Seaspray Flood Study – Hydraulics (R03)

March 2016

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PROJECT DETAILS

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Cover Photo: Floodway Regulator, Seaspray

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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 m^2 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 m^2. Triangular elements are used to migrate between quadrilateral sections and areas with different resolutions.

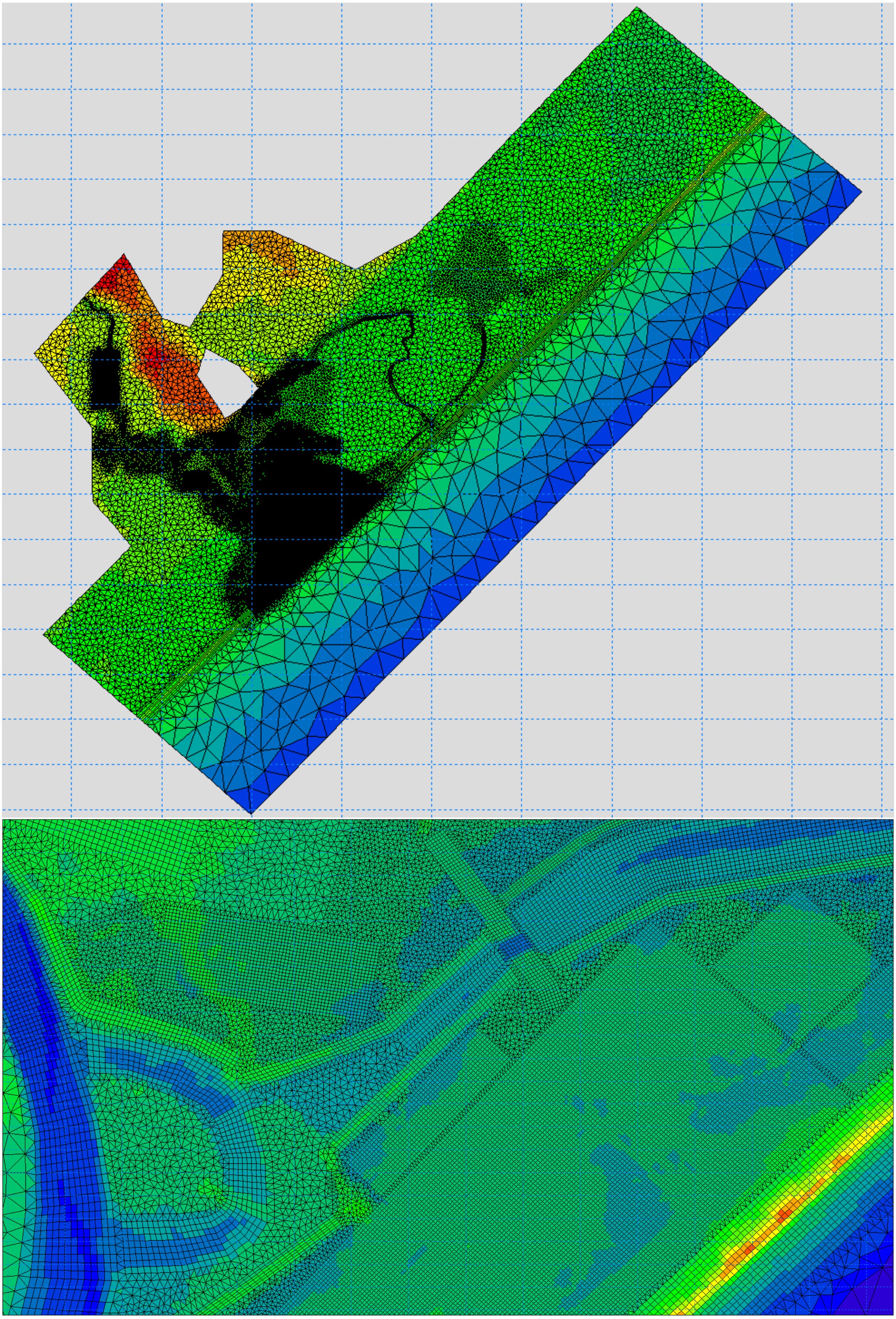

Figure 3-1 Hydrodynamic Model Domain and Mesh

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).

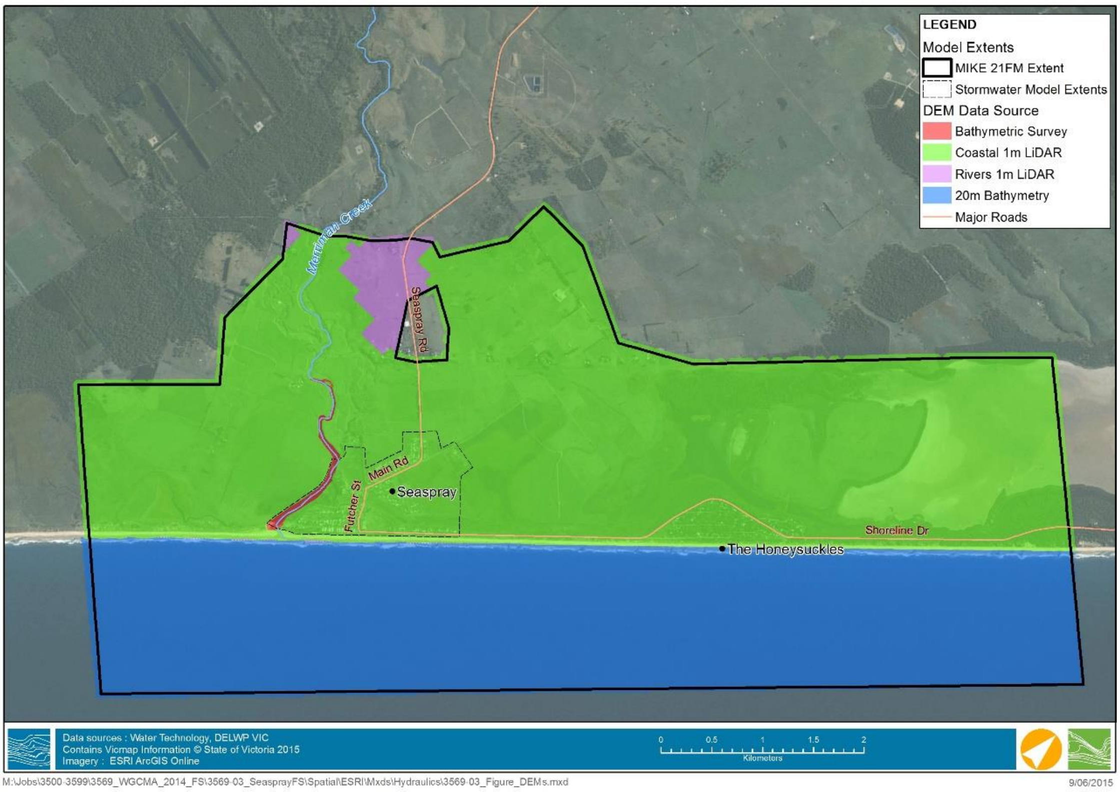

Figure 3-2 Extent of DEM Data Sources Used in Model

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 overlayed 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.


