	N/A	N/A	N/A	N/A
516347	5752887	3.047	2.031	RES	11 DAVIES ST		-0.293	-0.674	N/A	N/A	N/A
516354.4	5752213	2.687	1.905	RES	32 BEARUP ST		-0.286	N/A	N/A	N/A	N/A
516517.5	5752958	2.52	1.888	RES	24 MAIN RD		-0.285	N/A	N/A	N/A	N/A
517030.5	5752742	2.366	1.525	RES	21 SHORELINE DR		-0.284	N/A	N/A	N/A	N/A
516458.9	5753327	2.812	2.368	RES	36 ELLEN AVE		-0.28	N/A	N/A	N/A	N/A
516512.5	5752494	2.611	1.595	RES	4 BUCKLEY ST	STOOP	-0.267	-0.861	-0.861	-0.861	-0.861
516520	5752310	2.572	1.775	RES	15 FORESHORE RD		-0.266	N/A	N/A	N/A	N/A
516521.7	5752778	2.523	1.798	RES	3 HANSEN ST		-0.26	N/A	N/A	N/A	N/A
516403.1	5752763	2.884	2.088	RES	1 MAIN RD		-0.245	-0.502	N/A	N/A	N/A
516441.1	5752305	2.601	2.149	RES	20 BEARUP ST		-0.241	N/A	N/A	N/A	N/A
516415.1	5752390	2.664	2.033	RES	18 BUCKLEY ST	STOOP	-0.24	N/A	N/A	N/A	N/A
516339.5	5752868	2.984	2.193	RES	9 DAVIES ST	STOOP	-0.228	-0.614	N/A	N/A	N/A

517343.4	5753034	2.152	1.65	RES	57-59 SHORELINE DR		-0.221	N/A	N/A	N/A	N/A
516549.8	5752966	2.317	1.67	RES	2 CENTRE RD		-0.22	-0.488	N/A	N/A	N/A
516420.9	5753007	2.8	2.024	RES	28 DAVIES ST		-0.215	-0.581	N/A	N/A	N/A
516488.3	5752898	2.567	1.797	RES	18 MAIN RD		-0.214	-0.684	N/A	N/A	N/A
516508.1	5753064	2.499	1.871	RES	37 MAIN RD		-0.213	-0.452	N/A	N/A	N/A
516861.8	5752905	2.264	1.248	RES	23 CENTRE RD		-0.212	-0.433	N/A	N/A	N/A
516778.7	5752861	2.302	1.263	RES	18 CENTRE RD	MAIN DWELLING	-0.211	-0.469	N/A	N/A	N/A
517287.8	5752979	2.148	1.705	RES	51 SHORELINE DR		-0.21	N/A	N/A	N/A	N/A
516874.3	5752950	2.227	1.188	RES	3 CATTON ST		-0.205	-0.399	N/A	N/A	N/A
516464.7	5753239	2.735	2.035	RES	15 ELLEN AVE		-0.201	-0.66	N/A	N/A	N/A
516562.7	5752801	2.411	1.642	RES	1 NEWTON ST		-0.196	-0.554	N/A	N/A	N/A
516187.8	5752043	2.589	2.19	RES	11 TROOD ST		-0.19	N/A	N/A	N/A	N/A
516255.1	5752082	2.584	1.721	RES	48 BEARUP ST		-0.189	-0.698	-0.698	-0.698	-0.698
516474.3	5753166	2.698	1.98	RES	14 ELLEN AVE		-0.184	-0.621	N/A	N/A	N/A
516454.4	5752393	2.567	1.92	RES	13 BUCKLEY ST		-0.182	N/A	N/A	N/A	N/A
517049.2	5752752	2.242	1.59	RES	23 SHORELINE DR		-0.181	N/A	N/A	N/A	N/A
516368.3	5752158	2.551	2.187	RES	35 FORESHORE RD		-0.172	N/A	N/A	N/A	N/A
516556.9	5753321	2.455	1.922	RES	69 MAIN RD		-0.171	N/A	N/A	N/A	N/A
516242.1	5752200	2.57	1.962	RES	42 BUCKLEY ST		-0.169	N/A	N/A	N/A	N/A
516271.1	5752545	2.867	2.389	RES	7 ROWLEY ST		-0.159	-0.478	N/A	N/A	N/A
516609	5752848	2.307	1.45	RES	8 NEWTON ST		-0.158	-0.474	N/A	N/A	N/A

516709.6	5752856	2.258	1.381	RES	19 HANSEN ST		-0.154	-0.424	N/A	N/A	N/A
516538.9	5753152	2.308	1.868	RES	49 MAIN RD		-0.148	-0.428	N/A	N/A	N/A
516180.6	5752118	2.548	1.812	RES	1 TROOD ST		-0.147	N/A	N/A	N/A	N/A
516368.1	5752784	2.834	2.642	RES	2 DAVIES ST		-0.143	N/A	N/A	N/A	N/A
516218.8	5752622	3.016	2.348	RES	31-33 GOVERNMENT RD		-0.14	-0.363	N/A	N/A	N/A
516385	5752184	2.511	2.258	RES	31 FORESHORE RD		-0.139	N/A	N/A	N/A	N/A
516452	5752316	2.486	2.329	RES	18 BEARUP ST		-0.138	N/A	N/A	N/A	N/A
517210	5752916	2.103	1.571	RES	41 SHORELINE DR		-0.137	N/A	N/A	N/A	N/A
516527.8	5753124	2.31	1.906	RES	45 MAIN RD		-0.133	-0.412	N/A	N/A	N/A
516464.6	5752250	2.456	1.812	RES	21 FORESHORE RD		-0.132	N/A	N/A	N/A	N/A
516284.7	5752107	2.523	2.072	RES	44 BEARUP ST		-0.129	N/A	N/A	N/A	N/A
516375	5752690	2.814	2.119	RES	6 GOVERNMENT RD		-0.127	-0.433	N/A	N/A	N/A
516564.6	5753308	2.379	1.806	RES	67 MAIN RD		-0.124	-0.47	N/A	N/A	N/A
516385.9	5752853	2.802	2.219	RES	10 DAVIES ST		-0.12	-0.465	N/A	N/A	N/A
517239.7	5752937	2.069	1.578	RES	45 SHORELINE DR		-0.118	N/A	N/A	N/A	N/A
516208.3	5752161	2.514	1.895	RES	48 BUCKLEY ST		-0.113	N/A	N/A	N/A	N/A
516374.2	5752174	2.478	1.952	RES	33 FORESHORE RD		-0.101	N/A	N/A	N/A	N/A
516344.2	5752202	2.501	1.796	RES	34 BEARUP ST		-0.1	-0.542	-0.542	-0.542	-0.542
516715.9	5752899	2.18	1.328	RES	14 CENTRE RD		-0.098	-0.349	N/A	N/A	N/A
516200.5	5752627	3.027	2.53	RES	35 GOVERNMENT RD		-0.097	-0.305	N/A	N/A	N/A

516563.1	5753296	2.338	1.787	RES	65 MAIN RD		-0.095	-0.43	N/A	N/A	N/A
516482.7	5753033	2.498	1.999	RES	33 MAIN RD		-0.092	-0.339	N/A	N/A	N/A
516484.6	5753300	2.615	2.065	RES	32 ELLEN AVE		-0.091	N/A	N/A	N/A	N/A
516544.5	5753176	2.261	1.661	RES	51 MAIN RD		-0.089	-0.367	N/A	N/A	N/A
516380.4	5752705	2.773	2.199	RES	4 GOVERNMENT RD		-0.087	-0.391	N/A	N/A	N/A
516898.3	5752899	2.124	1.24	RES	2 CATTON ST		-0.083	-0.293	N/A	N/A	N/A
516177.6	5752628	3.086	2.603	RES	37-39 GOVERNMENT RD		-0.081	-0.326	N/A	N/A	N/A
517270.5	5752965	2.024	1.679	RES	49 SHORELINE DR		-0.08	N/A	N/A	N/A	N/A
516429.1	5753024	2.628	2.043	RES	30 DAVIES ST		-0.078	-0.428	N/A	N/A	N/A
517299.4	5752996	2.012	1.6	RES	53 SHORELINE DR		-0.077	N/A	N/A	N/A	N/A
516382.3	5752889	2.769	2.151	RES	12 DAVIES ST		-0.076	-0.436	N/A	N/A	N/A
516762.7	5753002	2.091	1.302	RES	25 HANSEN ST	STOOP	-0.074	-0.271	N/A	N/A	N/A
516445.5	5753076	2.612	2.155	RES	2 ELLEN AVE		-0.073	-0.431	N/A	N/A	N/A
516180.1	5752020	2.47	1.885	RES	13 TROOD ST		-0.073	N/A	N/A	N/A	N/A
516491.3	5752778	2.352	2.182	RES	1 HANSEN ST		-0.07	N/A	N/A	N/A	N/A
516668.7	5752794	2.217	1.612	RES	11-13 HANSEN ST		-0.068	-0.37	N/A	N/A	N/A
516357.2	5752839	2.795	2.35	RES	6 DAVIES ST		-0.064	-0.438	N/A	N/A	N/A
516950.9	5752679	2.227	1.417	RES	15 SHORELINE DR		-0.063	-0.848	-0.848	-0.849	-0.849
516390.7	5753003	2.717	2.068	RES	25 DAVIES ST		-0.06	-0.462	N/A	N/A	N/A
516289.2	5752251	2.462	1.802	RES	36 BUCKLEY ST		-0.059	N/A	N/A	N/A	N/A

516679.7	5752948	2.134	1.299	RES	11 CENTRE RD		-0.057	-0.305	N/A	N/A	N/A
516967.7	5752695	2.208	1.568	RES	17 SHORELINE DR		-0.057	N/A	N/A	N/A	N/A
516560.4	5752342	2.341	1.678	RES	9 FORESHORE RD		-0.053	N/A	N/A	N/A	N/A
516402.7	5753048	2.666	2.064	RES	31 DAVIES ST		-0.051	-0.437	N/A	N/A	N/A
516510.6	5753265	2.514	1.987	RES	26 ELLEN AVE		-0.048	-0.448	N/A	N/A	N/A
516572.5	5752357	2.326	1.636	RES	7 FORESHORE RD		-0.045	N/A	N/A	N/A	N/A
516233.5	5752068	2.433	1.82	RES	50 BEARUP ST		-0.038	N/A	N/A	N/A	N/A
517250.5	5752948	1.979	1.598	RES	47 SHORELINE DR		-0.031	N/A	N/A	N/A	N/A
516599.2	5752898	2.147	1.464	RES	11 NEWTON ST		-0.03	-0.317	N/A	N/A	N/A
516237.8	5752565	2.971	2.432	RES	11 ROWLEY ST		-0.028	-0.225	N/A	N/A	N/A
516531.1	5752514	2.336	1.694	RES	2 BUCKLEY ST	STOOP	-0.023	N/A	N/A	N/A	N/A
516441.8	5752378	2.416	2.004	RES	17 BUCKLEY ST	STOOP	-0.017	N/A	N/A	N/A	N/A
516402.3	5752947	2.668	1.922	RES	20 DAVIES ST		-0.017	-0.388	N/A	N/A	N/A
516501.7	5753042	2.3	1.97	RES	35 MAIN RD		-0.015	-0.211	N/A	N/A	N/A
516626.4	5752798	2.185	1.681	RES	9 HANSEN ST		-0.007	-0.319	N/A	N/A	N/A
516450.3	5752418	2.411	1.926	RES	14 BUCKLEY ST		-0.003	N/A	N/A	N/A	N/A
516188.3	5752064	2.401	1.944	RES	7 TROOD ST		-0.001	N/A	N/A	N/A	N/A
516850.1	5752849	2.09	1.098	RES	24 CENTRE RD		-0.001	-0.256	N/A	N/A	N/A
516303.3	5752138	2.387	1.966	RES	39 FORESHORE RD		0.007	N/A	N/A	N/A	N/A
516418.5	5752474	2.496	1.639	RES	15 IRVING ST		0.009	-0.582	N/A	N/A	N/A
516981.4	5752714	2.121	1.692	RES	19 SHORELINE DR		0.01	-0.742	-0.742	-0.743	-0.743

516511.9	5752921	2.268	1.737	RES	22 MAIN RD		0.011	-0.401	N/A	N/A	N/A
516887.5	5752969	1.994	1.291	RES	5 CATTON ST		0.013	-0.166	N/A	N/A	N/A
516293.2	5752124	2.377	1.962	RES	42 BEARUP ST		0.017	N/A	N/A	N/A	N/A
516486.4	5752531	2.445	2.053	RES	3 IRVING ST		0.017	N/A	N/A	N/A	N/A
516459.7	5752809	2.427	2.471	RES	10 MAIN RD		0.018	N/A	N/A	N/A	N/A
516504.5	5753203	2.419	2.077	RES	20 ELLEN AVE		0.021	-0.392	N/A	N/A	N/A
516466.4	5752835	2.393	2.188	RES	12 MAIN RD		0.022	N/A	N/A	N/A	N/A
516553.9	5753196	2.126	1.671	RES	55 MAIN RD		0.025	-0.225	N/A	N/A	N/A
516188.4	5752081	2.372	1.929	RES	5 TROOD ST		0.028	N/A	N/A	N/A	N/A
516736.9	5752828	2.078	1.436	RES	16 HANSEN ST		0.034	-0.242	N/A	N/A	N/A
516415	5752792	2.523	1.986	RES	5 MAIN RD		0.037	-0.141	-0.283	-0.283	-0.283
516736.6	5752844	2.065	1.32	RES	18 HANSEN ST		0.039	-0.23	N/A	N/A	N/A
516492.7	5753295	2.474	2.037	RES	30 ELLEN AVE	STOOP	0.041	N/A	N/A	N/A	N/A
516290.6	5752618	2.677	2.148	RES	25 GOVERNMENT RD		0.043	-0.262	N/A	N/A	N/A
516703.4	5752912	2.036	1.21	RES	12 CENTRE RD		0.044	-0.206	N/A	N/A	N/A
516412.5	5753083	2.514	2.098	RES	35 DAVIES ST		0.051	-0.29	N/A	N/A	N/A
516251.4	5752626	2.72	2.291	RES	29 GOVERNMENT RD		0.051	-0.235	N/A	N/A	N/A
516480.9	5752347	2.273	1.839	RES	14 BEARUP ST		0.054	N/A	N/A	N/A	N/A
516459.8	5752922	2.377	1.672	RES	21 MAIN RD		0.061	-0.13	N/A	N/A	N/A
516500.9	5752910	2.254	1.88	RES	20 MAIN RD		0.07	N/A	N/A	N/A	N/A
516321.4	5752613	2.617	2.174	RES	21 GOVERNMENT RD		0.071	-0.233	N/A	N/A	N/A

516474.1	5752412	2.298	1.848	RES	9 BUCKLEY ST	STOOP	0.073	N/A	N/A	N/A	N/A
516310.6	5752615	2.619	2.048	RES	23 GOVERNMENT RD		0.077	-0.231	N/A	N/A	N/A
516674.9	5752920	2.001	1.235	RES	10 CENTRE RD		0.081	-0.171	N/A	N/A	N/A
516537.9	5752767	2.172	1.925	RES	5 HANSEN ST		0.082	N/A	N/A	N/A	N/A
516451.1	5753095	2.446	2.208	RES	4 ELLEN AVE	STOOP	0.087	-0.278	N/A	N/A	N/A
516574.6	5753254	2.066	1.576	RES	61 MAIN RD		0.093	-0.157	N/A	N/A	N/A
516258.5	5752101	2.3	2.236	RES	46 BEARUP ST		0.096	N/A	N/A	N/A	N/A
516533.2	5753357	2.303	2.444	RES	73 MAIN RD		0.096	N/A	N/A	N/A	N/A
516187.7	5752051	2.301	2.198	RES	9 TROOD ST		0.098	N/A	N/A	N/A	N/A
516133	5752610	2.948	2.13	RES	43-45 GOVERNMENT RD	STOOP	0.099	-0.178	N/A	N/A	N/A
516597.2	5752778	2.108	1.803	RES	7 HANSEN ST	STOOP	0.104	-0.222	N/A	N/A	N/A
516335.6	5752611	2.573	1.952	RES	19 GOVERNMENT RD		0.105	-0.192	N/A	N/A	N/A
516468.1	5753319	2.421	2.431	RES	34 ELLEN AVE		0.107	N/A	N/A	N/A	N/A
516466.6	5752302	2.216	2.024	RES	18 BOCK ST		0.107	N/A	N/A	N/A	N/A
516524.7	5752393	2.181	1.911	RES	8 BEARUP ST		0.113	N/A	N/A	N/A	N/A
516435.2	5752231	2.217	1.865	RES	25 FORESHORE RD		0.117	N/A	N/A	N/A	N/A
516402.3	5752960	2.537	2.085	RES	22 DAVIES ST		0.122	-0.261	N/A	N/A	N/A
516482	5753182	2.378	2.064	RES	16 ELLEN AVE		0.125	-0.304	N/A	N/A	N/A
516775.5	5752938	1.932	1.359	RES	17 CENTRE RD		0.126	-0.104	N/A	N/A	N/A
516493.8	5752360	2.196	1.868	RES	12 BEARUP ST		0.126	N/A	N/A	N/A	N/A

516443.2	5752885	2.378	1.551	RES	17 MAIN RD		0.127	-0.098	N/A	N/A	N/A
516369	5752598	2.508	1.96	RES	15 GOVERNMENT RD		0.13	-0.131	N/A	N/A	N/A
516407.2	5753063	2.464	2.195	RES	33 DAVIES ST		0.132	-0.254	N/A	N/A	N/A
516370.7	5752964	2.568	2.104	RES	21 DAVIES ST		0.133	-0.281	N/A	N/A	N/A
516213.4	5752049	2.258	2.092	RES	49 FORESHORE RD		0.138	N/A	N/A	N/A	N/A
516390.3	5753012	2.51	2.081	RES	27 DAVIES ST		0.141	-0.263	N/A	N/A	N/A
517130.7	5752839	1.849	1.546	RES	35 SHORELINE DR		0.146	N/A	N/A	N/A	N/A
516383.6	5752423	2.338	1.808	RES	21 IRVING ST		0.153	-0.424	N/A	N/A	N/A
516495	5753224	2.351	2.106	RES	22 ELLEN AVE		0.157	N/A	N/A	N/A	N/A
516508.6	5752376	2.153	1.802	RES	10 BEARUP ST		0.158	N/A	N/A	N/A	N/A
516471.6	5752441	2.226	1.816	RES	10 BUCKLEY ST	STOOP	0.16	N/A	N/A	N/A	N/A
516371.9	5752953	2.537	2.081	RES	19 DAVIES ST		0.162	-0.239	N/A	N/A	N/A
516931.3	5752656	2.011	1.461	RES	11 SHORELINE DR		0.165	N/A	N/A	N/A	N/A
516445.4	5752488	2.323	1.997	RES	11 IRVING ST		0.17	N/A	N/A	N/A	N/A
516672	5753023	1.89	1.523	RES	16 NEWTON ST		0.17	-0.064	N/A	N/A	N/A
516424.3	5752820	2.38	2.042	RES	9 MAIN RD		0.173	N/A	N/A	N/A	N/A
516406.5	5752668	2.452	2.112	RES	5 GOVERNMENT RD		0.173	-0.082	N/A	N/A	N/A
516411.4	5752681	2.44	2.104	RES	3 GOVERNMENT RD		0.177	-0.072	N/A	N/A	N/A
516488	5752457	2.174	1.797	RES	8 BUCKLEY ST	STOOP	0.187	N/A	N/A	N/A	N/A
516434.9	5752477	2.308	2.114	RES	13 IRVING ST		0.187	N/A	N/A	N/A	N/A
516458.2	5753117	2.339	2.06	RES	8 ELLEN AVE		0.192	-0.226	N/A	N/A	N/A

516389.3	5752918	2.488	2.09	RES	16 DAVIES ST		0.192	-0.173	N/A	N/A	N/A
517062.1	5752769	1.847	1.369	RES	25 SHORELINE DR		0.193	N/A	N/A	N/A	N/A
516333.9	5752549	2.446	1.876	RES	1 ROWLEY ST		0.196	-0.067	N/A	N/A	N/A
516367.3	5752865	2.522	2.11	RES	8 DAVIES ST		0.2	-0.169	N/A	N/A	N/A
516392.9	5753031	2.444	2.049	RES	29 DAVIES ST		0.202	-0.21	N/A	N/A	N/A
517114.2	5752814	1.806	1.335	RES	31 SHORELINE DR		0.206	N/A	N/A	N/A	N/A
516470.8	5752403	2.16	1.801	RES	11 BUCKLEY ST	STOOP	0.207	N/A	N/A	N/A	N/A
516332.2	5752189	2.188	1.813	RES	36 BEARUP ST		0.212	N/A	N/A	N/A	N/A
516362.1	5752931	2.515	2.179	RES	17 DAVIES ST		0.212	-0.174	N/A	N/A	N/A
517078	5752783	1.815	1.434	RES	27 SHORELINE DR		0.214	N/A	N/A	N/A	N/A
516384.2	5752900	2.478	2.107	RES	14 DAVIES ST		0.214	-0.149	N/A	N/A	N/A
516407.4	5752448	2.276	1.864	RES	17 IRVING ST		0.218	-0.362	N/A	N/A	N/A
517129.5	5752826	1.78	1.547	RES	33 SHORELINE DR		0.219	N/A	N/A	N/A	N/A
516529.5	5752467	2.083	1.483	RES	3 BEARUP ST	STOOP	0.22	N/A	N/A	N/A	N/A
516473.7	5753152	2.3	2.179	RES	12 ELLEN AVE		0.221	N/A	N/A	N/A	N/A
516435.1	5753041	2.306	2.006	RES	32 DAVIES ST		0.224	-0.117	N/A	N/A	N/A
516460.1	5752428	2.174	1.87	RES	12 BUCKLEY ST	STOOP	0.224	N/A	N/A	N/A	N/A
516465.8	5752510	2.257	1.977	RES	7 IRVING ST		0.226	N/A	N/A	N/A	N/A
516401.4	5752653	2.4	1.982	RES	7 GOVERNMENT RD		0.226	-0.029	N/A	N/A	N/A
516591.7	5752876	1.908	1.472	RES	9 NEWTON ST		0.231	-0.076	-0.446	-0.446	-0.446
516574.6	5752953	1.859	1.533	RES	4 CENTRE RD		0.233	-0.03	N/A	N/A	N/A

