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Port Lincoln Flood Mapping Study

Hydrologic and Hydraulic Modelling Report

City of Port Lincoln

May 2017

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Document History and Status

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1 Executive summary

Tonkin Consulting was commissioned by the City of Port Lincoln to undertake the floodplain mapping of Port Lincoln. The primary purpose of the study has been to define the extent of inundation and categorise the potential hazard resulting from a series of design storm events within the township.

The study assesses a total catchment area of approximately 19.2 km^2 and includes the majority of the urbanised areas of Port Lincoln.

The following flood events were modelled:

- 20 year Average Recurrence Interval (ARI) flood event
- 50 year ARI flood event
- 100 year ARI flood event

The floodplain modelling was carried out using the TUFLOW computer program jointly funded and developed by WBM Oceanics Australia Pty Ltd and The University of Queensland in 1990. TUFLOW is a two-dimensional hydrodynamic model that links with the ESTRY one-dimensional network modelling system which together provide a linked 1D–2D model that can simulate the interaction between waterways and their floodplains.

The majority of the modelled catchment is relatively steep. Consequently, in most areas the overland flood flow paths are of limited depth. However, hazardous flows do occur along some roads due to the concentration of unobstructed, fast moving water. Comparatively, deep flooding occurs within the lower lying, flatter areas of the township. This area can generally be described as occurring along a strip, 800 m wide, starting near the Port Lincoln racecourse in the south and ending near Liverpool Street in the north.

The worst locations for flooding include Liverpool Street, Napoleon Street, and Edinburgh Street in the central business district; Le Brun Street, near the centre of the urban area; Stamford Terrace, particularly at the corner of Follett Street; and Windsor Avenue in the southern industrial area.

A flood damages assessment estimated that the township is exposed to an annual average damage (AAD) of almost $2.9 million per year.

The completed flood modelling provides the City of Port Lincoln with an unprecedented level of understanding of flooding within the township. The exposure of known and unknown flooding hotspots will enable the Council to focus their efforts to protect the community from flooding.

2 Introduction

This report is concerned with the preparation of a 1D–2D linked hydrodynamic flood model of the Port Lincoln township. The report outlines the steps undertaken to prepare the model inputs and the verification and validation of the model outputs.

The primary purpose of the work undertaken has been to define the extent and magnitude of flooding during events of differing annual recurrence intervals (ARI) and to identify areas of significant inundation. The risk to public safety, otherwise known as the 'flood hazard' has also been categorised for some of the recurrence intervals investigated.

The flood maps created herein will be used for the purposes of development assessment, town planning, emergency response planning, infrastructure planning and flood warning purposes.

The study area is presented in Figure 2.1.

2.1 Previous studies

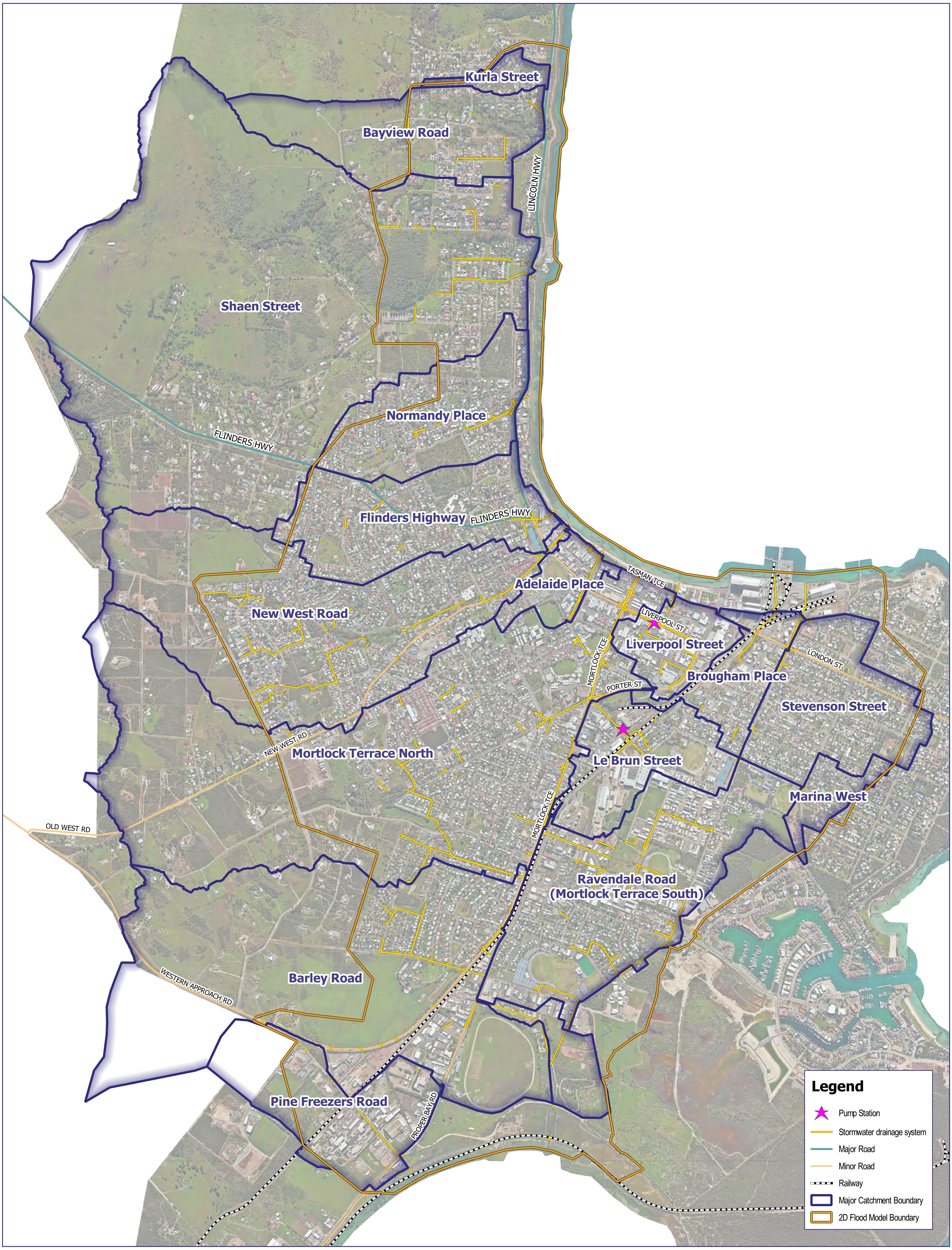
In 1994, Kinhill Engineers were engaged to undertake a comprehensive stormwater drainage study of the Port Lincoln township covering the residential and commercial zone areas. The study used ILSAX and RORB models to estimate catchment runoff and pipe hydraulics. The study identified flood locations and proposed strategies for managing flooding.

In 2007, Tonkin Consulting were engaged to undertake the development of a compliant Stormwater Management Plan (SMP) for Port Lincoln. The study used ILSAX models to assess current drainage infrastructure standards and proposed broad stormwater management goals specific to each catchment to address flooding and water quality. The SMP was approved in 2014.

2.2 Scope of works

The general scope of works for the study was to determine the extent of flood inundation during various flood events within the Port Lincoln township. The project included the following tasks:

- Locating the existing surface and subsurface drainage infrastructure, including measurement of conduit dimensions, junction dimensions and inlet pit types.
- Converting raw field measurements into a GIS database of the full pit and pipe network.
- Obtaining details of hydraulic structures, such as pump stations, inlet structures and parts of the pipe network that could not be accessed during field measurement.
- Obtaining an accurate digital elevation model (DEM) and aerial imagery of the study area.
- Preparing a hydrological model of the urban and rural catchments.
- Preparing a combined 1D–2D hydrodynamic computer model of the study area based on the existing level of development.
- Analysing the resulting flooding for the following storm events:
  - 5 year ARI storm event
  - 20 year ARI storm event
  - 100 year ARI storm event
- Producing flood inundation and hazard zone maps for various specified flood events within the Study Area.
- Prepare an assessment of damages caused by flooding.
- Issuing a modelling report, associated flood maps and data in electronic format.



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Road and Rail data by PBI, 2014

CITY OF PORT LINCOLN

PORT LINCOLN FLOOD MAPPING STUDY
Study Area

Figure 2.1

3 Asset discovery and data collection

3.1 Existing database

Council's existing GIS database of the drainage network was originally compiled by Tonkin Consulting using Council's drawing archive. This work was undertaken during the development of the *Port Lincoln Stormwater Management Plan* (Tonkin Consulting, 2014). The GIS database provided a basic understanding of the trunk drainage layout. However, many small diameter pipes were missing from the network and some trunk systems were completely absent. As the database was created for modelling with ILSAX, a lumped approach had been taken to modelling inlets. The lumped approach meant it was unclear how many individual inlets were actually present in the field. Due to these issues, it was determined that the existing database lacked the requisite detail and positional accuracy needed for a linked 1D–2D hydrodynamic model, and that additional data collection was required.

3.2 Data collection

Discussion with Council enabled development of a data collection project that would update the Council's GIS database for the dual purposes of estimating capital worth of the drainage infrastructure as well as detailed hydraulic modelling of the underground network.

As the number of assets in the network was not known with great certainty, a staging plan was implemented to ensure the most essential areas of the catchment were surveyed first. Not all areas were selected to be field surveyed. The area surrounding the marina had good records of installed drainage infrastructure so it was determined that these areas should be recreated directly from Council design drawings.

In addition to information required for asset management, the attributes listed below were recorded at each pit in the drainage network:

- depth from pit lid to each pipe invert,
- internal dimensions of the pit,
- inlet pit type and dimensions of the inlet,
- estimated direction of each pipe leading in to, or out of, a pit.

After initial asset data collection, the elevation of pit lids was surveyed to provide a reference elevation from which pipe inverts could be inferred.

It was not possible to access all drainage pits to gain the desired information. Additionally, some of the recorded data did not enable a reliable reconstruction of the drainage network. In these areas, additional information was sought in the form of design or as-built drawings. CCTV video footage was also used for a small number of pipes to supplement the information collected at drainage pits. In areas where no additional information could be obtained an engineering judgement was made as to the most likely layout of the underground drainage network.

Two pump stations are operated by Council: the Liverpool Street Pump Station and the Le Brun Street Pump Station. It was necessary to include both pump stations in the 2D model to ensure an accurate prediction of inundation in the low lying areas serviced by the pump stations. To ensure accurate modelling of the pump stations, Council records were examined to find operating rule sets and impeller properties. This information was then encoded within the model.

4 Hydrologic modelling

4.1 Overview

The hydrologic modelling aims to determine the rate of runoff given a particular rate of rainfall. This information is then applied to the hydraulic model which dynamically models the path of runoff through the study area.

The hydrologic modelling for this study involves determining runoff from local urban catchments and from external rural catchments. For local catchments a Time–Area method was applied to create hydrographs for each sub-catchment. For external catchments runoff–routing models were developed to produce inflow hydrographs at the boundary of the hydraulic model. These models were prepared in RORB.

4.2 Catchment description

The northern half of the modelled area is overlooked by the steep slopes of Winters Hill which fall eastwards to the low beaches or steep cliffs of Boston Bay. The southern parts of the modelled area fall towards marshland adjacent Porter Bay and Proper Bay. Within the township there are multiple low lying areas where water will naturally pond.

The modelled catchments are dominated by urban residential development on large allotments. The central business district is well developed and is almost entirely covered with impervious surfaces. Industrial development is clustered along the railway line that runs through the centre of the town leading to the main port facilities. On the highest parts of the overlooking hillsides land use primarily consists of rural living and agriculture.

There is a long history of flooding in low lying areas due to the absence of natural flow paths. High levels of commercial and industrial development have exacerbated flooding in these areas due to increases in the amounts of impervious surface within the contributing catchments.