03 Apr 1965
12 Jul 1974
13 Feb 1976
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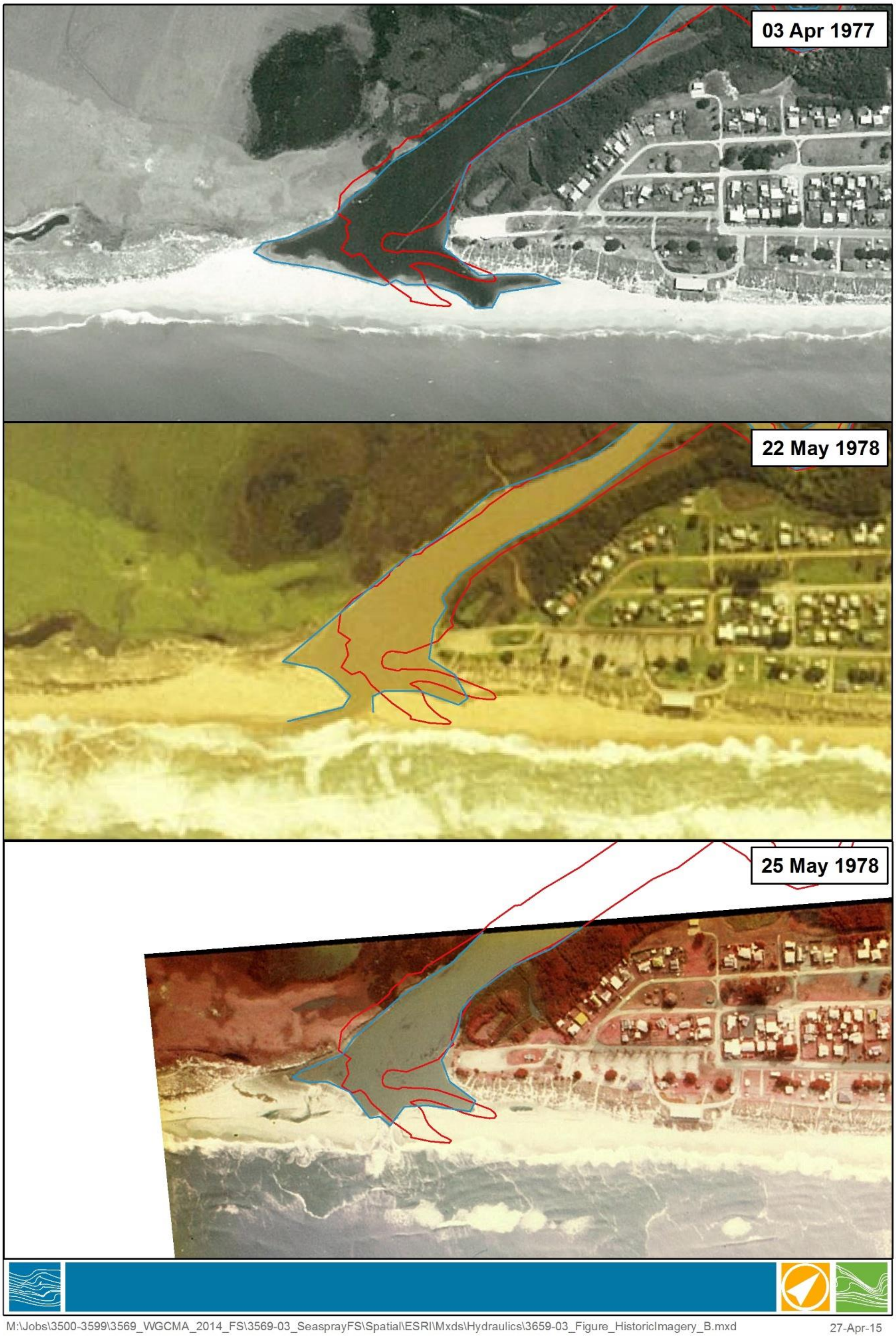
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22 May 1978
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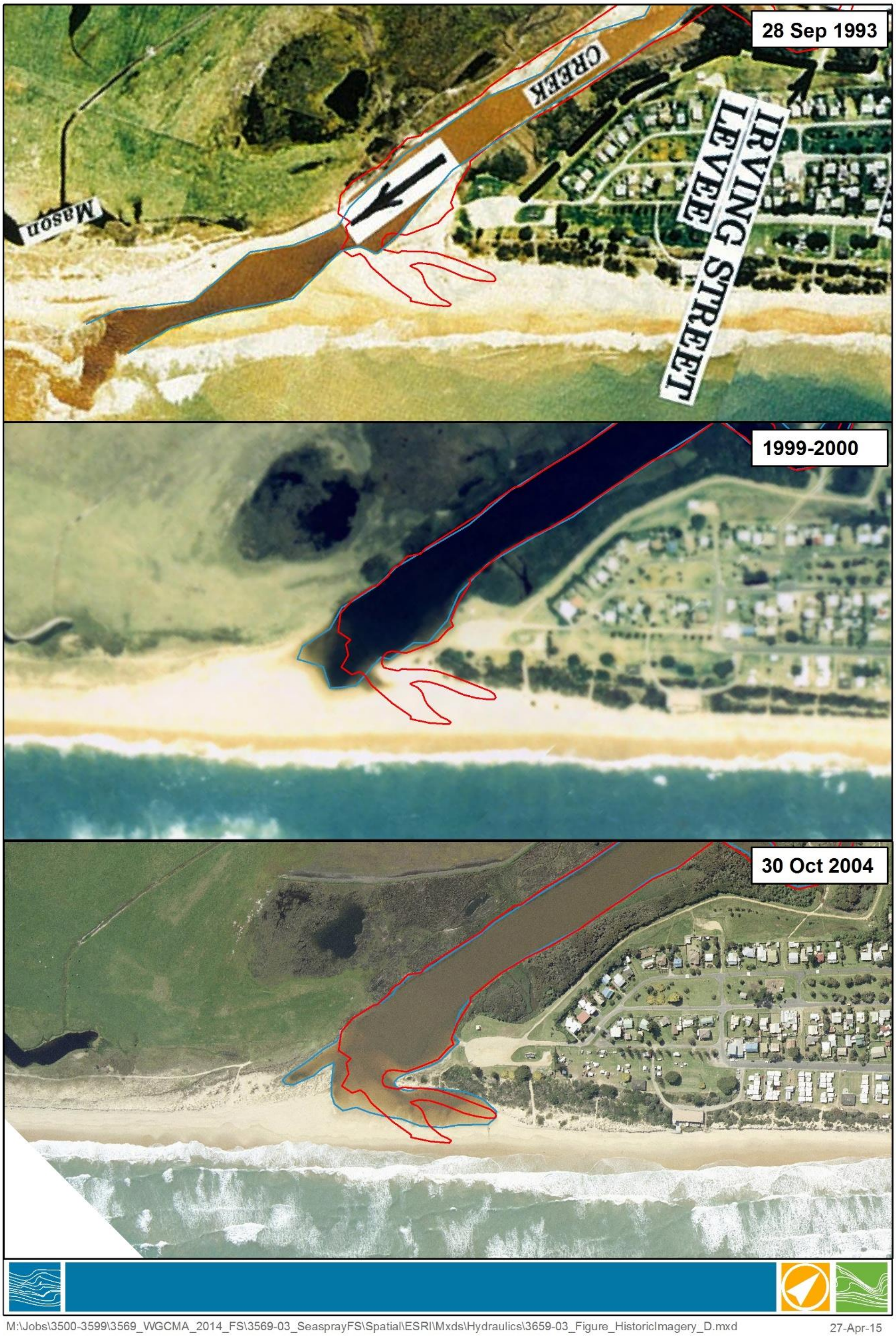
28 Sep 1993
CREEK
IRVING STREET
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27-Apr-15

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.

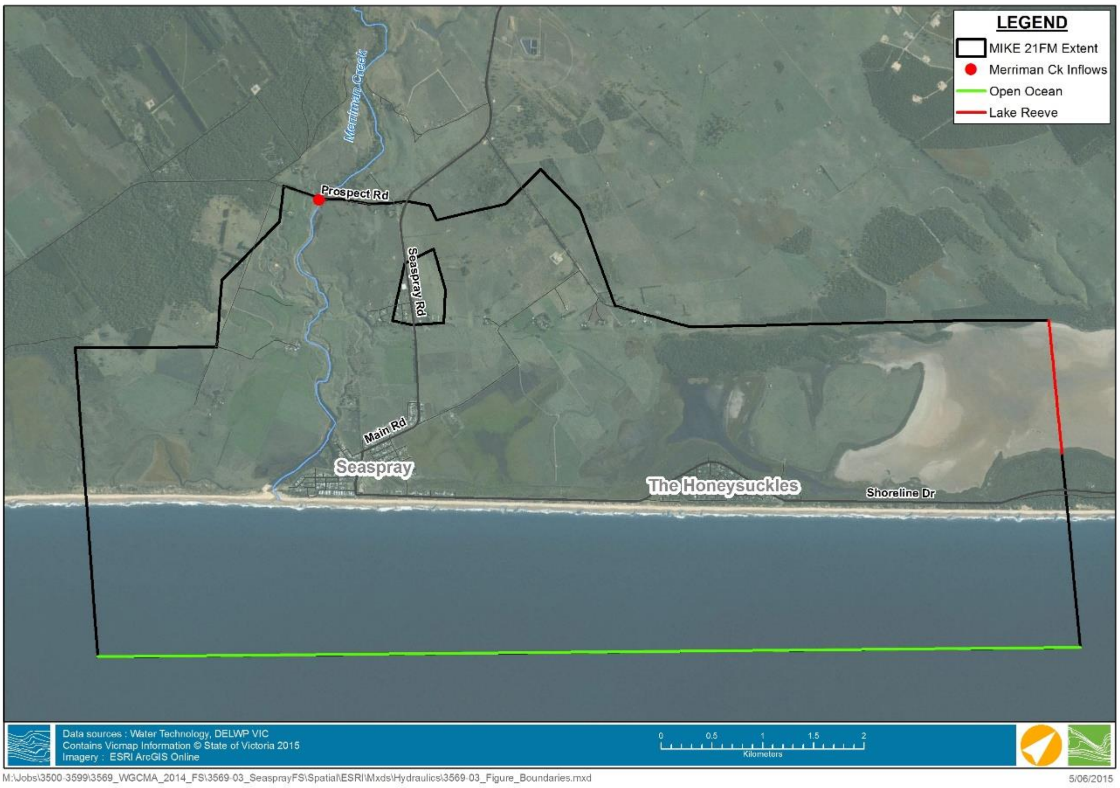

Figure 3-4 Model Boundaries

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 m^3/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.