516455.9	5752499	2.253	2.114	RES	9 IRVING ST		0.236	N/A	N/A	N/A	N/A
516480.6	5752981	2.114	1.717	RES	29 MAIN RD		0.237	0.057	N/A	N/A	N/A
516477.2	5753005	2.157	1.798	RES	31 MAIN RD		0.239	0.007	N/A	N/A	N/A
516944.4	5752951	1.763	1.166	RES	8 CATTON ST		0.242	0.067	N/A	N/A	N/A
516298.3	5752157	2.155	1.984	RES	40 BEARUP ST		0.244	N/A	N/A	N/A	N/A
516455.4	5753111	2.289	2.141	RES	6 ELLEN AVE		0.244	-0.149	N/A	N/A	N/A
516647.3	5752970	1.832	1.324	RES	9 CENTRE RD		0.246	-0.003	N/A	N/A	N/A
517092.6	5752798	1.771	1.408	RES	29 SHORELINE DR		0.247	N/A	N/A	N/A	N/A
516501.5	5753246	2.24	1.953	RES	24 ELLEN AVE		0.249	-0.187	N/A	N/A	N/A
516483.2	5752276	2.071	1.743	RES	19 FORESHORE RD		0.249	N/A	N/A	N/A	N/A
516533.6	5752323	2.051	1.619	RES	13 FORESHORE RD		0.25	N/A	N/A	N/A	N/A
516467.9	5753126	2.26	2.002	RES	10 ELLEN AVE		0.25	-0.156	N/A	N/A	N/A
516509.9	5753214	2.179	2.095	RES	22 ELLEN AVE	GRANNY FLAT	0.254	N/A	N/A	N/A	N/A
516523.8	5753149	1.982	1.811	RES	47 MAIN RD		0.256	-0.104	N/A	N/A	N/A
516736.1	5752931	1.82	1.384	RES	15 CENTRE RD		0.256	0.01	N/A	N/A	N/A
516568.5	5753272	1.931	1.697	RES	63 MAIN RD		0.258	-0.023	N/A	N/A	N/A
516481.3	5752853	2.102	1.87	RES	14 MAIN RD		0.26	N/A	N/A	N/A	N/A
516187.7	5752149	2.14	1.796	RES	50 BUCKLEY ST		0.261	N/A	N/A	N/A	N/A
516587.6	5752866	1.882	1.496	RES	7 NEWTON ST		0.269	-0.05	N/A	N/A	N/A
516368.6	5752672	2.416	2.138	RES	8 GOVERNMENT RD		0.271	-0.034	N/A	N/A	N/A
516333	5752848	2.492	2.262	RES	7 DAVIES ST		0.272	-0.116	N/A	N/A	N/A

516354.7	5752605	2.386	1.904	RES	17 GOVERNMENT RD		0.277	-0.006	N/A	N/A	N/A
516330.4	5752299	2.126	1.613	RES	30 BUCKLEY ST	STOOP	0.279	N/A	N/A	N/A	N/A
516508.2	5752299	2.028	1.733	RES	17 FORESHORE RD		0.283	N/A	N/A	N/A	N/A
516922.3	5752935	1.728	1.321	RES	6 CATTON ST		0.286	0.102	N/A	N/A	N/A
516261.9	5752228	2.111	1.752	RES	40 BUCKLEY ST	STOOP	0.291	N/A	N/A	N/A	N/A
516213	5752176	2.109	1.737	RES	46 BUCKLEY ST		0.292	N/A	N/A	N/A	N/A
516275	5752241	2.11	1.803	RES	38 BUCKLEY ST	STOOP	0.292	N/A	N/A	N/A	N/A
516602.2	5752941	1.788	1.513	RES	6 CENTRE RD		0.297	0.041	N/A	N/A	N/A
517311.4	5753003	1.635	1.522	RES	55 SHORELINE DR		0.298	N/A	N/A	N/A	N/A
516412.7	5752992	2.329	2.123	RES	24 DAVIES ST		0.299	-0.071	N/A	N/A	N/A
516493.9	5752852	2.03	1.797	RES	14 MAIN RD	GRANNY FLAT	0.3	N/A	N/A	N/A	N/A
516471.9	5752960	2.082	1.786	RES	27 MAIN RD		0.313	0.118	N/A	N/A	N/A
516382.7	5752346	2.087	1.697	RES	22 BUCKLEY ST	STOOP	0.318	N/A	N/A	N/A	N/A
516412.4	5752775	2.267	2.15	RES	3 MAIN RD		0.318	0.115	-0.123	-0.123	-0.123
516188.4	5752102	2.078	1.826	RES	3 TROOD ST		0.322	N/A	N/A	N/A	N/A
516935.7	5752669	1.846	1.487	RES	13 SHORELINE DR		0.326	N/A	N/A	N/A	N/A
516394	5752367	2.077	1.914	RES	20 BUCKLEY ST	STOOP	0.339	N/A	N/A	N/A	N/A
516607.1	5752834	1.815	1.515	RES	6 NEWTON ST	STOOP	0.343	0.019	N/A	N/A	N/A
517158.1	5752855	1.628	1.738	RES	37 SHORELINE DR		0.353	N/A	N/A	N/A	N/A
516318.6	5752175	2.043	2.006	RES	38 BEARUP ST		0.355	N/A	N/A	N/A	N/A
516469.7	5752335	1.961	1.753	RES	16 BEARUP ST		0.369	N/A	N/A	N/A	N/A

516419.3	5752808	2.177	2.145	RES	7 MAIN RD		0.385	0.205	N/A	N/A	N/A
516396.5	5752640	2.248	1.836	RES	9 GOVERNMENT RD		0.387	0.125	N/A	N/A	N/A
516234	5752621	2.436	2.349	RES	31-33 GOVERNMENT RD	GRANNY FLAT	0.389	0.142	N/A	N/A	N/A
516703.2	5752839	1.712	1.351	RES	17 HANSEN ST		0.401	0.124	N/A	N/A	N/A
516915.6	5752916	1.623	1.221	RES	4 CATTON ST		0.402	0.207	N/A	N/A	N/A
516359.5	5752908	2.33	2.175	RES	13 DAVIES ST		0.402	0.02	N/A	N/A	N/A
516368.3	5753018	2.316	2.068	RES	25 DAVIES ST	SECOND DWELLING	0.404	-0.058	N/A	N/A	N/A
516439.1	5752864	2.101	1.73	RES	13-15 MAIN RD		0.411	0.185	N/A	N/A	N/A
516492.8	5752436	1.944	1.922	RES	7 BUCKLEY ST		0.411	N/A	N/A	N/A	N/A
516464.6	5752953	2.015	1.684	RES	25 MAIN RD		0.415	0.199	N/A	N/A	N/A
516404.4	5752440	2.067	1.926	RES	19 IRVING ST		0.423	-0.153	N/A	N/A	N/A
516695.6	5752829	1.689	1.311	RES	15 HANSEN ST		0.431	0.148	N/A	N/A	N/A
516385.8	5752622	2.22	1.685	RES	11 GOVERNMENT RD		0.433	0.157	N/A	N/A	N/A
516910.2	5752622	1.756	1.4	RES	7 SHORELINE DR		0.433	N/A	N/A	N/A	N/A
516533	5752478	1.868	1.615	RES	3 BEARUP ST		0.433	-0.123	-0.123	-0.123	-0.123
516414.3	5752696	2.188	2.104	RES	1 GOVERNMENT RD		0.438	0.182	N/A	N/A	N/A
516494.9	5753190	2.026	1.917	RES	18 ELLEN AVE		0.44	0.024	N/A	N/A	N/A
516504.3	5753285	2.024	1.993	RES	28 ELLEN AVE		0.459	0.046	N/A	N/A	N/A
516777.3	5752873	1.625	1.386	RES	18 CENTRE RD	SECOND DWELLING	0.46	0.207	N/A	N/A	N/A
516317.6	5752284	1.923	1.624	RES	32 BUCKLEY ST	STOOP	0.478	N/A	N/A	N/A	N/A

516570.6	5753238	1.65	1.69	RES	59 MAIN RD		0.496	0.257	N/A	N/A	N/A
516329.1	5752845	2.255	2.27	RES	5 DAVIES ST		0.519	0.134	N/A	N/A	N/A
516511.1	5752447	1.807	1.745	RES	5 BUCKLEY ST		0.522	N/A	N/A	N/A	N/A
516646.3	5752881	1.579	1.323	RES	12 NEWTON ST		0.528	0.252	N/A	N/A	N/A
516499.9	5752850	1.774	1.775	RES	14 MAIN RD	GRANNY FLAT	0.539	N/A	N/A	N/A	N/A
516341.8	5752303	1.864	1.719	RES	28 BUCKLEY ST	STOOP	0.54	-0.15	-0.15	-0.15	-0.15
516885.9	5752870	1.523	1.31	RES	25 CENTRE RD		0.541	0.309	N/A	N/A	N/A
516488.5	5752868	1.801	1.89	RES	16 MAIN RD	STOOP	0.554	N/A	N/A	N/A	N/A
516351.7	5752323	1.834	1.7	RES	26 BUCKLEY ST	STOOP	0.571	N/A	N/A	N/A	N/A
516659.5	5753000	1.496	1.379	RES	14 NEWTON ST		0.575	0.332	N/A	N/A	N/A
516395.6	5752936	2.085	2.059	RES	18 DAVIES ST		0.576	0.199	N/A	N/A	N/A
516429.8	5752839	1.947	1.758	RES	11 MAIN RD		0.582	0.343	N/A	N/A	N/A
516385.2	5752608	2.009	1.73	RES	13 GOVERNMENT RD		0.617	0.365	N/A	N/A	N/A
516445.7	5752909	1.873	1.71	RES	19 MAIN RD		0.631	0.4	N/A	N/A	N/A
516676	5752828	1.481	1.414	RES	15 HANSEN ST	GRANNY FLAT	0.647	0.356	N/A	N/A	N/A
516454.3	5752937	1.813	1.87	RES	23 MAIN RD	STOOP	0.649	0.418	N/A	N/A	N/A
516758.9	5752891	1.349	1.328	RES	16 CENTRE RD		0.73	0.482	N/A	N/A	N/A
516869.7	5752608	1.406	1.335	RES	3/5 SHORELINE DR	BOTTOM STOREY (STORAGE/GARAGE)	0.789	0.016	N/A	N/A	N/A
516698.2	5752604	2.789	2.549	COM	1 FUTCHER RD	CARAVAN PARK	-0.56	N/A	N/A	N/A	N/A
516582.2	5752493	2.804	1.959	COM	1 FUTCHER RD	CARAVAN PARK RECEPTION	-0.541	N/A	N/A	N/A	N/A

516638.3	5752631	2.571	2.136	COM	1 FUTCHER RD	CARAVAN PARK	-0.336	-0.827	-0.833	N/A	N/A
516753.1	5752484	2.501	1.875	COM	1 FUTCHER RD	CARAVAN PARK	-0.269	N/A	N/A	N/A	N/A
516813.4	5752543	2.365	1.838	COM	1 FUTCHER RD	CARAVAN PARK	-0.143	N/A	N/A	N/A	N/A
516499.4	5752580	2.505	1.872	COM	1 IRVING ST	BUILDING BEHIND CFA	-0.029	-0.591	N/A	N/A	N/A
516648.1	5752646	2.254	1.969	COM	1 FUTCHER RD	CARAVAN PARK	-0.02	N/A	N/A	N/A	N/A
516598.6	5752466	2.281	1.793	COM	1 FUTCHER RD	CARAVAN PARK	-0.018	N/A	N/A	N/A	N/A
516802.6	5752558	2.213	1.816	COM	1 FUTCHER RD	CARAVAN PARK	0.007	N/A	N/A	N/A	N/A
516767	5752538	2.218	2.005	COM	1 FUTCHER RD	CARAVAN PARK	0.01	N/A	N/A	N/A	N/A
516765.5	5752633	2.19	1.901	COM	1 FUTCHER RD	CARAVAN PARK	0.021	N/A	N/A	N/A	N/A
516770.3	5752629	2.122	1.895	COM	1 FUTCHER RD	CARAVAN PARK	0.091	N/A	N/A	N/A	N/A
516805.6	5752535	2.111	1.89	COM	1 FUTCHER RD	CARAVAN PARK	0.112	N/A	N/A	N/A	N/A
516590.9	5752523	2.055	1.939	COM	18 FUTCHER RD	TENNIS CLUB	0.198	N/A	N/A	N/A	N/A
516777.4	5752620	2.014	1.841	COM	1 FUTCHER RD	CARAVAN PARK	0.199	N/A	N/A	N/A	N/A
516832.8	5752563	2.005	1.716	COM	1 FUTCHER RD	CARAVAN PARK	0.21	N/A	N/A	N/A	N/A
516827	5752570	1.998	1.711	COM	1 FUTCHER RD	CARAVAN PARK	0.219	N/A	N/A	N/A	N/A
516788.2	5752663	1.975	1.508	COM	1 FUTCHER RD	CARAVAN PARK	0.229	-0.553	N/A	N/A	N/A
516784	5752580	1.982	1.813	COM	1 FUTCHER RD	CARAVAN PARK	0.237	N/A	N/A	N/A	N/A
516343.5	5752148	2.109	2.033	COM	37 FORESHORE RD	MILK BAR	0.281	N/A	N/A	N/A	N/A
516511.1	5752553	2.095	2.076	COM	1 IRVING ST	CFA BUILDING	0.336	N/A	N/A	N/A	N/A
516728.7	5752942	1.652	1.331	COM	15 CENTRE RD	STUDIO	0.419	0.177	N/A	N/A	N/A

Seaspray Flood Study - Summary Report (R05)



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Cover Photo: Flooding in Seaspray in 1993

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EXECUTIVE SUMMARY

This report details the input data, approach and outcomes for the Seaspray Flood Study and is intended to be read as an accompaniment to the previous reports:

- Seaspray Flood Study – Data Review (R01)
- Seaspray Flood Study – Hydrology (R02)
- Seaspray Flood Study – Hydraulics (R03)
- Seaspray Flood Study – Assess and Treat Risk (R04)

The Seaspray Flood Study has been initiated by the West Gippsland Catchment Management Authority (WGCMA) in order to define the extent and characteristics of flooding in the township of Seaspray so that future planning decisions may be soundly based and measures may be put in place to minimise risk to the community.

The study provides information on flood behaviour and flood intelligence in and round the township of Seaspray. The study involved a rigorous technical analysis of the drivers for flooding, which included coastal, riverine and stormwater flooding, and provides confidence in the use of this information to guide emergency management and future floodplain management in the catchment.

Community consultation was undertaken at various stages of the study, primarily in order to gather data and accounts of flooding and to benchmark the outputs of the calibration and preliminary design event mapping. The flood information provided by residents was invaluable in the development of the study outcomes.

A hydrologic analysis of Merriman Creek and the associated catchment was undertaken to determine design flood hydrographs for the 10%, 5%, 2%, 1% and 0.5% annual exceedance probability (AEP) flood events which affect the township as well as the probable maximum flood (PMF). A rigorous approach has been applied to test and validate the design flows by utilising a number of hydrologic approaches including Flood Frequency Analysis, and development of a detailed hydrologic (RORB) model. The adopted design flood inflows for the study are listed in Table 1.

Table 1 **Design peak flows at Prospect Road**

AEP	Flow (ML/day)
10%	10,280
5%	19,790
2%	29,030
1%	41,130
0.5%	55,640

To place the design peak flows in a historical context, the approximate AEP (and Average Recurrence Interval, ARI) of significant historical flood events are provided in Table 2. Widespread flooding occurred throughout Seaspray during the September 1993 event. The November 1995 event is the largest gauged event recorded at the Prospect Road Gauge. Despite the 1995 event being the largest on record, flood mitigation works following the 1993 prevented flooding in Seaspray.

Table 2 Merriman Creek, Approximate AEP/ARIs for significant historical flood events

Historical event (year)	Approximate AEP/ARI (based at Prospect Road)
September 1993	2% / 50 years
November 1995	2% / 50 years
June 2012	<10% / 10 years

A digital elevation model (DEM) was developed from available LiDAR survey. Using the DEM, a hydraulic model was established to simulate flood behaviour within the study area. Flood behaviour was assessed for flooding originating from tributaries as well as local catchment runoff within the floodplain. The hydraulic model was calibrated to one historic flood event (September 1993). The completion of the Flood Mitigation Scheme for the town in 1987 significantly altered the flood behaviour through Seaspray. The introduction of flood defence levees and the construction of the Lake Reeve Floodway as part of this scheme, removed the ability to achieve any relevant calibration to flood events prior to the scheme's implementation. Additionally, the more recent changes to flood levees and culvert upgrades following the 1993 flood event have shown a positive influence in reducing flood impacts within Seaspray. As such, more recent flood events have resulted in limited impact on properties and infrastructure, and has led to insufficient data with which model comparisons can be based.

Flood maps of all the design flood events and output data for all events have been produced as a key deliverable of this study.

A flood risk analysis was undertaken, including extraction of flood intelligence and flood warning information, review of the operation of the existing flood management scheme, and development of a Flood Overlay (FO) and Land Subject to Inundation Overlay (LSIO). Recommendations for improved management and maintenance are provided in the Flood Risk Report (R04) and include operational controls for the manual opening of the entrance berm if closed prior to a flood event, maintenance of the floodway, and operation of the existing regulator structure.

Conceptual design and feasibility assessment was undertaken for several mitigation options to further improve the flood resilience of Seaspray. Improvement works to the existing levee network, and the extension of Griffioens Levee has been shown to provide considerable reduction in flood damages for the 1% AEP flood event and a positive return from a benefit-cost analysis.

The model resolution of this study is suitable to inform land use planning and flood insurance pricing at a property scale. The timing of the flood peak and associated data is suitable to inform emergency response services.

ACKNOWLEDGEMENTS

Numerous organisations and individuals have contributed both time and valuable information to the Seaspray Flood Study. The study team acknowledges the contributions made by these groups and individuals, in particular:

- (West Gippsland CMA)
- (Department of Environment Land Water and Planning)
- (Wellington Shire Council)
- (Thiess/Ventia Hydrographic Services)
- (VicSES)
- (Bureau of Meteorology)

The study team also wishes to thank all those stakeholders and members of the public who participated in the steering group and community information sessions and provided valuable records (including historic photos) and discussed their experiences and views on flooding in Seaspray.

GLOSSARY OF TERMS

Annual Exceedance Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be of extreme magnitude.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level. Introduced in 1971 to eventually supersede all earlier datums.
Average Recurrence Interval (ARI)	Refers to the average time interval between a given flood magnitude occurring or being exceeded. A 10 year ARI flood is expected to be exceeded on average once every 10 years. A 100 year ARI flood is expected to be exceeded on average once every 100 years. The AEP is the ARI expressed as a percentage.
Cadastre, cadastral base	Information in map or digital form showing the extent and usage of land, including streets, lot boundaries, water courses etc.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design flood	A design flood is a probabilistic or statistical estimate, being generally based on some form of probability analysis of flood or rainfall data. An average recurrence interval or exceedance probability is attributed to the estimate.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or overland runoff before entering a watercourse and/or coastal inundation resulting from elevated sea levels and/or waves overtopping coastline defences.
Flood frequency analysis	A statistical analysis of observed flood magnitudes to determine the probability of a given flood magnitude.
Flood hazard	Potential risk to life and limb caused by flooding. Flood hazard combines the flood depth and velocity.
Floodplain	Area of land which is subject to inundation by floods up to the probable maximum flood event, i.e. flood prone land.
Flood storages	Those parts of the floodplain that are important for the temporary storage, of floodwaters during the passage of a flood.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.
Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.

Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
Intensity frequency duration (IFD) analysis	Statistical analysis of rainfall, describing the rainfall intensity (mm/hr), frequency (probability measured by the AEP), duration (hrs). This analysis is used to generate design rainfall estimates.
LiDAR	Spot land surface heights collected via aerial light detection and ranging (LiDAR) survey. The spot heights are converted to a gridded digital elevation model dataset for use in modelling and mapping.
MIKE FLOOD	A hydraulic modelling tool used in this study to simulate the flow of flood water through the floodplain. The model uses numerical equations to describe the water movement.
Peak flow	The maximum discharge occurring during a flood event.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Average Recurrence Interval.
Probable Maximum Flood	The flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a particular drainage area.
RORB	A hydrological modelling tool used in this study to calculate the runoff generated from historic and design rainfall events.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
Stage hydrograph	A graph that shows how the water level changes with time. It must be referenced to a particular location and datum.
Topography	A surface which defines the ground level of a chosen area.

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1. INTRODUCTION

1.1 Overview

The West Gippsland Catchment Management Authority (WGCMA) commissioned Water Technology to undertake the Seaspray Flood Study. This study involved detailed hydrological and hydraulic modelling of the township of Seaspray, the adjacent floodplain areas from Prospect Road on Merriman Creek to Lake Reeve and took into account the downstream ocean conditions.