4.3 Catchment delineation

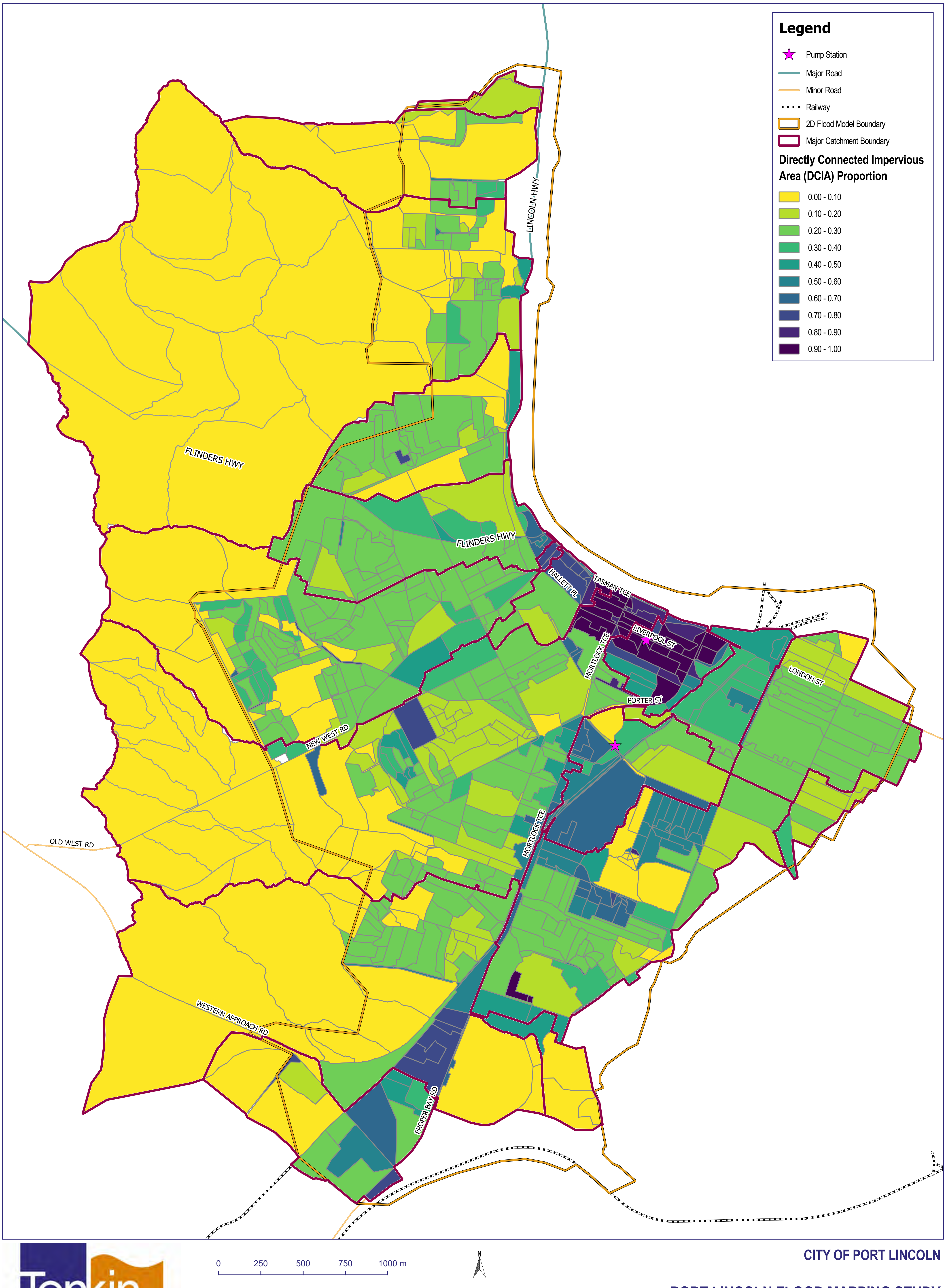
Previous work undertaken by Tonkin Consulting for the Port Lincoln SMP provided a base set of sub-catchments. The sub-catchments of the SMP were larger than that required for the TUFLOW modelling, therefore, work was undertaken to further refine the catchments. Delineation of additional sub-catchments was performed manually using the following information for guidance:

- the digital elevation model (DEM) and surface contours,
- aerial photography,
- GIS data including property boundaries (cadastre), road network, and stormwater inlets.

For each inlet to the underground drainage system, at least one sub-catchment has been delineated. Some large sub-catchments have been divided into smaller areas to improve representation of pluvial flooding in the road network.

4.4 Catchment imperviousness

Imperviousness of the urban areas is predominantly characterised by residential development. It was agreed with Council to use the same catchment imperviousness developed during the Port Lincoln SMP. Figure 4.1 illustrates the variation in directly connected impervious area across the wider catchment (refer Section 4.7 for more detail).



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CITY OF PORT LINCOLN

PORT LINCOLN FLOOD MAPPING STUDY
Catchment imperviousness by DCIA proportion

Figure 4.1

4.5 Catchment response time

The time of concentration for each sub-catchment was calculated within MapInfo (a GIS software) based on the following information:

- The distance between the inlet pit receiving runoff and the most distant vertex of the digitised sub-catchment boundary relative to the inlet pit.
- The change in elevation between the two aforementioned points.

During past flood studies conducted by Tonkin Consulting it was noted that the actual flow path is on average 10% longer than a direct line between the most distant vertex and the receiving inlet. Therefore, the automatically determined flow path length was multiplied by a factor of 1.1 to account for this difference.

The flow path slope was calculated by dividing the change in elevation across the catchment by the modified length of the flow path. A minimum slope of 0.2% is applied if the catchment slope is calculated to be less than this amount to prevent excessively large times and to represent likely minimum road grades.

In addition to the gutter-flow time, an allowance of 5 minutes for roof-to-gutter travel time for residential sub-catchments (or 10 minutes for commercial/industrial) was included as recommended in *Stormwater Drainage Design in Small Urban Catchments: A Handbook for Australian Practice* (Argue, 1986).

The equation to calculate time of concentration was as follows:

$$Time\ of\ Concentration\ (mins) = \frac{1.1 \times Flow\ path\ length\ (m)}{39.6 \times \sqrt{Max[Flow\ path\ slope\ (\%), 0.2\%]}} + (5\ or\ 10\ mins)$$

For urban areas, the pervious area time of concentration was calculated as the impervious area time of concentration plus 15 minutes.

For rural areas not modelled in RORB, a combination of sheet-flow travel time (fixed at 25 minutes) plus an overland flow travel time (set as 1.5 times that of the paved area), was used to determine a time of concentration for the pervious areas of each rural sub-catchment.

4.6 Rainfall depth and intensity

Design rainfall was determined from the 1987 Intensity–Frequency–Duration (IFD) dataset provided by the Australian Bureau of Meteorology (BoM).

The parameters to be used in the generation of rainfall depth is shown in Table 4.1.

Table 4.1 ARR87 rainfall intensity parameters

Parameter	2 year ARI	50 year ARI
1 hour rainfall intensity (mm/hr)	14.78	32.25
12 hour rainfall intensity (mm/hr)	2.97	5.46
72 hour rainfall intensity (mm/hr)	0.72	1.37
Average skew coefficient	0.65	
2 year ARI short duration geographic factor	4.51	
50 year ARI short duration geographic factor	15.09	
Latitude	34.725°S	
Longitude	135.850°E	

4.7 Catchment runoff estimation

Hydrographs were created using the Time–Area method and the ILSAX hydrological model. The ILSAX hydrological model splits each sub catchment into three sub areas: directly and indirectly connected impervious area, and pervious area. The pervious area losses were based on an Initial Loss – Continuing Loss model. The initial and continuing losses were varied depending on the ARI of the event.

Table 4.2 Loss parameters used for frequent events

Parameter	Unit	Value
Impervious area depression storage (all ARIs)	mm	1
5 year ARI pervious area depression storage (equivalent to an initial loss)	mm	20
20 year ARI pervious area depression storage (equivalent to an initial loss)	mm	25
100 year ARI pervious area depression storage (equivalent to an initial loss)	mm	35
Pervious area continuing loss (all ARIs)	mm/hr	3

4.8 RORB modelling parameters

The RORB runoff–routing model was used to estimate hydrographs from three rural catchments that would not be modelled as part of the 2D model. The layout of the three RORB models is shown in Figure 4.2.

The routing in RORB is based on two parameters – the non-linearity exponent, m, and the routing parameter, k_c.

The k_c value for each catchment was derived using Equation 3.25 from ARR87 (below), where A is the area of the catchment in km^2:

$$k_c = 0.6A^{0.67}$$

This equation applies to the south eastern area of South Australia and provides a value of k_c for catchments with an area less than 100 km^2.

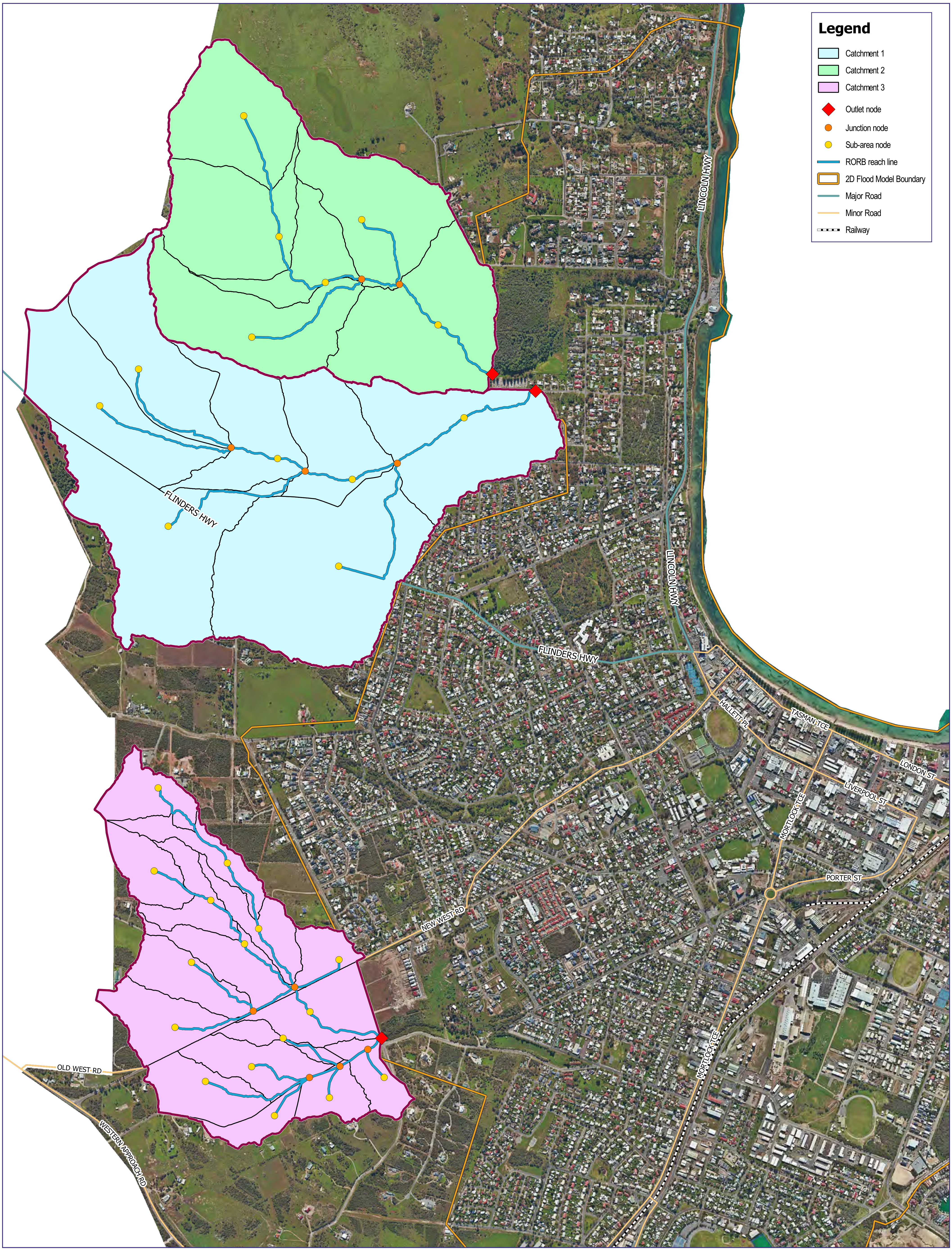
Calibration guidance in ARR87 suggests that m should be held constant at 0.8, whilst k_c is varied, unless there is good data to suggest another value of m is more appropriate. Since the local Port Lincoln catchments are ungauged it is considered that there is no evidence available to suggest an alternative value. Therefore, a value of 0.8 for the non-linearity exponent is recommended for this project.

The adopted initial and continuing losses for the RORB models were equal to those of the urban areas.

Table 4.3 RORB model parameters

Catchment	Description	Area (km^2)	k_c	m
1	Shaen Street rural catchment south	2.288	1.40	0.80
2	Shaen Street rural catchment north	1.539	1.13	0.80
3	Mortlock Terrace rural catchment (Grantala Road)	1.276	1.01	0.80

The peak flow of the RORB models compared favourably with regional regression equations from the Mt Lofty Ranges. The Mt Lofty Ranges and Port Lincoln share the same latitude and experience many of the same weather patterns and climatic conditions. Therefore, it is considered that this comparison is a valid check of the RORB output.



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Roads and rail data by PBI, 2014

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PORT LINCOLN FLOOD MAPPING STUDY
RORB model layout

Figure 4.2

5 Hydraulic modelling

5.1 Introduction

Hydraulic modelling uses the outputs of hydrologic modelling to determine the extent, depth and behaviour of flood flows within the study area. The resulting outputs provide an estimate of areas subject to flooding.

A detailed 1D–2D flood model was created for this study. The model was run to simulate storm events within the study area and generate flood inundation and hazard maps for the existing level of development.

5.2 Modelling software

The modelling was carried out using the TUFLOW computer program. The program simulates depth averaged, one- and two-dimensional, free surface flows, such as those that occur from floods and tides (BMT WBM, 2016). TUFLOW has the ability to dynamically link to the ESTRY one-dimensional (1D) model, which enables the creation of models containing both 1D and 2D domains.

The TUFLOW simulation engine uses a finite difference, alternating direction implicit (ADI) solution scheme developed by Stelling (1984) that solves the full 2D free surface flow equations. The solution scheme includes viscosity and sub-grid-scale turbulence terms that other schemes do not. The ESTRY component uses a Runge-Kutta explicit solution scheme to solve the full 1D Saint-Venant flow equations (BMT WBM, 2016).

TUFLOW was initially developed to model tidal estuaries. However, Tonkin Consulting assisted in pioneering the use of TUFLOW for urban flood inundation mapping. The drainage network is modelled in 1D and dynamically linked at each inlet/outlet structure to the floodplain represented in 2D. This allows for the integrated modelling of the drainage network and floodplain in urban areas.

5.3 Digital elevation model

A digital elevation model (DEM) of the study area was prepared by a subconsultant using data captured by LiDAR. LiDAR is a remote sensing method that uses laser pulses to measure the distance to features in the terrain. The reflected laser pulses are captured and processed to create a 3D model of the landscape.