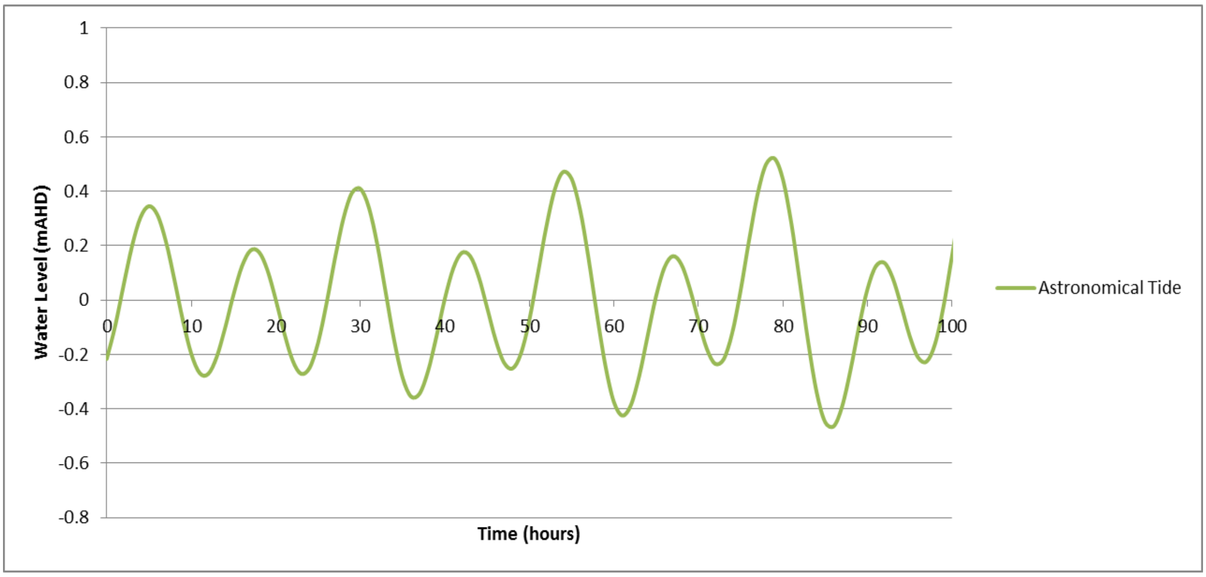

Figure 3-5 Astronomical Tide Time Series

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

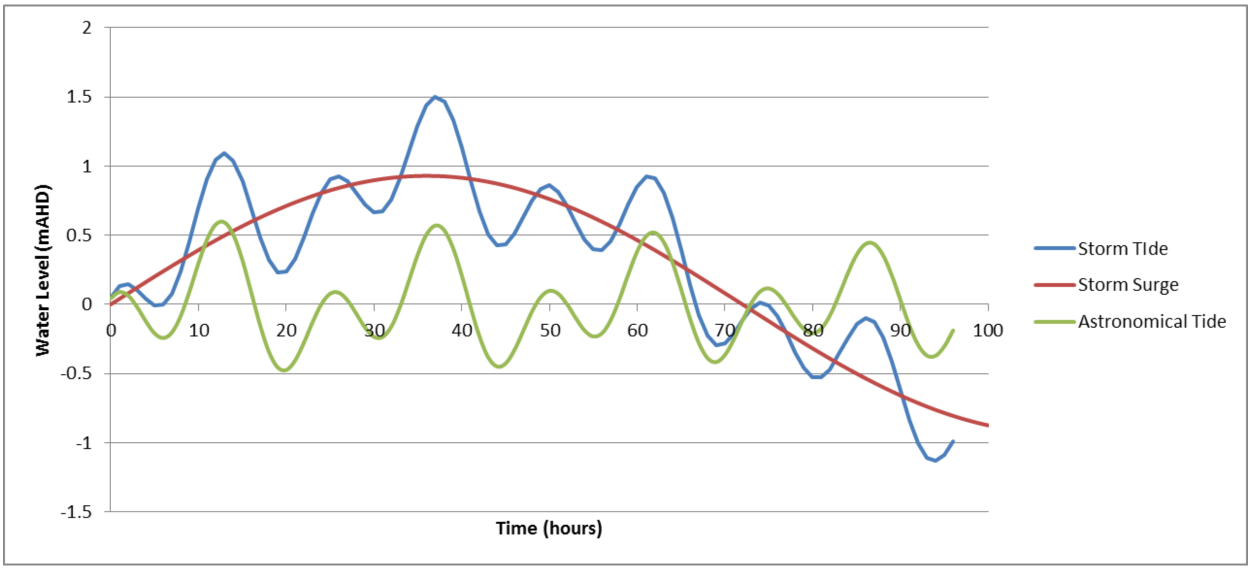

Figure 3-6 1% AEP Storm Tide under current mean sea level and climate

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.

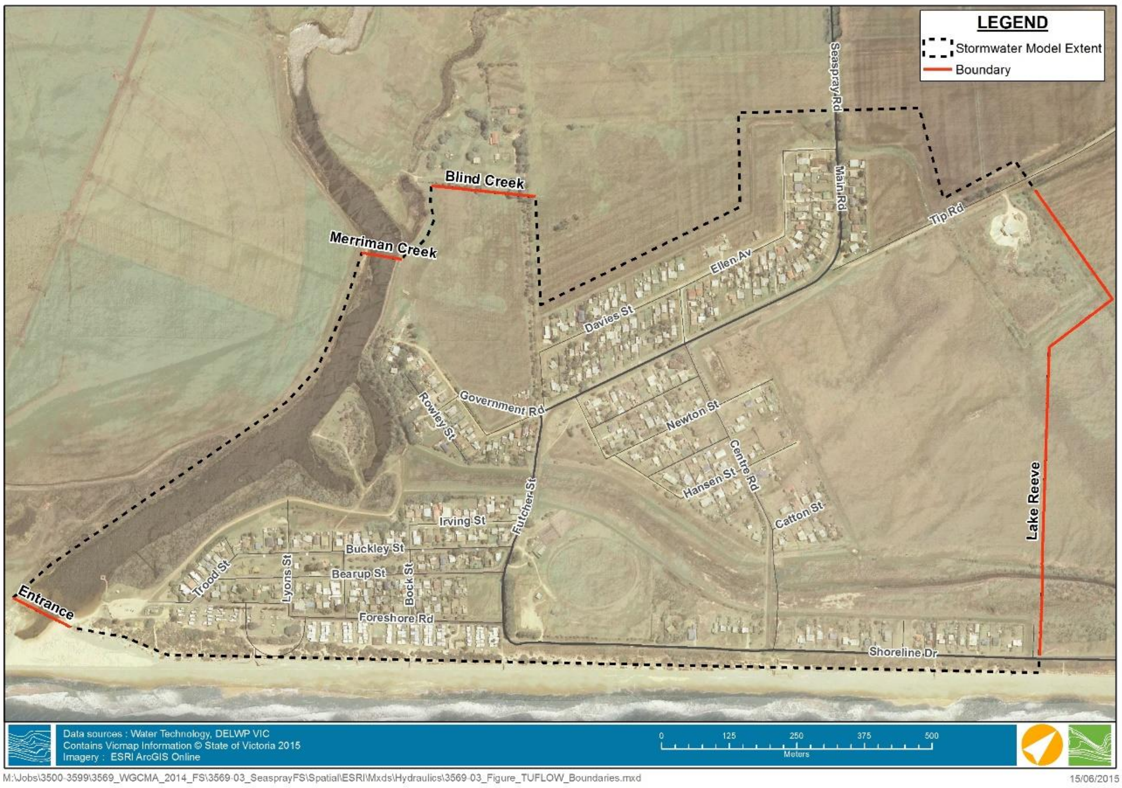

Figure 3-7 Stormwater Model Boundaries

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.