The objective of the Seaspray Flood Study was to define the extent and characteristics of flooding in and around Seaspray so that emergency management and future planning decisions may be soundly based and measures may be put in place to minimise risk to the community.

The study addressed the following aspects:

- Examine contributing factors to flood events around Seaspray including Merriman Creek, Lake Reeve, stormwater and the ocean;
- Assess the sensitivity of the impacts of climate change, bushfire, sea level rise and Lake Reeve influences;
- Determine flood levels and extents for a range of flood modelling scenarios within the study area;
- Provide detailed flood intelligence outputs;
- Develop conceptual design of feasible flood mitigation options.

1.2 Study Catchment and Floodplain

Seaspray is a coastal town with 320 permanent residents which rises to more than 1,000 during summer. The township lies at the western end of the Ninety Mile Beach west of the Gippsland Lakes and approximately 200 km east of Melbourne. As detailed in the Urban Design Framework developed for the township by Wellington Shire Council (Wellington Shire Council, 2007), Seaspray is an important access point to 90 Mile Beach, and the areas of small lot subdivision that extend easterly for 28 km to Golden Beach and Paradise Beach.

The town is located on the estuarine floodplain of the Merriman Creek catchment. Merriman Creek flows along the western boundary of Seaspray before it passes through the 90 Mile Beach coastal dune system and enters Bass Strait. As part of the flood mitigation scheme completed in 1987, an engineered floodway running through the middle of the town, has the ability to distribute water from the Merriman Creek estuary to Lake Reeve in the east.

The study area includes Merriman Creek downstream of Prospect Road, the lagoon entrance with Bass Strait, and the floodway extending all the way to the western end of Lake Reeve as shown in Figure 1-1.

In this study area, flooding can occur as a result of a range of potential mechanisms, including;

- Overbank flows from Merriman Creek,
- Coastal inundation due to elevated ocean levels,
- Flooding from the Gippsland Lakes via Lake Reeve,
- Localised stormwater flooding.

Each of these flooding mechanisms have been evaluated during this project.



Figure 1-1 Study Area

1.3 Supporting Documents

A number of reports were prepared at each stage of the study. These reports were produced as separate standalone volumes, and a summary of each is provided in Table 1-1. In addition to these documents, flood intelligence outputs and GIS layers have been provided for each of the design flood events.

Table 1-1 Supporting documents

Report	Document Number	Title	Summary
1	R01	Data Report	Review of flood related information for the study area, a review of available topographic and structure data (bridges and culvert information), and verification of topographic data. The report also provided a proposed outline of the hydrologic analysis and hydraulic modelling methodology
2	R02	Hydrology Report	Hydrologic modelling and analysis report, summarising results of flood frequency analysis, RORB modelling, estimation of design event, and probable maximum flood hydrographs
3	R03	Hydraulics Report	Hydraulic modelling report providing details of hydraulic model construction and calibration, sensitivity tests, and results of design event simulations.

Report	Document Number	Title	Summary
4	R04	Assess and Treat Risk Report	Flood risk report detailing the flood intelligence, flood mitigation options and associated flood damages for a range of design events.

2. DATA REVIEW

On inception of the project a detailed review was undertaken of all available flood related information as well as topographic data, structure information, and hydrological data. Details of this review are provided in Report 1, while a short overview is provided herein.

2.1 Flood Information

2.1.1 Flood Related Studies

Following extensive flooding in Seaspray in May and then June 1978, the State Rivers and Water Supply Commission published a flood report for the floods. The report included mapping of approximately 18 surveyed flood levels from the May 1978 flood and 27 flood levels from the June 1978 flood, as well as aerial photography of the June 1978 flood.

The Seaspray Flood Study was then commissioned by the State Rivers and Water Supply Commission in 1980 (Camp Scott Furphy, 1980). A flood frequency analysis was undertaken with limited gauge data and a 1% AEP peak discharge of 330 m³/s was adopted, equal to the June 1978 peak discharge. Flood modelling and damage assessment was undertaken and a flood mitigation scheme recommended, which included the following works and measures:

- Proclamation of flood-prone lands and drainage courses and incorporation into local planning scheme.
- Construction of an approved flood mitigation scheme including levees along Merriman Creek and Lake Reeve Floodway, excavation of the Lake Reeve Floodway and construction of a regulating structure between the Lake Reeve Floodway and Merriman Creek.
- Operation and maintenance arrangements including the clearing of Merriman Creek mouth and operation of the Lake Reeve Floodway.
- Improved flood warning arrangements.

The construction of the scheme was completed in 1987. A levee audit in 1996 (Findlay Irrigation Design Services and BM Consulting Civil Engineers, 1996) found the levees in generally good condition, however in many cases the levee crest was below the design level by up to 0.3 m, and in one area there was a gap in the levees that had allowed floodwaters into the protected area in the 1993 flood.

A flood report was also published for the 1993 flood (Hydrotechnology, 1995), which contained comprehensive details of flood extents, heights and impacts in Seaspray. The flood review of 2007 (EGCMA and WGCMA, 2010) contained a brief reference to the coastal erosion that occurred in Seaspray.

A minor investigation was undertaken for the Duke Energy track crossing of the Lake Reeve floodway in 2002 (ID&A 2002). A Coastal Hazard and Vulnerability Assessment and Flood Study was undertaken for the Seaspray Caravan Park in 2010 (Cardno Lawson Treloar, 2010), which found the existing levee system only protected land up to the 70 year ARI flood event, primarily due to a revision of the 1% AEP flow estimate to more than double the 1980 estimate (679 m³/s). The study

also assessed flooding under a range of storm surge and sea level rise conditions, finding that vulnerability to coastal flooding was low.

The recently completed Gippsland Lakes/90 Mile Beach Local Coastal Hazard Assessment Project (Water Technology, 2014a) assessed the sensitivity of Seaspray to flooding from the Gippsland Lakes under present and future sea level rise conditions based on hydrodynamic modelling. The work found that the floodplains around Seaspray only became inundated during the 10% AEP flood event under the +0.8 m sea level rise scenario. Flood levels did not overtop the existing levees. Coincident flooding from Merriman Creek was not considered. However, coastally driven inundation of the township due to overwash of the dune barrier system was also assessed.

2.1.2 Historic Flood Information

Significant historic flood events have been compiled from available sources and are listed in Table 2-1. The largest flood on record at the Prospect Road gauge, just upstream of Seaspray, was the October 1995 flood, however very little information is available for that event. More information is available for the 1993 event, including digitised flood extent, surveyed flood levels, floor levels and numerous flood photographs, collated for Hydrotechnology's report on the 1993 Gippsland flood. A large amount of data is also available for the 1978 floods which were the largest on record when they occurred, but were later exceeded in 1993 and 1995.

Table 2-1 Historic Flood Events

Event	Description	Data available
1880	Whole area flooded with fresh water	Gippsland Times letter
1895	Salt water flooding of whole area	Gippsland Times letter
1900	Town flooded by salt water	Gippsland Times article
March 1908	Salt water flooding of camping reserve	Gippsland Times article
September 1916		
1930	Two salt water flooding events in the first half of 1930	Gippsland Times article
April 1932	Creek overflowed and swamped a large portion of Seaspray	Gippsland Times article
December 1934	"Flood of unusual magnitude"	Gippsland Times article
April 1935	Every house in north end of town surrounded by water	
July 1936	Flood covering both ends of township	Gippsland Times article
May 1944	Mouth blocked	
July 1949	Mouth blocked	
February 1951	Reported as the worst flood in 60 years. 75% of homes in Seaspray were flooded	Gippsland Times articles
June 1952	Reported as a "record flood" Overflow from Merriman Creek and Blind Creek, with Monkey Creek also in flood Mouth opened readily as floodwaters rose. 75% of homes in Seaspray were flooded	Discharge estimated at 194 m ³ /s in 1980 Flood Study (Camp Scott Furphy 1980) One flood level (VFD) Gippsland Times articles
January 1962	Tidal flooding	
February 1971		Gauged streamflow at 227001 (peak discharge 326 m ³ /s)
May 1975	Tidal flooding	
May 1978	Overflow from Merriman Creek and Blind Creek	Digitised flood extent (VFD)

Event	Description	Data available
	12 out of 55 permanently occupied residential properties were flooded at or above floor level 9 out of 42 seasonally occupied residences were flooded at or above floor level	Approx. 18 surveyed flood levels (Camp Scott Furphy 1980)
June 1978	42 out of 55 permanently occupied residential properties were flooded at or above floor level 38 out of 42 seasonally occupied residences were flooded at or above floor level Complete inundation of over 180 built-on allotments Only 15 to 20 allotments out of 320 were not subject to any inundation	Peak discharge estimated at 330 m ³ /s in 1980 Flood Study (Camp Scott Furphy 1980) Digitised flood extent (VFD) Approx. 27 surveyed flood levels
April 1990		Digitised flood extent (VFD)
September 1993	1.5 times larger than the previous largest flood and previous 1% AEP estimate. Land affected by flooding from Blind Creek and overtopping/outflanking of levees. 44 houses flooded above floor level Around 140 built-on allotments affected by flooding in total	Peak discharge estimated at 450 m ³ /s Digitised flood extent (VFD) Approx. 12 surveyed flood levels CFA logs and transcripts 44 surveyed floor levels Numerous flood photographs (Hydrotechnology 1995)
October 1995	Second-largest flood on record.	Flood peak 370 m ³ /s
2007	Coastal erosion of dunes	
June 2012		Digitised flood extent (VFD) Lake Reeve western end only.

2.1.3 Flood Mitigation Scheme

The Seaspray flood mitigation scheme was constructed in response to the 1978 flooding following the 1980 flood study. The flood mitigation scheme was originally intended to protect residential land and allow extra time to get the entrance open. The scheme consists of:

- Levees along Merriman Creek and surrounding developed areas
- Excavation of the Lake Reeve Floodway
- Regulating structure and spillway between Merriman Creek and the Lake Reeve Floodway
- Remodelling of Merriman Creek including cut-off of the meander (now The Island)

Figure 2-2 shows the location of the levees and floodway structures.

Wellington Shire Council is responsible for the operation of the floodway. The opening of the entrance is considered the first priority when a flood alert is issued. If the mouth is not open then it must be mechanically excavated. At times the mouth has been difficult to get open and has tended to reclose due to wave conditions. If the mouth cannot be opened then flows exceeding 45 m³/s are likely to overtop the levees.

The regulator structure can be lowered to allow flood flows to flow towards Lake Reeve to allow additional time to get the entrance open. The capacity of the floodway is 15 m³/s. In recent years the regulator has been kept closed in all events, with the aim to allow flood levels to build up in the lagoon to help drive entrance scouring.

A low flow bypass pipe connects Merriman Creek with the Lake Reeve end of the floodway, but is not currently operable due to corrosion.

The flood mitigation scheme refers to a fixed spillway at 2.4 m to Masons Creek to the south west of the lagoon. However there is no evidence of this spillway on the ground or in the DEM. The ground

surface has an overflow level of approximately 2.3 m to Masons Creek. The scheme also refers to filling of the land west of the lagoon to compensate for increased flood levels, however the land appears to have been protected with a levee with crest height approximately 1.9 m to 2.5 m AHD rather than fill.

The construction of the scheme was completed in 1987. Some additional works were undertaken after the 1996 audit (Findlay Irrigation Design Services and BM Consulting Civil Engineers 1996) including the levee extension up to Blind Creek and the Eastern Prior Stream culvert upgrade. The levees were surveyed for the Urban Levee Review project (SKM, 2014).

2.1.4 Existing Flood Warning Arrangements

Bureau of Meteorology gauges at Prospect Road and Stradbroke West were installed following the 1980 flood study for flood warning purposes. Flood alert flow levels at the Prospect Road gauge are listed in the Mitigation Scheme Operating Procedure but it is unknown whether these have been formally implemented by the Bureau of Meteorology. No flood class levels have been defined for either gauge.

The Mitigation Scheme Operating Procedure also includes recommendations for opening of the estuary entrance when flood conditions are expected in Merriman Creek, based on the upstream gauge flows.

2.2 Topographic Data

2.2.1 Available Datasets

Aerial LiDAR (Light Detection and Ranging) survey was available for the Seaspray area from two difference sources:

- Vicmap Elevation Coastal 1m DEM (referred to as the 'Coastal DEM')
- 2009-10 Victorian State Wide Rivers LiDAR Project - West Gippsland CMA (referred to as the 'Rivers DEM')

It was identified during the data review that further bathymetric data information was required for the lower sections of Merriman Creek and also for the new caravan park area which was relocated to its present location after the LiDAR data capture. Field survey was subsequently obtained for the areas within lower Merriman Creek and Wellington Shire Council provided design levels and layout drawings for the caravan park.

For ocean areas, VicMap 20m bathymetry dataset was available. Details of each of the topographic data sets are provided in the Data Report (R01).

2.2.2 DEM Development

The model topography was developed from four data sources as shown in Figure 2-1. The Coastal DEM (green) was used where available, with the exception of the main river channel where surveyed bathymetric levels collected for this project were used (red). VicMap 20m bathymetry (blue) was used seaward of the shore line and the Rivers DEM (pink) was used in a small area of the upstream section of the model where the Coastal DEM did not have coverage. Modifications were made directly to the mesh for the various entrance conditions representation. Further discussion on the quality and availability of LiDAR is provided in the Data Report (R01).

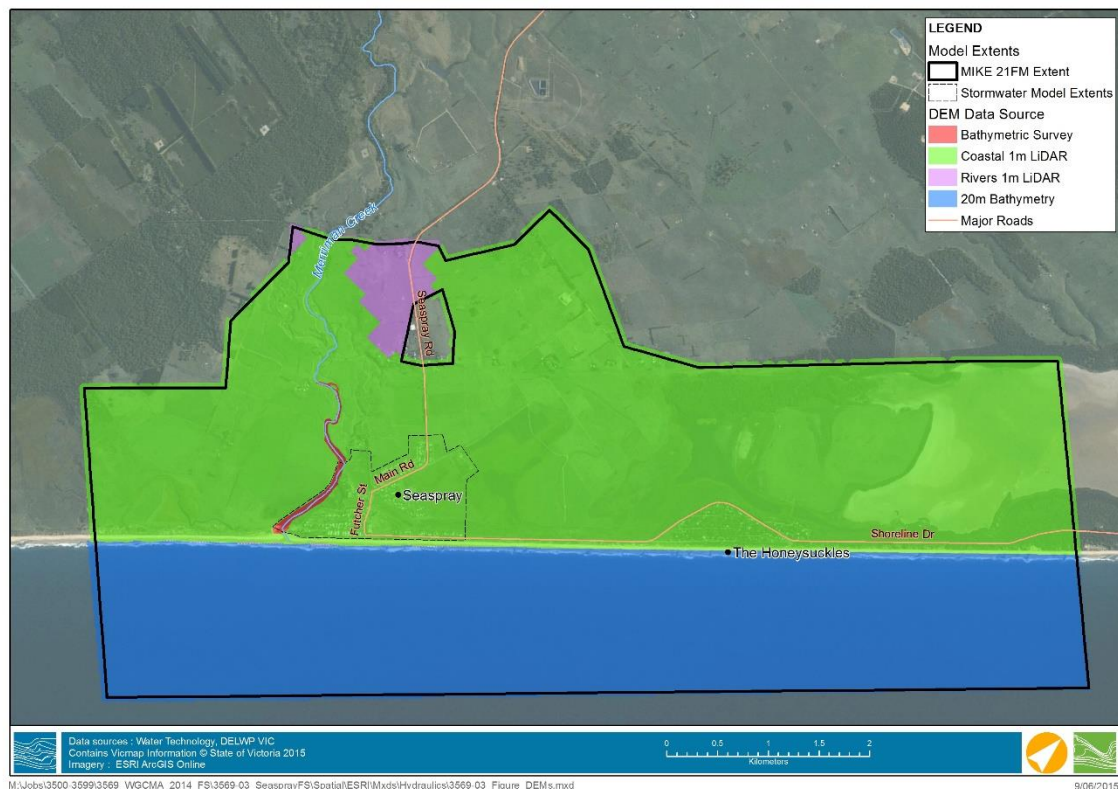


Figure 2-1 Extent of DEM Data Sources Used in Model.

2.3 Structure Information

The following structures are those that have been identified for incorporation to the hydraulic model.

2.3.1 Drainage Information

GIS layers of local pits and pipes were provided by Wellington Shire (Figure 2-2). The town of Seaspray is divided into two separate drainage areas. The area east of Fletcher Road drains to a pump station on Centre Road on the northern side of the floodway, which pumps stormwater out to the floodway. The area west of Fletcher Road drains to a pump station on Irving Street, which pumps to Merriman Creek. Each pump station has two pumps which have a maximum capacity of 170 L/s each. During a high rainfall event both pumps can be operated together giving a maximum discharge rate from each pump station of 340 L/s.

The caravan park has been constructed recently on the site of the former oval and has significant internal drainage infrastructure, including grated pits and pipes running along internal roads, and two retarding basins. Details of the caravan park infrastructure including pit and pipe layout, and detention basin locations was provided by Wellington Shire Council.



Figure 2-2 Drainage system at Seaspray

2.3.2 Levees

The Urban Levees Review (SKM, 2014) identified the following four levees within Seaspray, which were subsequently surveyed along with a preliminary condition assessment:

- Levee A – situated along the west bank of Merriman Creek between the floodway and Ninety Mile Beach.
- Levee B and C – situated along the north bank of the floodway (to the east of Merriman Creek) between Merriman Creek and the end of Catton Street, where it runs parallel to Centre Road to Main Road (Seaspray Road) and then along Main Road to the junction with Tip Road.
- Levee D – situated along the south bank of the floodway (to the east of Merriman Creek) between Merriman Creek and the most easterly house along Shoreline Drive.

An additional section of levee (Levee E), which is an extension of Levee C, was not surveyed. Each of the surveyed levees is shown in Figure 2-3. The levees labelled 2000 Levee are from the Victorian Flood Database (VFD).

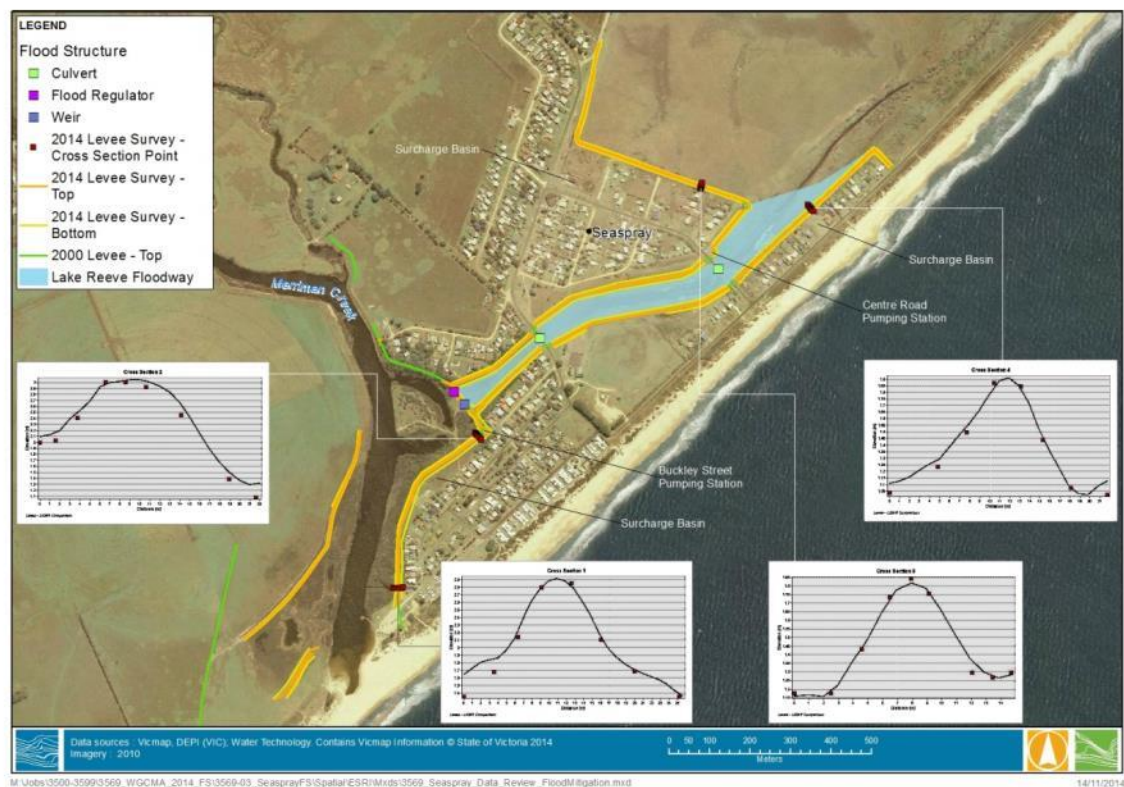


Figure 2-3 Location of Levees and Flood Mitigation Structures within and around Seaspray

2.3.3 Bridges and Culverts

The main bridges and culverts were included in the model as structures defined by their dimensions. The dimensions for the bridges and culverts were determined during a site visit as listed in the Data Review Report (R01).

For the 1993 calibration event, the Eastern Prior culvert under Seaspray Road was a single 450 mm reinforced concrete pipe. After the flood, this was replaced with five 2000 x 1500 mm reinforced concrete box culverts. The modelling of the 1993 event therefore applied a 450 mm pipe culvert, while the sensitivity and design runs applied the new box culvert dimensions.