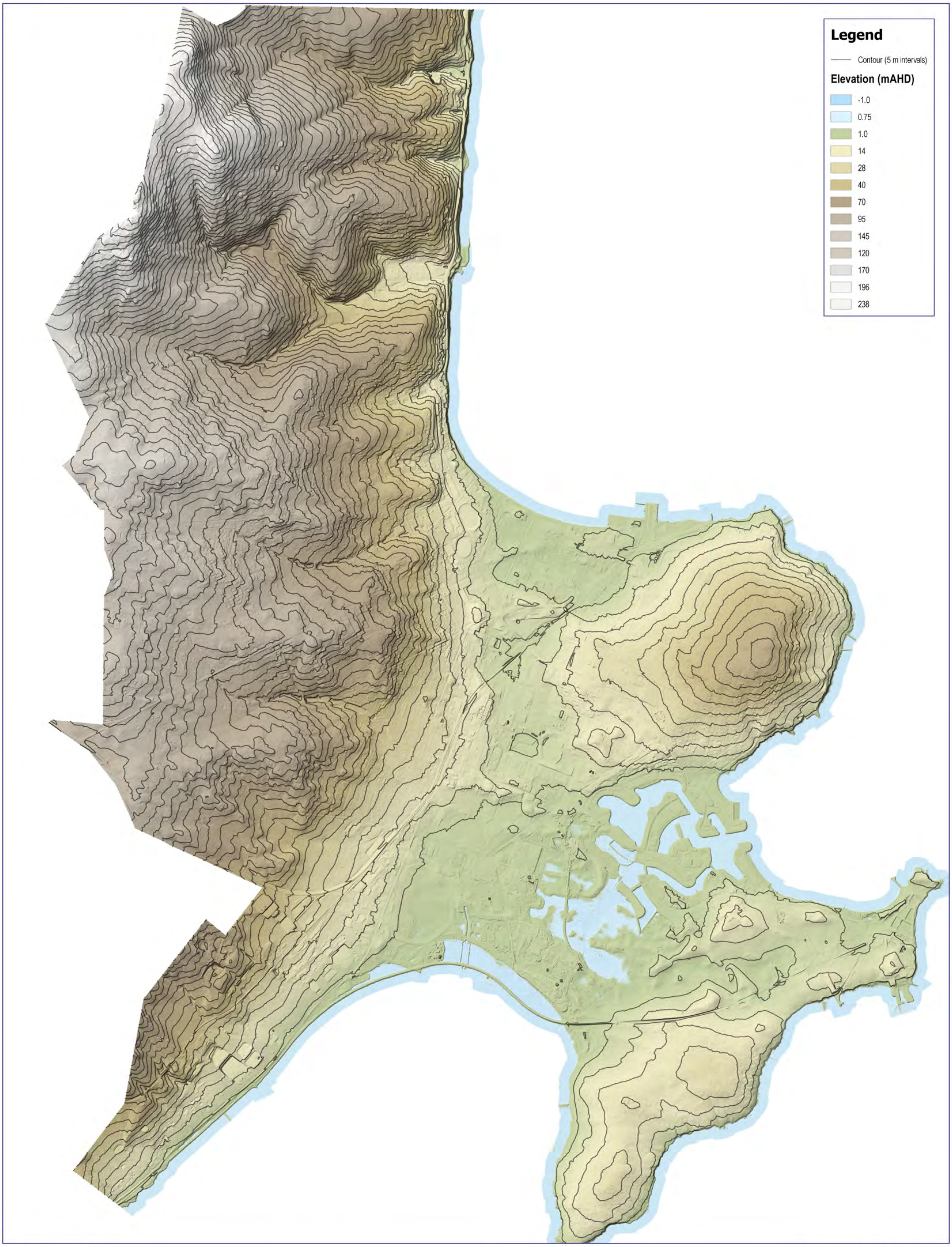
The spatial accuracy of the DEM was tested against test points obtained by field survey. Comparison with the test points showed that the desired ±0.1 m at the 68% confidence level vertical accuracy of the DTM was achieved.

Tonkin Consulting reviewed the DEM to ensure it was free of major errors. This review found some issues at joints between each DEM tile, which were fixed before the DEM was used for modelling. The full DEM obtained (before modification for modelling) is presented in Figure 5.1.

5.4 TUFLOW model setup

5.4.1 Computational grid cell size

Determining an appropriate cell size for the computation grid used by TUFLOW requires a compromise between the resolution of flood mapping and the simulation time required to run the models. Smaller 2D cell sizes more accurately reproduce detailed topography and the hydraulic behaviour of the flood, but significantly increase the amount of memory and computational power required to run the model. An understanding of the specific requirements for each study is needed in order to select an appropriate 2D cell size.



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Data Acknowledgement:
Elevation model from AAM Pty Ltd, 2016

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PORT LINCOLN FLOOD MAPPING STUDY
LiDAR derived Digital Elevation Model

Figure 5.1

A cell size of 4 m is considered by Tonkin Consulting as a good compromise between resolution and computational power and has been used for many studies previously undertaken by Tonkin Consulting. In this instance however, the steep nature of the catchment warranted a better representation of the topography than was possible to achieve using a 4 m cell size. A number of cell sizes were trialled but ultimately a 2 m grid size was adopted.

5.4.2 Computational time step

The selection of an appropriate time step for the 2D domain of TUFLOW is critically important to the accuracy of the model output. Time steps that are too large may result in models that are unstable. Time steps that are too small may unnecessarily increase simulation times. An appropriate time step will balance simulation time with the model's stability and numerical accuracy.

For this study, a time step of 0.5 seconds was adopted for the 2D domain. This achieved an acceptable balanced between simulation time and accuracy of the model results.

Ninety nine per cent of computational effort is expended solving the 2D surface flow equations and very little effort is needed to resolve the 1D domain. Consequently, the 1D domain time step has a negligible impact on simulation times. A time step of 0.1 seconds was used for the 1D domain.

5.5 Boundary and initial conditions

5.5.1 Outflow boundary conditions

Where water interacts with the boundaries of the model, special attention is required to ensure the correct hydraulic conditions at the boundary are recreated.

Where shallow sheet flow was expected to reach a model boundary, the boundary condition at that location was set to allow flow to freely leave the model. For channelised flows, the boundary condition was set to represent the hydraulic conditions downstream using an automatically generated, stage–discharge relationship based on the topography and expected hydraulic grade at that location.

For this model there were few boundaries that required special attention as the majority of runoff was eventually directed to the sea. The sea boundary is set to a static elevation of 0.79 mAHD which is equivalent to the Mean High Water Spring (MHWS) tide. The MHWS tide is generally accepted as an appropriate tide level for flood modelling when the joint probability of fluvial and tidal interaction is not being considered.

5.5.2 Inflow boundary conditions

Inflow hydrographs were generated for each ARI and duration of storm event analysed, as outlined in Section 4. In the urban areas, the inflow hydrograph for each sub-catchment was generally applied to an inlet pit, grate, or headwall. Inlet capacity tables were used to accurately represent the capacity of the network to capture surface flow from the road network. This allowed runoff to pass directly into the drainage network until the pit or pipe capacity was exceeded, with the excess spilling into the street network (2D floodplain).

Inflow hydrographs for the creeks along the upstream boundary of the study area were extracted from the RORB models (see Section 4).

5.5.3 Initial conditions

The catchment was assumed to be dry before the onset of the storm event. Consequently, no initial conditions were applied to alter storage levels of basins within the model.

To ensure stability of the model, the initial condition of areas of the model affected by marine waters, including any underground systems with an invert directly connected to the marine waters, were set to the same level as the sea outflow boundary.

5.6 Existing stormwater drainage infrastructure

5.6.1 Modelling of the pipe network

The drainage network consists mostly of underground drainage network discharging directly to the marine waters of Boston Bay. Two pump stations assist drainage of low lying areas. There are also a number of detention basins of varying volume within the drainage network.

Base drainage infrastructure data was previously developed by Tonkin Consulting as described in Section 3. This database was then converted into the required format needed by TUFLOW.

Invert elevations for the underground drainage were determined using the following procedure:

1. Where a surveyed pit lid elevation and a measured depth to the conduit invert was available, the invert elevation was determined by subtracting the measured depth from the surveyed pit lid elevation.
2. Where only a measured depth to the conduit invert was available, the invert elevation was determined by subtracting the measured depth from the DEM elevation at the pit location.
3. Where no measured depth to the conduit invert was available, design drawings were used if available.
4. Where no other information regarding the pipe invert or depth was available, an invert level equal to the DEM elevation minus 0.6 m nominal cover minus the conduit height was assumed.

The final inverts assigned to all pipes were reviewed for consistency. In areas where technique four was used to determine inverts, some manual manipulation was undertaken to ensure drainage networks graded downhill.

In addition to the above, the drainage network was checked as follows:

- Pipe diameters and box culvert sizes were reviewed to check for consistency with standard dimensions and that sizes generally increased in the downstream direction.
- Checks were carried out to ensure all drains were digitised in the downstream direction. For flood modelling it is preferable that drains be drawn in the downstream direction, so that flow results are positive in the downstream direction.
- Checks were made to ensure connectivity of the drainage network.

Particular effort was made to ensure the following structures were accurately represented:

- Shaen Street detention basin outlet structure,
- Shaen Street relief weir opposite Flaxman Street,
- Mortlock Terrace diversion weir,
- Mortlock Terrace outlet structure,
- Mallee Park basin outfall

5.6.2 Modelling of pump stations

There are two pump stations within the study area: one at Liverpool Street, the other at Le Brun Street. Pump stations can be represented in TUFLOW with great detail. For this model a detailed height–storage relationship was developed for the storage chamber of each pump station. A pump discharge curve was used to ensure pumps operated at the correct rate as water levels in the storage chamber changed. Operating rules were also implemented to ensure pumps operated at the correct times.

5.6.3 Modelling of inlets

Inlet pits were modelled using head–flow relationships to provide a good estimate of the inlet capacity of each pit. The head–flow relationships adopted were based on standard “Transport SA” pit capacity tables. Different curves were created for single, double and triple side entry pits (SEPs) as well as 900×900 grated inlet pits (GIPs).

5.6.4 Gutter flow

While the adoption of a 2 m grid cell size improved the modelling of flood behaviour, it was found that the model resolution was not sufficient to accurately represent the kerb profile of roads in steep areas. It was observed that the expected depth of flow in streets was not being achieved before water spilled into adjacent land. This resulted in a lower than expected prediction of the road network conveyance. To counteract this, the cells on the lower side of the roads in the affected areas were raised to approximately 0.15 m above the level of the water table. This ensured low flows followed road kerbs and allowed for better representation of the road network capacity within the model.

5.6.5 Allowance for blockages

During large storm events, objects could be swept into inlet pits, headwalls and creek channels, exacerbating flooding in the local area. Siltation, particularly at beach outlets, could also reduce the capacity of the stormwater network exacerbating flooding in the local area. Due to the broad scale objective of this flood study, no specific allowance has been made to account for blockages that may occur during storm events.

5.7 Bed resistance

The TUFLOW model requires bed resistance be specified by the modeller. In this model, the Manning’s n roughness coefficients is used to define the bed resistance. The bed resistance is a primary determinant of water depth within the 2D model domain.

Areas that have few obstructions to flow, such as the road reserve, have relatively low Manning’s n values. Conversely, areas with many obstructions, such as buildings, fences, and dense vegetation, have high Manning’s n coefficients.

The Manning’s n roughness coefficients used in this model are listed in Table 5.1. These values were selected based on current literature and the prior experience of Tonkin Consulting.

Table 5.1 Adopted bed resistance parameters

Land Use	Manning’s n
Houses/Residential areas, obstructions to flow	0.200
Medium and high density residential and commercial areas	0.300
Parklands with scattered trees	0.045
Grassed areas and bare ground	0.035
Roads (including verges)	0.020
Unlined creek channels	0.040-0.065
Plastic conduits	0.011
Concrete conduits	0.014

5.8 Modelling uncertainty

While every care has been taken in preparation of the TUFLOW model and the choice of the adopted parameters, all hydrological and hydraulic modelling has an inherent level of uncertainty. This inherent uncertainty is due a number of factors which may include any of the following:

- The accuracy and resolution of the DEM used and the interpretation of this information by the hydraulic model (refer Section 5.3)
- Dynamic changes to topography due to erosion or deposition of soil during a flood event; which can lead to changes in the distribution of flow. These processes have not been included in this model.
- Uncertainty in the rainfall pattern and catchment conditions prior to a flood. Actual flood events are dependent on the antecedent moisture conditions prior to rainfall, initial detention storage levels at the onset of rainfall runoff, and the intensity and uniformity of the rainfall event itself. The floods modelled by this study are based on design storm bursts which attempt to reproduce the expected average temporal pattern of a storm burst within specified rainfall zones. As such, individual rainfall events may exhibit different behaviour than those modelled.
- Estimation of input parameters to the model (such as runoff coefficients, time of concentration, Manning's roughness, and entry and exit losses).
- Blockage or failure of drainage infrastructure during a flood event.

5.9 TUFLOW simulations

Three different flood events were modelled:

- 5 year ARI flood event
- 20 year ARI flood event
- 100 year ARI flood event

For each flood event, a number of different storm durations were modelled in order to obtain the peak flood level at different points within the catchment. The durations modelled were:

- 15 minutes
- 30 minutes
- 1 hour
- 1.5 hours
- 3 hours
- 6 hours
- 9 hours
- 12 hours

Due to the relativity steep and short catchments—predominantly urban in nature—longer storm durations were not modelled as the peak flood level was achieved with the listed durations.

6 Modelling results

During each model run, the peak flood depth and hazard category (20 and 100 year ARI events only) was recorded across the 2D model domain. The results from each duration were spliced together to create a maximum depth and hazard envelope for each flood event.

Flood inundation and hazard maps were produced so that the impact of flooding could be visually analysed. The flood inundation and hazard data was overlaid onto aerial imagery, with the drainage network and street names shown to allow for easy identification and assessment of flooding. The flood depth data was classified into discrete intervals to allow for easy discrimination of flood depths. Flooding less than 25 mm deep is not shown as it is not considered relevant to the wider flood map.

The flood inundation and hazard maps have been produced and are presented in Appendix A of this report.