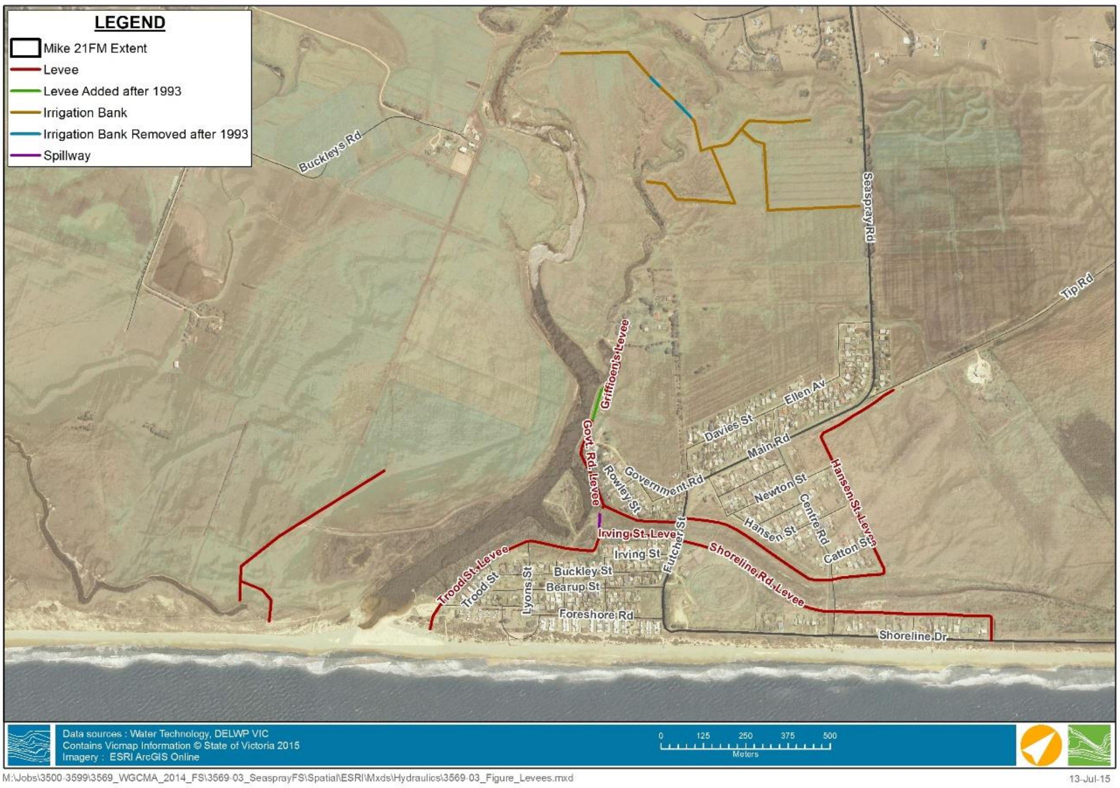

Figure 3-9 Levees and Irrigation Banks

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 Surveyed 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.

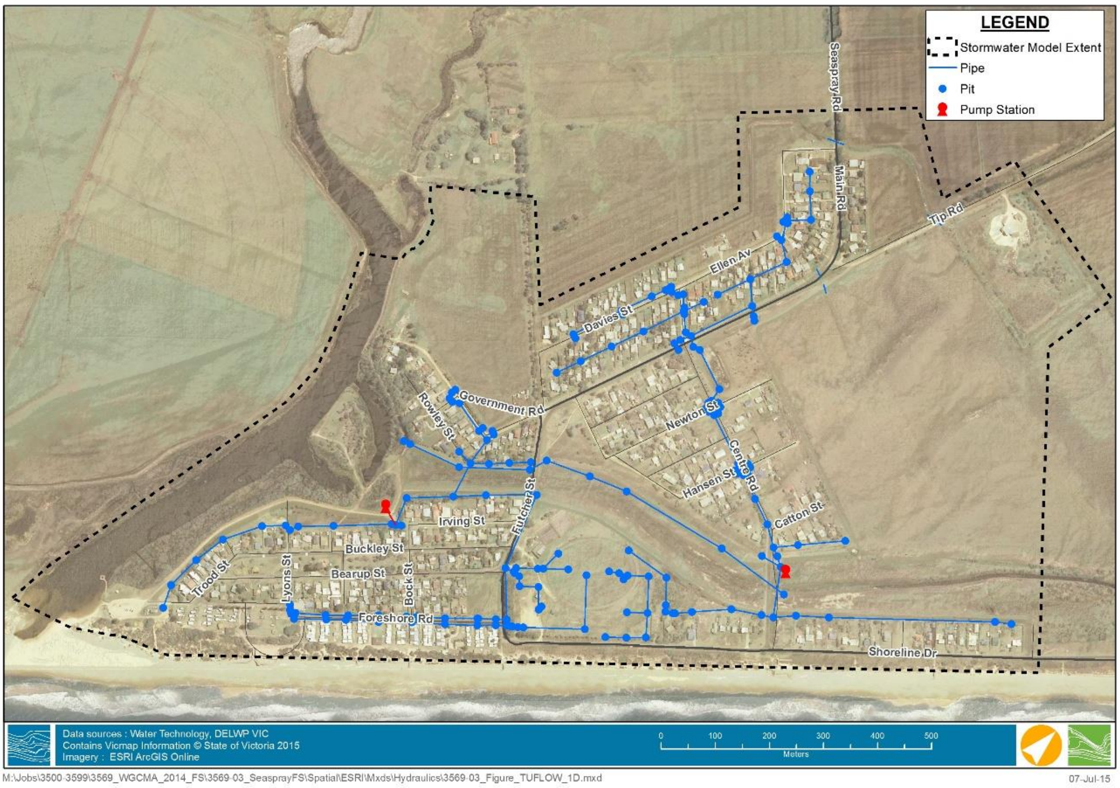

Figure 3-11 Seaspray Pit and Pipe Network

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.

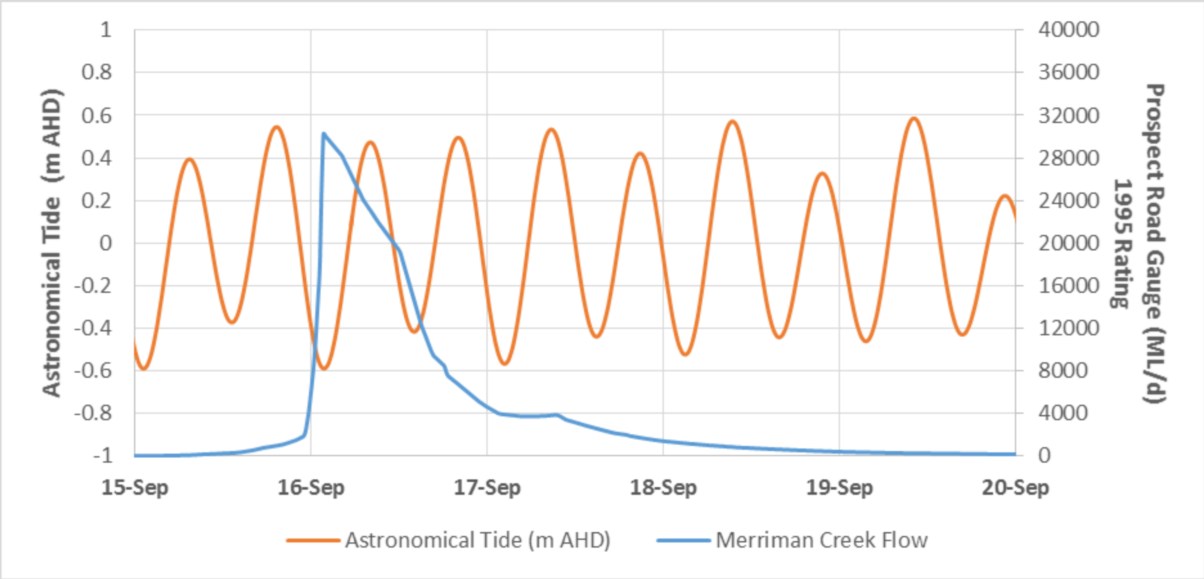

Figure 5-2 Catchment Inflows and Astronomical Tide for September 1993 Calibration

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 m^3/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 Futcher 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.