2.3.4 Regulator and Adjacent Weir

There is a regulator and weir structure at the western end of the Lake Reeve Floodway. The minimum level of the weir is 2.2 m AHD. It is the current operating practice to retain all boards within the structure, raising the overall elevation to 2.7 m AHD. This level, representing the highest setting of the regulator, preferentially directs flow toward the lagoon entrance and away from the Lake Reeve Floodway. The impact and control of the regulator structure has been investigated within a sensitivity analysis, which is described in detail in Report 3 (R03).

2.4 Hydrological Data

Hydrological data required for the study included streamflow, rainfall, along with river water level information.

2.4.1 Rainfall Data

The average annual rainfall at Seaspray is 620 mm. A steep rainfall gradient exists over the catchment with average annual rainfall reaching 1500 mm in the headwaters. At the catchment centroid the average annual rainfall is around 670 mm.

Numerous daily rainfall sites are in operation in and around the catchment. Key stations, including current stations and stations operating over the 1978, 1993 and 1995 floods, are listed in Table 2-2.

Table 2-2 Daily rainfall stations around Merriman Creek catchment

Gauge No.	Location	Period	Years	Distance from catchment centroid
85006 226017	Le Roy (Taylors Rd Quarry)	2000-2009	9	34
85009 226815	Traralgon Epa	2000-2014	14	34
85017	Callignee South	1932-1985	54	26
85033	Giffard	1906-2014	108	21
85071	Rosedale	1878-2005	124	20
85072	East Sale Airport	1943-2014	72	30
85073	Seaspray (Burong)	1989-2014	112	25
85076	Stradbroke West (Inglenook)	1891-1982	91	6
85101	Tarra Valley	1952-1990	10	34
85105	Hazelwood North	1939-1990	37	35
85148	Woodside (Lake View)	1952-1997	46	23
85152	Won Wron Prison	1967-2005	38	27
85160	Darriman (Tarralangi)	1952-2014	62	13
85170	Traralgon L.V.W. & S.B.	1967-1990	23	33
85236	Callignee North	1956-2014	57	28
85281	Traralgon Creek At Koornalla	2000-2014	14	31
85298	East Sale Comparison	1996-2005	9	31
85299	Koornalla Traralgon Ck Rd	2000-2014	14	31

Pluviograph (sub-daily rainfall) stations in and around the Merriman Creek catchment are listed in Table 2-3. The 1993 and 1995 events were captured at East Sale Airport and Calignee North pluviographs. The 1978 events were captured at all listed stations.

Table 2-3 Pluviograph stations around Merriman Creek catchment

Gauge No.	Location	Period	Years	Distance from catchment centroid (km)
85072	East Sale Airport	1953-2011	58	30
85236 226817	Calignee North	1961-2013 1999-2014	51 15	28
85170	Traralgon LVW & SB	1961-1979	18	33
85265	Macks Creek	1975-1978	3.4	26

Gauge No.	Location	Period	Years	Distance from catchment centroid (km)
85264	Novacs	1968-1978	6.8	24
85007 226818	Balook	1999-2014	15	33
585047 227239	Stradbroke West	2006-2014	8	10
585186 226814	Mount Tassie	1998-2014	16	30

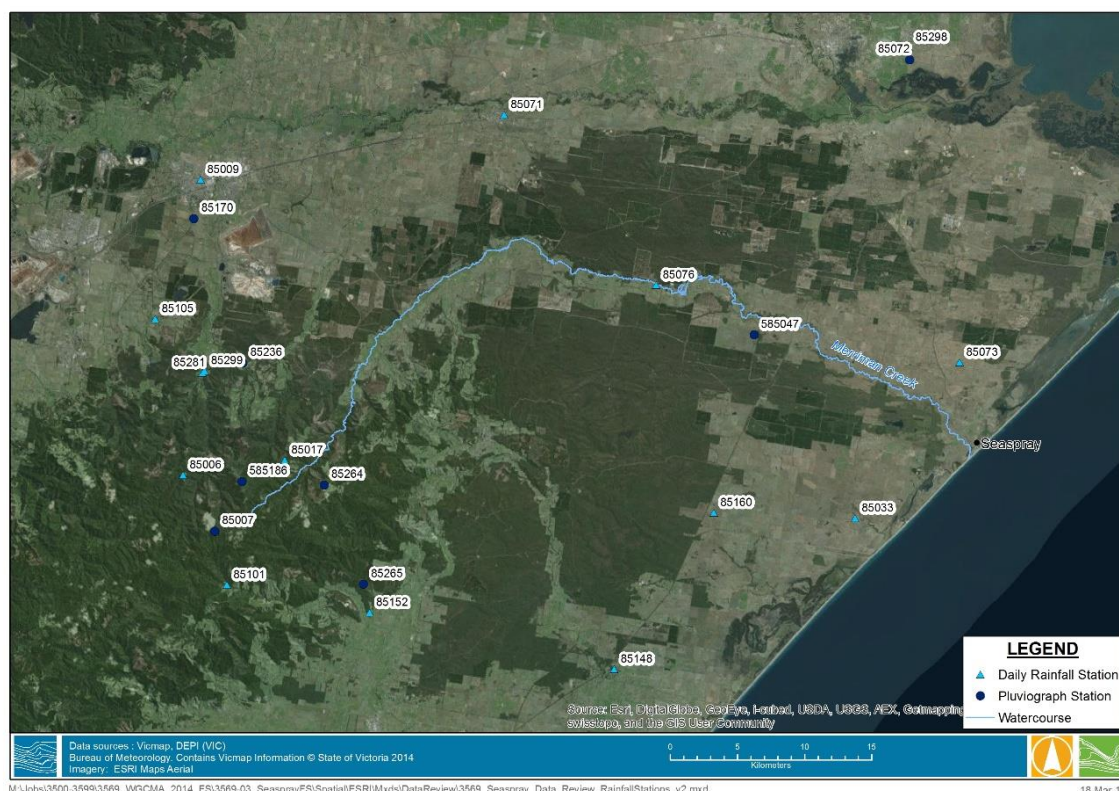


Figure 2-4 Rainfall Stations in and around the Merriman Creek Catchment

2.4.2 Streamflow Data

Gauge Locations

Four streamflow gauges operate in the catchment (Table 2-4). The Prospect Road gauge is located close to the town of Seaspray and captures most of the catchment. The Stradbroke West gauge is located upstream and captures just under half of the catchment. The ungauged Monkey Creek joins Merriman Creek between the Stradbroke West and Prospect Road gauges. The Calignee South gauge is located in the headwaters of the catchment, capturing around 7% of the catchment.

Table 2-4 Streamflow gauges in Merriman Creek catchment

Gauge No.	Location	Period	Years	Catchment Area (km ²)
227240	Merriman Creek @ Prospect Road Seaspray	1983-2014	31	529
227001	Merriman Creek @ Seaspray (old Prospect Road site)	1966-1971	5	525
227239	Merriman Creek @ Stradbroke West	1983-2014	31	256
227205	Merriman Creek @ Calignee South	1946-1952 and 1965-2014	54	36
227242	Merriman Creek @ Seaspray Township	1984-2014	31	

Rating Curves

Prospect Road Seaspray is the closest gauge to the Seaspray Flood Study mapping area, and the design flows determined at this location provide the design event inflows to the hydraulic model. Therefore a detailed review of the Prospect Road Gauge rating curve was undertaken prior to completion of the hydrologic assessment.

The Merriman Creek at Prospect Road gauge (227240) has 31 years of instantaneous flow data. There were also 5 years of data from 1966-1971 at a previous gauging station (227001, referred to as Merriman Creek at Seaspray) located near the current Prospect Road gauge. Little is known about the reliability of the rating at the old gauge site, although numerous physical gauging measurements were undertaken for flows up to around 1,470 ML/d. The rating for the current gauge is officially considered reliable up to flows of 8,210 ML/d, however the small number of physical gauging measurements show considerable scatter across the whole range, particularly for low flows. Therefore flow estimates are likely to have a high degree of uncertainty.

Plotting of the published water levels and flows from the gauge clearly showed a number of rating curves were used over the 32 year period (refer Report 2, R02). This is because the creek at this location has a sand bed which has been subject to significant instability over the record period. A changing water level – discharge relationship occurs as the sand bed accretes and erodes. The long term water level record was also reviewed and it was found that following the flood event in 1993, gauged levels were elevated, even during low flow periods, for a number of years. The low flow water level, after suddenly increasing in 1993, slowly settled back to current levels over a period of around ten years. This is indicative of sand accretion in the channel after the 1993 event. The 1995 rating curve had a lower predicted flow for a given water level than the rating curve used in 1993, such that the recorded 1995 peak flow was lower than the 1993 peak, even though the water level was higher.

Two dimensional hydraulic modelling of the reach of Merriman Creek around the gauge site was undertaken to verify the appropriateness of the rating curves for the calibration events. This is detailed in Report 2 (R02).

2.4.3 Water Level Data

A water level gauge operates in the Merriman Creek lagoon at Seaspray (Table 2-5). The gauge record spans from 1984-2014 but is incomplete, with 1986-1989 and 1993-1995 missing and a patchy record from 2011-2014. None of the major floods are recorded at this gauge.

Table 2-5 Water level gauge at Seaspray

Gauge No.	Location	Period	Years
227242	Merriman Creek @ Seaspray Township	1984-2014 (incomplete)	~20

2.5 Metocean Conditions

2.5.1 Wind Climate

Along the coastline at Seaspray, winds are predominantly westerly, with minor southerly and easterly components. As the broad orientation of the Outer Barrier is to the NE, the majority of the strongest and most persistent winds blow offshore, however, there is some variation along the study area. At Seaspray the shoreline is aligned to the NE (45°) such that only winds from the south to east have appreciable onshore components.

2.5.2 Wave Climate

Waves and the variability associated with their height, period and direction comprise the principle source of energy for mobilising the sediments along the shoreline at Seaspray, which influences the conditions at the Merriman Creek entrance.

The wave climate of the East Gippsland coast is largely sheltered from direct exposure to the highly energetic wave climate of the Southern Ocean by the Tasmanian landmass, although waves from this source do refract around the eastern side of Tasmania. Larger waves are therefore principally generated within eastern Bass Strait by south westerly to southerly winds and in the South Tasman Sea by east to south-easterly winds.

2.5.3 Coastal Water Levels

Ocean water level variations are primarily caused by a combination of the inverse barometric pressure affect, coastally trapped waves and astronomical tides.

Astronomical Tides

The gravitational interactions associated with the sun and moon on the earth's oceans generate regular water level variations commonly referred to as the tide. The Eastern Bass Strait coastline experiences a micro-tidal climate with the tidal range increasing slightly towards the south-west of the study area due to resonance of the tide within Bass Strait. The astronomical tidal planes for Seaspray are detailed in Report 1 (R01).

Storm Tides

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

Estimated recurrence intervals of peak storm tide levels for the open coast at Seaspray for various sea level rise scenarios have been developed by the CSIRO (McInnes et. al., 2009) and are displayed in Table 2-6.

Table 2-6 Estimated Storm Surge and Storm Tide Recurrence Intervals for Open Coast at Seaspray, based on the IPCC 2007 A1FI Scenario 2, from McInnes, et al. (2009)

Period (years)	Storm Tide (m)			
	Current Climate	+0.15 m SLR	+0.47 m SLR	+0.82 m SLR
10	1.22 ±0.12	1.42	1.87	2.33
20	1.32 ±0.12	1.54	1.98	2.44
50	1.43 ±0.12	1.66	2.09	2.53
100	1.50 ±0.14	1.73	2.18	2.64

3. PROJECT CONSULTATION

3.1 Overview

An important element of the flood mapping study was the active engagement of residents in the study area. This engagement was developed over the course of the study through community consultation sessions and meetings with a Steering Committee. The aims of the community consultation were as follows:

- To raise awareness of the study and to identify key community concerns; and
- To provide information to the community and seek their feedback/input regarding the study outcomes including the existing flood behaviour and proposed flood mapping extents.

3.2 Steering Committee

The flood mapping study was led by a Steering Committee consisting of representatives from West Gippsland Catchment Management Authority (WGCMA), Department of Environment Land Water and Planning (DELWP), Wellington Shire Council (WSC) and Victorian State Emergency Service (VicSES).

The Steering Committee met on three occasions at key points throughout the study, to review and manage the development of the study.

3.3 Community Consultation

The main aim of the community engagement process was to provide information regarding the development of the study and to seek feedback, both verbally and through the use of online methods. All community meetings were supported by media releases to local papers and meeting notices.

The public consultation process was coordinated by the West Gippsland Catchment Management Authority. The following community meetings were held as part of the consultation process:

- Initial community meeting, 26th November 2014. This first public meeting was held to outline the objectives of the study to the community and to receive any flood information the community may be able to provide;
- Second community meeting, 1st April 2015. This meeting presented the results of the calibration event flood modelling. Community feedback was sought on the flood modelling results, particularly the flood behaviour aspects.
- Third community meeting, 14th September 2015. This meeting presented the results of the design event modelling and proposed mitigation options.

The community provided knowledge of a range of previous floods. Many of those present were able to provide specific flood intelligence for the 1993 flood, and also provided information on how the entrance channel across the berm is managed.

4. FLOOD BEHAVIOUR

4.1 Overview

Flooding at Seaspray occurs as a result of floodwaters from Merriman Creek upstream, local stormwater flows, elevated ocean water levels and potentially through flooding from the Gippsland Lakes via Lake Reeve.

The flood behaviour associated with these flooding mechanisms has been assessed using a range of industry standard approaches and tools:

- Hydrological analysis – this involves the analysis of the magnitude of previous flood events in the catchment, the development of a rainfall-runoff model for the Merriman Creek catchment, and the prediction of the likelihood of future flood events of a given magnitude,
- Hydraulic analysis – the physical understanding of what a given flood event may look like in and around Seaspray was assessed through a hydraulic analysis. A hydraulic model was used to predict the extent of flooding, flood depths and flow velocities for a range of possible future flood events.

The different flood mechanism and the results of the hydrologic and hydraulic analysis for the study area are discussed in detail in the following sections. Detailed discussions are also provided in Report 2 & 3.

4.2 Hydrology

4.2.1 Streamflow Gauging

There are two streamflow gauge stations available for use in flood frequency analysis and calibration data for the hydrologic and hydraulic models. A detailed analysis of each gauge was undertaken and is presented in the Section 3.4 of Report 2, the Hydrology report.

4.2.2 Flood Frequency Analysis

A flood frequency analysis was used to estimate the magnitude of flood events at the four selected gauges in terms of a probability of occurrence. This allows the quantification of previous flood events and also enables the estimation of the frequency of future flood events.

The flood frequency analysis was based on an annual series of maximum flows at each gauge for the full record of data. Historic flood peaks were also included based on flood information received for each of the gauges and relationships between the gauges. Comparison to previous estimates and regional equations was also undertaken. Further details are provided in Section 3.5 of Report 2.

4.2.3 Hydrologic Modelling

A hydrological model of the catchment was developed for the purpose of extracting design flows to be used as boundary conditions in the hydraulic model. The rainfall-runoff program, RORB (Version 6) was used for this study.

RORB is a non-linear rainfall runoff and streamflow routing model which is used for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas, connected by a series of conceptual reach storages. Design storm rainfall is input to the centroid of each subarea. Specified losses are then deducted, and the excess routed through the reach network. The RORB model layout is shown in Figure 4-1.

The two streamflow gauges within the catchment were used to calibrate the RORB model. Parameter selection was based on calibration to both gauges and comparison to accepted regional methods, and the design flows were validated against the flood frequency analysis.

Design flow hydrographs were developed using the calibrated routing parameters, and loss parameters adjusted to reconcile the flood peak to the flood frequency analysis. The adopted peak design flows are provided in Table 4-1.

Table 4-1 Adopted Peak Flows for Merriman Creek at Prospect Road

AEP	RORB Design Flow (ML/d)	Critical Duration
10%	10,280	48hr
5%	19,790	48hr
2%	29,030	36hr
1%	41,130	48hr
0.5%	55,640	48hr

4.2.4 Sensitivity Analysis

Climate Change

The impacts of climate change were tested by increasing the rainfall intensity by 5% per degree of warming, in line with latest guidance from Australian Rainfall and Runoff (Engineers Australia 2014). A scenario of 2°C of warming (i.e. 10% increase in rainfall intensity) was adopted for this sensitivity test. This is consistent with 'Climate Change in Australia Projections' (CSIRO, 2015) report which suggests for an intermediate climate scenario a temperature increase of between 1.1 °C to 2.0 °C is likely for the Southern Slopes region, which includes Merriman Creek catchment.

The 10% increase in rainfall intensity was to the RORB model. Due to the hydrologic response of the catchment, the proportional increases in catchment flows are significantly larger than the applied percentage increase in rainfall. This is most notable in the more frequent events where the 10% AEP design flood event produces a 46% increase in flow. As the magnitude of the design event increases, the percentage increases in flow are reduced to a value more closely aligned to the percentage increase in rainfall. Further detail is provided in Report 2.

Bushfires

The impacts of bushfires were tested by adjusting the fraction impervious values of the Merriman Creek sub-catchments to reflect an agreed severity of bushfire. For example, Blackham et al (2012) provides values of equivalent percentage impervious for different levels of burn severity, based on BAER (2009). This involved increasing the impervious fraction for all Farming Zone (Forestry) and Public Conservation and Resource Zone areas across the catchment.

As these land use types constitute a significant proportion of the catchment area the increase of impervious fraction due to bushfire had a significant impact on the peak flows generated from the catchment at the catchment outlet. For example, for the 1% AEP flood event, a high intensity bushfire event could increase the flow in Merriman Creek at Prospect Road by 82%. Further detail is provided in Report 2.

4.2.5 Probable Maximum Flood

The Probable Maximum Flood (PMF) is the flow generated from the theoretical maximum precipitation for a given duration under current climate conditions. A PMF Estimate for Merriman Creek at Seaspray (at the lagoon mouth) was prepared using the Quick Method of Nathan et al. (1994). This method applies a set of empirical equations to compute a triangular PMF hydrograph.

The equations are applicable to southeast Australian catchments from 1 to 10,000 km² that do not have large lakes or storages. As the Merriman Creek catchment contains a reasonably large swamp

storage (around 35 ha in area increasing to around 90 ha when full), a correction to the catchment area was required before the equations are applied.

The resulting PMF peak flow estimate is given in Table 4-2.

Table 4-2 PMF peak flow estimates at Seaspray

	Design Flow (m³/s)	Time to Peak (hrs)
PMF	5,818	8.1

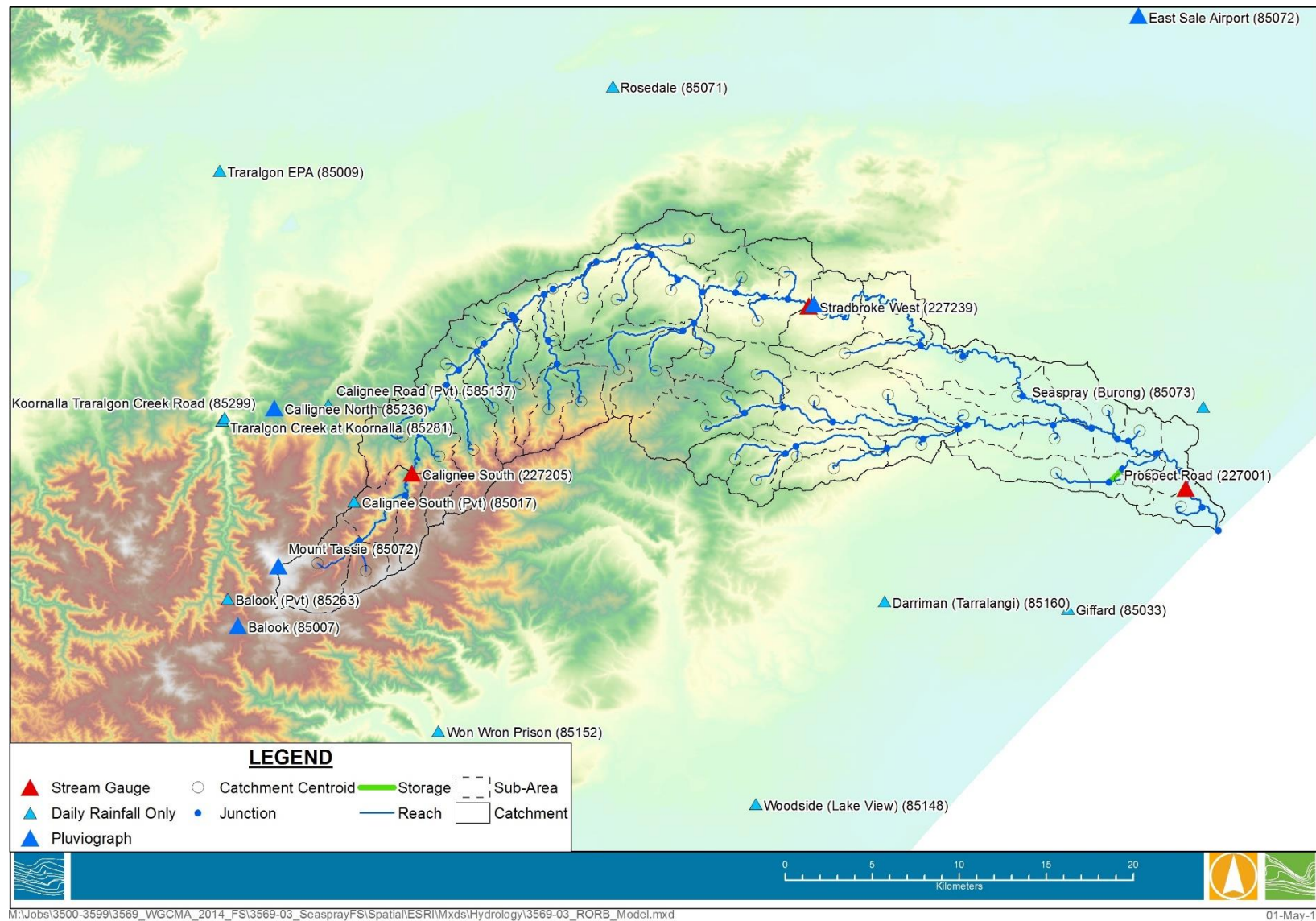


Figure 4-1 RORB Model Structure

4.3 Hydraulics

4.3.1 Overview

This section discusses the application of the hydraulic model to simulate flood behaviour (extents, depth, velocities) for a range of flood magnitudes. The study area experiences a range of different flood mechanisms, including:

- Overbank flows from Merriman Creek,
- Coastal inundation due to elevated ocean levels,
- Flooding from the Gippsland Lakes via Lake Reeve,
- Localised stormwater flooding.