6.1 Model verification

A number of techniques were employed to verify the model implementation. Manual and automated checks of the pipe network to detect connectivity issues in addition to comparison of recorded peak flow against expected pipe capacity ensured confidence in the correct modelling of the pit and pipe network.

6.2 Validation of results

To help validate the TUFLOW model results, the peak recorded flow rate in key drains was compared with the theoretical capacity of the drains. In the majority of cases, the results compared favourably, providing confidence in the modelling of the underground drainage network. In a few cases, peak flow in the drainage network was significantly lower than the estimated capacity. It is believed that this is likely due to the upstream catchment producing a much lower peak flow compared with the capacity of the pipe.

As discussed in Section 5.6.3 visual inspection of the results showed that gutter flow in steeper areas was poorly modelled. Modifications were made to the DEM to better represent the full capacity of roadside gutters and the flow of surface water along the street network.

Draft flood inundation results were discussed with Council staff, however, no areas of unexpected flooding were noted.

6.3 Key flood prone areas

6.3.1 Liverpool Street and Bligh Street

The Liverpool Street catchment drains to a trapped low spot at the intersection of Liverpool Street and Eyre Street and is serviced by the Liverpool Street pump station. The pump station rising main discharges into the Mortlock Terrace box culvert system. The pump station was designed to deliver a peak flow of 10,000 gallons per minute (approximately 760 L/s).

In the 5 year ARI event, there is ponding of water up to 450 mm deep at the intersection of Liverpool Street and Eyre Street. This inundates the left most lane of traffic on both sides of Liverpool Street at the lowest part of the catchment. No building floors are inundated above floor level. However, vehicle movement through ponded water is known to cause waves and currents that push water into buildings. At the intersection of Bligh and Liverpool streets, the northwest corner of the roundabout is inundated to a depth of approximately 150 mm; no buildings are flooded. Generally, flooding in the 5 year ARI event is contained within the road corridor.

In the 20 year ARI event flooding is significantly worse in both locations. Several buildings are inundated above floor level and ponding is no longer predominantly within the road corridor. Ponding of water is over 500 mm deep at the Liverpool Street and Eyre Street intersection.

Of note in the 20 year ARI event is the runoff that flows from the rail corridor into Railway Place and contributes to flooding at the intersection of Liverpool Street and Porter Street. Water ponds east of Porter Street and is prevented from flowing into the trapped low spot of Liverpool Street by the Porter Street road crown. At the intersection of Bligh and Liverpool streets, the entirety of the roundabout is inundated.

In the 100 year ARI event (refer Figure 6.1) flooding is increased to almost 1.0 m deep at the intersection of Eyre Street and Liverpool Street. Due to the depth of ponding flood hazard is categorised as medium to high. At least fifteen buildings are affected by above floor flooding. Other properties are affected by ponded water in car parking areas of the property. The ponded area extends south to Napoleon Street. At the intersection of Bligh and Liverpool streets, ponding reaches a depth of approximately 600 mm. Flooding at the intersection of Liverpool Street and Porter Street overtops the Porter Street road crown. In addition to external runoff arriving from the rail corridor, there is runoff contributed from the Mortlock Terrace system which surcharges water at the intersection of Mortlock Terrace and Edinburgh Street. The surcharged water flows towards Napoleon and Liverpool streets and exacerbates flooding at the Eyre Street intersection.

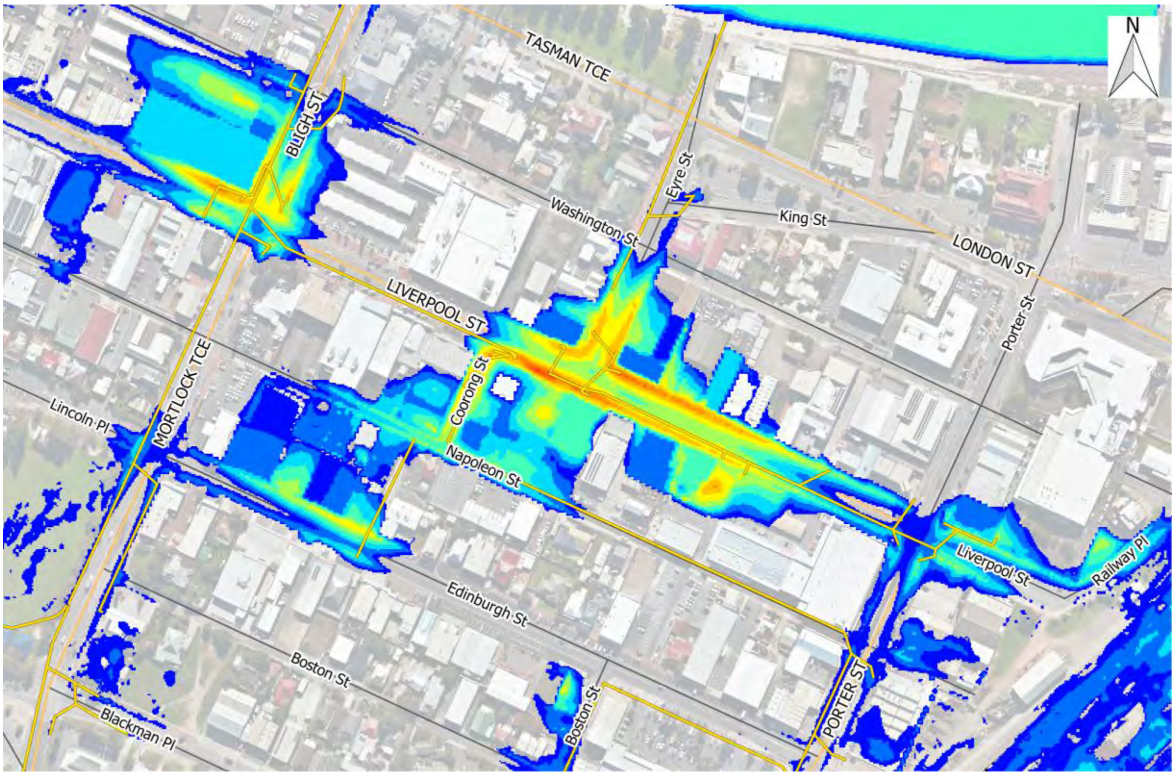


Figure 6.1 100 year ARI flood inundation in Liverpool Street

6.3.2 Light Street and Flinders Highway

This catchment starts at Hill Top Drive and drains through a shallow valley towards Boston Bay. Flinders Highway runs in an east–west direction slightly to the south of the valley centre. There is little underground drainage in the area. The largest underground system is located at the very bottom of the catchment and is designed to remove surface runoff from Flinders Highway and Light Street before the runoff crosses the Lincoln Highway.

In the 5 year ARI event there is nuisance surface flow (sheet flow less than 100 mm deep) throughout the catchment. Key locations affected are properties at Kaye Drive, Frobisher Street and Lear Place; properties along Flinders Highway between Trigg Street and Oxford Terrace; and properties between Nigel Street and Gloucester Terrace. Due to the depth of flow it is not certain whether houses would have above floor flooding.

In the 20 year ARI event the behaviour of flooding is worse, and sheet flow through properties is 120-160 mm deep. Flow along Flinders Highway is on average 150 mm deep. Due to the steepness of the highway, the velocity of flow is high and the hazard classification for the highway is rated as extreme.

In the 100 year ARI event the behaviour of flooding is worse, and sheet flow through properties is 200-250 mm deep (refer Figure 6.2).

An important behaviour to note in all events is that the flow of water along the valley centre is through private property because the road network is offset to one side.

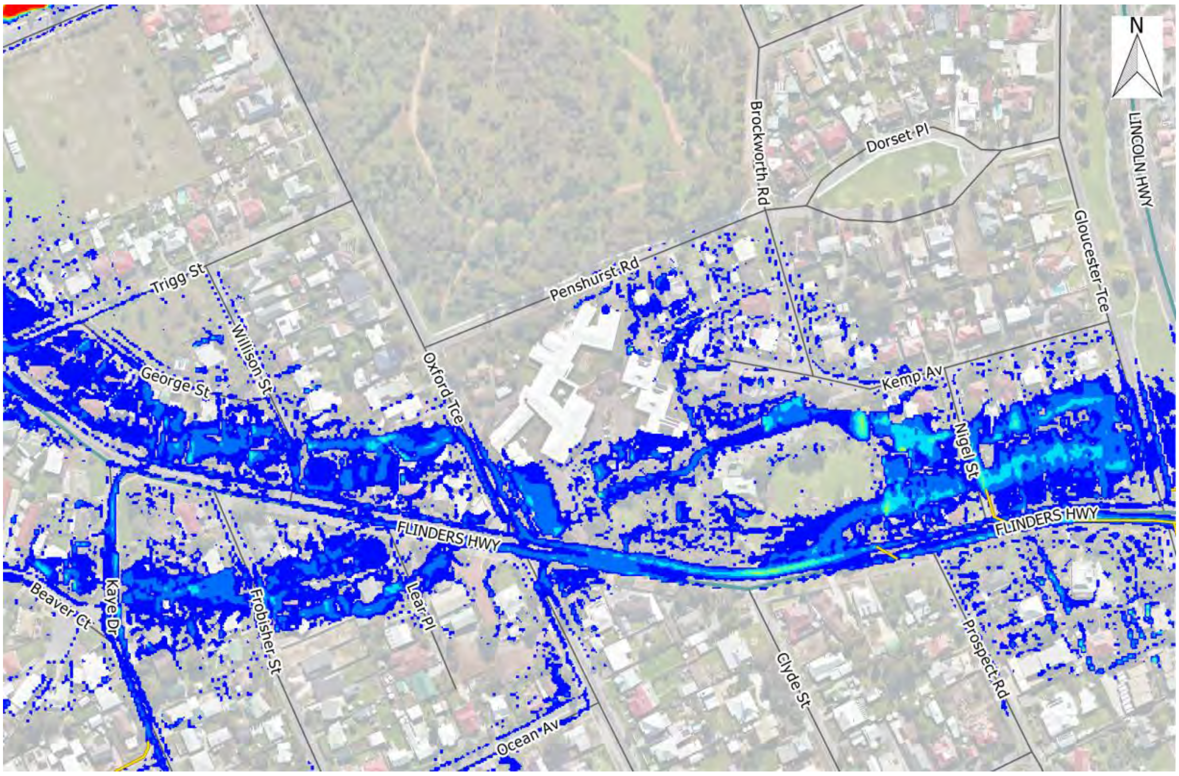


Figure 6.2 100 year ARI flood inundation along Flinders Highway

6.3.3 Le Brun Pump Station

Le Brun Street Pump Station services a 52 ha catchment that drains to a trapped low spot at the intersection of Simmons Street and Le Brun Street. The pump station has two pumps controlled by variable speed drives. The pump station discharges to a large junction chamber located in the roundabout at the intersection of Le Brun Street and Mortlock Terrace. It is believed that the pump station is designed to a 5 year ARI standard.

In the 5 year ARI event, ponding up to 300 mm occurs in Simmons Street, but is contained to the road reserve.

In the 20 year ARI event the pump station cannot cope with the increased runoff and the ponding reaches up to 900 mm deep—a few buildings are flooded as a result.

In the 100 year ARI event ponding reaches 1.3 m deep in Simmons Street (refer Figure 6.3). Many buildings are inundated. Homes between Mark Street and Luke Street due to sheet flow from St Andrews Terrace.

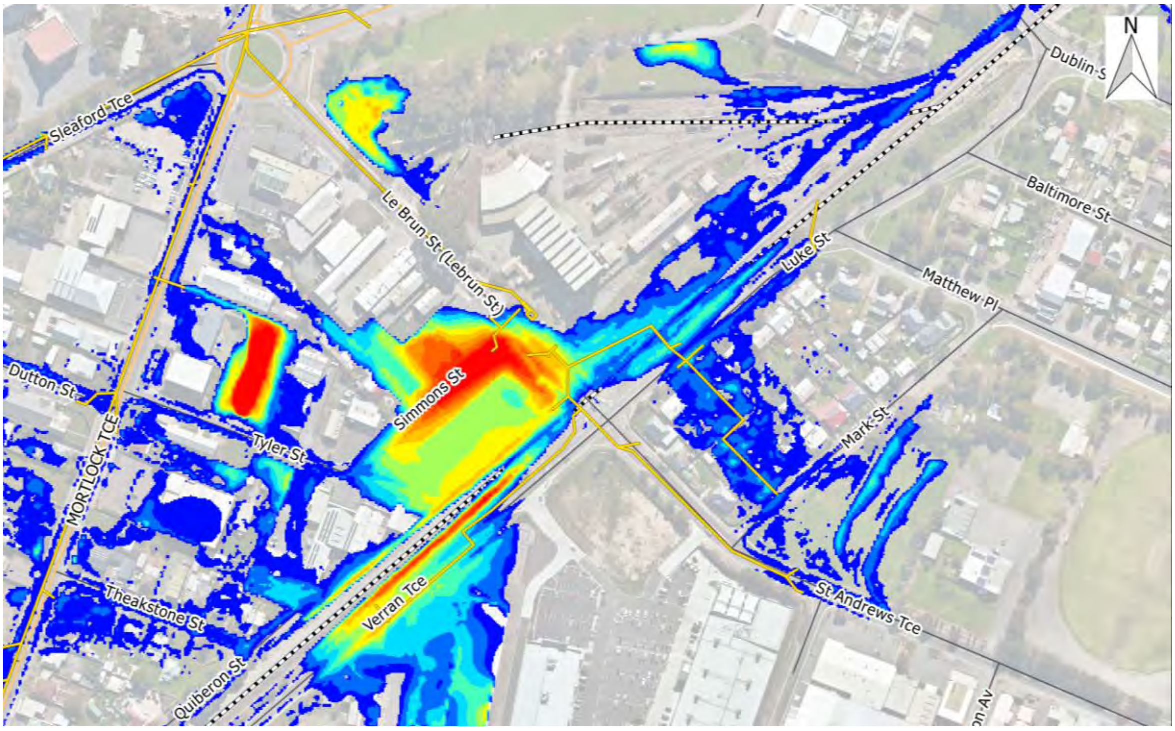


Figure 6.3 100 year ARI flood inundation at Le Brun Street Pump Station

6.3.4 Stamford Terrace, Windsor Avenue and Mallee Park detention basin

This catchment is characterised by a mixture of land uses including residential allotments, recreation areas and industrial allotments. The upper half of this catchment drains into the Mallee Park detention basin. The lower half of the catchment is serviced by the Stamford Terrace underground system (DN1050) that discharges to an open channel drain adjacent the Port Lincoln race course.