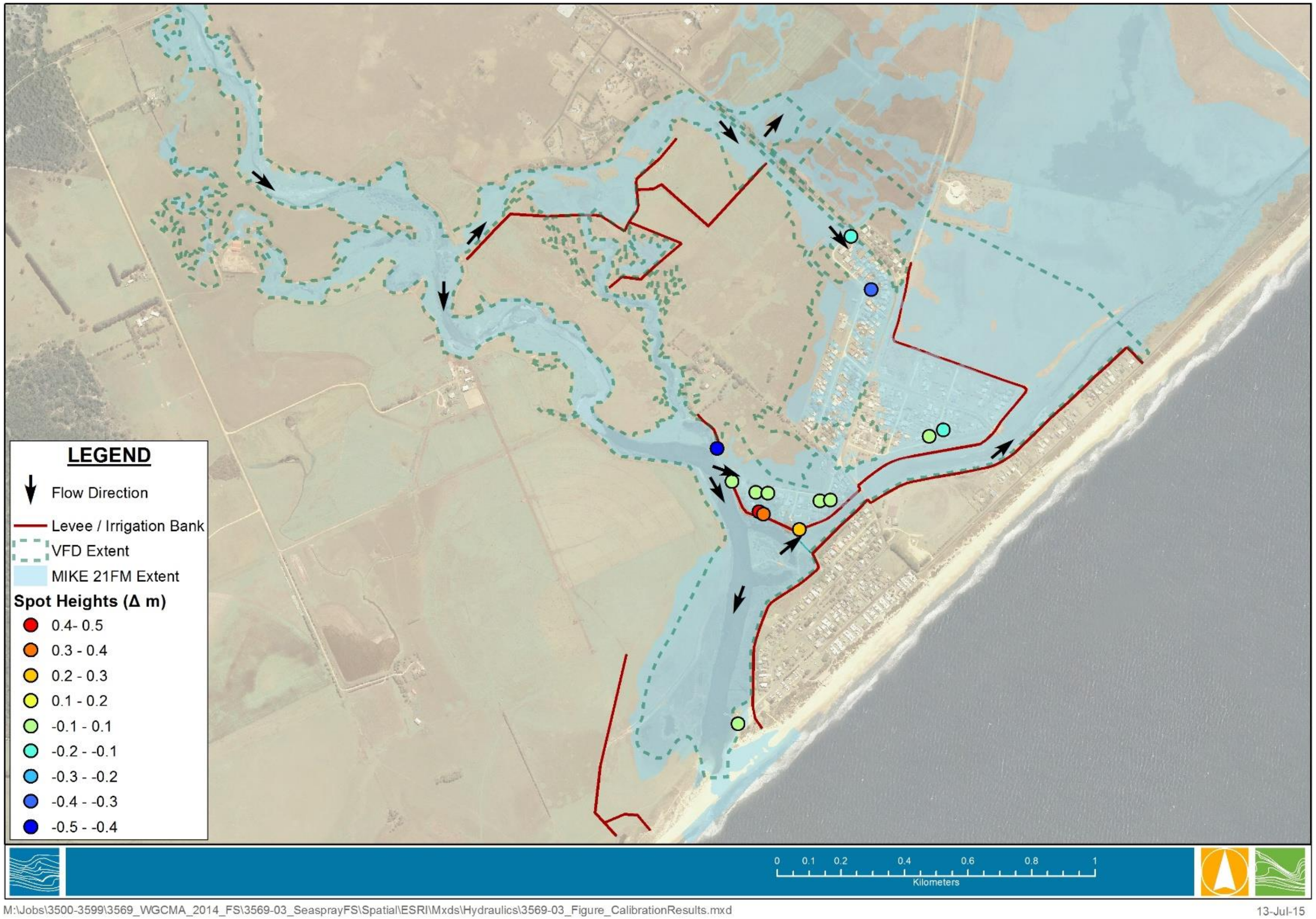

Figure 5-3 Modelled vs. Recorded Flood Extents and Spot Heights

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 Futcher 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 Futcher 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 is 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.

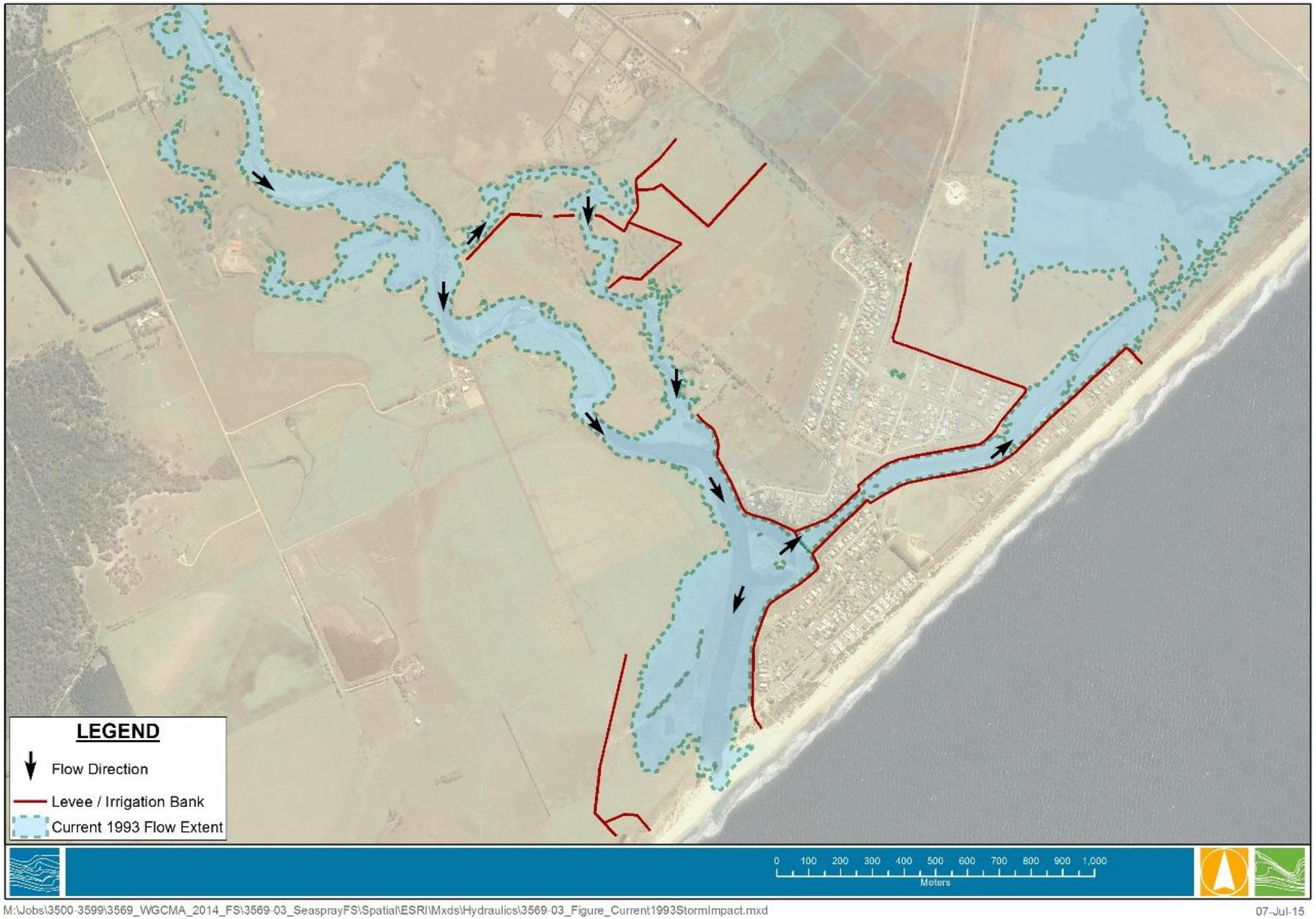

Figure 5-4 Modelled Flood Extents for 1993 Flows with Current Topography

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 Futcher 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.

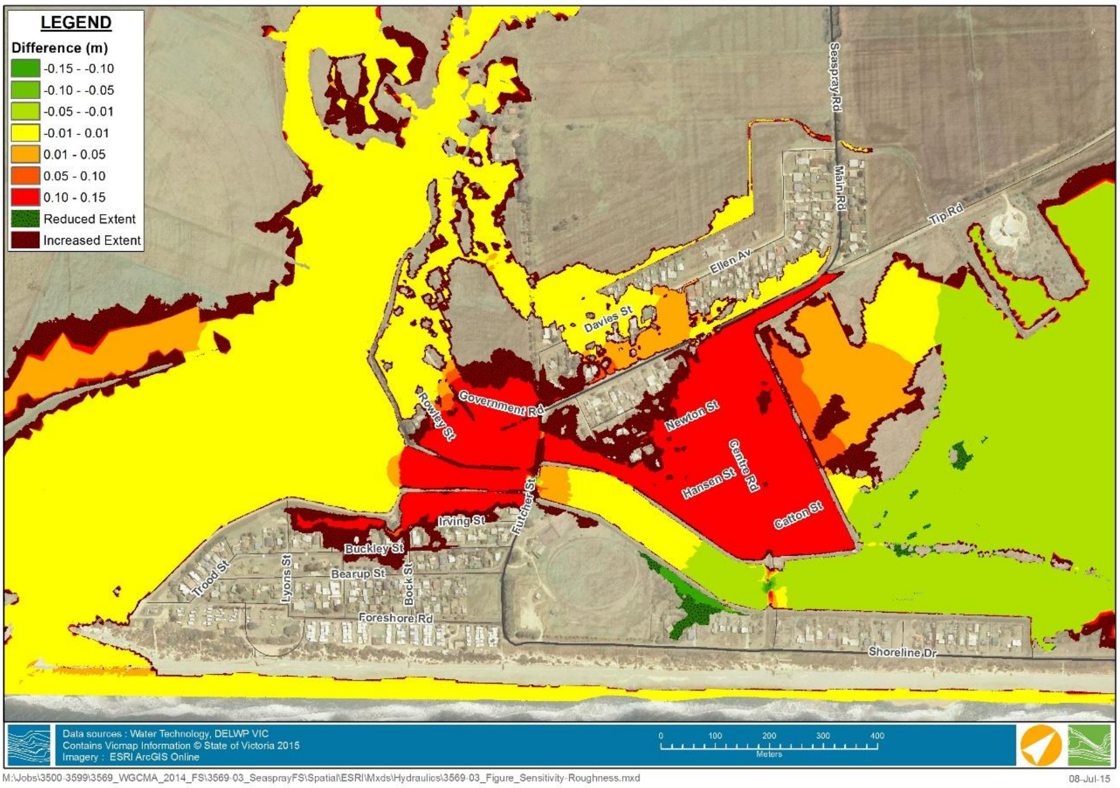

Figure 6-1 Difference plot of water surface elevation with increased roughness

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 undertake 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 coincidently, 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 Futcher 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.

