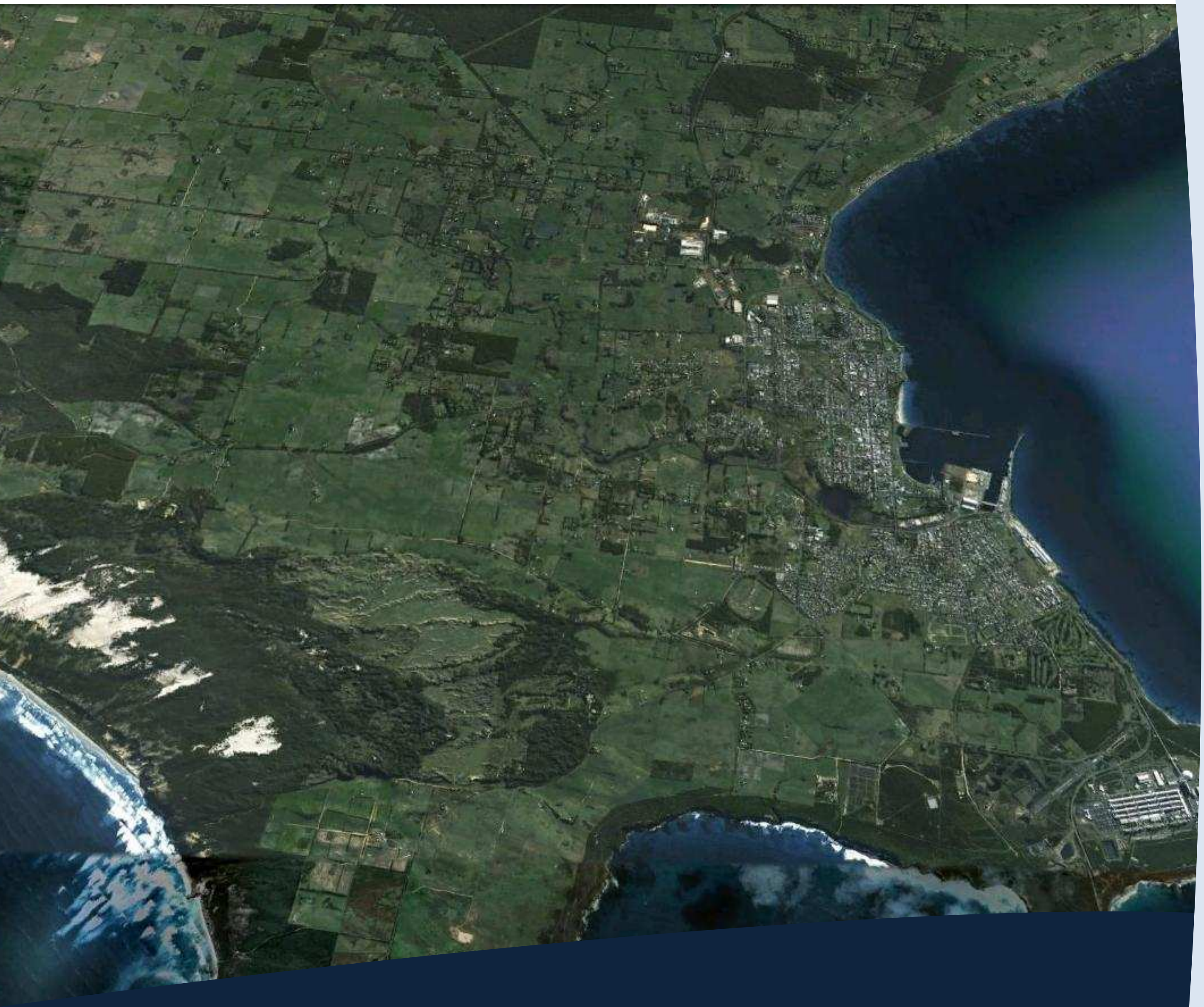

Portland Flood Study

Implementation Works

RM5521 Final v1.0

Prepared for Glenelg Hopkins CMA

June 2011

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GLOSSARY

Annual Exceedence Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Recurrence Interval (ARI)	The average or expected value of the period between exceedances of a given discharge or event. A 100-year ARI event would occur, on average, once every 100 years.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design flood	A significant event to be considered in the design process; various works within the floodplain may have different design events e.g. some roads may be designed to be overtopped in the 1 in 1 year or 100% AEP flood event.
Development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or overland runoff before entering a watercourse and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.
Floodplain	Area of land which is subject to inundation by floods up to the probable maximum flood event, i.e. flood prone land.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.

Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.
Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff, pipe and overland stream flow.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedence Probability.
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. For this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Topography	A surface which defines the ground level of a chosen area.

1 INTRODUCTION

Cardno have undertaken the Portland Flood Investigation for the Glenelg Hopkins Catchment Management Authority (GHCMA). The flood study aims to provide a definitive flood investigation to obtain a robust 1% probability flood extent for the Portland township. The flood extent will be used to provide flood advice, make planning decisions, declarations on flood levels and amendments to the Planning Scheme Zoning and Overlay Maps.

There is no streamflow data available for Wattle Hill Creek and Finn Street Creeks and accordingly a crucial component of the flood study is providing a robust estimate of the design 1% probability flood extent. Cardno have utilised previous studies and performed detailed hydrological assessment of the catchment to provide a detailed understanding of the limitations and uncertainties associated with the estimated flood events.

1.1 Study Area

Portland is located in the south west of Victoria approximately 70 km from the South Australian border. Portland was established in 1834 by Tasmanian Edward Henty, the town was Victoria's first permanent European settlement. It is located on Portland Bay and was established as the only deep water port between Melbourne and Adelaide. The main industries in Portland are the large aluminium smelter run by Alcoa and the port. The township has a population of approximately 9,900 and the local waterways include Wattle Hill Creek, Finn Street Creek and Fawthrop Lagoon. The detailed study area is shown in Figure 1.1.