The flooding in this catchment is generally due to the limited capacity of the outlet pipe between Windsor Avenue and the race course open channel.

The intersection of Follet Street and Stamford Terrace is a known problem area (refer Figure 6.4). This spot suffers flooding in the 5, 20 and 100 year ARI events. In the 5 year ARI event a single dwelling is affected by above floor flooding. In the 20 and 100 year ARI events, 3 or 4 dwellings are affected.

In the 5 and 20 year ARI events, there is very little flooding in Hermay Crescent and Kuranda Street, however, in the 100 year ARI event there is significant ponding up to 400 mm deep (refer Figure 6.4). This results in up to 10 dwellings being affected by above floor flooding.

In the 5, 20 and 100 year ARI events, there is significant ponding of water in a trapped low spot southeast of the Ravendale Sports Complex. This location is on private property and not a formal basin. Water would remain trapped in this location for many days after the flood event.

In both the 20 and 100 year ARI events, the land between Windsor Avenue and the Port Lincoln race course is subject to widespread inundation averaging 200-300 mm deep (refer Figure 6.5). This inundation is caused primarily by runoff that overtops the Western Approach Road. Runoff from the north of the Western Approach Road flows east towards the intersection of Barley Road and Yandra Terrace. At the intersection runoff becomes trapped and eventually overtops the roadway and flows east towards Windsor Avenue.

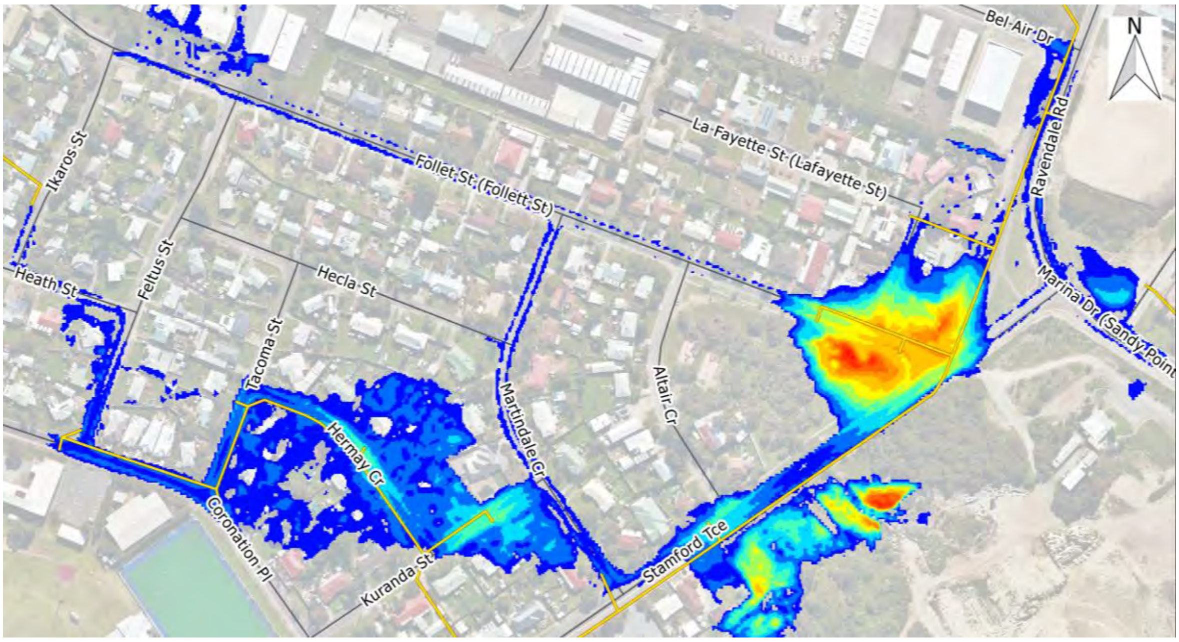


Figure 6.4 100 year ARI flood inundation in Follet Street and Kuranda Street

The intersection of Buberis Court and Proper Bay Road is a known problem spot (refer Figure 6.5). This location is the lowest part of Proper Bay Road and consequently surface runoff ponds in this location. The intersection is serviced by a DN450 pipe that connects to a DN1200 pipe that passes around the northern boundary of the race course. If there is enough rain, water escapes from the low spot and flows into the race course. This behaviour occurs in events with an ARI as low as 5 years.

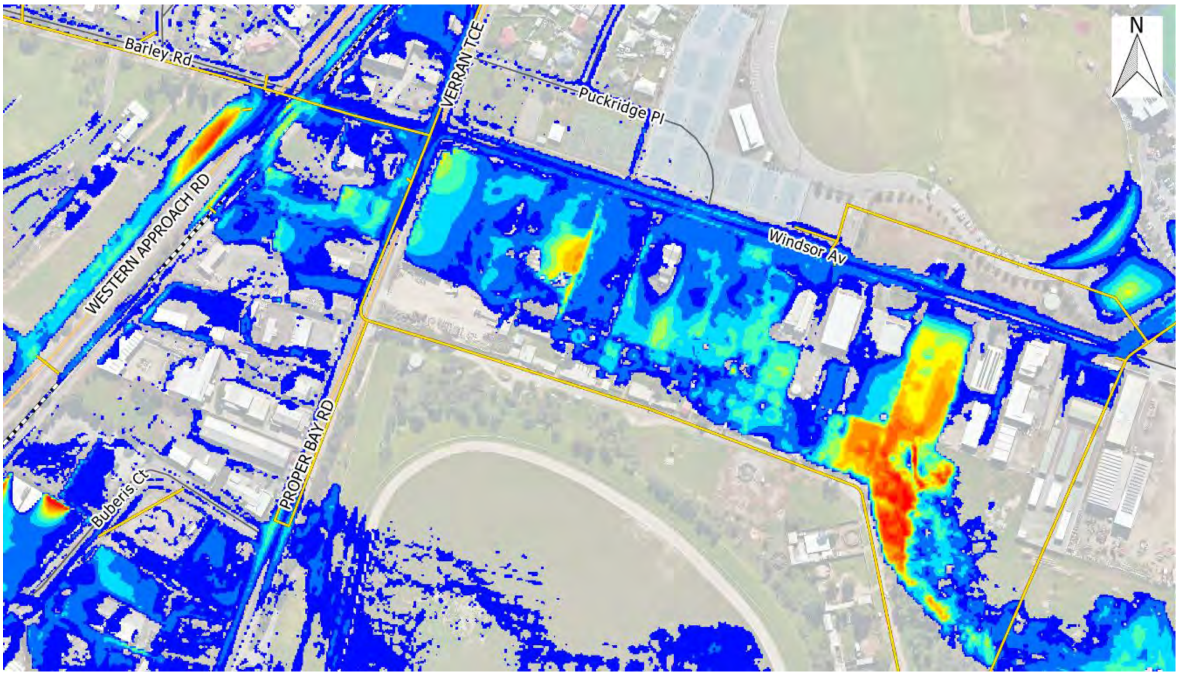


Figure 6.5 100 year ARI flood inundation in Windsor Avenue and Buberis Court

The industrial areas along Pine Freezers Road are affected by shallow sheet flow (up to 100-200 mm deep) during the 20 year and 100 year ARI events, likely due to limited drainage infrastructure in the area.

6.3.5 Shaen Street and Lincoln Highway

At 462 ha, the Shaen Street catchment is the largest under consideration in this study. The catchment consists of two large valleys which converge just downhill of the Happy Valley Cemetery. The catchment is dominated by open land and rural living areas. The Shaen Street catchment is serviced by a large detention basin just downstream of the valley confluence. The detention basin has a DN1950 corrugated metal pipe outlet that transitions to a DN1800 and then DN1650 metal pipe as the grade of the pipe line increases along Shaen Street. The outfall of the detention basin connects to an existing DN900 concrete pipe beneath the Lincoln Highway which discharges to Boston Bay. A large junction chamber has been constructed at the junction of the DN1650 and DN900 pipes. This chamber is designed to surcharge flow in excess of the capacity of the DN900.

In the 5 year ARI event, runoff from Happy Valley spills over Eric Avenue, as there is no underground drainage at this location. The runoff flows through the cemetery and Garden of Remembrance to the low spot of Bernard Place. The low spot is serviced by a DN450 concrete pipe, however, the capacity of the pipe is exceeded and runoff flows through the residential allotments in this location.

In the 20 year ARI event, the above behaviour is repeated but the depth of inundation is increased to 150-250 mm as runoff flows through the residential allotments north of the low spot. It is likely that the dwellings in this location are affected by above floor flooding during the 20 year ARI event. Lincoln Highway is overtopped by shallow sheet flow and the north bound lane is inundated to a depth of 300 mm. Despite the shallow depth of the overtopping flow, the velocity of the flow is high and this causes the flow to be categorised as high hazard.

In the 100 year ARI event, flow through residential allotments at the low spot becomes categorised as high to extreme hazard. Dwellings become inundated between 250-350 mm deep (refer Figure 6.6). Lincoln Highway is overtopped by deep fast moving flows and is consequently categorised as extreme hazard. As the Lincoln Highway is one of the main thoroughfare into Port Lincoln flooding in this location could pose difficulties accessing the township during emergencies.

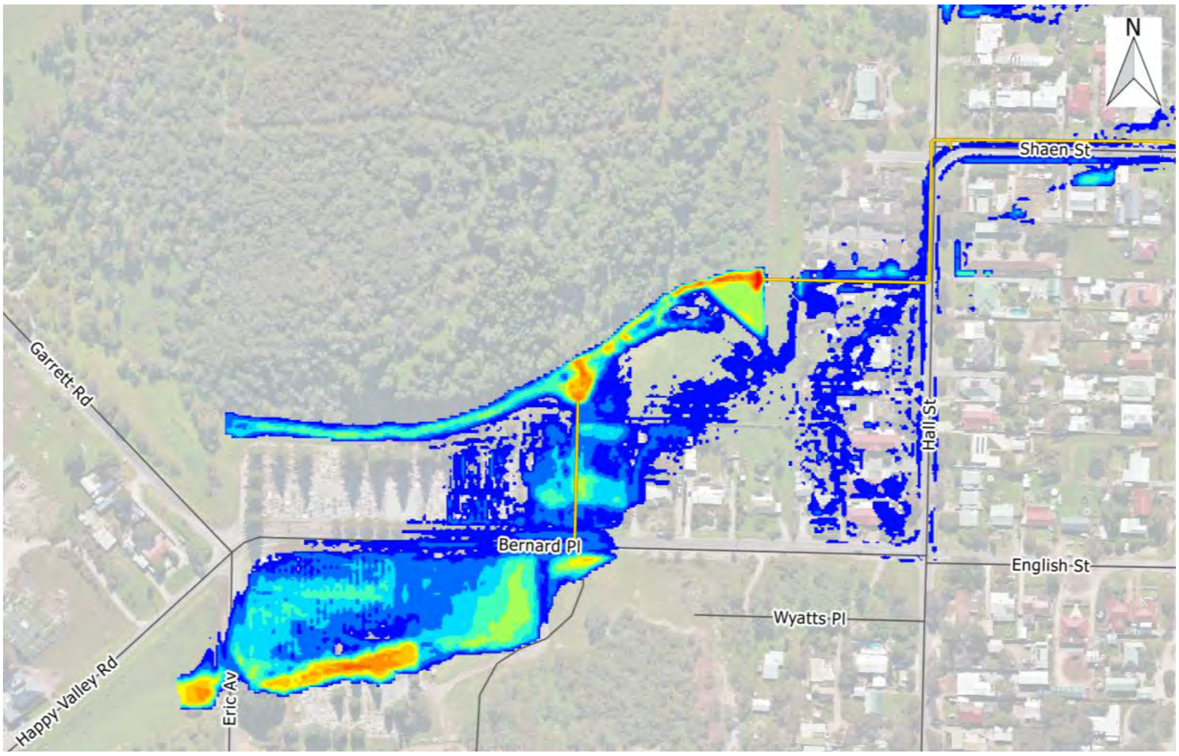


Figure 6.6 100 year ARI flood inundation in Shaen Street catchment

7 Flood damages assessment

7.1 Introduction

The flood damages assessment uses the Rapid Appraisal Method (RAM) developed by the Victorian Department of Natural Resources and Environment (DNRE, 2000). This approach allows for a rapid and consistent evaluation of the floodplain management measures in a cost-benefit analysis framework. The simplicity of this approach (compared to other methods) enables clear and concise documentation of the process to allow for reproduction in the future.

7.2 Data pre-processing

The RAM relies primarily on geographic datasets which require pre-processing before the damages assessment can be undertaken. In particular, cadastral data, land use types and property valuations are used.

7.2.1 Potential damage categories

A potential damage category was assigned to each allotment based on the land use type. The potential damage categories describe the relative damage potential of a property and reduces the number of property types to be assessed. The adopted strategy treats residential properties separately to other land use types. The flood damage categories used were low, medium, high and residential. Table 7.1 summarises how the different categories were assigned to the land use types. For properties that had no land use data, a land use code was assigned based on the aerial imagery.