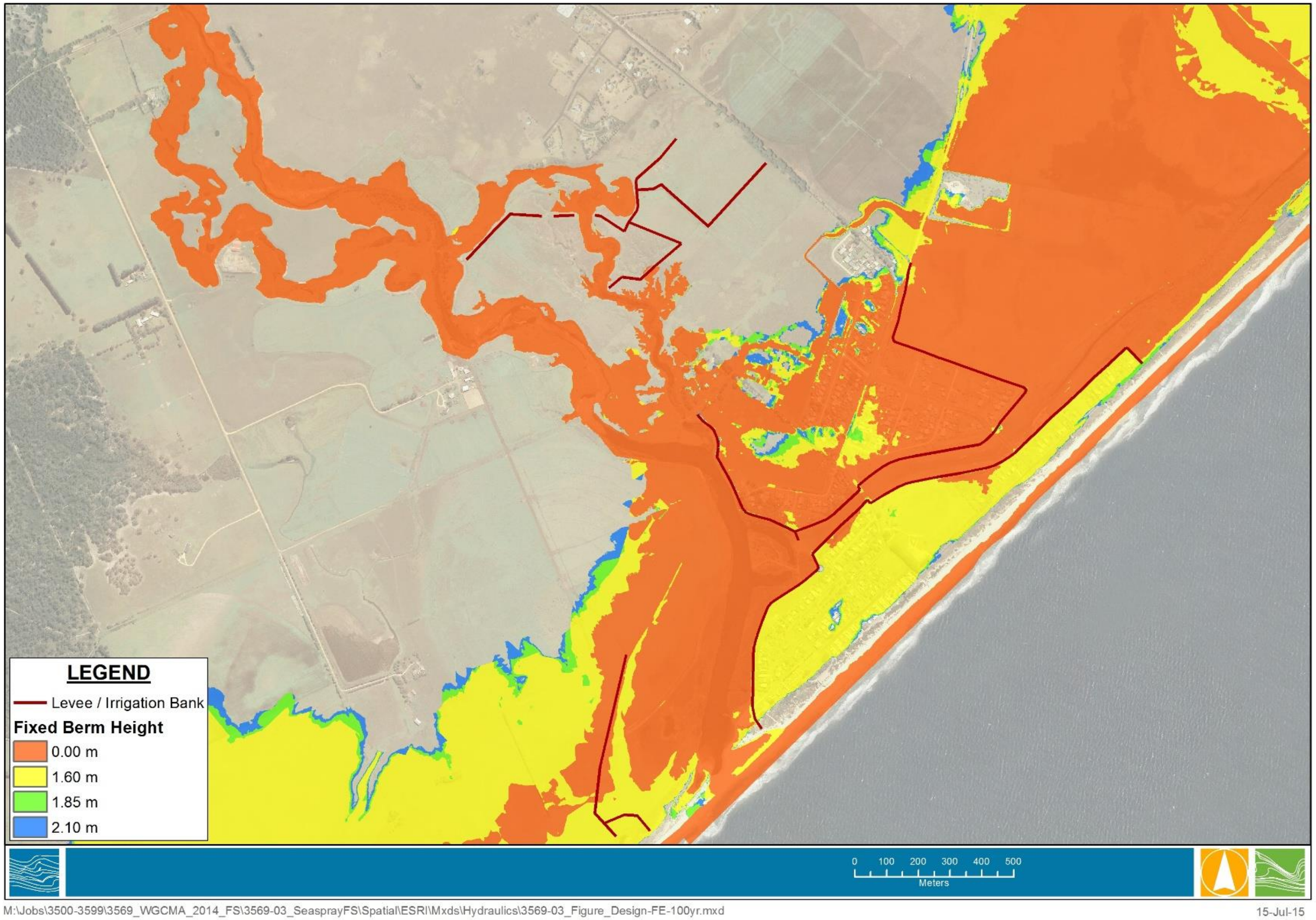

Figure 6-3 Inundation Extents (1% AEP) for Fixed Entrance Berm Heights of 0.0m, 1.6m, 1.85m, and 2.1 m AHD

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.

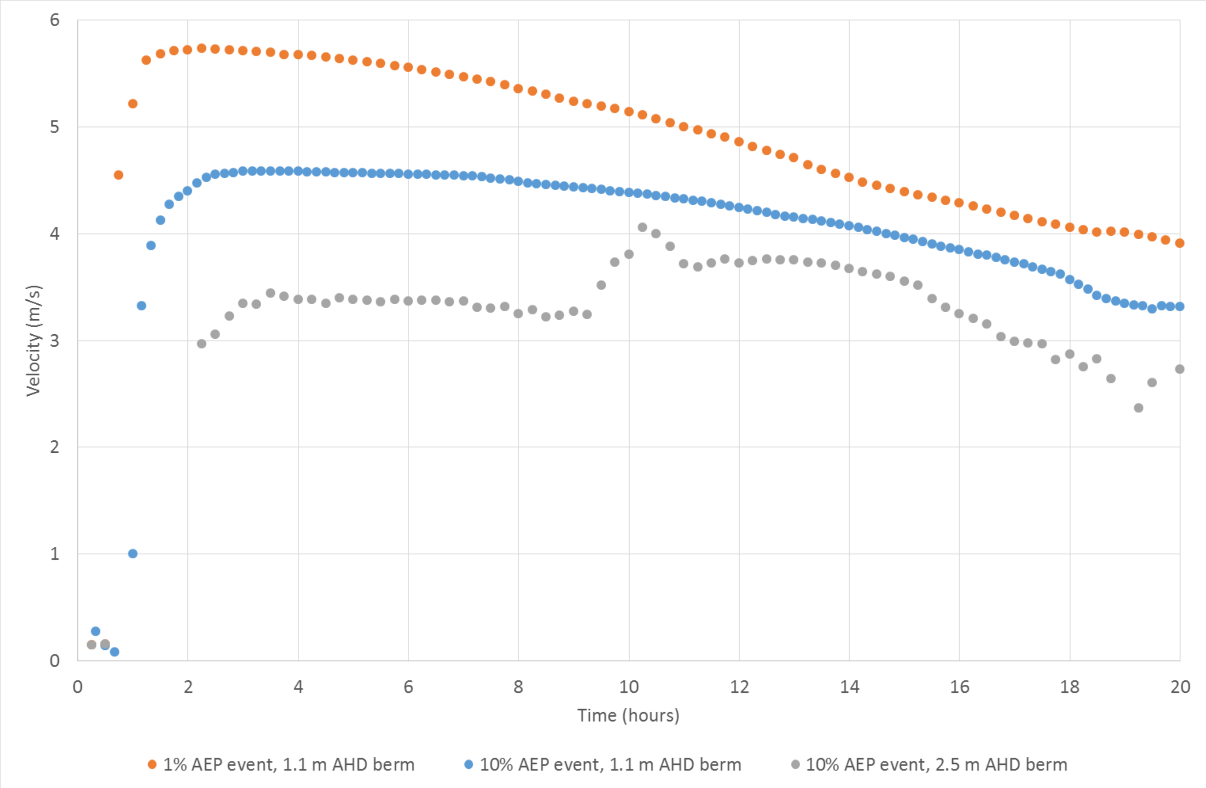

Figure 6-4 Modelled velocities across the entrance berm

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.