The catchment relevant to flooding in Portland includes the catchments of Wattle Hill Creek and Finn Creek. Both of which discharge into Fawthrop Lagoon, and then through a canal to the Southern Ocean. The total catchment area is given as 164.4 km^2, of which approximately 10% by area is contributed by Finn Creek.

1.2 Scope of Works

The key aim of the project is to develop a greater understanding of flooding risk in the study area by undertaking a comprehensive analysis with all available data. Project outputs will drive the provision of flood advice, planning and development decisions, the declaration of flood levels and amendments to the extents of Floodway and Land Subject to Inundation zones and/or overlays within the Glenelg Planning Scheme areas.

Objectives of the project therefore are as follows:

(a) Hydrologically assess the catchment draining to Portland using all available data. This assessment should explore different methods of estimating flood flows to Portland.

(b) Assess the event probabilities of the 1946 event and another at least one other large rainfall event

(c) Produce design flood hydrographs for the full range of durations for the 10%, 5%, 2%, and 1% probability events.

(d) Identify and define any additional survey requirements.

(e) Create and calibrate a hydraulic model to 3 historic flood events, including the 1946 event, from local observed flood heights and estimated peak flow rates determined from the hydrologic analysis.

(f) Run the calibrated hydraulic model to determine design flood levels and extents allowing for sea level rise.

(g) Perform sensitivity analysis on the modelling results.

(h) Determine the critical sea levels for existing infrastructure under 1% probability flood and storm surge conditions.

(i) Produce flood planning maps based on sound rationale.

The original scope of works included provisions to undertake the 20% and 0.5% AEP flood events however these flood events were removed from the analysis. Similarly, the 1.2 m sea level rise scenario was removed from the analysis due to budget constraints.

2 AVAILABLE DATA

2.1 Summary of data sources

The following data was used in this study:

- Reports:
  - o Portland Floodplain Management Study (Rural Water Commission, 1988).
  - o Glenelg Flood Investigation (Cardno, 2008).
  - o Surry River Estuary Flood Study (WaterTech, 2008).
  - o Port Fairy Regional Flood Study (WaterTech, 2008a).
- Cross sections and structure details from VicTrack and VicRoads.
- Rainfall data for regional gauges (Bureau of Meteorology, 2010).
- State survey marks taken from LandVic SMES website.
- Digital cadastre information from GHCMA.
- Portland RORB model from GHCMA.
- Regional RORB parameters from numerous regional flood assessment reports.
- Aerial LIDAR survey from the Future Coasts mapping project (provided by the GHCMA)
- Tide Level data for Portland from 1982-2010 at 6 minute intervals (provided by the National Tidal Centre)
- Victorian Tide Tables for Portland (Port of Melbourne Corporation, 2010)

2.2 Site inspection

A site inspection was undertaken to become familiar with the local topography and physical features within the catchment. The field inspection was undertaken in July 2010. The location of the following floodplain features was noted:

- Bridges
- Roadways
- Culverts.

Appendix A includes photographs of the study area.

2.3 Sea Level and Tide Information

There is a high accuracy tide level gauge at the Port of Portland which was installed in 1991. Data was also sourced from the previous tide gauge at Portland dating back to 1981. This provides a significant data record to determine the expected sea levels at a range of recurrence intervals. The Victorian Tide Tables also provide additional information on the expected astronomical tide levels and the Highest Astronomical Tide (HAT). This information is shown in Table 2.1

Cardno undertook a frequency analysis of the tidal data to determine sea levels for various recurrence intervals (Table 2.2). These are the actual sea level, caused by the combination of astronomical and meteorological including storm surge.

Table 2.1 – Tidal Information at Portland (after POMC, 2010)

	Level (m AHD)
Highest Recorded Tide (6/6/2003)	1.124
Highest Astronomical Tide	0.71
Mean Higher High Water	0.47
Mean Lower High Water	0.21
Australian Height Datum	0
Mean Higher Low Water	-0.15
Mean Lower Low Water	-0.41
Lowest Astronomical Tide	-0.597

Table 2.2 – Sea Level ARI at Portland

ARI	Level (m AHD)
1	0.726
2	0.953
5	1.029
10	1.074
20	1.113
50	1.235
100	1.280

3 SURVEY DATA AND DIGITAL TERRAIN MODEL

LIDAR was supplied from GHCMA from the high quality Future Coasts program. This LIDAR covered the full model and allowed the development of a fine grid DTM to define the existing overland drainage network.

Additional plans for the area were received from GHCMA. These included river cross-sections and details of hydraulic structures along Wattle Hill and Finn Creeks. Structures within the system (such as weirs, levees and bridges) were also updated using the plans provided to ensure they were correctly defined within the model.

3.1.1 Ground Survey

Cardno have undertaken the survey utilising a sub-contractor Surfcoast Survey and Drafting Services P/L to undertake additional field survey to capture all structures not defined in available plans, capture additional river cross sections and to capture point survey markers to assess the accuracy of the survey and LIDAR data. Appendix B contains examples of the data obtained during the field survey program.

This additional ground survey was used to define structures present within the system as well as improve the channel definition in the upstream areas of the catchment.

3.1.2 Digital Terrain Model Validation

To assess the suitability of the aerial survey data for the floodplain modelling, a comparison of elevations between ground survey and aerial survey was undertaken. The Fawthrop Lagoon area has dense vegetation and the comparison allows for any suitable adjustments to be made to the aerial survey data in order to obtain a better representation of the land surface.

The aerial survey data has a quoted accuracy of +/- 150 mm from the natural surface at one standard deviation. In general, the survey data is within this accuracy along hard surfaces (roads) and areas with short vegetation. The check shows that only a small percentage of sampled points meet this criterion in the area upstream of the causeway at the entrance to Fawthrop Lagoon where the reeds and dense vegetation are significant. The dense vegetation causes the aerial survey to overestimate the ground surface level when compared to field survey. Downstream of the causeway, the aerial survey represents the ground surface reasonably. Table 3.1 and Table 3.2 show the differences between the aerial and surveyed ground levels in these areas. This is also shown in Figure 3.1.