Table 7.1 Summary of the potential damage category allocation

Property type	Potential damage category
Residential	Residential
Retail	High
Industrial	High
Public reserves	Low
Education institutions	Medium
Public utilities	Medium
Recreation areas	Low
Agricultural	Low

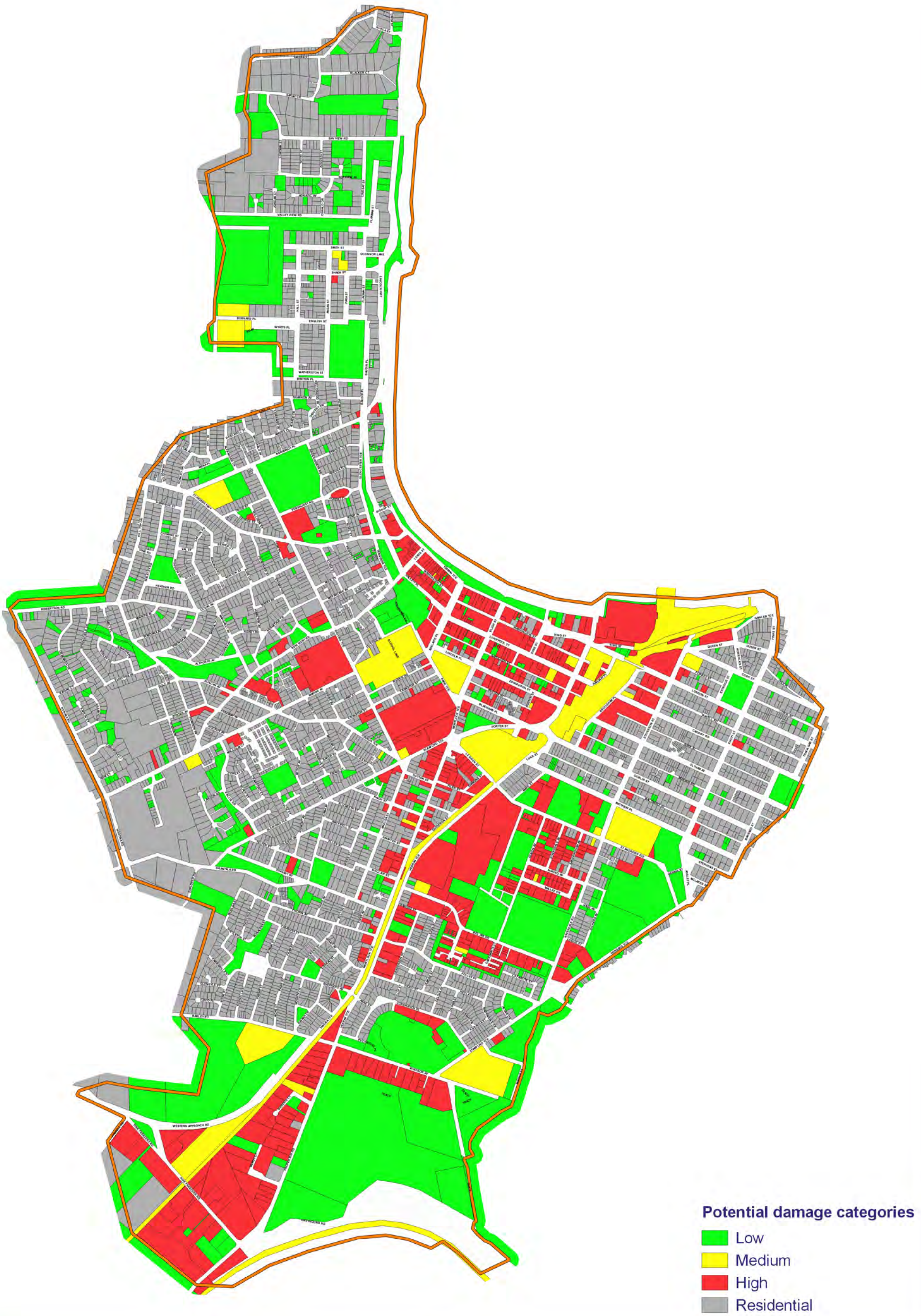
The allocation process was verified by comparing the assigned potential damage categories with the aerial imagery. The final potential damage category allocation is shown in Figure 7.1.

7.2.2 Property valuation data

Valuation data was only required for the residential properties. The property valuations were associated with each property in the GIS database. For the residential properties that had no valuation data, the average valuation of all residential properties within the assessment area was used. The average residential valuation adopted was $300,000.

7.2.3 Excluded properties

There were a number of reasons why properties were excluded from the damages analysis, as described in the following list.



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Data Acknowledgement:
Cadastre by City of Port Lincoln

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PORT LINCOLN FLOOD MAPPING STUDY
Potential damage category allocation

Figure 7.1

- **multi-storey properties:** only the ground level properties of a multi-storey complex were included. Properties above ground level were excluded, as including them would result in double (or more) the damage costs when, in reality, flood levels would need to be above 2 m to affect these properties.
- **small areas:** there were a significant number of parcels with areas less than 50 m^2. These areas predominantly included individual car parks and strata titles. If left in the dataset, they would have contributed a large residential damage when, in reality, very little property damage would occur.
- **subdivisions:** there was a double up in the number of parcels for several subdivisions, including both the total property as well as the new divisions. In situations where the aerial imagery showed one large property, the total property remained and the subdivisions were excluded. Vice versa for situations where the aerial imagery showed the subdivisions.
- **bodies of water:** any areas that are supposed to have large depths of water, such as the coastal area south of the Port Lincoln Racecourse, were excluded.
- **roads:** there is usually no roads included in cadastral data, so any that were included were excluded.

7.3 Direct damages

The direct damages are those caused by direct impact from flooding, including physical or functional damage. The direct costs were calculated for three different groups; residential properties, non-residential properties with an area less than 1,000 m^2, and non-residential properties with an area greater than 1,000 m^2.

7.3.1 Residential properties

Residential properties were only considered damaged if the flood depth at the centroid of the allotment was greater than 0.1 m. This is based on the assumption that the finished floor level (FFL) of all residential dwellings are 0.1 m above ground level.

The flood damage costs were calculated based on the property capital value (CV) relative to the average residential capital value (CV_{ave}), the depth of flooding above the FFL (d), and a base flood damage amount. Equation 7.1 shows the function used.

Equation 7.1

$$\text{Flood damage costs} = \$30{,}000 + \$30{,}000 \times d \times \left(\frac{CV}{CV_{ave}} \right)$$

A base flood damage value of $30,000 was adopted from work that Tonkin Consulting conducted in 2008, adjusted for inflation to get the present (2017) value. The past work by Tonkin Consulting selected this value from reviewing several flood damage assessments undertaken in Adelaide.

The above costs are only for damages to the dwelling. However, due to the high velocities of the flood waters an additional cost was added to account for the damage to property landscaping. For residential properties affected by flooding (i.e. where the flood depth was greater than 0.1 m) to an extent greater than 5% of the total property area, a flat rate of $4,000 was added to the direct damages.

7.3.2 Small non-residential properties

Non-residential properties with an area less than 1,000 m^2 were considered flooded if the depth of flooding at the centroid of the allotment was greater than 0.1 m.

A flat rate was used to determine the damages for this group of properties, shown in Table 7.2.

Table 7.2 Flat rate for flood damaged small non-residential properties

Potential damage category	Flat rate per property
Low	$4,000
Medium	$32,000
High	$80,000

7.3.3 Large non-residential properties

Non-residential properties with an area greater than 1,000 m^2 were considered flooded if any portion of the allotment was inundated to a depth above 0.1 m.

For each of the flooded properties, the total flooded area was determined and multiplied by the appropriate unit rate, shown in Table 7.3.

Table 7.3 Cost per square meterage of flood damage in large non-residential properties

Potential damage category	Damage cost per m^2
Low	$5
Medium	$40
High	$100

7.4 Indirect damages

Indirect damages consist of any loss in revenue caused by the effects of flooding. The indirect damage costs were calculated using the factors shown in Table 7.4. Indirect costs were estimated to be 60% of direct costs for medium and high damage potential categories, due to the higher disruption to services, transport and commerce (Kates, 1965; URS, 2005). Indirect costs were 15% of direct costs for residential and low damage potential categories, for opposing reasons.

Table 7.4 Indirect damage factors

Potential damage category	Indirect factor (%)
Residential	15
Low	15
Medium	60
High	60

7.5 Potential to actual damages conversion

The direct and indirect damages are not equivalent to realised damages due to mitigating factors such as the community's preparedness to flooding. Without taking this into consideration the damages are potential, not actual. Given that the community is experienced with flooding (i.e. they have dealt with flooding in the last five years) and the rapid response time of the catchment (warning time for floods is typically less than two hours), a potential to actual conversion factor of 0.8 has been adopted. This value is based on Table 3.5 of the Rapid Appraisal Method for Floodplain Management report (DNRE, 2000).

7.6 Exclusions

The following damages have not been included as part of the flood damages assessment:

- damage to vehicles
- damage to roads

- economic costs due to injury or loss of life

These cannot be easily assessed as part of a cadastral based flood assessment and have therefore not been included.

7.7 Flood damage costs

The flood damage costs were evaluated using the following six zones:

- **Zone 1:** the residential areas north of Flinders Highway (222 hectares)
- **Zone 2:** the residential area north of New West Road and south of Flinders Highway (244 hectares)
- **Zone 3:** the commercial precinct surrounding Liverpool Street (63 hectares)
- **Zone 4:** the predominantly residential area north-east of Matthew Place (116 hectares)
- **Zone 5:** the residential and industrial central region of Port Lincoln (335 hectares)
- **Zone 6:** the industrial precinct south of Cronin Avenue and Follett Street (310 hectares)

These zones are represented in Figure 7.3.

7.7.1 Actual damages

Costs for the actual flood damages are summarised in Table 7.5 and shown in Figure 7.2, broken down into each potential damage category (low, medium, high and residential).

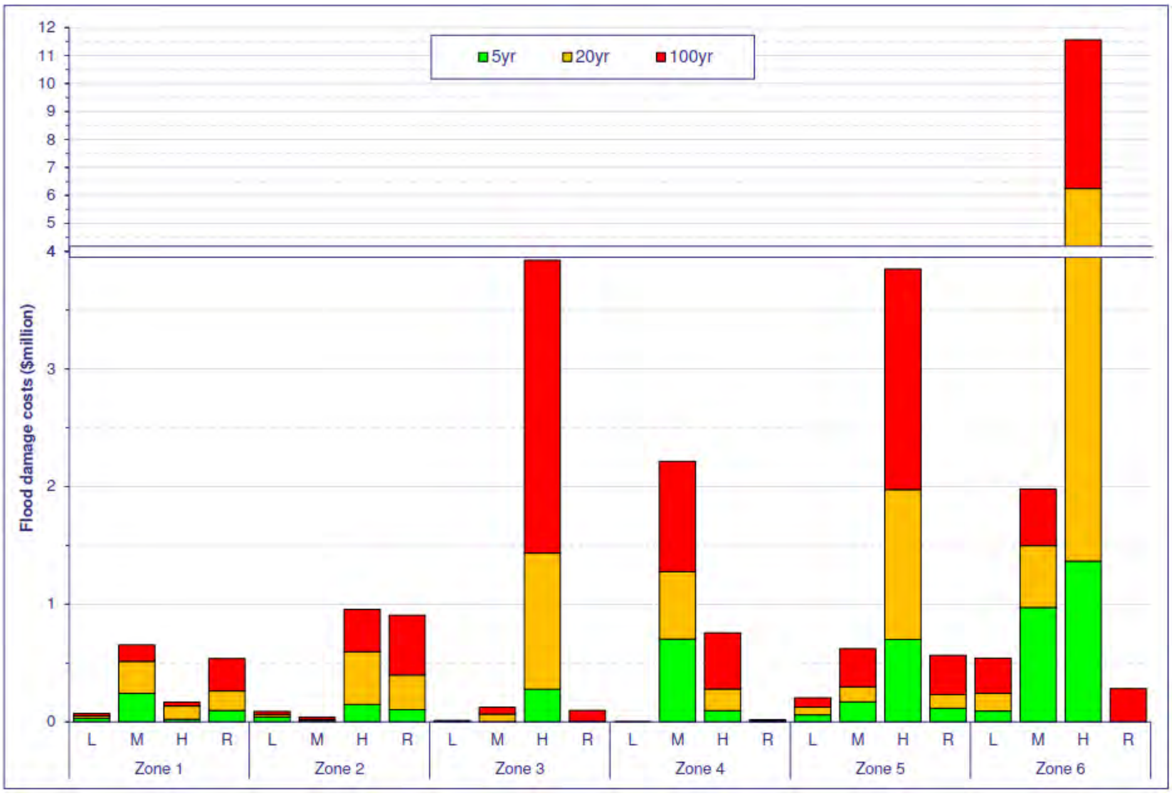
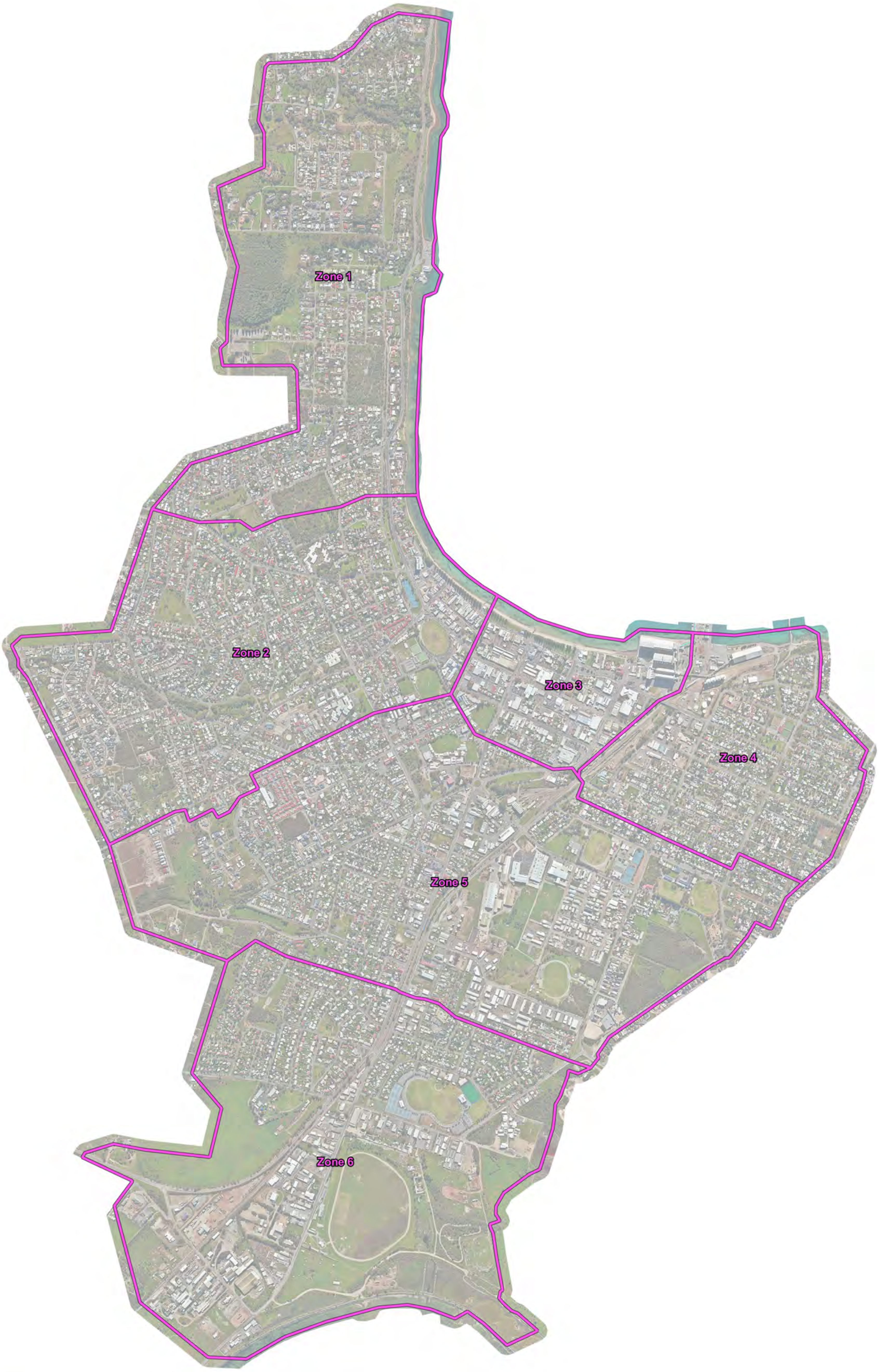