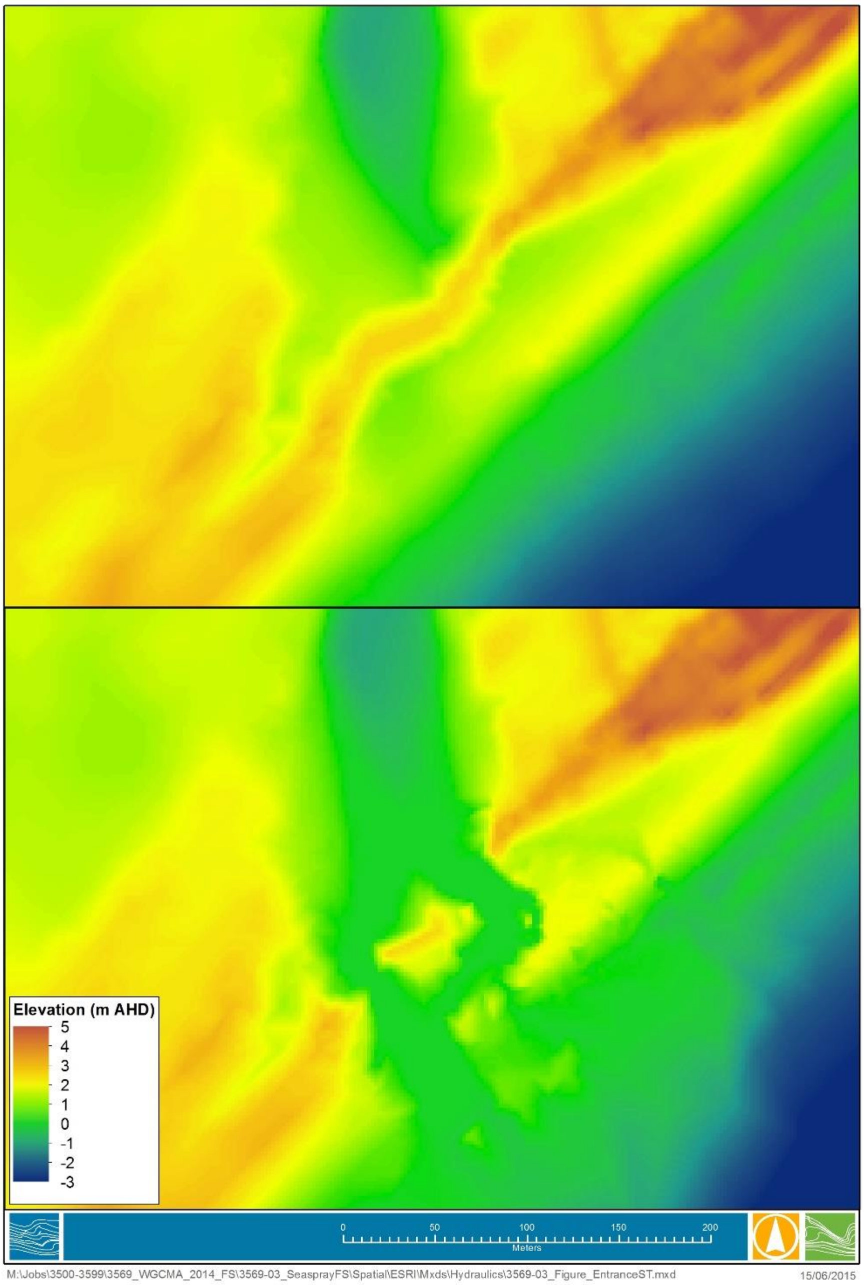

Figure 6-5 Entrance berm within dynamically scouring channel during 10% AEP flood event

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 ±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.

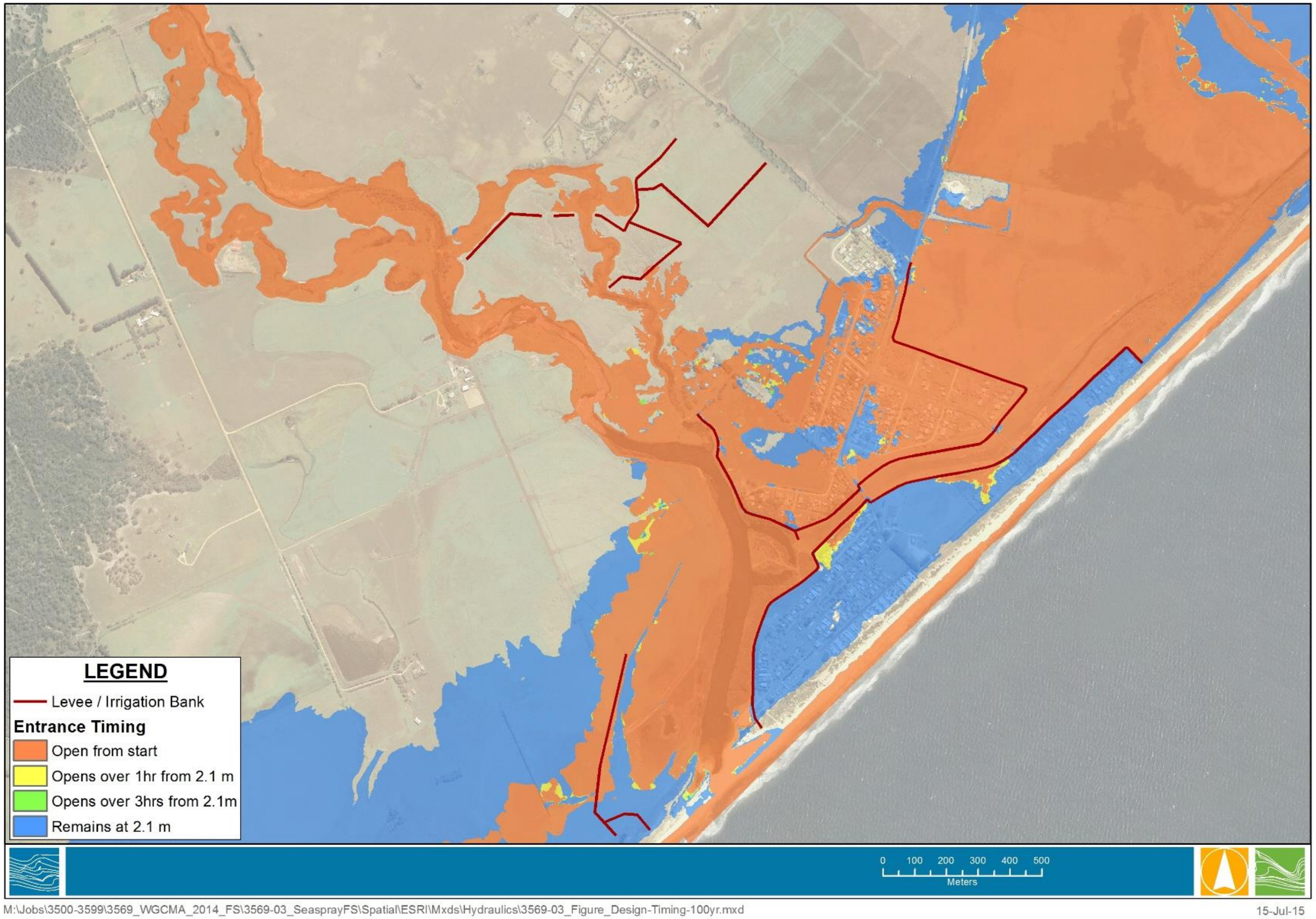

Figure 6-6 Initial Berm Height Sensitivity – scour initiated at maximum berm height (1% AEP event)