Table 3.1 – Differences in survey and grid elevations downstream of causeway

Eastings	Northings	Grid Elevation (m)	Survey Elevation (m)	Difference (m)
553540.37	5754974.36	1.47	1.4	0.07
552919.99	5754605.42	3.183	3	0.183
552979.63	5754575.40	3.106	2.99	0.116
553133.77	5754669.32	3.387	3.33	0.057

Table 3.2 – Differences in survey and grid elevations upstream of causeway

Eastings	Northings	Grid Elevation (m)	Survey Elevation (m)	Difference (m)
551728.55	5755553.73	2.351	1.05	1.301
551889.55	5755602.73	1.372	0.7	0.672
551837.55	5755472.73	1.317	0.8	0.517
551761.05	5755595.73	2.009	1.75	0.259
551702.55	5755662.23	3.364	3.45	-0.086
551887.55	5755578.73	1.304	0.65	0.654
551790.05	5755564.73	2.982	1.25	1.732
551939.55	5755634.73	1.932	1.2	0.732
551889.55	5755487.73	1.326	0.8	0.526
551883.55	5755432.73	1.365	0.95	0.415
551696.55	5755862.73	1.539	1.2	0.339
551751.55	5755828.73	1.495	1.1	0.395
551690.05	5755768.23	1.518	1.1	0.418
551789.55	5755712.23	1.708	1.35	0.358
551841.05	5755793.73	1.841	1.05	0.791
551862.05	5755894.23	1.391	1.2	0.191
551991.05	5755811.73	1.964	1.5	0.464
551931.05	5755735.73	1.553	1.1	0.453
551739.55	5755963.73	1.368	1.15	0.218
551977.05	5756014.23	3.454	3.5	-0.046
551727.05	5755215.73	2.909	3.2	-0.291
551798.05	5755358.73	1.171	0.8	0.371
551897.05	5755349.24	1.135	1.45	-0.315
551835.05	5755334.23	2.652	2.65	0.002
551799.55	5755261.73	2.675	2.95	-0.275
551847.05	5755650.74	2.023	1.55	0.473
551945.05	5755622.23	1.932	1.35	0.582
551683.55	5755705.73	3.239	2.85	0.389
551675.55	5755952.73	3.131	3	0.131
551687.05	5756078.73	5.459	5	0.459
551881.54	5755596.95	1.392	0.683	0.709
551737.68	5755778.70	1.356	1.118	0.238
551867.30	5755471.76	1.334	0.864	0.47
551713.65	5755961.00	1.328	1.105	0.223

On average, the LIDAR data is 40 cm above surveyed elevations in the area upstream of the causeway around Finn St Creek with the largest difference being greater than 1 m. Downstream of the causeway, around the canal, the average difference is 0.1 m, which is within the stated accuracy of the aerial survey. As a result of the analysis, the model grid has been lowered by up to 0.5 m based on the difference between the available survey data and the aerial survey data. It is considered that this is a reasonable approach as the aerial survey is unlikely to have penetrated the thick vegetation.

Figure 3.2 shows the areas that have been lowered and the amount of change. The average difference between survey and model elevations after alterations is 0.14 m. This difference was considered appropriate as the thick vegetated layer would be unlikely to convey significant amounts of flow and these differences are unlikely to impact on the final flood extents and impacts.

4 HYDROLOGY

Hydrology is a key component of the Portland Flood Study as there is no recorded streamflow gauge within the catchment. As such, design flood hydrographs have been derived using catchment information and rainfall data captured from within and around the Portland catchment. As the catchment is ungauged, a range of hydrological methods have been applied and analysis has been undertaken on the uncertainties and sensitivity to specific model parameters for the range of methods. This approach provides a robust method that can calculate the uncertainty associated with the hydrology of the Portland catchment. In turn, this will allow an understanding of the confidence that can be associated with the hydraulic model results which will be used for future planning in the Portland region.

Previous reports and studies were referenced as part of this assessment, including:

- Portland Floodplain Management Study (Rural Water Commission, 1988)
- Glenelg Flood Investigation (Cardno, 2008)
- Surry River Estuary Flood Study (Water Technology, 2008)
- Port Fairy Regional Flood Study (Water Technology, 2008a)

For Portland, the main historic event that has known flood levels is the 1946 event and therefore this event forms the basis of the hydrological assessment. A number of steps were undertaken as part of the hydrology assessment, including:

- Review of existing models and studies.
- Regional rainfall assessment of the 1946 event
    - CRC Forge assessment
    - Individual gauge assessment
- Regional k_c, m, Initial Loss (IL) and Continuing Loss (CL) assessment.
- Rainfall runoff model from the Surry River.
- RORB model sensitivity assessment.
- RORB model design events.
- Use of the hydraulic model to assess volume and flow rates of the 1946 event.

4.1 Regional Rainfall Assessment

Five nearby rainfall gauges recorded the 1946 flood event (16th - 18th March 1946). The peak rainfall occurred was recorded on the 16th or the 17th of March, depending on location. The Portland rainfall gauge (90070), which is located approximately 4 km from the town, is the closest to the study area. The spatial location of each of the rainfall gauges is shown in Figure 4.1 and the rainfall records and the daily rainfall totals are summarised in Table 4.1 and Table 4.2 respectively. The spatial distribution of 24 hour, 48 hour and 72 hour rainfall for the event is shown as isohyets in Figure 4.2.

Table 4.1 – Rainfall records for the Portland area

Gauge Name	Gauge No.	Record	Length (years)
Portland	90070	1872 - 2008	136
Cape Bridgewater	90013	1905 – 2008	105
Tyrendarra (Ellangowan)	90038	1906 - 2006	99
Mount Richmond	90050	1940 - 2008	68
Heywood Post Office	90047	1887 - 1971	84

Table 4.2 – Recorded March 1946 rainfall for the Portland area

Gauge Name	Gauge No.	Distance (km)	Rainfall (mm)			
			16th March	17th March	18th March	Total
Portland	90070	4.0	104.1	74.2	12.4	195.3
Cape Bridgewater	90013	17.3	26.4	39.9	7.4	75.5
Tyrendarra (Ellangowan)	90038	26.0	71.9	148.6	33.8	258.6
Mount Richmond	90050	27.5	101.1	73.2	12.2	189.8
Heywood Post Office	90047	28.0	88.9	122.7	23.4	242.6

In Portland, the 1946 event brought almost 200 mm of rainfall over three days, however to the west of Portland at Cape Bridgewater the rainfall was less severe. To the north and east of Portland the rainfall totals range between 190 mm to 258 mm.