Figure 7.2 Breakdown of the actual flood damages for various ARI events



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Aerial imagery by AAM Pty Ltd, used with permission of City of Port Lincoln

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PORT LINCOLN FLOOD MAPPING STUDY
Flood damages assessment zones

Figure 7.3

Table 7.5 Actual flood damages in millions of dollars

	Flood Damage Category	Average Recurrence Interval 5	20	100
Zone 1	Low	$ 0.03	$ 0.05	$ 0.07
	Medium	$ 0.24	$ 0.51	$ 0.65
	High	$ 0.02	$ 0.13	$ 0.17
	Residential	$ 0.10	$ 0.26	$ 0.54
	Total	**$ 0.40**	**$ 0.96**	**$ 1.44**
Zone 2	Low	$ 0.04	$ 0.06	$ 0.09
	Medium	$ 0.01	$ 0.02	$ 0.04
	High	$ 0.15	$ 0.60	$ 0.96
	Residential	$ 0.10	$ 0.40	$ 0.91
	Total	**$ 0.31**	**$ 1.08**	**$ 1.99**
Zone 3	Low	$ 0.00	$ 0.00	$ 0.01
	Medium	$ 0.00	$ 0.07	$ 0.12
	High	$ 0.28	$ 1.43	$ 3.92
	Residential	$ 0.00	$ 0.00	$ 0.10
	Total	**$ 0.28**	**$ 1.51**	**$ 4.16**
Zone 4	Low	$ 0.00	$ 0.00	$ 0.00
	Medium	$ 0.70	$ 1.28	$ 2.21
	High	$ 0.10	$ 0.28	$ 0.76
	Residential	$ 0.00	$ 0.01	$ 0.02
	Total	**$ 0.80**	**$ 1.57**	**$ 2.99**
Zone 5	Low	$ 0.06	$ 0.13	$ 0.21
	Medium	$ 0.17	$ 0.30	$ 0.62
	High	$ 0.70	$ 1.97	$ 3.85
	Residential	$ 0.12	$ 0.23	$ 0.57
	Total	**$ 1.05**	**$ 2.63**	**$ 5.24**
Zone 6	Low	$ 0.09	$ 0.24	$ 0.54
	Medium	$ 0.97	$ 1.50	$ 1.98
	High	$ 1.37	$ 6.25	$ 11.56
	Residential	$ 0.00	$ 0.00	$ 0.28
	Total	**$ 2.43**	**$ 7.99**	**$ 14.36**
Grand Total		**$ 5.27**	**$ 15.74**	**$ 30.18**

For the range of storm events analysed, the majority of flood damages occur within high category properties. This typically consists of commercial and industrial properties such as offices, retail stores and manufacturing warehouses. On the other hand, there is very little damage to low category properties such as reserves or car parking lots.

Most of the residential damages occur along or in the vicinity of:

- **Zone 1:** Sarah Crescent and Normandy Place

- **Zone 2:** Nigel Street, George Street, Kaye Drive, Frobisher Street, Rodda Avenue, Casuarina Court and Orabanda Drive
- **Zone 5:** Tobruk Terrace, Dutton Street, Tyler Street and Luke Street
- **Zone 6:** Hermay Crescent

Ultimately, a large portion of the flood damages reside in Zone 6, predominantly affecting the industrial allotments west of Proper Bay Road and south of Windsor Avenue. This is due to Zone 6 having the largest area of flooding. Additionally, the zone comprises of a lot of high value properties.

The commercial precinct surrounding Liverpool Street (Zone 3) also has a large amount of flood damages relative to the size of the zone. The street has a significant area of flooding affecting commercial buildings, which have a high damage cost.

Table 7.6 summarises the number of flood damaged properties (any property that was considered flooded, as described in Section 7.3), broken down into each zone and each damage category (low, medium, high and residential).

Table 7.6 Number of flood damaged properties

Zone	Average Recurrence Interval											
	5				20				100			
	L	M	H	R	L	M	H	R	L	M	H	R
1	25	3	1	20	27	3	2	43	27	3	2	61
2	23	2	8	21	24	2	13	55	26	2	18	107
3	2	2	14	0	3	4	22	1	6	4	36	3
4	0	5	3	1	0	6	7	3	2	7	7	5
5	14	3	26	8	17	4	38	22	26	4	55	50
6	14	7	38	0	18	7	58	0	22	7	62	23
Total	**78**	**22**	**90**	**50**	**89**	**26**	**140**	**124**	**109**	**27**	**180**	**249**

Figure 7.4 shows the rate of change of flood damage with respect to a change in the annual exceedance probability (AEP). The annual exceedance probability is the probability of an event being equalled or exceeded within a year. It is different to the annual recurrence interval (ARI) which is the average time period between occurrences equalling or exceeding a given value. Although similar in nature they are not interchangeable; converting between the two requires use of the Langbein formula (Langbein, 1949).

7.7.2 Annual average damage

The annual average damage (AAD) is an estimate of the average annual cost of flood damages over a long period of time. It balances small frequent flood damages with large but less frequent flood damages and provides a convenient way to compare different floodplain management measures. It is a probability-weighted mean of the actual flood damages and is equivalent to the area beneath the flood damage-probability curve (refer Figure 7.4).

It was assumed that there are no flood damages for events that have an ARI smaller than 2 years (not shown in Figure 7.4).

The AAD for the study area was calculated to be $2.86 million, as shown in Table 7.7.

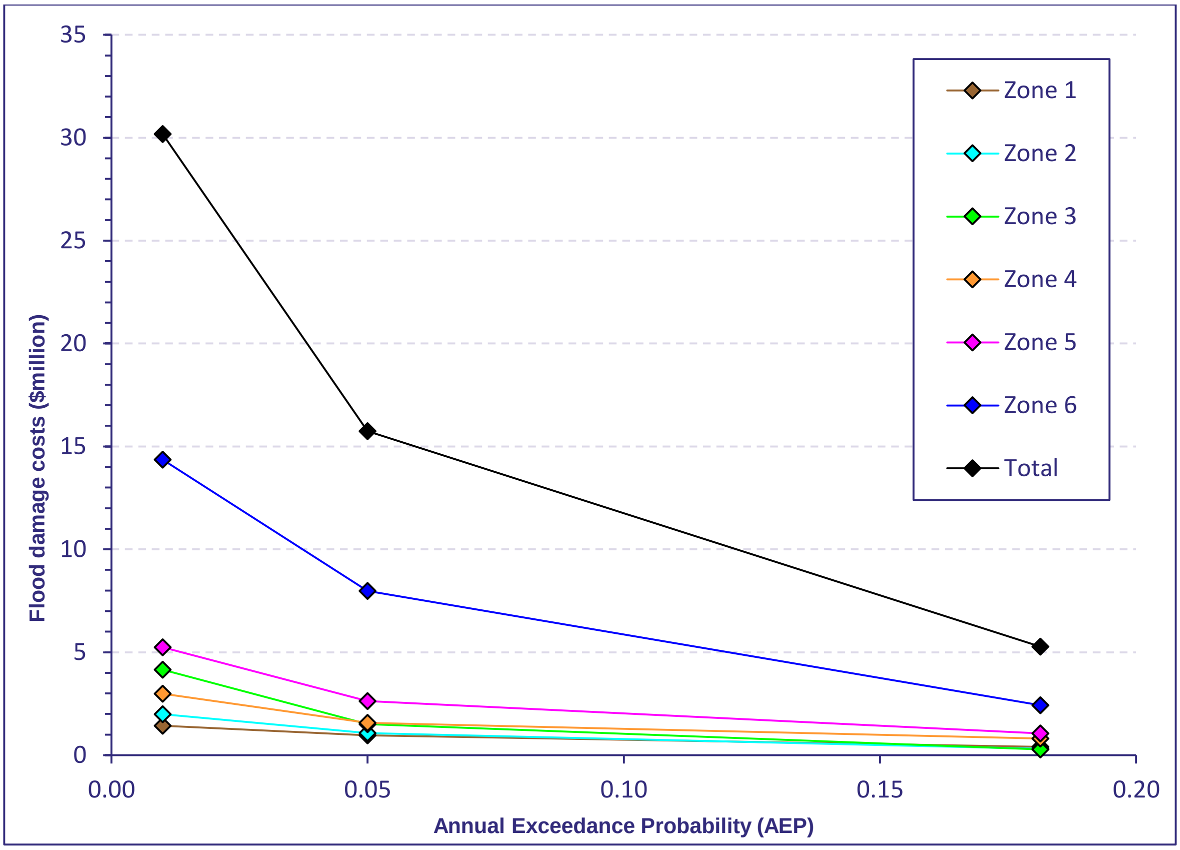

Figure 7.4 Flood damage probability curve

Table 7.7 Annual average damage costs

Zone	Annual Average Damage
1	$ 180,000
2	$ 190,000
3	$ 260,000
4	$ 330,000
5	$ 510,000
6	$ 1,390,000
Total	**$ 2,860,000**