Each of these mechanisms has been assessed in detail during the hydraulic modelling component of the study.

The hydrologic analysis previously discussed, provided flood inflow hydrographs for the hydraulic model. These inflow hydrographs were routed through the calibrated hydraulic model. Ocean boundary levels were applied as a boundary condition. Stormwater influences were modelled independently and then incorporated into the broader assessment. This enabled the modelling of flood depths, extents and velocities over a range of flood magnitudes and conditions. It also provided a tool for understanding the flood behaviour across the study area.

A detailed description of the hydraulic model setup, calibration, validation, sensitivity tests and design event simulation is provided in Report 3. This section summaries the general model development and key outcomes from the hydraulic modelling investigation.

4.3.2 Hydraulic Modelling

A two dimensional (2D) flexible mesh hydraulic model was developed for the study area using MIKE21FM (Mike by DHI). MIKE21FM two-dimensional flexible mesh model systems is a 2D state-of-the-art tool for floodplain modelling. Further details on the capabilities of the MIKE modelling system can be found at <http://www.dhisoftware.com>.

Adopting a flexible mesh modelling approach allowed the hydraulic model to incorporate greater detail in areas of importance, whilst maintaining computational efficiency through a larger element size in less sensitive regions of the modelled area. This allows features within the broader floodplain and the township area to be resolved in varying detail in the same model whilst maintaining appropriate run times.

The model extent covered the full study area, as shown in Figure 1-1, and inflows were included from Merriman Creek, as well sensitivity tests with flows from Lake Reeve. Ocean levels provided the downstream boundary of the model. Topography data was provided by LiDAR and field survey.

To assess specific stormwater flooding mechanisms, a separate local 1D-2D TUFLOW rainfall-on-grid model was established for Seaspray itself. TUFLOW was chosen due to the ease of incorporating pipes and pits within the model. The localised TUFLOW model incorporated inflow boundaries extracted from the larger MIKE21FM model with the addition of rainfall-on-grid excess across the TUFLOW model area. Stormwater drainage assets were included within the model, incorporating key pipes, pits and pumps.

The modelling process involved the following stages:

- Model setup and calibration,
- Validation and sensitivity tests,
- Design flood simulations,

The calibration, validation, and sensitivity assessments are an iterative investigative process and all outcomes from these stages inform the final design flood simulations.

4.3.3 Understanding Flood Behaviour

Table 4-3 describes the key flood characteristics at Seaspray and surrounds for each design event.

Table 4-3 Design Flood Events and Associated Flood Consequences

Design Flood Event (% AEP)	Gauge Height at Prospect Road (m)	Flood Consequences
10	3.78	River flooding confined to banks of Merriman creek and Blind creek and the urban floodway toward Lake Reeve. There is some flow across the floodplains west of Merriman Creek at the Estuary. There is some pooling throughout the town due to localised rainfall with minor impacts houses along Foreshore Rd and Rowley St. Depths of around 200mm on parts of Foreshore Rd and Futchers St. Total Properties Flooded = 56 Properties Flooded above floor level = 0
5	4.82	River flooding confined to banks of Merriman creek and Blind creek and the urban floodway toward Lake Reeve. Some breakout to the north of the Urban Floodway, East of the town, into undeveloped farmland. Pooling throughout Seaspray due to localised rainfall. Depths of around 200mm on Foreshore Rd and parts of Futchers St and Centre Rd. Around 25 houses are within the flood extent. Properties along Rowley St begin to become inundated. Total Properties Flooded = 57 Properties Flooded above floor level = 0
2	5.57	River mostly confined to banks with some flow breaking the banks but remaining close to the main channels. Larger breakouts occurring to the east of the town, to the north of the urban floodway onto low lying farmland. Localised rainfall causing flooding within town. Around 45 houses within the flood extent. Properties along Shoreline Dr and Catton St road begin to become impacted. Deeper flooding along Foreshore Rd, Futchers St and Centre Rd. Total Properties Flooded = 58 Properties Flooded above floor level = 0
1	6.36	River breaking from banks of Merriman creek and Blind Creek adjacent to Griffioens Levee. Breakouts from the Urban Floodway also occur, leading to flow over Government Road Levee from the urban floodway. Deep flows to the east of Seaspray flowing towards Lake Reeve. Overland flow to the west of the Merriman creek towards the estuary. Localised rainfall causing pooled water over a number of properties. Around 195 houses within the flood extent, many between Main Rd and Catton St. Inundation of a large number of roads, with deep water on Centre Rd, Catton St and Hansen

		St. Total Properties Flooded = 177 Properties Flooded above floor level = 37
0.5	NA*	Breakout at turn in Merriman Creek North east of Buckleys Rd. Breakouts from Blind Creek and Merriman Creek adjacent to Griffioens Levee. Breakouts to the west of Merriman Creek moving westward. Seaspray Rd inundation, north of the town, potential isolation. Shoreline drive inundated between Seaspray and The Honeysuckles, causing potential isolation of the Honeysuckles. All houses within Seaspray township within the flood extent except for several along Seaspray Rd, north of Tip Rd, and the south of Bearups St between Lyons St and Bock St. Total Properties Flooded = 322 Properties Flooded above floor level = 180

5. FLOOD RISK AND TREATMENT

5.1 Flood Risk

Flood risk is the product of the consequence and the frequency of a flood event occurring. To assess the flood risk within Seaspray, the hydraulic modelling outputs (depth and velocity) can be used to identify both the frequency of flood impacts (in terms of annual exceedance probability) and the consequence of the magnitude of the flood impacts.

The method used to delineate high flood hazard is broadly based on the new Australian Rainfall and Runoff Project 10 'Appropriate Safety Criteria for People'. Criterion for delineating the flood overlay considers both vehicle and people safety, and are as follows:

- Depth > 0.3 m
- Velocity > 1.5 m/s
- Depth x velocity > 0.3 m²/s.

If any of the above criteria is exceeded, the area is considered to be High Hazard. Low Hazard areas are locations within the flood extent, but do not exceed any of the above criteria. The maximum flood hazard for the 1% AEP flood event is shown in Figure 5-1.



Figure 5-1 High and Low Hazard Areas in the 1% AEP flood event

5.2 Flood Risk Treatment

5.2.1 Flood Warning

Two streamflow gauges located upstream of the township (Prospect Road and Stradbroke West) provide valuable information on the approaching streamflow towards Seaspray. This information can be used to provide a warning to an approaching flood to the Seaspray Township. The time between peak flows at the two gauges can provide a good indication of the travel time and the available flood warning time, this is outlined in Table 5-1.

While Table 5-1 shows the time between peak flows at the streamflow gauges, flood impacts in Seaspray can occur well before the flood peak arrives at the Prospect Road gauge. Localised rainfall can lead to stormwater flooding within Seaspray independently of any catchment flooding in Merriman Creek. High Risk areas, including Centre Road between Catton Street and Newton Street can be inundated from local rainfall, well before the flood peak from the catchment reaches Seaspray. The source of flooding is important to consider for emergency response procedures such as road closures, sandbagging and if required, evacuation.

Table 5-1 Historical Travel Time between Flood Peaks (based on gauge flows)

Historical Event	Time from Peak at Stradbroke West to Peak at Prospect Road
1993	~ 4 Hours
1995	~ 9 Hours
2012	~ 26 Hours

There are currently no flood warning gauge levels for Seaspray despite the presence of stream flow gauges at Prospect Rd and further upstream at Stradbroke West. Based on the analysis undertaken for this study, recommendations are provided in Table 5-2 to establish flood warning levels at the Prospect Road gauge.

Table 5-2 Proposed Flood Warning Levels

Flood Warning Level	Prospect Rd Height
Minor	2.0m
Moderate	3.0m
Major	4.5m

5.2.2 Flood Management Scheme Operation

Regulator Structure

The existing operating procedure for the Seaspray flood mitigation scheme, including the floodway regulator and manual opening of the estuary entrance is detailed in the operating manual maintained by Wellington Shire Council (Wellington Shire Council, 1996).

The intention of the regulator structure is to provide flood protection from both riverine and coastal flooding. In practical terms, if the sand berm is closed the regulator should maintain a pool level at 2.0 m AHD to 2.2 m AHD, which will preferentially direct river flows down the floodway to limit flows overtopping the berm and scouring an entrance channel. This therefore would prevent ingress of water from the ocean during extreme storm tide events. However, the analysis of coastal flooding reported in the Hydraulics Report R03 has concluded that under existing conditions, there is no risk

posed to Seaspray from coastal flooding as the 1% AEP storm tide elevation at Seaspray is 1.5 m AHD under existing mean sea level and therefore the existing levees along Merriman Creek would prevent inundation for storm tides, regardless of the operation of the regulator structure.

The existing operating procedure (Wellington Shire Council, 1996) recommends that for riverine flooding events the regulator structure is to have boards incrementally added manually throughout the duration of the event to limit and control flow down the floodway. In practice this has proven dangerous and impractical, as reported in the failed attempts to place flood boards during the 1993 flood event. Maintaining all boards in place within the regulator structure does however force flood water towards the estuary entrance and encourage earlier natural scouring of the entrance berm if the entrance channel is initially closed.

It is therefore recommended that the operating procedure be updated so that in future all boards remain in place to provide protection against riverine flooding, as there is no flood benefit to removing them during an event.

Entrance Opening

Estuary closure as a result of sedimentation of the entrance and creation of a sand berm is a continued concern for local residents, with pooling of water within the estuary a perceived flood risk if riverine flooding is to occur. Results of sensitivity analysis within the Hydraulics Report (R03) showed that once overtopped, the sand berm will naturally erode within 1-3 hours through natural erosion processes. A safe operating height of 2.1m AHD is proposed as a maximum berm level crest for the sand berm to open naturally without negative flood impacts in Seaspray. It is proposed that the berm be maintained at a maximum height of 2.1m AHD which allows 100mm of freeboard before the earthen weir adjacent the regulator structure is overtopped and water is diverted down the floodway. The height of 2.1m AHD also provides 400mm of freeboard to the crests of the levees along Merriman Creek. If the entrance sand berm is greater than 2.1m AHD, manual opening of the entrance is recommended if weather warnings indicate significant rainfall is likely to occur in the catchment. This differs from the existing operating procedure for the structure as detailed in the operating manual. It is therefore recommended that the operating manual be updated to reflect this revised information.

There is currently no accurate measure of the berm height, although the water level behind the berm is recorded by the gauge at Seaspray. A water level gauge board located at the boat ramp or on the berm would allow Council, WGCMA and the local community to have up to date information on the sand levels and provide a trigger for implementing potential artificial openings of the entrance prior to predicted flood events.

5.2.3 Land Use Planning

The Victoria Planning Provisions (VPPs) contain a number of controls that can be employed to provide guidance for the use and development of land that is affected by inundation from floodwaters. These controls include the Floodway Overlay (FO), the Land Subject to Inundation Overlay (LSIO), the Special Building Overlay (SBO), the Urban Floodway Zone (UFZ) and the Environmental Significance Overlay (ESO).

For the study area, FO and LSIO overlays have been developed based on accepted safety criteria (refer Report 4 for details). The resultant draft FO and LSIO maps are provided in Figure 5-2.



Figure 5-2 Draft LSI0 and FO Map for Existing Conditions

5.2.4 Structural Mitigation Options

Three structural mitigation options were presented to the steering committee and community meeting held in September. The three options are listed below, and are described in further detail in Report 4.

1. **Increase Conveyance of Eastern Prior Stream** – It was proposed to assess whether deepening of Eastern Prior Stream to the invert of the culvert at Seaspray Road would allow water to flow along Eastern Prior Stream and reconnect with Lake Reeve. This aimed to reduce the flood flows directly towards Seaspray.
2. **Increase flood storage along Mason Creek** – It was proposed that additional storage of flood waters could be achieved by utilising the natural depression along Mason Creek towards Lake Denison. The aim was to reduce flood levels within the Merriman Creek estuary and consequently overtopping of the adjacent levees.
3. **Increase Existing Flood Levee Function** - The existing network of levees constructed around Seaspray as part of the flood mitigation scheme have ensured that Seaspray was not impacted during the 1995 or 2012 floods. Design modelling of the 1% AEP flood event, does however, show that the existing levee scheme remains vulnerable to overtopping and breakout flows during a 1% AEP event. Raising of levees in specific locations could be undertaken to eliminate this overtopping.

Modelling results indicated that Options 1 and 2 provide negligible reduction in flood risk to Seaspray and are not effective flood mitigation options.

The levee assessment (Option 3) has identified locations across the levee network in and around Seaspray that are vulnerable to overtopping (Figure 5-3) and how much the vulnerable sections of levee should be raised to provide the required level of flood protection. This work allows a targeted approach to raise underperforming sections of the levee and improve the levee network to mitigate flood levels in the 1% AEP flood event.



Figure 5-3 Vulnerability of existing levees in the 1% AEP flood event

5.3 Flood Damages Assessment

A flood damages assessment was undertaken on existing conditions and is shown in Figure 5-4. The 1% AEP damage calculated was **\$2,259,374** with 36 residential properties flooded above floor and 1 commercial property also flooded above floor. The average annual damage (AAD), a measure of the flood damage, per year over an extended period was estimated for existing conditions to be **\$46,124**. The AAD is an estimate of the cost of flooding to the community that includes both public and privately owned assets.

Damages were costed for mitigation Option 3, the upgrade of the existing levee mitigation scheme. This option aims to prevent flooding of Seaspray under the 1% AEP design flow from Merriman Creek. It does not address local stormwater flooding. The results (Figure 5-5) showed that the number of residential properties flooded above floor in a 1% AEP event are reduced to 4, and no commercial properties are now flooded above floor level. The flooding of these properties is limited to localised rainfall and stormwater flooding. This reduced the flood damage estimate for a 1% AEP flood event to **\$429,895**. The AAD was also reduced to **\$33,413**.

EXISTING CONDITIONS					
ARI (years)	200yr	100yr	50yr	20yr	10yr
AEP	0.005	0.01	0.02	0.05	0.1
Residential Buildings Flooded Above Floor	166	36	0	0	0
Commercial Buildings Flooded Above Floor	14	1	0	0	0
Properties Flooded Below Floor	142	177	58	57	56
Total Properties Flooded	322	214	58	57	56
Direct Potential External Damage Cost	\$1,218,495	\$738,020	\$60,090	\$48,791	\$48,733
Direct Potential Residential Damage Cost	\$9,992,403	\$1,997,493	\$0	\$0	\$0
Direct Potential Commercial Damage Cost	\$170,813	\$3,779	\$0	\$0	\$0
Total Direct Potential Damage Cost	\$11,381,711	\$2,739,292	\$60,090	\$48,791	\$48,733
Total Actual Damage Cost (0.8*Potential)	\$9,105,369	\$2,191,434	\$48,072	\$39,033	\$38,986
Infrastructure Damage Cost	\$160,563	\$67,940	\$35,906	\$32,534	\$19,876
Indirect Clean Up Cost					
Indirect Residential Relocation Cost					
Indirect Emergency Response Cost					
Total Indirect Cost	\$0	\$0	\$0	\$0	\$0
Total Cost	\$9,265,932	\$2,259,374	\$83,978	\$71,567	\$58,862

Figure 5-4 Existing Condition Flood Damages Summary

MITIGATION CONDITIONS					
ARI (years)	200yr	100yr	50yr	20yr	10yr
AEP	0.005	0.01	0.02	0.05	0.1
Residential Buildings Flooded Above Floor	158	4	0	0	0
Commercial Buildings Flooded Above Floor	1	0	0	0	0
Properties Flooded Below Floor	125	212	58	57	56
Total Properties Flooded	284	216	58	57	56
Direct Potential External Damage Cost	\$462,331	\$324,188	\$60,090	\$48,791	\$48,733
Direct Potential Residential Damage Cost	\$11,480,148	\$168,532	\$0	\$0	\$0
Direct Potential Commercial Damage Cost	\$19,781	\$0	\$0	\$0	\$0
Total Direct Potential Damage Cost	\$11,962,260	\$492,720	\$60,090	\$48,791	\$48,733
Total Actual Damage Cost (0.8*Potential)	\$9,569,808	\$394,176	\$48,072	\$39,033	\$38,986
Infrastructure Damage Cost	\$100,203	\$35,719	\$35,906	\$32,534	\$19,876
Indirect Clean Up Cost					
Indirect Residential Relocation Cost					
Indirect Emergency Response Cost					
Total Indirect Cost	\$0	\$0	\$0	\$0	\$0
Total Cost	\$9,670,011	\$429,895	\$83,978	\$71,567	\$58,862

Figure 5-5 Levee Mitigation Flood Damages Summary

The levee improvement mitigation option shows a positive benefit to cost ratio due to the relatively limited extent of levee raising required and associated costs compared to the reduction of flood impacted properties in the 1% AEP flood event (Table 5-3). The magnitude of the 0.5% AEP flood event is such that raising the existing levees is not able to prevent Seaspray from becoming inundated. Overbank flow from Blind Creek continues to outflank the existing levee arrangement with breakout flows occurring all the way along Blind Creek to Eastern Prior stream.

Table 5-3 Benefit-Cost Analysis of Levee Mitigation Scenario

	Existing Conditions	Levee Mitigation Option
Annual Average Damages	\$46,124	\$33,413
Annual Maintenance Cost		\$1,559
Annual Cost Saving		\$11,152
Net Present Value (6%)		\$156,821
Capital Cost of Mitigation		\$83,781
Benefit-Cost Ratio		1.87

6. STUDY DELIVERABLES

6.1 Overview

The study deliverables provide a comprehensive set of data that support the study outcomes. The deliverables are supplied on a study USB and consist of background data and outputs as listed below:

- Digital copies of study reports in PDF format.
- Study survey data (LIDAR, structures, cross-sections and floor levels)
- Other input data including rainfall and flow data
- A property database including flood information
- Digital copies of the maps (PDF format)
- GIS datasets for the model results (ArcGIS format)
- The hydrologic and hydraulic model input files

There is a readme.txt file on the disk that describes the directory structure of the data contained on the disk.

6.2 Mapping Outputs

Details are provided of the study outputs for emergency response, and land use planning mapping including:

- Data sets: grids and shapefiles
- Planning layer
- Flood response inundation maps
- VFD layer updates

6.2.1 Data Sets

The following datasets have been provided. All GIS files were provided in ESRI format.

Grids

Gridded datasets of model results were provided for the following:

- PMF – maximum hazard and water surface elevation,
- Design events (10%, 5%, 2% 1% & 0.5% AEP events) – maximum depth, hazard, velocity and water surface elevation.
- September 1993 Calibration event – maximum depth, water surface elevation and extent

Shapefiles/Tabfiles

ESRI shapefiles were provided for the following:

- Flood depth contours
- Flood extents
- Floor levels
- Mapping limits
- Water surface elevation (flood level) contours

6.2.2 Maps

The flood response inundation maps have been produced for the following design flood events:

- PMF – maximum depth and hazard,
- Flood Hazard - 10%, 5%, 2% 1% & 0.5% AEP events,
- Flood Depth - 10%, 5%, 2% 1% & 0.5% AEP events,
- Flood Velocity - 10%, 5%, 2% 1% & 0.5% AEP events,
- Flood Levels - 10%, 5%, 2% 1% & 0.5% AEP events.

Each map includes:

- Flood extent,
- Flood level contour at 0.2 m and 1m intervals,
- Depth of inundation,
- Identification of essential services,
- Road/street names
- Cadastral base
- Land marks, including all physical man-made features particularly those affecting flood flows and distribution.

Soft copies were provided as PDFs. Related GIS files were provided in ESRI format.

6.2.3 Flood Extent Mapping (VFD Compliant)

All flood mapping data was prepared to the VFD metadata specifications.

6.2.4 Land Use Planning Maps

A draft FO and LSIO map has been produced. A copy of this map is included on the study USB.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Overview

The Seaspray Flood Study provides a comprehensive analysis and review of flood risk in and around the township of Seaspray. The study has involved:

- Collection and review of a range of data relevant to the definition of flooding within the study area.
- A rigorous hydrologic analysis to develop robust design flood estimates for the study.
- Development of a detailed hydraulic model that is capable of predicting flood impacts from a range of mechanisms across the entire floodplain under a range of conditions.
- Thorough sensitivity testing of the hydraulic results including the assessment of relative impacts associated with the different flood mechanisms including riverine flooding, elevated ocean water levels, stormwater and flooding from the Gippsland Lakes via Lake Reeve.
- Design and calibration model result outputs.
- Examination of a range of potential risk management options including structural mitigation measures for Seaspray.
- Quantification of flood risk in terms of flood damages.

7.2 Key Outcomes

In undertaking this study a number of important aspects of flood risk relevant to Seaspray have become apparent. These are summarised as follows.