In order to assess the ARI of each of these events two methods were used:

- A rainfall frequency analysis (RFA) was undertaken for each individual gauge using the annual 24, 48 and 72 hour peak rainfall totals for the available record at each station. The RFAs provided a methodology for estimating the return period of the 1946 event. The results of the frequency analysis are found in Appendix C. It should be noted that the rainfall totals that have been obtained from the gauges are 'restricted' totals i.e. they are a total rainfall from 9am to 9am (24 hour period). The CRC-FORGE estimated rainfall totals are 'unrestricted' as they are the maximum 24, 48 and 72 hour period or rainfall unrestricted by gauge measurement times. The CRC-FORGE rainfall totals were adjusted to be 'restricted' totals to make these rainfall totals directly comparable.

- CRC-FORGE (Focussed Rainfall Growth Estimation) rainfall estimates were derived using a catchment area of 164 km^2. For any site within Victoria CRC-FORGE provides an estimate of the recurrence interval of various rainfall totals for events with a magnitude between the 1 in 50 and 1 in 2000 year ARI's with durations from 12 to 72 hours. These rainfall estimates can provide a second method for estimating the severity of the 1946 event. The average **point** rainfall was utilised and no areal reduction factors were applied to the rainfall totals. The CRC-FORGE rainfalls have been reduced by factors 1.16, 1.11 and 1.07 for the 24hr, 48hr and 72hr totals respectively to account for the 'unrestricted' nature of the data (Nandakumar et. al., 1997; see also Boughton and Jakob, 2008).

Recorded rainfall volumes for the five gauges adjacent to the Portland catchment are summarised in Table 4.3.

Table 4.3 – Approximate ARI of the 1-day, 2-day and 3-day hour duration rainfall totals for the 1946 event (restricted duration)

Gauge No.	1-day total (mm)	ARI (CRC FORGE)	ARI (RFA Gauge)	2-day total (mm)	ARI (CRC FORGE)	ARI (RFA Gauge)	3-day total (mm)	ARI (CRC FORGE)	ARI (RFA Gauge)
90070	**104.1**	253	351	**178.3**	1145	1059	**190.7**	874	1751
90013	**39.9**	< 50	3	**66.3**	< 50	4	**73.7**	< 50	6
90038	**148.6**	1791	185	**220.5**	> 2000	6349	**254.3**	> 2000	> 10000
90050	**101.1**	204	309	**174.3**	970	913	**186.5**	770	1517
90047	**122.7**	614	77	**211.6**	> 2000	4821	**235.0**	> 2000	8837

4.1.1 Portland Gauge (90070)

The Portland (90070) gauge was the closest gauge to the Wattle Hill Creek and Finn Street Creek catchments. For the 48 and 72 hour events CRC-FORGE equates the recorded rainfall volumes to approximately a 1000 year ARI event. The RFA indicates a higher recurrence interval of between 1000 and 1750 year ARI.

For the 72 hour event at the 90070 gauge the 1946 event is largest and is 90% higher than the second largest event (190.7 mm of rainfall in 1946 compared to the next largest of 108.3 mm in 1932). As a result, the RFA places the 1946 event on the extrapolated portion of the fitted relationship with no other rainfall totals for the distribution to be fitted around this magnitude. Although there are no events of similar magnitude, the RFA confidence intervals show that that there is 95% confidence that the ARI of the 1946 event is between a 600 and 4000 year event. Whilst this range of ARI is significant, it does reinforce the notion that the 1946 event was much rarer than the previously assessed 100 year ARI. It is also evident from this assessment that the most critical duration of the 1946 event was between the 48 hour and 72 hour storms.

4.1.2 Other Gauges (90013, 90038, 90047 and 90050)

Cape Bridgewater (90013) was the next closest gauge and recorded significantly lower rainfall totals that for Portland. As can be seen in Figure 4.2 it seems the rainfall event was not as significant at Cape Bridgewater. Tyrendarra (90038) and Heywood Post Office (90047) gauges are located north-east of Portland. At these gauges rainfall volumes were higher than those recorded at the Portland gauge and both gauges had estimates of greater than 2000 year ARI. It seems that the intensity of this rainfall event increased to the north-east of Portland.

To the north-west of Portland was the Mount Richmond (90050) gauge. This gauge recorded a similar rainfall volume as the Portland gauge for the 48 hour and 72 hour duration events. CRC-FORGE estimates the ARI of the 48 and 72 hour event as 800 year. Similar to the results at the Portland gauge, the RFA estimate at Mount Richmond was approximately the 1000 year ARI.

Overall the rainfall assessment indicated that the 1946 event was significantly higher at Portland than any other event on record (record ran from 1872 to 2008). The ARI for the 48 to 72 hour duration event was estimated at between a 500 and 2000 year ARI. This suggests that the flood event is higher than the previously estimated 100 year ARI.

4.2 Hydrological Modelling - RORB

As part of the Glenelg Flood Investigations Project (Cardno, 2008) a RORB model of the Wattle Hill and Finn Creek catchments was created. This model has been used to assess the flows likely under rare and extreme events for design purposes. The RORB hydrological model version 6.15 (Laurenson, Mein and Nathan, 2010) was used for this study. RORB calculates flood hydrographs from storm rainfall hyetographs and can be used for modelling natural, part urban and fully urban catchments. RORB is an industry standard model that has been used widely in previous studies.

Figure 4.3 shows the RORB model used for this investigation. RORB Sub-catchments were delineated by following the contours of the natural surface derived from NASA SRTM data. A fraction impervious of zero was used within the RORB model due to the mainly rural nature of the catchment.