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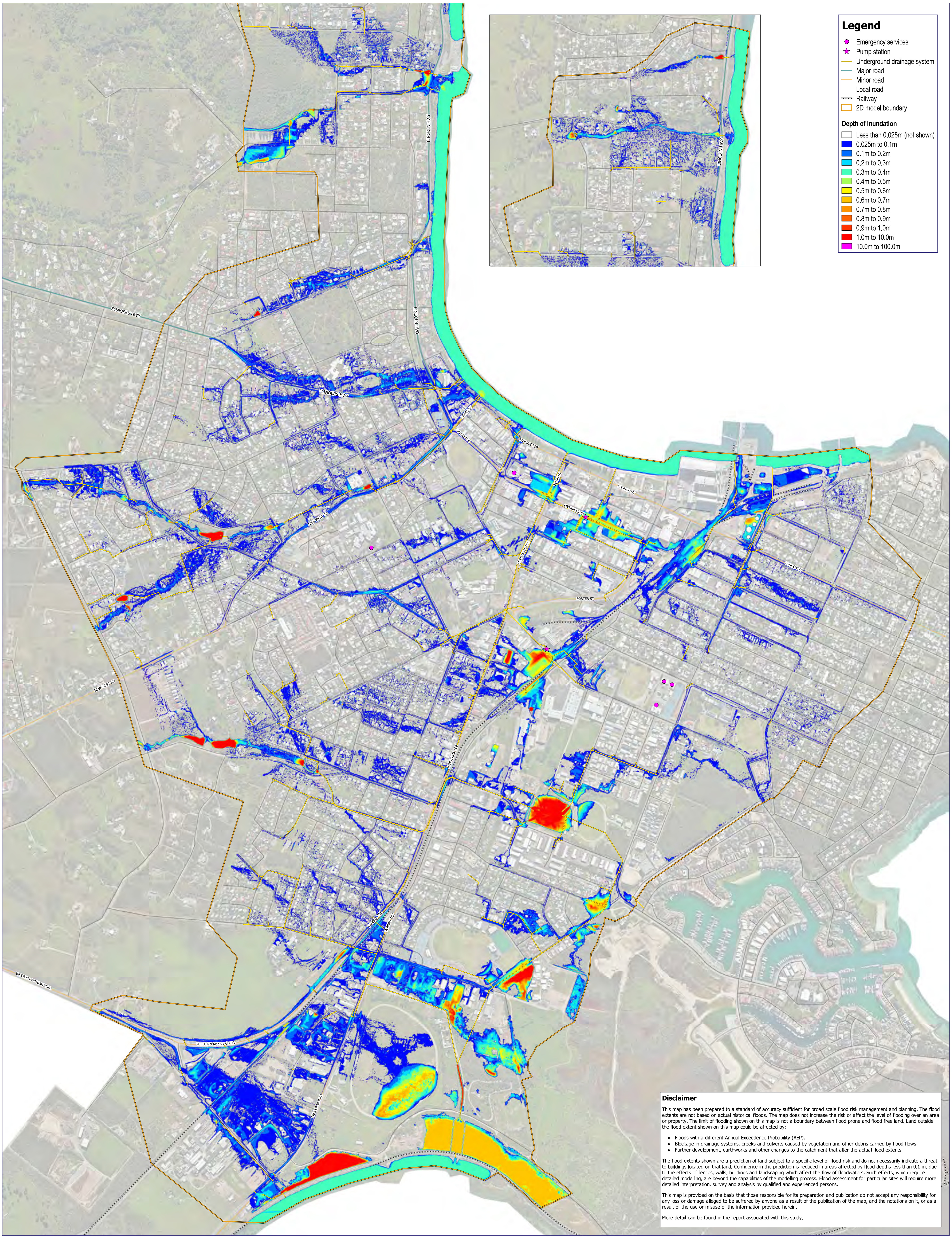
Appendix A

Flood inundation and hazard maps



Disclaimer

This map has been prepared to a standard of accuracy sufficient for broad scale flood risk management and planning. The flood extents are not based on actual historical floods. The map does not increase the risk or affect the level of flooding over an area or property. The limit of flooding shown on this map is not a boundary between flood prone and flood free land. Land outside the flood extent shown on this map could be affected by:

- Floods with a different Annual Exceedence Probability (AEP).
- Blockage in drainage systems, creeks and culverts caused by vegetation and other debris carried by flood flows.
- Further development, earthworks and other changes to the catchment that alter the actual flood extents.

The flood extents shown are a prediction of land subject to a specific level of flood risk and do not necessarily indicate a threat to buildings located on that land. Confidence in the prediction is reduced in areas affected by flood depths less than 0.1 m, due to the effects of fences, walls, buildings and landscaping which affect the flow of floodwaters. Such effects, which require detailed modelling, are beyond the capabilities of the modelling process. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

This map is provided on the basis that those responsible for its preparation and publication do not accept any responsibility for any loss or damage alleged to be suffered by anyone as a result of the publication of the map, and the notations on it, or as a result of the use or misuse of the information provided herein.

More detail can be found in the report associated with this study.

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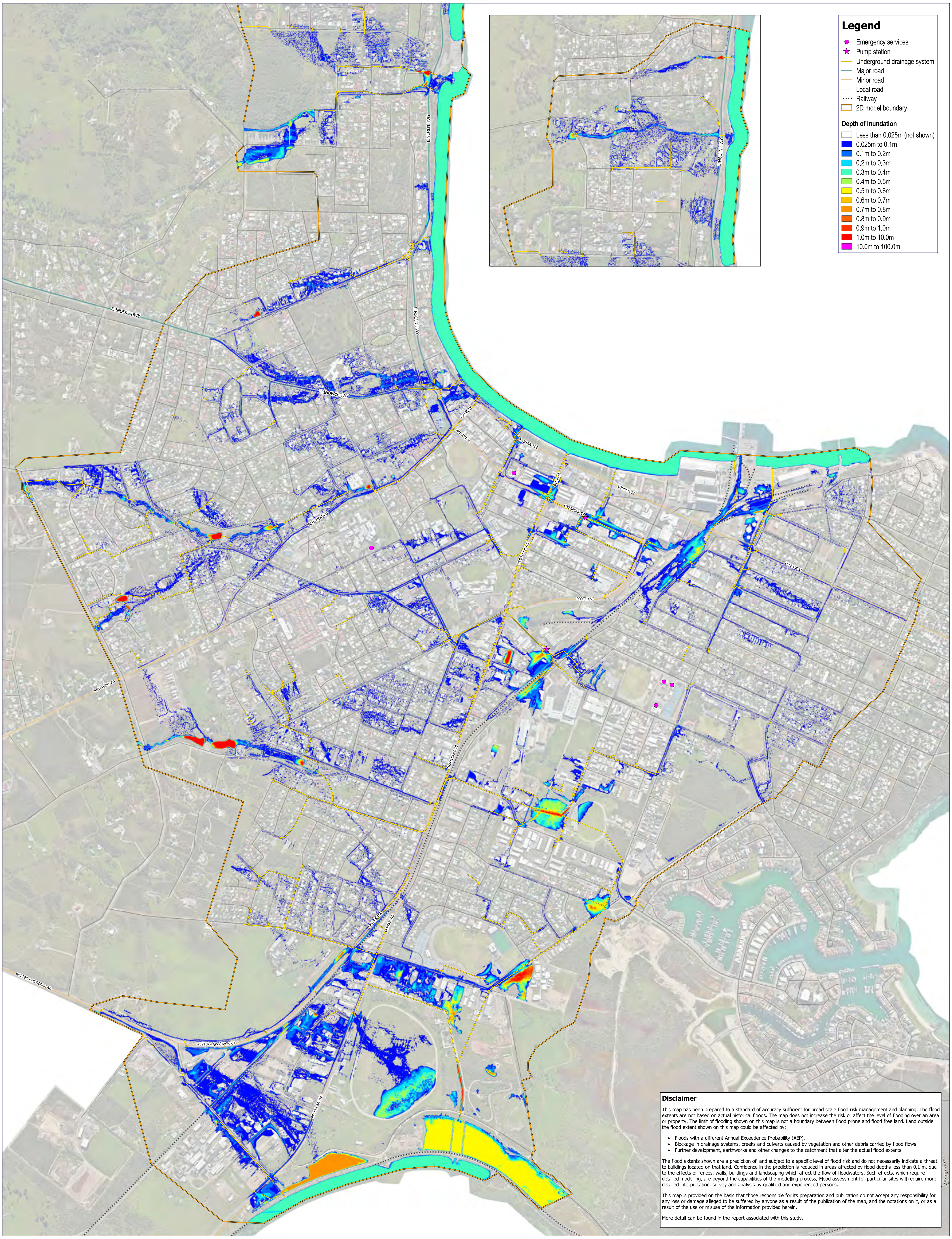
CITY OF PORT LINCOLN

PORT LINCOLN FLOOD MAPPING STUDY
Existing development 100 year ARI flood inundation

Map 1



Disclaimer

This map has been prepared to a standard of accuracy sufficient for broad scale flood risk management and planning. The flood extents are not based on actual historical floods. The map does not increase the risk or affect the level of flooding over an area or property. The limit of flooding shown on this map is not a boundary between flood prone and flood free land. Land outside the flood extent shown on this map could be affected by:

- Floods with a different Annual Exceedence Probability (AEP).
- Blockage in drainage systems, creeks and culverts caused by vegetation and other debris carried by flood flows.
- Further development, earthworks and other changes to the catchment that alter the actual flood extents.

The flood extents shown are a prediction of land subject to a specific level of flood risk and do not necessarily indicate a threat to buildings located on that land. Confidence in the prediction is reduced in areas affected by flood depths less than 0.1 m, due to the effects of fences, walls, buildings and landscaping which affect the flow of floodwaters. Such effects, which require detailed modelling, are beyond the capabilities of the modelling process. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

This map is provided on the basis that those responsible for its preparation and publication do not accept any responsibility for any loss or damage alleged to be suffered by anyone as a result of the publication of the map, and the notations on it, or as a result of the use or misuse of the information provided herein.

More detail can be found in the report associated with this study.

0 200 400 600 800 1000 m

1:8,000
at A1

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CITY OF PORT LINCOLN

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Job Number: 20150098
Filename: 20150098M002A.qgs
Revision: A
Date: 2017-05-01T15:12:58
Drawn: JDN

Data Acknowledgement:
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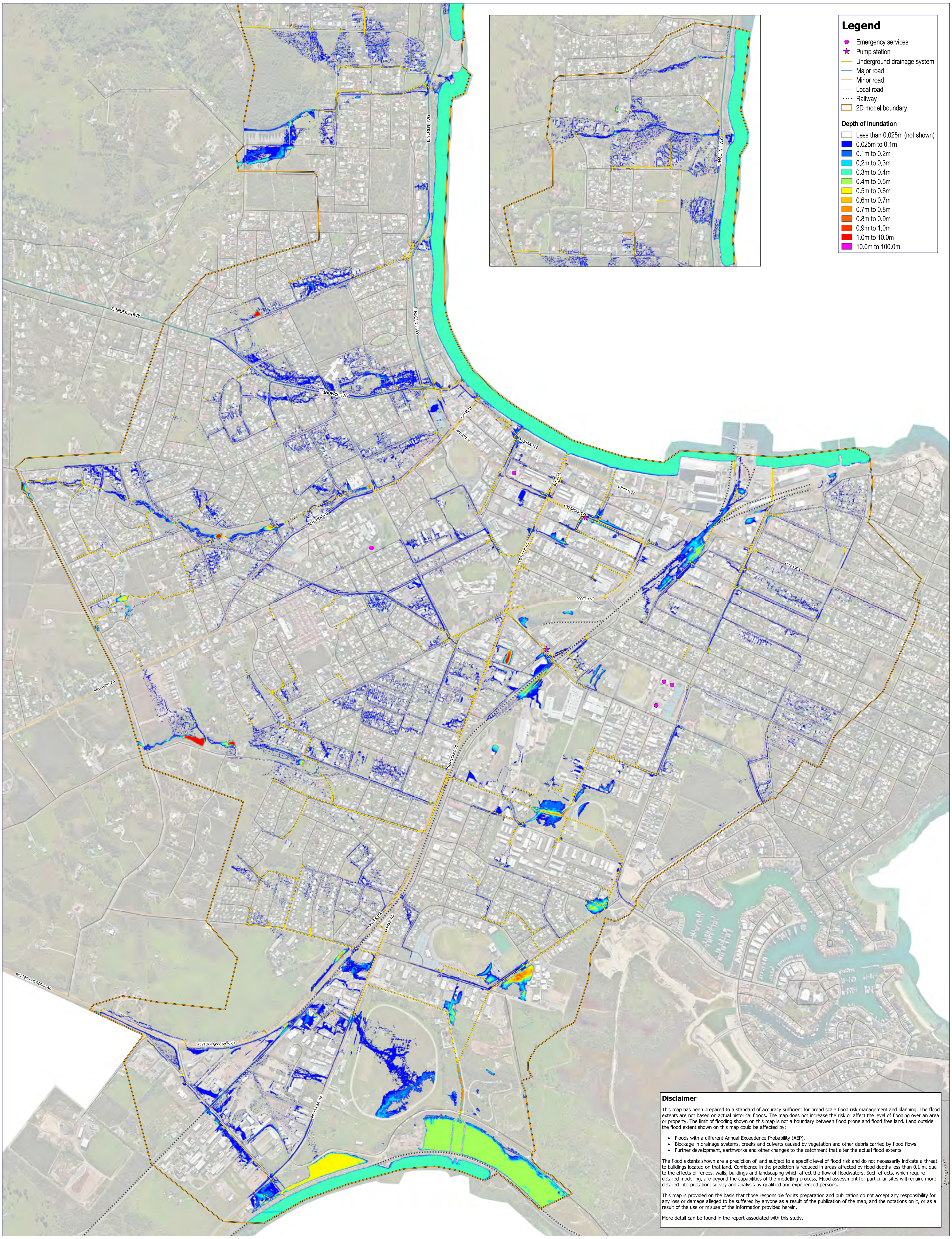
Road and Rail data by PBI, 2014

PORT LINCOLN FLOOD MAPPING STUDY
Existing development 20 year ARI flood inundation



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More detail can be found in the report associated with this study.

0 200 400 600 800 1000 m

1:8,000
at A1

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Job Number: 20150098
Filename: 20150098M003A.qgs
Revision: A
Date: 2017-05-01T16:07:37
Drawn: JDN

Data Acknowledgement:
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CITY OF PORT LINCOLN

PORT LINCOLN FLOOD MAPPING STUDY
Existing development 5 year ARI flood inundation

Disclaimer

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More detail can be found in the report associated with this study.

0 200 400 600 800 1000 m

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at A1

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CITY OF PORT LINCOLN

PORT LINCOLN FLOOD MAPPING STUDY
Existing development 100 year ARI flood hazard

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Job Number: 20150098
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Date: 2017-05-01T16:18:18
Drawn: JDN

Data Acknowledgement:
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Map 4

Disclaimer

This map has been prepared to a standard of accuracy sufficient for broad scale flood risk management and planning. The flood extents are not based on actual historical floods. The map does not increase the risk or affect the level of flooding over an area or property. The limit of flooding shown on this map is not a boundary between flood prone and flood free land. Land outside the flood extent shown on this map could be affected by:

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Job Number: 20150098
Filename: 20150098M001A.qgs
Revision: A
Date: 2017-05-01T16:17:38
Drawn: JDN

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PORT LINCOLN FLOOD MAPPING STUDY
Existing development 20 year ARI flood hazard