Hydrology & Flooding Mechanisms – flooding in the Merriman Creek catchment is primarily a function of sustained, high intensity rainfall events in the upper catchment over a period of 36 – 48 hours. Coastal flooding and flooding from the Gippsland Lakes through increased water levels in Lake Reeve are successfully mitigated through the existing levee network and do not result in inundation of Seaspray under existing conditions. Stormwater flooding from local rainfall can create localised pooling of water within Seaspray for events of greater magnitude than the 10% AEP rainfall event.

Hydraulic Characteristics of Flooding at Seaspray – Peak flood levels in 2% AEP and smaller flood event are well confined within Merriman Creek and the floodway. The existing levee network prevents overbank flows from Merriman Creek inundating Seaspray in events equal to and smaller than the 2% AEP flood event. The 1% AEP event results in breakout flows flanking Griffioens Levee at the junction of Merriman Creek and Blind Creek, leading to inundation of Seaspray and water ponding on the inside of flood mitigation levees. Government Road levee is also overtopped from the floodway. Severe levee failure and inundation across most of Seaspray occurs in the 0.5% AEP flood event.

Flood Risk Treatment & Mitigation – locations along the existing levee network that are vulnerable to overtopping in the 1% AEP flood event have been identified. A targeted approach to raising vulnerable sections, in addition to an extension of Griffioens Levee along the bank of Blind Creek, can result in significant reduction of flood damages for the 1% AEP flood event.

Operation of the existing regulator structure and an updated entrance opening procedure would also aid the management of flood risk to the community.

7.3 Recommendations

Following the investigations undertaken for the study and the conclusions reached it is recommended that:

- The WGCMA and Wellington Shire undertake a planning scheme amendment process to incorporate new LSIO and FO mapping into the Wellington Planning Scheme as soon as possible.
- The WGCMA and Wellington Shire consider all recommendations provided within the accompanying “Wellington Planning Scheme Flood Controls Review – Seaspray Flood Investigation” provided by Planning and Environmental Design, for inclusion into a revision of the Wellington Planning Scheme.
- Hydraulic modelling be undertaken to assist Ventia in developing ‘level to level’ correlation between the Gippsland Water pump location gauge and the existing Prospect Road gauge
- Wellington Shire Council update the existing operating procedure for the regulating structure at Seaspray to reflect the learning from this study and confirm responsibilities for implementation of the procedure during flood events. This may include installation of a new water level/berm height gauge near the Merriman Creek entrance area.
- The Wellington Shire Council and WGCMA continue to engage the community in the treatment of flood risks through regular flood awareness programs such as the VICSES FloodSafe program, starting with the development of a local flood guide.
- The Wellington Council and WGCMA explore further the recommendations for enhanced flood response through co-operation with SES and Police, utilising the flood inundation maps and flood intelligence tools included in the Municipal Flood Emergency Plan (MFEP). Consideration should be given to the use of the MFEP during an emergency.
- The Wellington Shire Council and WGCMA explore further the recommendations for the development of community portal for flood information at Seaspray in conjunction with the BoM and SES.

8. REFERENCES

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MEMORANDUM

To

From

Date 02 December 2016

Subject Seaspray Sea Level Rise Addendum

The following memo outlines a series of additional modelling scenarios undertaken to assess potential sea level rise impacts on flooding in the township of Seaspray. This work has been undertaken as an addendum to the Seaspray Flood Study (Water Technology, 2016).

1 INTRODUCTION

As part of the recent flood study investigation for Seaspray (Water Technology, 2016), it was determined that under existing conditions catchment derived flows dominate flooding of the township. Subsequent to the study, Water Technology has been requested to further investigate the impact of sea level rise (SLR) on future inundation extents across the township.

2 SCENARIOS

Four simulations were undertaken to determine the impacts of sea level rise on inundation of Seaspray. These were:

1. 1% AEP catchment flows + 0.8 m SLR
2. 1% AEP catchment flows + 10% AEP storm tide elevation
3. 1% AEP catchment flows + 0.8 m SLR + 10% AEP storm tide elevation
4. 1% AEP catchment flows + 0.8 m SLR + 1% AEP storm tide elevation

The original hydraulic model developed for the flood study was utilized in this modelling, with modifications only made to the ocean boundary conditions.

3 BOUNDARY CONDITIONS

The model domain and boundary locations is illustrated in Figure 3-1. Inflow hydrographs were applied to Merriman Creek at the Prospect Road Gauge, and a water level boundary used offshore to represent the tide, mean sea level, and storm surge.

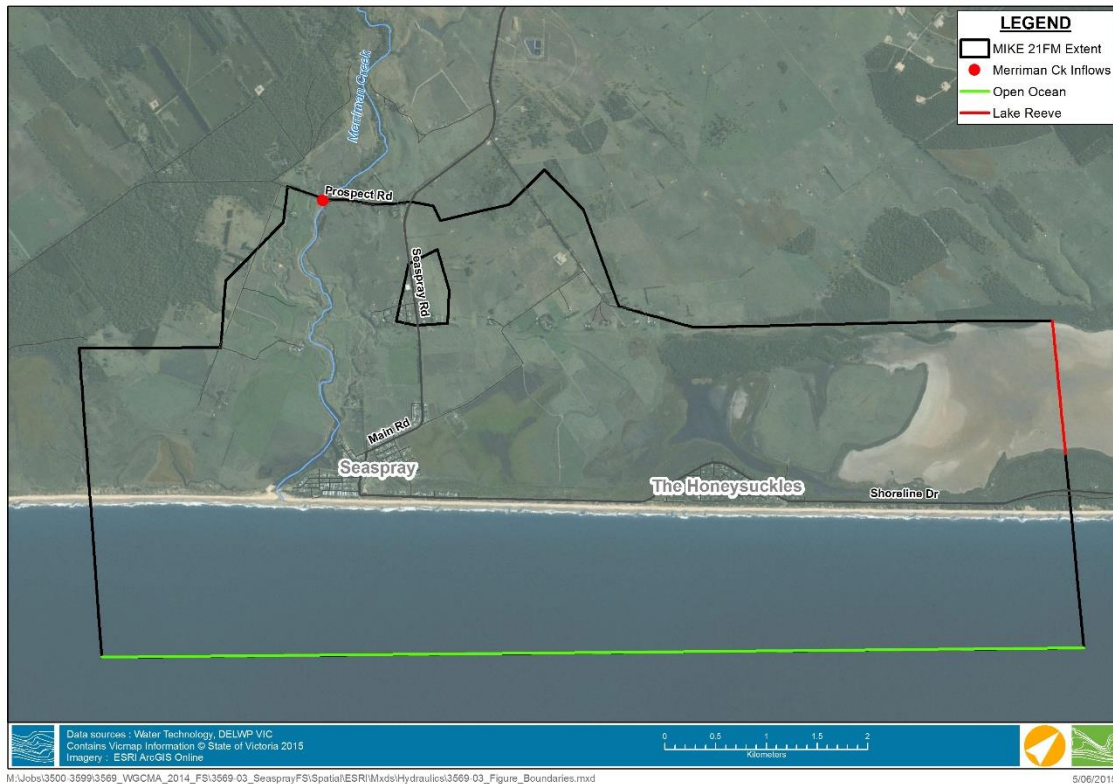


Figure 3-1 Model Boundaries

3.1 Catchment Flows

The hydrograph for the 1% AEP catchment flow derived in the original flood study work was applied for this modelling (Figure 3-2). The hydrograph peaks at around 470 cumecs (40,750 ML/d) 28 to 30 hours into the simulation, coinciding with the peak storm tide.

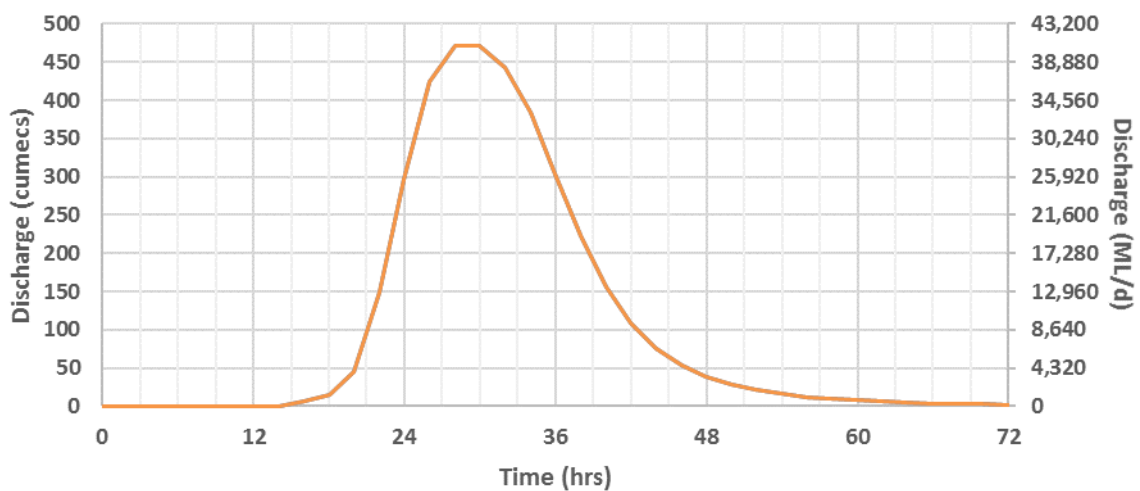


Figure 3-2 1% AEP Inflow Hydrograph



3.2 Lake Reeve

The eastern boundary bordering Lake Reeve had a constant water level equal to the mean sea level applied for these simulations. A mean sea level of 0 m AHD was used for the 10% AEP storm surge simulation, while a mean sea level of 0.8 m AHD was used for the other two simulations to represent mean sea level rise to the end of the century.

3.3 Offshore Boundary

The predicted 10% AEP storm tide at 2100 of 2.33 m AHD and predicted 10% storm surge at 2100 of 1.96 m were adopted from McInnes et. al. (2009), future climate scenario 2 (IPCC 2007 A1F1 scenario in combination with 'high' wind speed scenario). These values are for storm tide and storm surge levels at Seaspray. McInnes (2009) used a 2100 scenario applying a sea level rise increment of +0.82 m rather than the +0.8 m currently recommended for planning purposes in Victoria. For this assessment it is considered that the difference is negligible in terms of potential impact on coastal inundation. We have reported the results using the +0.8 m sea level rise increment to avoid confusion with current planning practice.

The offshore water levels for the three scenarios are illustrated in Figure 3-3.

- Scenario 1 employs a tidal signal raised by 0.8 m to represent the predicted sea level conditions at 2100.
- Scenario 3 combines this tidal signal with a sine curve, representing a storm surge. The peak water level in this scenario is derived from the 10% AEP storm tide level of 2.33 m AHD defined in McInnes et. al. (2009).
- Scenario 2 employs the same water level as Scenario 3 with a vertical shift down of 0.8 m to represent the same storm surge event in the absence of sea level rise. This results in a peak water level approximately 0.3 m higher than the current 10% AEP storm tide. All four scenarios have a peak water level offshore water level occurring at the same time, and the same tidal signal underlying the boundary. This allows the impact of sea level rise and storm surge to be directly compared. The peak water level in this scenario (1.51 m AHD) is very similar to that of the current 1% AEP storm tide (1.50 m AHD).
- Scenario 4 combines a tidal signal with a sine curve, representing a storm surge to produce a storm tide peak water level of 2.64 m AHD as defined by McInnes et. al. (2009) for the 2100 1% AEP storm tide. This includes a sea level rise of 0.82 m as with all 2100 projections in McInnes et. al. (2009).

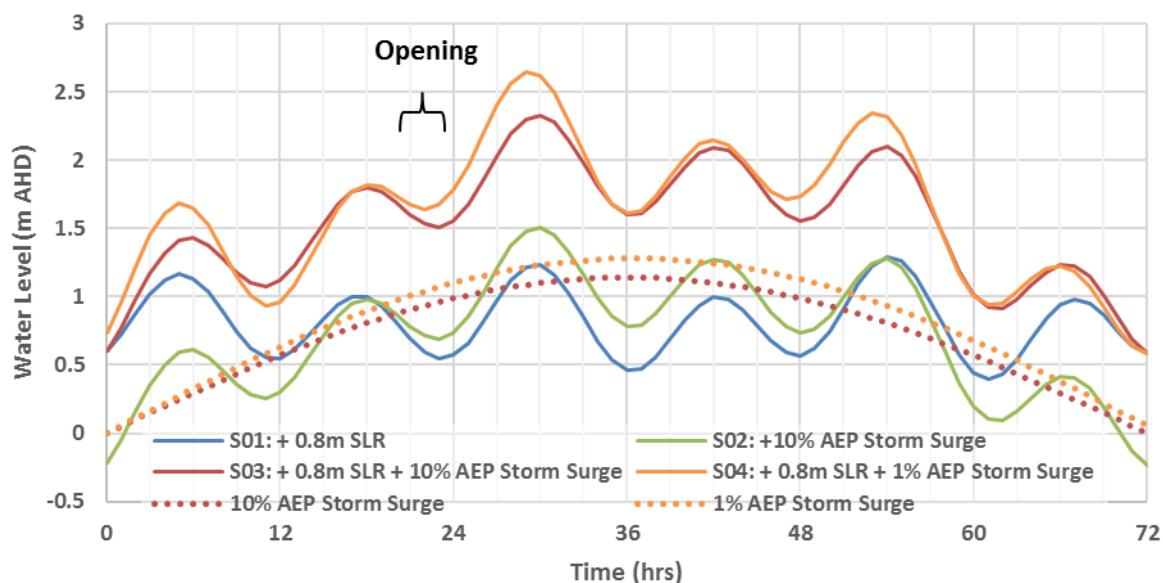


Figure 3-3 Offshore Water Level Boundaries



3.4 Entrance Conditions

Entrance conditions used in this modelling were consistent with design simulations undertaken in the Seaspray Flood Study (Water Technology, 2016). The entrance berm was initially closed with a height of 2.1 m AHD, opening over a 3-hour period to an elevation of 0 m AHD. The opening of the entrance starts from when water levels inside the estuary reach the top of the berm (2.1 m AHD) approximately 20 hours into the simulation. This means that by the time the peak storm tide occurs the entrance is completely open. The same timing for the opening was applied in all three scenarios.

Further details on the effects of entrance condition on flood level is provided in the flood study report.

4 ASSUMPTIONS

A number of simplifying assumptions have been identified in this modelling and are discussed here.

4.1 Entrance Berm Conditions

The same entrance berm conditions were applied for all model simulations, including initial berm height and shape, and rate of opening. The processes and specific geometry of entrance opening and closing are complex, as discussed in the flood study report. The berm is likely to undergo some adjustment in response to mean sea level rise, such as an increase in the mean berm height when closed, or lateral translation of the entrance berm into the estuary. Any such adjustments to berm conditions have not been included in this study.

4.2 Timing of Catchment Flows and Peak Storm Tide Water Level

All scenarios have considered the same timing of both catchment flows and peak storm tide water levels, allow simple comparison between each scenario. Timing of flood and storm tide peaks can vary greatly, however, for the purposes of this modelling the timing has been aligned to match peak catchment flows and storm tide water levels, producing the worst-case timing for each scenario.

4.3 Lake Reeve Water Levels

A constant water level boundary has been applied on the Lake Reeve side for all scenarios. For scenarios 1, 3, and 4, which included a sea level rise assumption of 0.8 m, a constant water level of 0.8 m AHD was applied, while Scenario 2 had a water level of 0.0 m AHD (no sea level rise). The impact of the water level applied at this boundary was minimal, with flood waters reaching Lake Reeve after water levels peak in Seaspray.

5 RESULTS

The results from scenario modelling are discussed in this section. Maximum water surface elevation, depth, and velocity figures for each scenario are presented in Appendix A. The 1% AEP and 0.5% AEP flood extents from the original modelling are included for reference. The original modelling included 1% AEP and 0.5% AEP catchment flow respectively, with the current 1% AEP storm tide at the ocean boundary. The gridded maximum data is also provided along with this memo.

5.1 Scenario 1 – 1% AEP catchment flows + 0.8 m SLR

Scenario 1 resulted in a similar flood extent as the original 1% AEP modelling (Appendix A). This scenario considered a tidal signal raised by 0.8 m to represent sea level rise, resulting in higher water levels offshore and in Lake Reeve prior to the flood event, and a peak water level offshore of 1.23 m AHD following the hydrograph peak. The higher initial water levels appear to have little impact on flood extents, with flood waters only reaching Lake Reeve around the time when the peak of the hydrograph is reaching the entrance meaning the tail water in Lake Reeve has minimal impact on the flooding in Seaspray.



The area north of the floodway is impacted by riverine flooding from the north west around the bottom end of Blind Creek. The floodway is filled from the west, driven by high water levels south of Blind Creek. When the levees on the northern side of the floodway are breached from the north, this leads to a breach on the southern side also, as was observed in the original modelling. The resulting peak water level in the floodway was less than in the 1% AEP design simulation, leading to reduced overtopping of Shoreline Road Levee and water levels east of the caravan park 0.17 m lower. Maximum velocities throughout the model were very similar to the original 1% AEP modelling.

High tides can be expected to exceed 1.50 m AHD approximately 4 time per month with 0.8 m SLR, and these occurrences can be predicted well in advance.

5.2 Scenario 2 – 1% AEP catchment flows + 10% AEP storm tide elevation

Scenario 2 is very similar to the 1% AEP design scenario from the original flood modelling owing to the similarity between peak storm tide levels (Appendix A). This scenario considered a peak offshore water level of 1.51 m AHD, corresponding to the 2100 10% AEP storm tide less sea level rise, while the original modelling considered a peak offshore water level of 1.50 m AHD, corresponding to the existing 1% AEP storm tide.

This slight increase in water level was reflected in the lower section of the river, downstream of Blind Creek where water levels were around 1 to 2 mm higher. There was also a small increase in overtopping of Irving Street levee resulting in water levels being 0.02 m higher.

This scenario differentiates the flooding impacts associated with sea level rise and from storm surge components. Results of this modelling suggest that the absolute tail water level rather than mechanism (sea level rise or storm surge) impacts on flooding in the southern area of Seaspray.

A key difference between these scenarios is the probability and predictability, with a 10% AEP storm tide having a 10% probability of occurring in any one year, while high tides can be expected to exceed 1.50 m AHD approximately 4 time per month by the end of the century with 0.8 m SLR.

5.3 Scenario 3 – 1% AEP catchment flows + 0.8 m SLR + 10% AEP storm tide elevation

Scenario 3 produced significantly greater flood extents and depths through Seaspray than either Scenario 1 or Scenario 2 (Appendix A). It produced significantly more flooding through town than the 1% AEP design catchment flow scenario from the original flood modelling which included the current 1% AEP storm surge. The impact of the increased tail water level tapers from the downstream end up the river. Increases in peak water levels along Trood Street Levee reduce from 0.20 m at the southern end to 0.09 m at the northern end on the outside of the floodway spillway. From the floodway spillway to Fatcher street, the increase in peak water level in the floodway reduces from 0.11 m to 0.09 m. East of Fatcher Street the increase in peak water level in the floodway remains below 0.06 m.

Flood depths inside the Trood Street Levee increased by 0.50 m, and inside Shoreline Road Levee increased 0.26 m. Most properties south of the floodway were directly impacted by flooding in this scenario and all would have access roads inundated. Flood depths north of the floodway increased by less than 0.05 m with the exception of a small area bordered by the floodway, Fatcher Street, and Hansen Street, with an increase of 0.06 m.

Peak flow velocities were around 0.10 m/s slower than the 1% AEP design scenario in the main river channel south of Blind Creek, as could be expected from an increased tail water level. Velocities in the floodway were around 0.10 m/s faster, and the floodway was filled in this case from the spillway rather than from flows entering from the north.

In this scenario breaching of the Irving Street and Shoreline Road Levees was initiated prior to floodwaters overtopping the levees on the northern side of the floodway from the north.



5.4 Scenario 4 – 1% AEP catchment flows + 0.8 m SLR + 1% AEP storm tide elevation

Scenario 4 considered the combined 1% AEP catchment flow combined with a 1% AEP storm tide towards the end of the century with 0.8 m SLR. This scenario resulted in flood extents similar to the design 1% AEP catchment flow (with existing 1% AEP storm tide) north of the floodway, and extents closer to the design 0.5% AEP catchment flow (with existing 1% AEP storm tide) south of the floodway (Appendix A).

The flooding mechanism for this scenario is similar to Scenario 3, with breaching of the levee on the southern side of the floodway initiating prior to catchment flows breaching the levee on its northern side. The opening of the berm at the entrance was initiated by water levels within the estuary reaching 2.1 m as with all the other scenarios considered.

5.5 Comparison of Scenarios

Flood depths and extents resulting from Scenarios 1 and 2 were very similar to each other, and also similar to the 1% AEP design scenario from the original flood modelling which considered a 1% AEP catchment flow combined with the existing sea level 1% AEP storm tide. This main difference is that these results indicate a significant increase in probability of flooding in town towards the end of the century, with high tides alone producing tail water levels equal to the existing 1% AEP storm tide (1.5 m AHD) 4 times a month with 0.8 m SLR.

The similarity between flood extents in Scenarios 1 and 2 suggests that the absolute tail water level rather than mechanism (sea level rise or storm surge) impacts most on flooding in the southern part of Seaspray.

Scenario 3 produced increased flooding compared to Scenarios 1 and Scenario 2, especially in the area south of the floodway. The high water levels resulting from the combined sea level rise and 10% AEP storm tide event to the end of the century will have a significant impact on flooding in Seaspray.

Scenario 4 produces further increased flooding south of the floodway compared to Scenario 3. Flood extents north of the floodway match closely with the original modelling 1% AEP flood event which considered the existing 1% AEP storm tide, while flood extents south of the floodway match more closely with the 0.5%. This shows the impact of tail water conditions are much greater south of the floodway than north.