4.2.1 Regional k_c, m and Loss Parameters

As there is no streamflow gauge within the catchment, there is no opportunity to calibrate a rainfall and runoff model (RORB) using known flow hydrographs and rainfall. As such, regional RORB calibrations were studied to find acceptable k_c, m and loss parameters for Portland. Within the region, there are nine appropriate catchments and the calibrated parameters are summarised in Table 4.4.

It is important to note that the catchment characteristics of the Portland area and from the examination of the 1:25,000 topography (Vicmap, 2008) and a map of the geomorphological units for the Glenelg-Hopkins CMA Region (Victorian Resources Online, 2008). The northern and north-eastern part of the catchment, from the creek line to the catchment boundary, is on 'volcanic derived plains with well developed drainage and deep regolith'. This shows that Wattle Hill Creek is well defined in the upper and middle reaches with limited floodplain reaches. The catchment west and south of the creek line is in 'sedimentary derived karst plains with depressions' and these areas are relatively flat and contain artificial drainage channels.

The catchments that exhibit the most similar catchment geomorphology traits are the nearby Surrey River catchments and Fitzroy River catchments. Unfortunately no calibrated RORB model could be obtained for the Fitzroy River and the Dav parameter (required for estimating the k_c could not be obtained from the reports containing this calibrated model. It should also be noted that even though the Surry River catchment is adjacent to the Portland catchment, Water Technology (2008) suggest that this catchment produced much lower flows than other catchments in the region due to unique catchment characteristics hence the calibrated parameters may not be appropriate to utilise for this analysis.

Table 4.4 – Regional kc, m and loss parameters for the Portland area

Catchment	k_c	m	Dav[1] (km)	Initial Loss (mm)	Continuing Loss (mm/hr)	Adjusted k_c
Moyne River	46	0.8	31.06	15	1.3	18.1
Russell Creek	6.45	0.8	7.15	20	1.26 to 2.13	11.0
Merri River	58	0.8	64.24	20	1.26 to 2.13	11.1
Wando River	15	0.8	15.82	1.1 to 9	0.17 to 1.19	11.6
Henty Creek	32	0.8	18.96	0 to 20	1.2 to 2.2	20.7
Dundas River	15.75	0.8	13.24	3 to 12	0.19 to 3	14.6
Glenelg R @ Casterton	115	0.96	118.68	25	2.5	11.9
Sunday Creek	17.29	0.8	7.89	17.5	2.25	26.8
Surry River	75	0.8	unknown	4	1.3	N/A

[1] The average distance from the centroid of all of the sub-catchments to the outlet.

4.2.2 Regional k_c assessment

As k_c varies proportionally with the flow path length it is possible to adjust the k_c value to suit the target catchment, in this case Portland. It is important therefore to use the adjusted k_c value, with the relationship used to adjust k_c parameter is shown in Equation 1. This relationship was applied to scale the k_c value (calibrated at another catchment) to the Portland model by using the average distance from sub-catchments to the model outlet.

$$k_{c\,Portland} = \frac{D_{av\,Portland}}{D_{av\,Catchment}}\ x\ k_{c\,Catchment}$$ **Equation 1**

Where:

k_c Portland is the k_c parameter for Portland.
k_c Catchment is the k_c of the regional catchment.
D_{av} Portland is the average distance from the sub-catchment centroid to the outlet for Portland.
D_{av} Catchment is the average distance from the sub-catchment centroid to the outlet for regional catchment.

From the assessment it can be seen that the adjusted k_c parameters range from 11.0 up to 26.8 with the majority of the k_c parameters being between 11 and 18. Previous studies for the Portland catchment have used a k_c value of 12.5; this value is considered valid and will therefore be adopted for the current study. Due to the range of calculated values, a sensitivity analysis will be carried out on the k_c parameter to show the range of flood peaks under the design flood conditions.

In addition to the regional k_c parameters that have been examined, the regional estimate equation (ARR98) generates a k_c of 13.5 and 25.5 for Victorian catchments with a mean annual rainfall (MAR) of less than 800 mm and greater than 800 mm respectively. The MAR for Portland is 830 mm which is on the cusp between the two estimates. The range of parameters explored (k_c of 11 to 18) includes this lower estimate of the k_c parameter which is more consistent with the regional estimated of k_c parameters.

4.2.3 Regional loss assessment

Initial and continuing losses can be estimated from regional catchments. No adjustment is needed to fit them to the Portland catchment – an appropriate value is to be chosen based on a regional assessment. Table 4.2 shows initial loss ranges from 0 to 25 mm and as such an initial loss of between 10 and 20 mm is appropriate for design storms. Continuing loss (Table 4.2) shows wide variety between the regional models, but consensus is between 1.5 and 2.5 mm/hour.

4.2.4 Adopted RORB model parameters

Regional assessment of the RORB models indicates that the following parameters should be utilised:

- **k_c** in the range of 11 to 16, with an adopted value of **12.5**.
- An '**m**' parameter of **0.8**.
- Initial losses between 10 and 20 mm.
- Continuing losses between 1.5 and 2.5 mm/hour.

4.3 RORB Model Sensitivity Analysis

As the RORB model could not be calibrated in the catchment, it is important to perform sensitivity analysis on the RORB model input parameters in order to gain an understanding of the uncertainties associated with them.

Regional parameter assessment showed that the values that should be investigated include the k_c, initial losses and continuing losses as these all had a range of possible values. It is unlikely that a value other than 0.8 would be utilised for the 'm' parameter as this is the typical value and was adopted for almost all of the regional models. As discussed in Section 4.2, the range of the parameters to be explored will be for:

- k_c of 11, 12.5 and 16.
- Initial losses of 10 mm, 15 mm and 20 mm.
- Continuing losses of 1.5, 2.0 and 2.5 mm/hour.

The sensitivity was setup as nine runs utilising this range of parameters and the peak of the flood hydrographs. The k_c values of 11, 12.5 and 16 were run using three sets of loss parameters:

- Initial loss of 10 mm and continuing loss of 1.5 mm/hour.
- Initial loss of 15 mm and continuing loss of 2.0 mm/hour.
- Initial loss of 20 mm and continuing loss of 2.5 mm/hour.