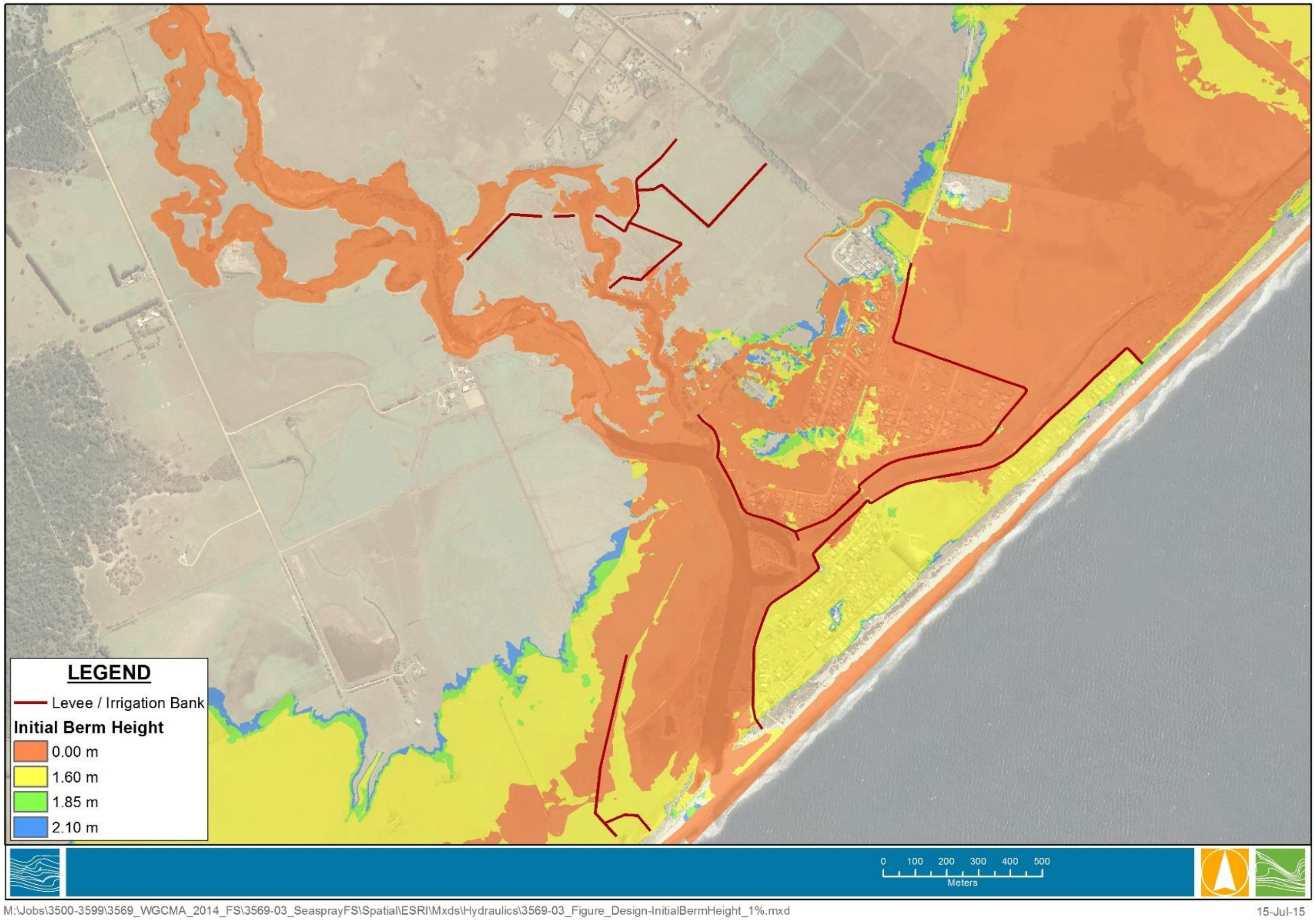

Figure 6-7 Initial Berm Height Sensitivity - scour initiated at peak water level in the estuary (1% AEP event)

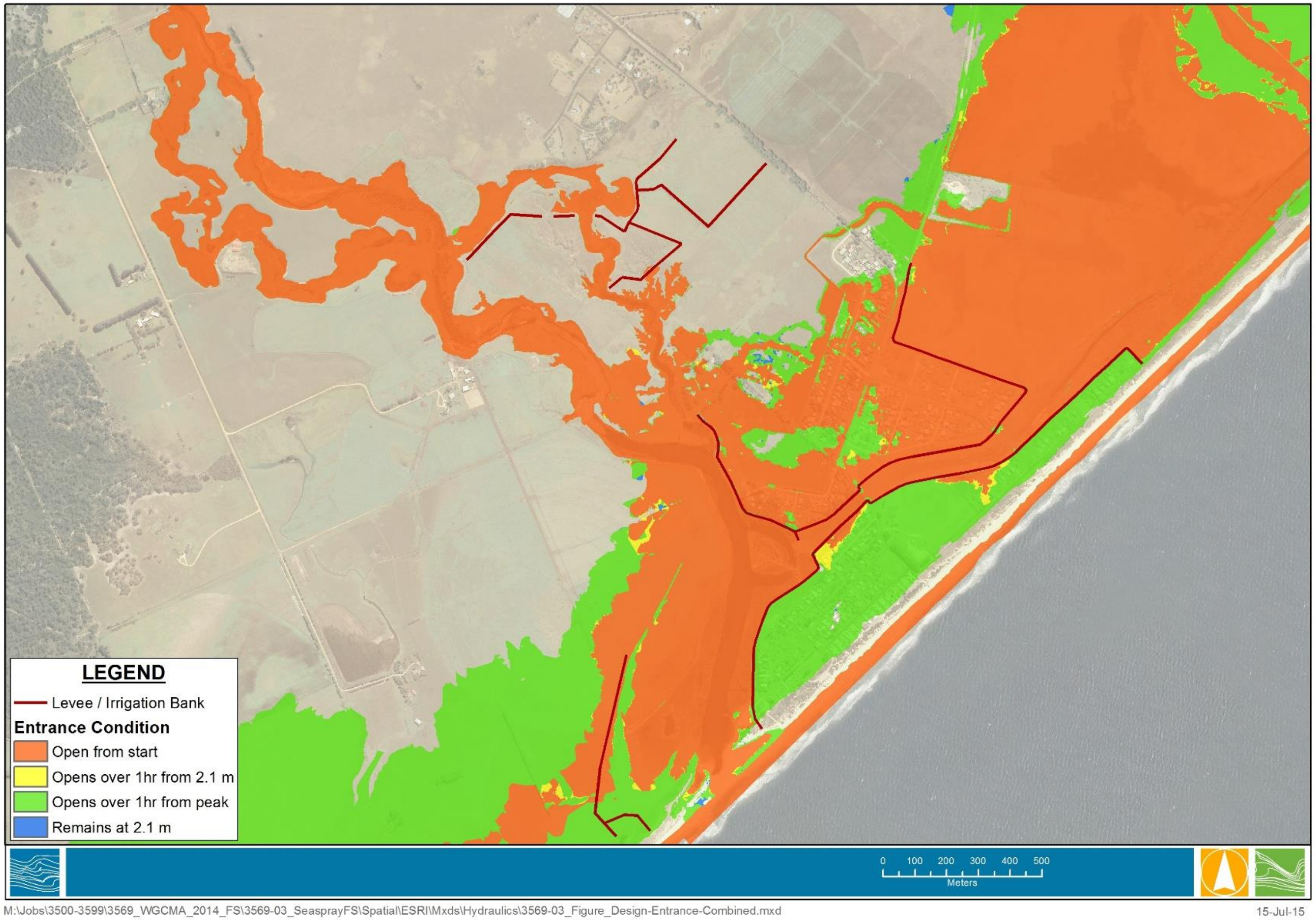

Figure 6-8 Effect of Entrance Opening Conditions (1% AEP, Combined)

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 ±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.

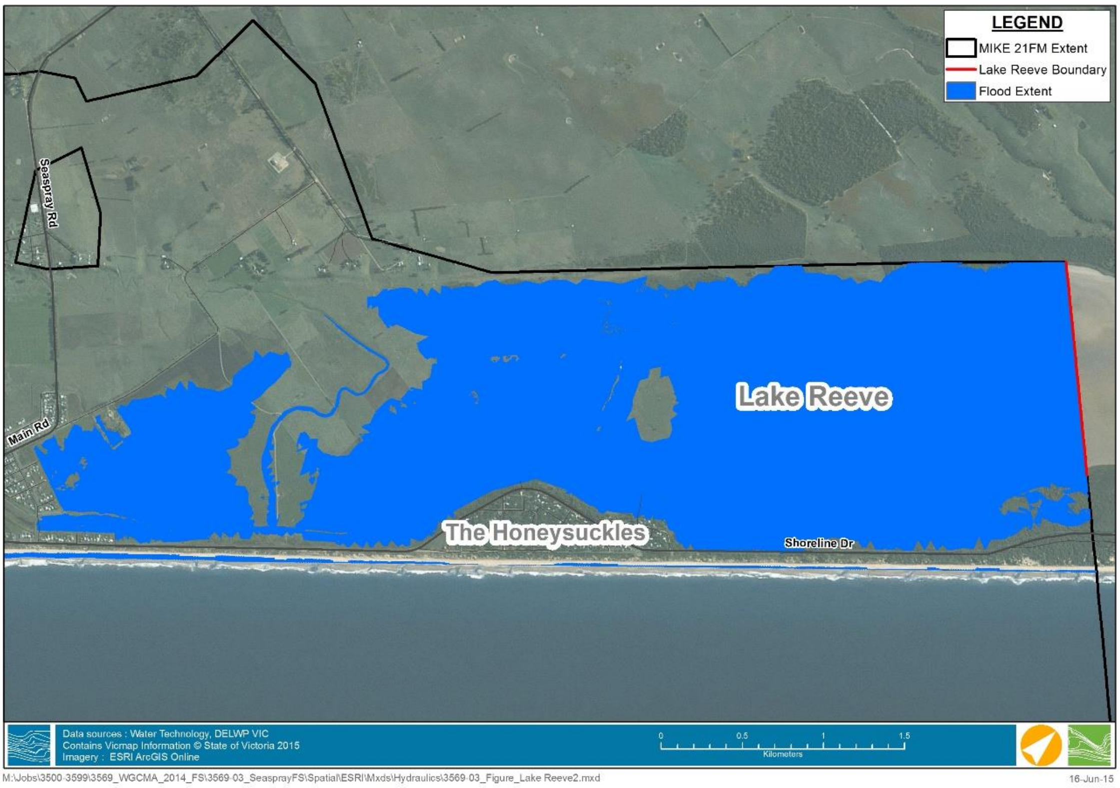

Figure 6-9 Flood Extent with water levels at Lake Reeve raised to 1.37 m AHD

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).

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