5.6 Summary

Key finds of this sea level rise flood assessment are as follows:

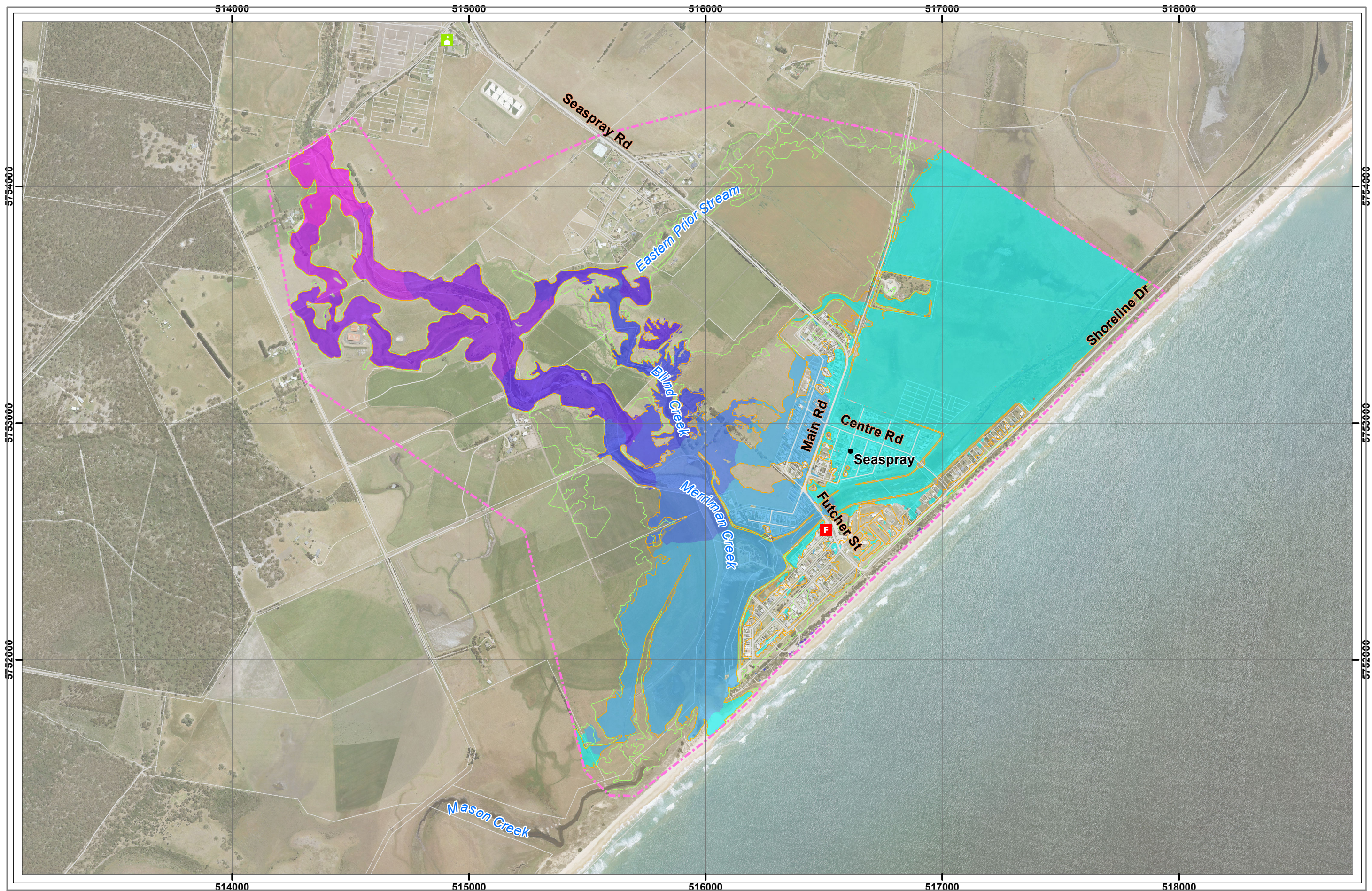
1. The entire area south of the floodway is considered to be at risk of inundation under a 1% AEP event by the end of the century.
2. Increased tail water levels due to storm surges and sea level rise result in increased flooding of Seaspray township, especially south of the floodway.
3. Any raising of levees associated with the current flood protection arrangements in Seaspray should consider the additional requirements associated with sea level rise and storm tide towards the end of the century. The levees can be designed to be upgraded as sea level rise occurs.



APPENDIX A FLOOD MODELLING RESULTS







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Legend:

- Fire Station
- School
- Major Road
- Minor Road
- Extent of Flood Mapping
- 1% AEP Flood Extent
- 0.5% AEP Flood Extent

Elevation (m AHD)

2 3 4 5 6 7 8 9

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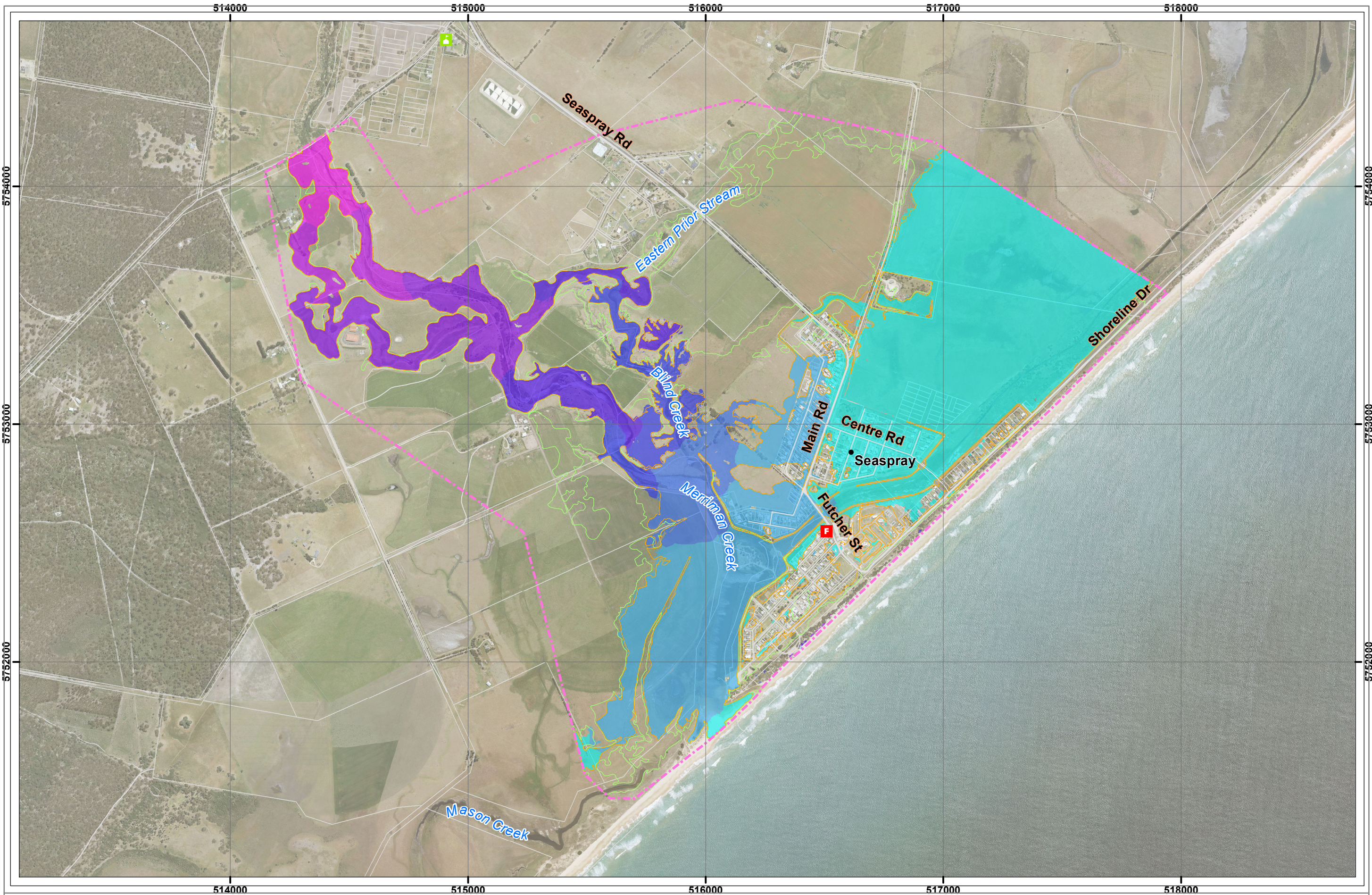
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0 100 200 300 400 500 Meters

Maximum Water Surface Elevation
S01: 1% AEP Flow + 0.8 m SLR

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- Fire Station
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- Major Road
- Minor Road
- Extent of Flood Mapping
- 1% AEP Flood Extent
- 0.5% AEP Flood Extent

Elevation (m AHD)

2 3 4 5 6 7 8 9

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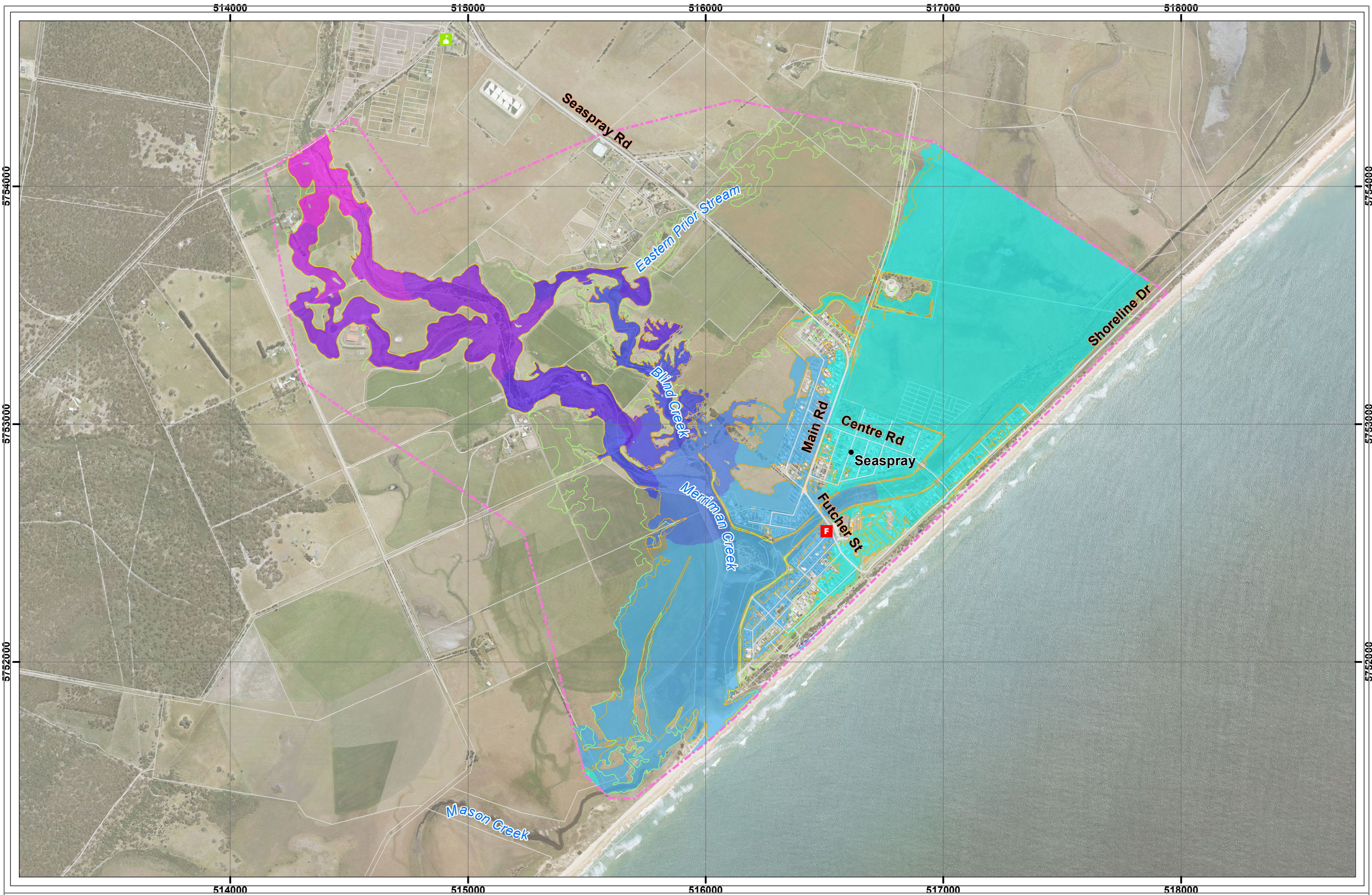
1:15,000 at A3

0 100 200 300 400 500 Meters

Maximum Water Surface Elevation
S02: 1% AEP Flow +10% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S03-082mSLR-10pcST_h.mxd

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Legend:

- Fire Station
- School
- Major Road
- Minor Road
- Extent of Flood Mapping
- 1% AEP Flood Extent
- 0.5% AEP Flood Extent

Elevation (m AHD)

2 3 4 5 6 7 8 9

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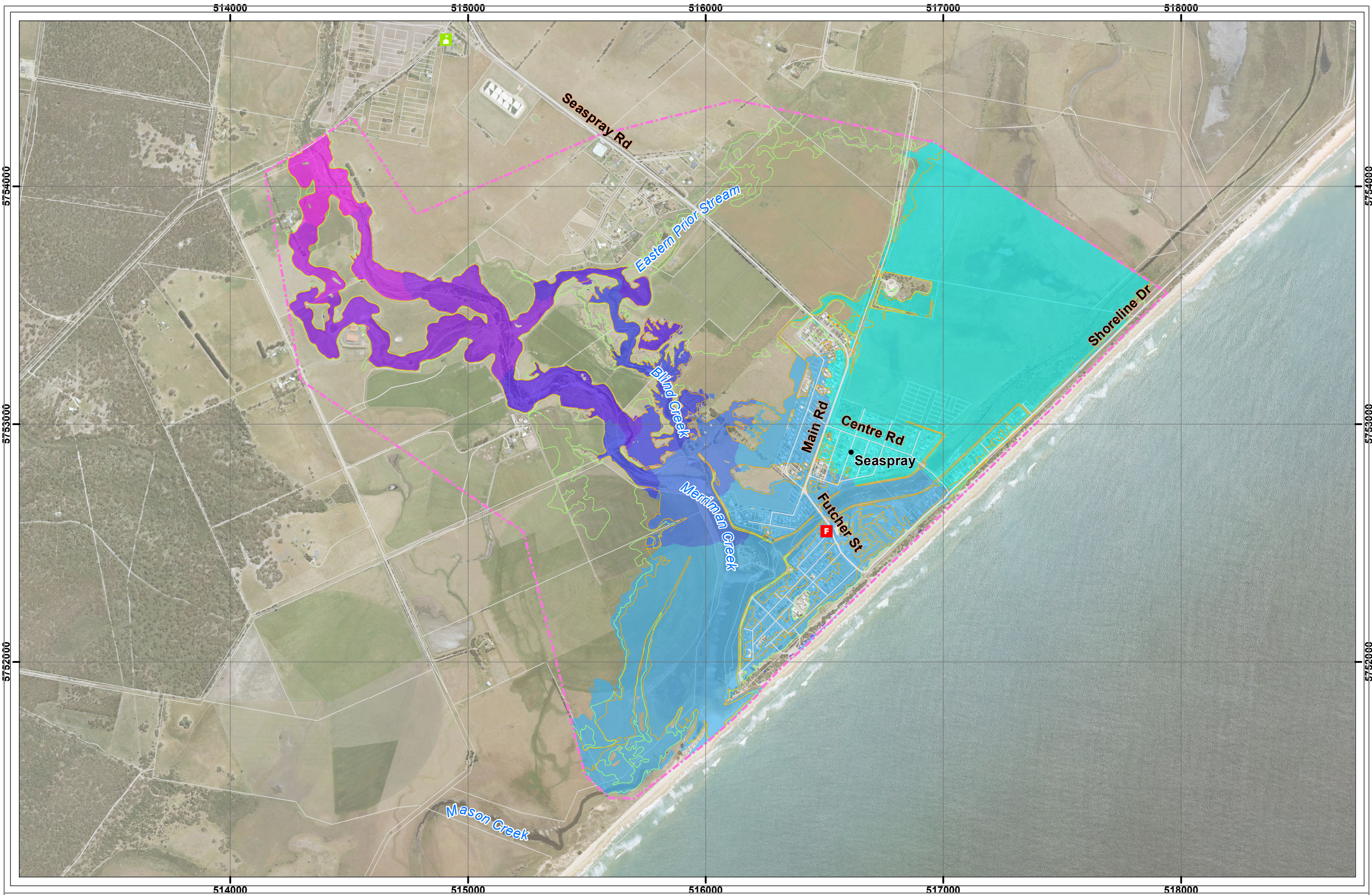
1:15,000 at A3

0 100 200 300 400 500 Meters

Maximum Water Surface Elevation
S03: 1% AEP Flow + 0.8 m SLR + 10% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S04-082mSLR-1pcST_h.mxd

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Legend:

- F** Fire Station
- S** School
- Major Road
- Minor Road
- Extent of Flood Mapping
- 1% AEP Flood Extent
- 0.5% AEP Flood Extent

Elevation (m AHD)

2 3 4 5 6 7 8 9

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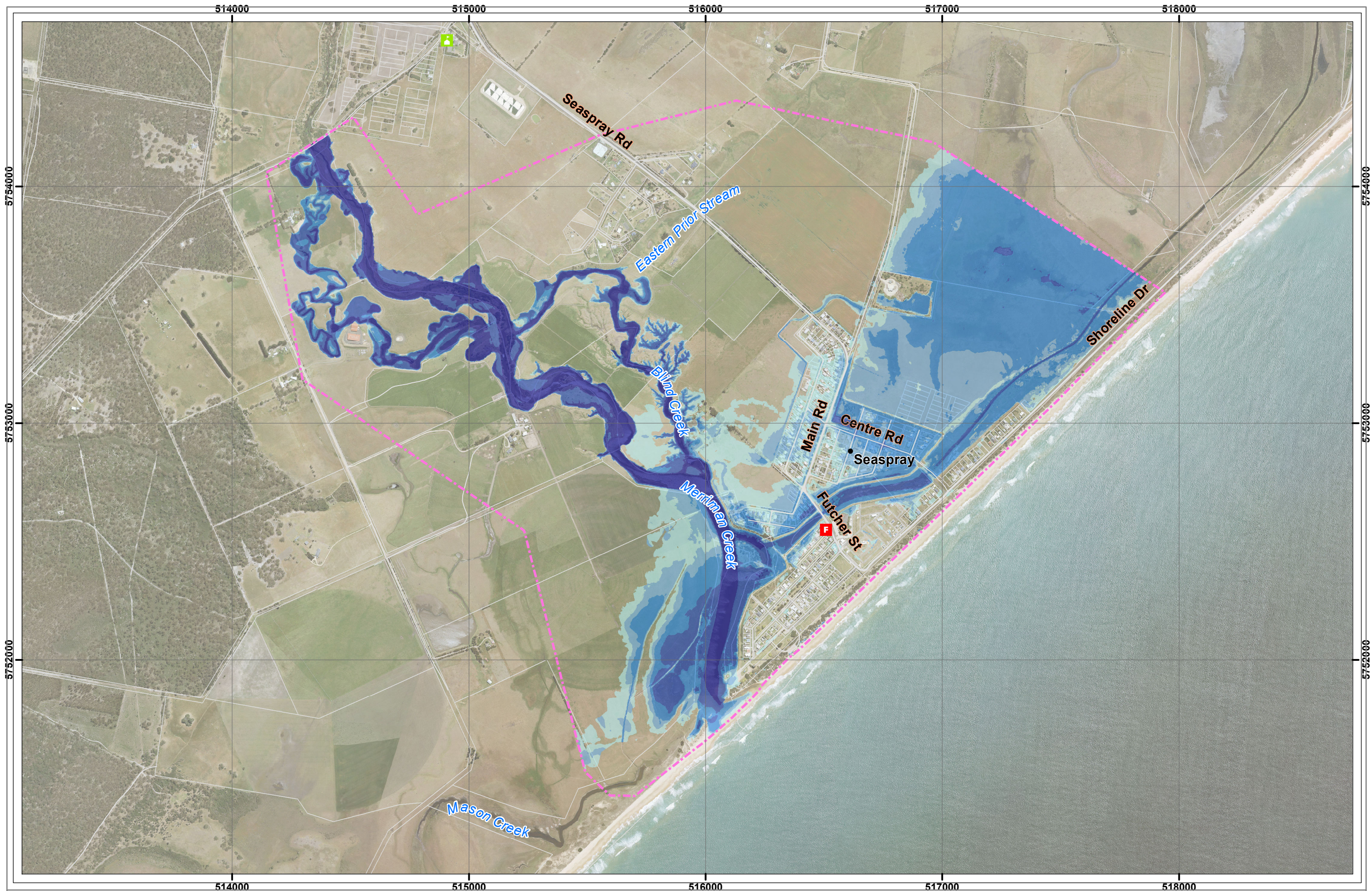
1:15,000 at A3

0 100 200 300 400 500 Meters

Maximum Water Surface Elevation
S04: 1% AEP Flow + 0.8 m SLR + 1% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S04-082mSLR-1pcST_d.mxd

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Legend:

- Fire Station
- School
- Major Road
- Minor Road
- Extent of Flood Mapping

Flood Depth (m)