For the design runs the rainfall Intensity Frequency Duration (IFD) information was utilised for Portland and is summarised in Table 4.5. The model was run using filtered rainfall patterns, a uniform areal distribution and utilising the Siriwardena and Weinnmann areal reduction factors.

Table 4.5 – IFD parameters for Portland

Location	Portland
2y1h	15.25
2y12h	3.50
2y72h	1.00
50y1h	25.00
50y12h	5.00
50y72h	1.60
Skew	0.62
F2 Value	4.34
F50 Value	14.6
Zone	6

The resulting flood hydrograph peaks are summarised for the 5 to 500 year ARI events using the Portland IFD rainfall information in Table 4.6, Table 4.7 and Table 4.8.

Table 4.6 – Flood peaks for k_c = 11 and 'm' = 0.8 for Portland

Initial Loss	10 mm			15 mm			20 mm		
Continuing Loss	1.5 mm/hr			2.0 mm/hr			2.5 mm/hr		
ARI	Wattle Hill	Finn Ck	Portland	Wattle Hill	Finn Ck	Portland	Wattle Hill	Finn Ck	Portland
5y	59.0	12.9	62.0	31.6	7.6	32.8	9.8	3.0	9.8
10y	70.5	15.4	75.1	39.9	9.6	41.5	18.4	5.4	19.0
20y	93.1	19.9	99.1	59.8	13.5	63.2	32.4	8.4	33.8
50y	125.5	27.0	133.3	86.2	19.8	91.7	52.9	13.1	56.1
100y	152.1	32.5	161.8	110.3	25.1	117.4	76.7	18.3	81.9
200y	180.8	38.4	192.8	138.1	31.1	147.3	103.4	25.1	110.8
500y	222.1	49.2	237.8	178.8	39.4	191.1	135.9	32.5	145.5

Table 4.7 – Flood peaks for k_c = 12.5 and 'm' = 0.8 for Portland

Initial Loss	10 mm			15 mm			20 mm		
Continuing Loss	1.5 mm/hr			2.0 mm/hr			2.5 mm/hr		
ARI	Wattle Hill	Finn Ck	Portland	Wattle Hill	Finn Ck	Portland	Wattle Hill	Finn Ck	Portland
5y	52.8	11.9	55.5	27.7	7.0	28.9	8.7	2.7	8.5
10y	63.1	14.2	66.3	35.1	9.0	36.7	16.2	5.0	16.6
20y	81.8	18.4	87.3	53.6	12.5	56.1	28.9	7.8	30.2
50y	110.5	25.2	117.9	75.8	18.1	80.7	46.9	12.0	49.2
100y	134.2	30.5	143.2	96.9	23.1	103.4	67.5	17.0	71.8
200y	160.0	36.1	170.6	121.5	28.9	129.7	90.8	23.1	97.2
500y	197.7	44.6	210.8	157.9	36.8	168.7	119.4	30.1	127.9

Table 4.8 – Flood peaks for k_c = 16 and 'm' = 0.8 for Portland

Initial Loss	10 mm			15 mm			20 mm		
Continuing Loss	1.5 mm/hr			2.0 mm/hr			2.5 mm/hr		
ARI	Wattle Hill	Finn Ck	Portland	Wattle Hill	Finn Ck	Portland	Wattle Hill	Finn Ck	Portland
5y	41.7	10.4	44.3	21.6	5.9	22.4	6.9	2.2	6.4
10y	49.5	12.2	52.5	27.6	7.8	28.6	12.8	4.1	12.7
20y	64.4	15.8	68.0	41.8	10.8	44.0	22.7	6.6	23.5
50y	86.8	21.6	91.7	60.0	15.6	63.0	37.5	10.2	38.4
100y	105.6	26.3	111.7	76.2	19.7	80.1	53.9	14.6	56.2
200y	125.8	31.4	133.5	95.1	24.5	101.0	72.1	19.5	75.9
500y	154.7	38.6	165.5	123.1	31.7	131.7	94.3	25.4	100.0

For the 100 year ARI, the flows at Portland range from a minimum of 56.2 m^3/s up to 161.8 m^3/s, with the mid range peak flow corresponding to 103.4 m^3/s. This suggests that the range of parameters utilised for the RORB model can cause a range of flows that vary by +/- 45 % around the mean peak flow of 103.4 m^3/s. This is a normal range of uncertainty given that there is no event to calibrate the model to and indicates that an additional method for assessing the parameters should be explored.

The Rural Water Commission (RWC, 1988) estimated the flow rate into Fawthrop Lagoon during the 1946 event was 114 m^3/s. This assumed a 5 day flood hydrograph, peaking on the second day, and the estimate was undertaken using a storage balance approach to replicate the conditions during the

1946 flood event. This number relates well to the mid-range result for the RORB runs for the 100 year ARI. From the rainfall analysis however, the 1946 would be estimated at around the 500 year ARI event, which for the mid range parameters is estimated at a peak of 168.7 m^3/s. This flow rate will be assessed using the hydraulic model to determine if it can reproduce the anecdotal 1946 flood levels.

Overall, the sensitivity analysis shows approximately a +/- 45 % difference between the selected scenario hydrograph peak for the design run (k_c = 12.5, 'm' = 0.8, IL = 15 mm, CL = 2.0 mm/hour) as compared to the extreme upper and lower flood peak estimates. This range was expected as rainfall runoff models can be sensitive to the loss rates and input parameters.

4.3.1 Regional Streamflow Comparison

The nearby catchments of the Fitzroy River and Surry River are the most geomorphologically similar catchments to the Portland region. The details of the available streamflow records for each of these gauges is summarised in Table 4.9.

Table 4.9 – Nearby catchment streamflows

Station name	Gauge No.	Area (km^2)	Period	Years
Surry River @ Heathmere	237207	300	1970 – date	39
Fitzroy River @ Heywood	237202	234	1948 – date	61

A flood frequency analysis was undertaken on each of these gauges and the estimated flows for the full range of ARIs were developed. The 5, 10, 20, 50, 100, 200 and 500 year ARIs are summarised in Table 4.10.