<0.25 <0.50 <1.00 <2.00 >2.00

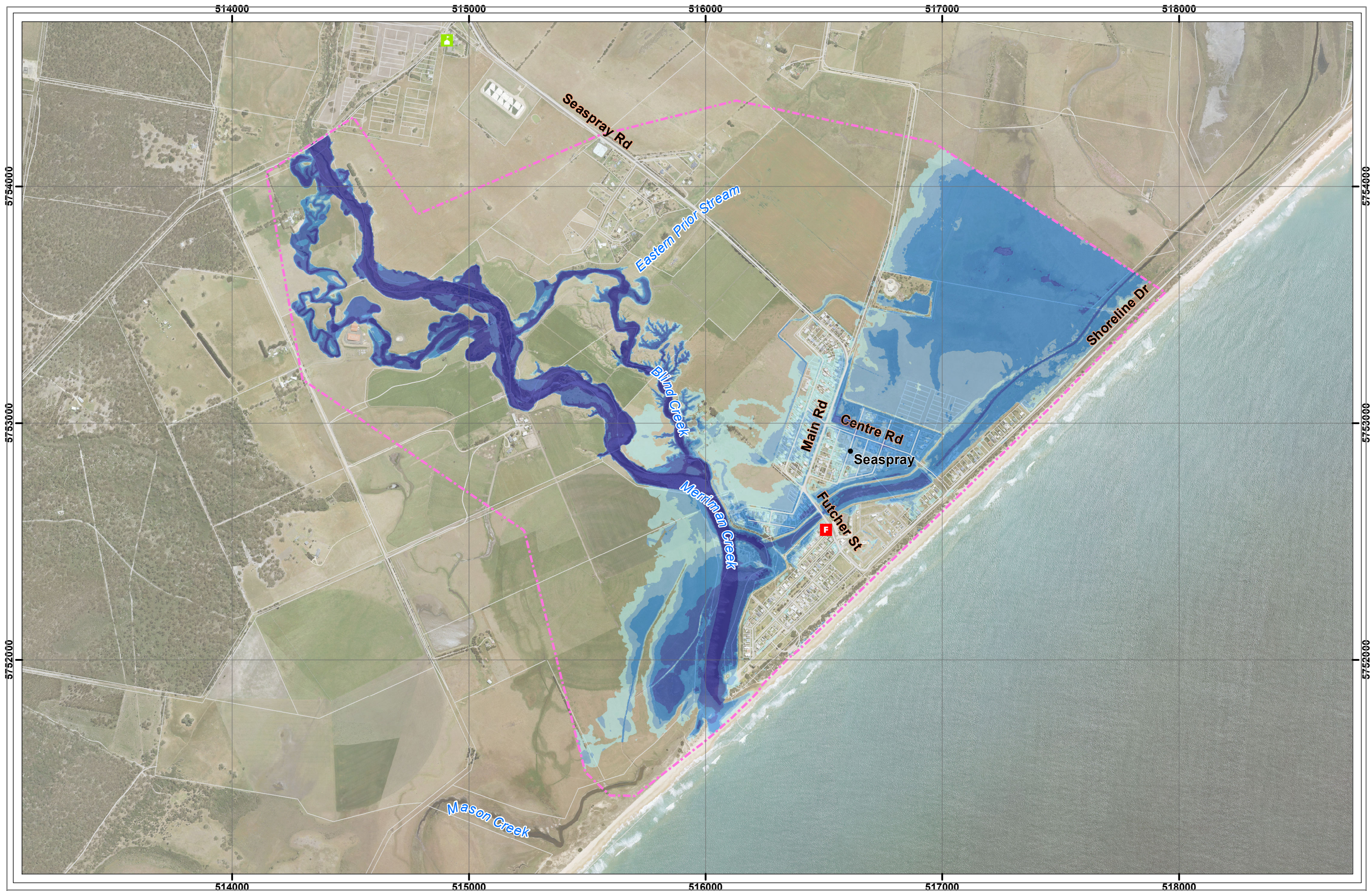
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1:15,000 at A3

Meters

Maximum Depth
S01: 1% AEP Flow + 0.8 m SLR

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRIMxds\4730_Mapping_A3_S01-082mSLR_v.mxd
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Legend:

- F** Fire Station
- S** School
- Major Road
- Minor Road
- Extent of Flood Mapping

Flood Depth (m)

<0.25 <0.50 <1.00 <2.00 >2.00

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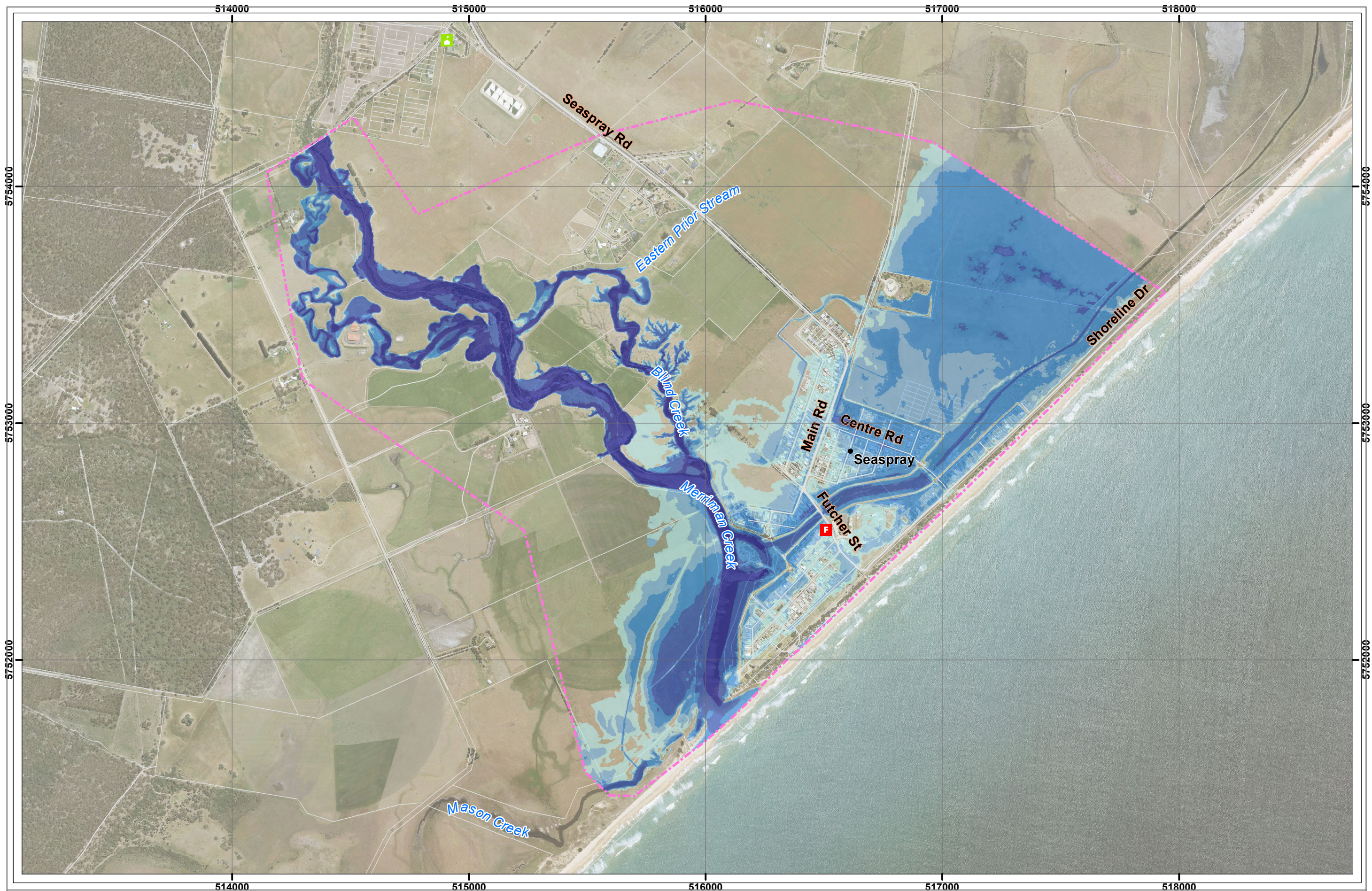
1:15,000 at A3

Meters

Maximum Depth
S02: 1% AEP Flow + 10% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S02-10pcST_d.mxd

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F Fire Station **S** School **---** Major Road **---** Minor Road **---** Extent of Flood Mapping

Flood Depth (m)

< 0.25 < 0.50 < 1.00 < 2.00 > 2.00

1:15,000 at A3

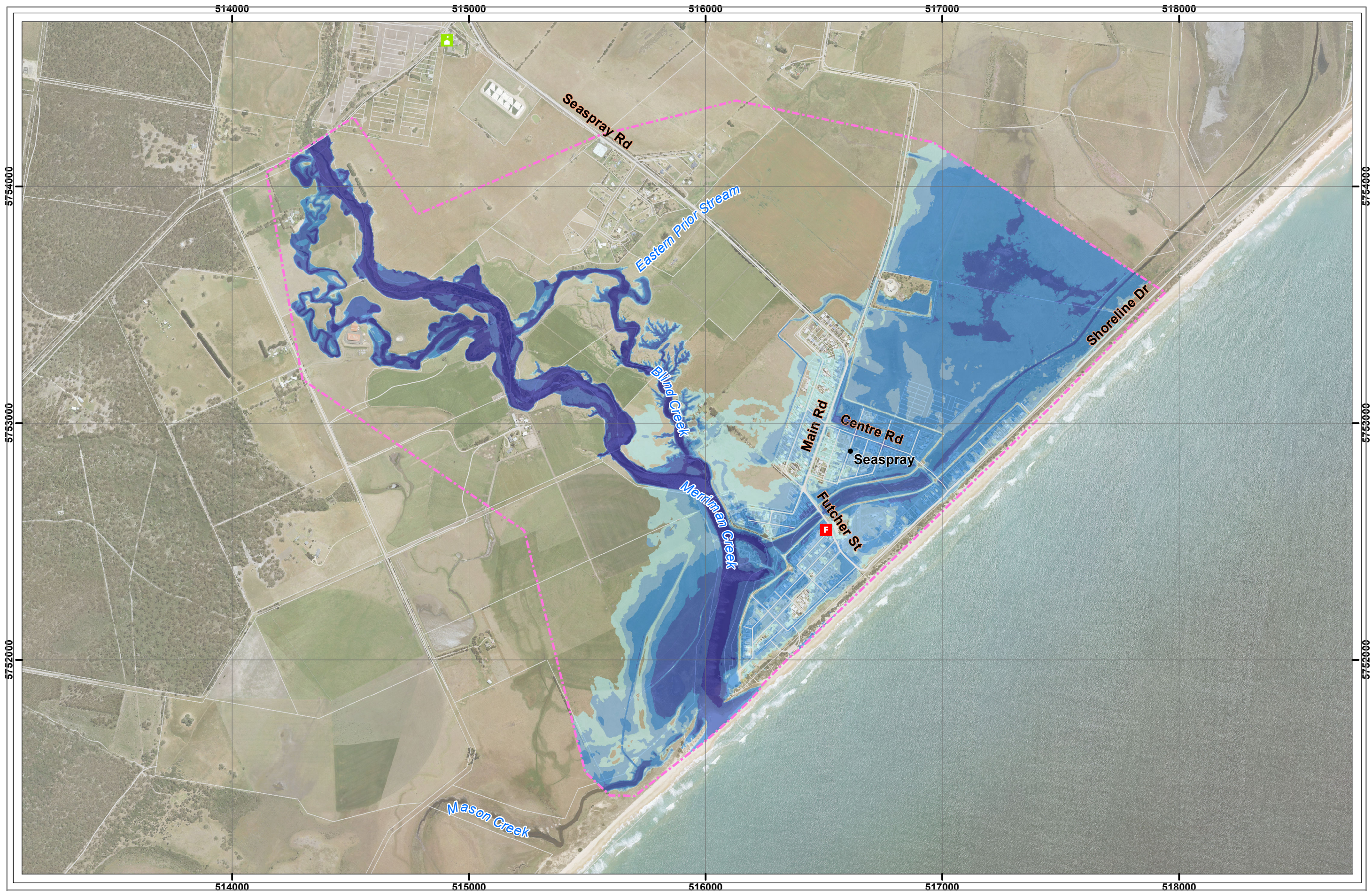
0 100 200 300 400 500 Meters

Maximum Depth

S03: 1% AEP Flow + 0.8 m SLR + 10% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S02-10pcST_d.mxd

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- School
- Major Road
- Minor Road
- Extent of Flood Mapping

Flood Depth (m)

<0.25 <0.50 <1.00 <2.00 >2.00

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1:15,000 at A3

Meters

Maximum Depth
S04: 1% AEP Flow + 0.8 m SLR + 1% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S03-082mSLR-10pcST_d.mxd
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Legend:

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- Major Road
- Minor Road
- Extent of Flood Mapping

Velocity (m/s)

0.5 1 1.5 2

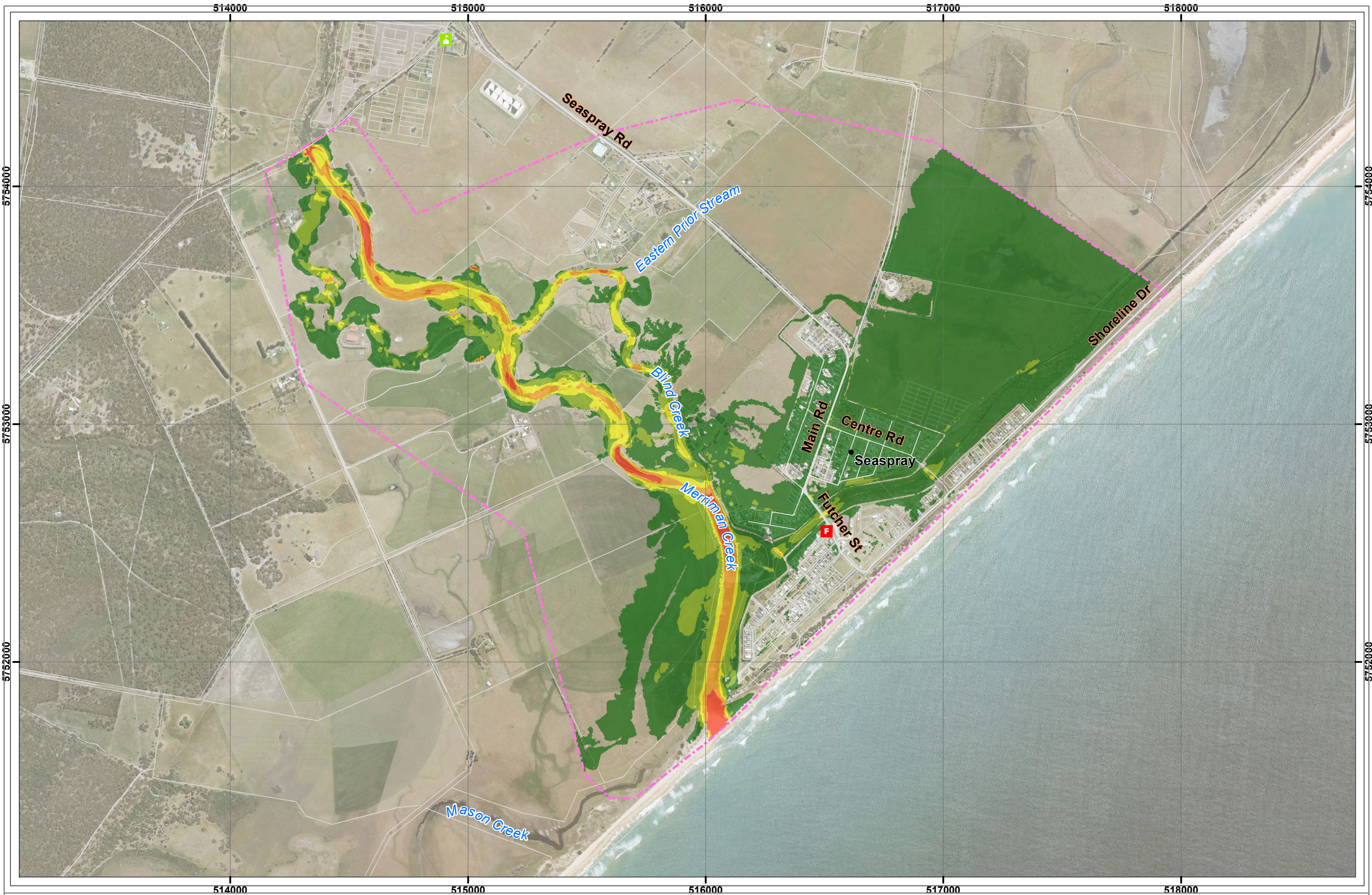
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1:15,000 at A3

0 100 200 300 400 500 Meters

Maximum Velocity
S01: 1% AEP Flow + 0.8 m SLR

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S02-1pcST_v.mxd
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- Minor Road
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Velocity (m/s)

0.5 1 1.5 2

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1:15,000 at A3

0 100 200 300 400 500 Meters

Maximum Velocity
S02: 1% AEP Flow + 10% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mapx\4730_Mapping_A3_S02-1pcST_v.mxd
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- School
- Major Road
- Minor Road
- Extent of Flood Mapping

Velocity (m/s)

0.5 1.0 1.5 2.0

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1:15,000 at A3

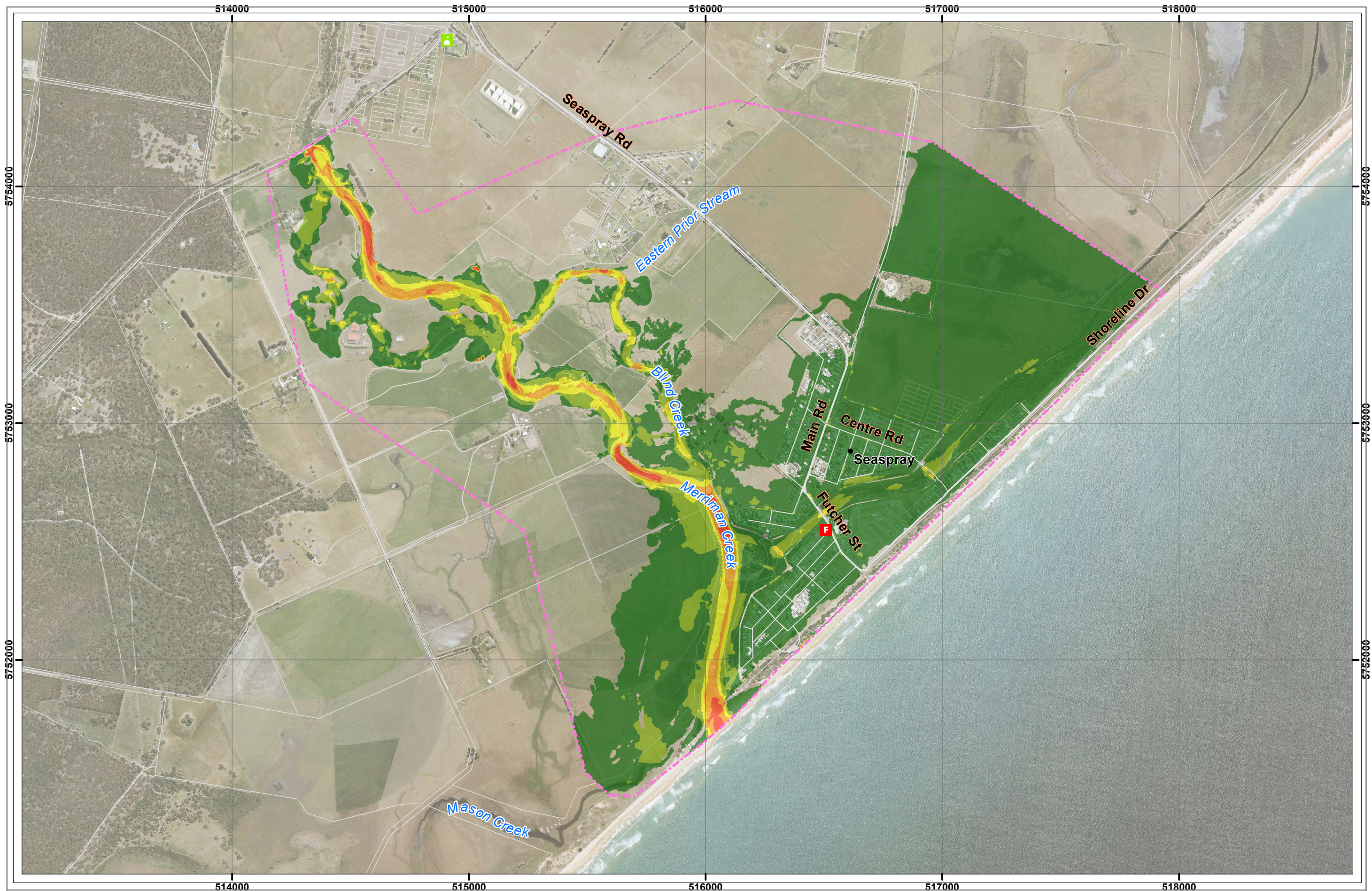
0 100 200 300 400 500 Meters

Maximum Velocity

S03: 1% AEP Flow + 0.8 m SLR + 10% AEP Storm Tide

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Legend:

- Fire Station
- School
- Major Road
- Minor Road
- Extent of Flood Mapping

Velocity (m/s)

0.5 1.0 1.5 2.0

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1:15,000 at A3

0 100 200 300 400 500 Meters

Maximum Velocity

S04: 1% AEP Flow + 0.8 m SLR + 1% AEP Storm Tide

REFERENCE: M:\Jobs\4700-4799\4730_Seaspray_SLR_Addendum\Spatial\ESRI\Mxd\4730_Mapping_A3_S04-082mSLR-1pcST_v.mxd

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