The comparison shows that for similar nearby catchments the most appropriate calibration parameters for the Portland RORB model are the mid-ranged parameters (k_c = 12.5, m = 0.8, IL = 15 mm and CL = 2.0 mm/hr). The runoff produced from the Portland catchment is consistent with the Fitzroy River catchment but exceeds the flows produced from the Surry River catchment. The Fitzroy River produced more runoff during events per catchment area than the Surry River and this may well be partly because of the extended record for generating the flood frequency assessment, as well as the underlying difference in catchment characteristics. The flood frequency assessment supports the mid-range parameter case for the Portland region, and the Portland flows are on the higher, more conservative side of the estimates.

Table 4.10 – Nearby catchment streamflow comparison

Gauge	Surry River (237207)	Fitzroy River (237202)	Portland (Low[1])	Portland (Med[2])	Portland (High[3])
Area (km^2)	300	234	164	164	164
ARI	Flow (m^3/s)	Flow (m^3/s)	Flow (m^3/s)	Flow (m^3/s)	Flow (m^3/s)
5y	26	30	6	29	62
10y	34	43	13	37	75
20y	43	56	24	56	99
50y	54	76	38	81	133
100y	63	93	56	103	162
200y	72	111	76	130	193
500y	83	139	100	169	238

[1] Low – k_c = 16, m = 0.8, IL = 20 mm, CL = 2.5 mm/hr

[2] Med – k_c = 12.5, m = 0.8, IL = 15 mm, CL = 2.0 mm/hr

[3] High – k_c = 11, m = 0.8, IL = 10 mm, CL = 1.5 mm/hr

In addition to these comparisons nearby catchments can also be utilised to predict a relative 100 year ARI event for the Portland catchment. The nearby catchments that have been considered include the Surry River, Fitzroy River, Moyne River and the Merri River. The catchment areas for each of these gauges varies and the estimated 100 year ARIs can be converted to a catchment area for Portland using a ratio of the catchment areas to the power of 0.7 (Grayson et al, 1997). The results of this translation into a relative 100 year ARI for Portland from the regional gauges is shown in Table 4.11.

Table 4.11 – Regional 100 year ARI estimates for Portland

Catchment	Area (km^2)	Method	100 year ARI (m^3/s)	Scaled 100 year ARI for Portland (m^3/s)
Portland	**164**	**RORB (DESIGN)**	**103**	**103**
Surry River at Heathmere	300	FFA	63	41
		RORB	83	54
Fitzroy River at Heywood	234	FFA	108-113	84-88
		RORB	109	85
Moyne River at Toolong	570	FFA	258	108
Merri River at Woodford	899	FFA	405	123

The results show that the Portland 100 year ARI flow estimate of 103 m^3/s is within the mid-range of the four selected regional catchments and this reinforces the calibrated RORB model results for the Portland catchment.

Although the Surry River catchment is adjacent to the Portland catchment and may exhibit similar catchment response, it appears that the Surry River produces relatively low runoff totals in a regional context. The catchment produces the lowest estimate for the regional 100 year ARI assessment (which is consistent with the results in Table 4.10) and produces relatively low runoff volumes, again this suggests that there are specific catchment parameters that are unique to the Surry River catchment and that this catchments parameters may not translate well to the Portland catchment. A report by Water Technology (2008) explains that Surry River catchment has unique characteristics which result in the catchment producing less runoff than the neighbouring catchments.

The Fitzroy River catchment is also adjacent to the Portland catchment and has similar catchment characteristics. Table 4.11 shows that the adjusted 100 year ARI peak flows are approximately 15% below the Portland RORB estimate. In hydrological terms this is within an acceptable range and the 100 year ARI adjusted flows reaffirm the validity of the estimated Portland design flows. Similarly, the Moyne River and Merri River results show that 100 year ARI peak flows can be expected in the range predicted for Portland.

Overall, the regional comparison of the four catchments to the Portland catchment shows that the Portland design events are within an acceptable range which is suggested by the regional catchments as being between 85 - 120 m^3/s.

4.4 Daily Rainfall Runoff Model

A potential method that can be used to estimate daily flows is through the use of a long-term rainfall runoff simulation. These models attempt to replicate the broad rainfall runoff behaviour of a catchment based on recorded rainfall and flows. This method was attempted in order to provide an additional estimate of the peak flow in the 1946 event. The Wattle Hill Creek has no stream gauging so the rainfall runoff model that was calibrated to the nearby Surry River catchment. The Surry River is located approximately 18 km to the north east of Portland with the River outlet located at Narrawong. There is a streamflow gauge on the Surry River at Heathmere and the nearby daily rainfall recorded at Tyrendarra (90038) has an extended continuous record. Potential evaporation was sourced from the Bureau of Meteorology (BoM) as a monthly total and was disaggregated uniformly over the month to form a daily evaporation series.

The rainfall runoff model was calibrated using the Rainfall Runoff Library v1.0.5 using the Sacramento rainfall runoff model. The model was ultimately not very successful with a poor calibration being obtained. Attempts were made to fit the relationship with a focus on the peaks in order to better replicate the large flow events however the rainfall runoff model did not replicate the peak events well. Given that a reliable calibration could not be obtained for the Surry River it was decided that this rainfall runoff approach was not a useful method for estimating the peak flow rate for the 1946 event. The use of a SIMHYD model was explored but it also failed to produce a reliable calibration.

In addition to the poor calibration, the 1946 event was also the most extreme 72 hour event on record at the Portland rainfall gauge (90070) (with the volume for the 72 hour 1946 event being almost double the next largest 72 hour event). Because of this, it is unlikely that a rainfall runoff model calibrated to the general peak events would be able to accurately replicate the extreme 1946 flood peak. Ultimately the other hydrological methods employed during this study provide appropriate guidance to the magnitude and ARI of the 1946 and there is not enough confidence in the rainfall runoff model results to use them to estimate the runoff from the 1946 rainfall event at Portland.

4.5 Hydraulic Model Analysis of 1946 Flood

As a secondary check for the rainfall runoff RORB design hydrographs, the selected design flood hydrograph for the 500 year ARI was run through the hydraulic model. The anecdotal flood levels could then be employed as a check for the adopted design flood rates for the 1946 event. The 500 year ARI event flows were utilised for the k_c of 12.5, ‘m’ of 0.8 and a range of loss rates. Three 500 year ARI hydrographs were assessed including:

- Initial loss of 10 mm and continuing loss of 1.5 mm/hour.
- Initial loss of 15 mm and continuing loss of 2.0 mm/hour.
- Initial loss of 20 mm and continuing loss of 2.5 mm/hour.

The selected loss rates are derived from the use of Australian Rainfall and Runoff (AR&R) for the recommended loss rates for Victoria. The loss rate estimates span the recommended range of losses for a catchment for Victoria with the characteristics of Portland.

The results for the 500 year ARI 72 hour duration runs are shown in Table 4.12. The peak levels show that the low loss, medium loss and high loss RORB flood hydrographs range from approximately 2.0 m depth in the lagoon up to 2.9 m with the initial loss of 15 mm and continuing loss of 2.0 mm/hr replicating the 1946 event accurately. The levels for Fawthrop Lagoon were matched closely in the medium loss event and highlights that these are the most suitable parameters to use for the calibration of the 1946 event.

Table 4.12 – Calibration results for the peak levels in Fawthrop Lagoon for 1946

500 year ARI 72 hour duration	Fawthrop Lagoon Maximum Level
Observed	**2.52 mAHD**
IL 10 mm & CL 1.5 mm/h	1.97 m AHD
IL 15 mm & CL 2.0 mm/h	2.55 mAHD
IL 20 mm & CL 2.5 mm/h	2.89 mAHD

The resulting flood depth plot is shown in Figure 4.4 for the initial loss of 15 mm with a continuing loss of 2.0 mm/hour. The peak water levels indicate that the downstream flood level of 2.52 mAHD was matched accurately using the hydraulic model with the model predicting a peak water level of 2.55 mAHD.

Importantly the modelling of the 500 year ARI flow rates through the hydraulic model shows that the current model produces the appropriate levels in Fawthrop Lagoon. This is despite the fact that the hydrology indicates a peak flow rate of 168.7 m^3/s which was larger than the estimated flows of 114 m^3/s by the RWC. This shows that the flows predicted in this modelling for the 1946 event are replicating the levels during this event very well and gives increased confidence in the selected hydrologic parameters.

4.6 Adopted Design Flows

From the rainfall runoff model sensitivity analysis the most appropriate set of parameters produced design flows at Wattle Hill Creek, Finn Street Creek and at Portland is shown in Table 4.13. These loss rates and resulting flood hydrographs are reinforced by the simulation of the 1946 event as the 500 year ARI event through the hydraulic model.

Table 4.13 – Design flood peaks for k_c = 12.5 and 'm' = 0.8 for Portland

Initial Loss	15 mm		
Continuing Loss	2.0 mm/hr		
ARI	Wattle Hill	Finn Ck	Portland
5y	27.7	7.0	28.9
10y	35.1	9.0	36.7
20y	53.6	12.5	56.1
50y	75.8	18.1	80.7
100y	96.9	23.1	103.4
200y	121.5	28.9	129.7
500y	157.9	36.8	168.7

The design flood hydrographs will be utilised in the hydraulic modelling to determine the flood extents in the revised hydraulic model. It should be noted that through the hydrology assessment the assigned range of error or uncertainty in these design flows would be in the range of +/- 30%. The Portland system is also noted to be a 'volume-sensitive' system as such, the longer duration events may cause the peak flood levels. To ensure that the full range of design durations is explored, the design durations to be run through the hydraulic model included the 6, 9, 18, 24, 30, 36, 48 and 72 hour events.

The design hydrographs for Wattle Hill Creek, Finn Creek and at Portland itself are shown for the 6, 9, 18, 24, 30, 36, 48 and 72 hour events for the 5, 10, 20, 50, 100, 200 and 500 year ARIs in Appendix D. The hydrographs show that the peak flow was associated with the 9 hour duration event for the 5, 10 and 20 year ARI and was as a result of the 6 hour duration for the 50, 100, 200 and 500 year ARI. Even thought he peak flow rates were caused by these events the full set of durations was run through the model to ensure that the peak depths, water surface elevations and flood extents were captured.

4.7 Probable maximum Flood (PMF)

The Generalised Southeast Australia Method (GSAM) was used to generate the Probable Maximum Precipitation (PMP) for the 24, 36, 48, 72 and 96 hour durations. The GSAM is a method developed for the south eastern regions of Australia for estimating PMP events. The principal inputs to the GSAM are summarised in Table 4.14. These parameters are determined using the information data sets provided with the method. The PMP is calculated for both the summer and autumn periods and the larger precipitation is used in the modelling. Table 5.15 shows the non-adjusted initial rainfall depths for the Portland area for the summer and autumn periods.

Table 4.14 – GSAM input parameters for the PMP estimate

Parameter		Value
Topographic Adjustment Factor		1.208
EPW $_{\text{seasonal catchment average}}$	Summer	56.96
	Autumn	45.60
EPW $_{\text{seasonal standard}}$	Summer	80.80
	Autumn	71.00
MAF $\left(\frac{EPW_{seasonal\,catchment\,avg.}}{EPW_{seasonal\,standard}}\right)$	Summer	0.705
	Autumn	0.642

Table 4.15 – Non-adjusted initial depths for the area around Portland

Duration (hours)	Initial Depth (D_{summer})	Initial Depth (D_{autumn})
24	780 mm	535 mm
36	863 mm	662 mm
48	920 mm	780 mm
72	962 mm	990 mm
96	1000 mm	1060 mm

The PMP is subsequently determined by using Equation 2.

$$PMP = D_x \; x \; TAF \; x \; MAF_x \qquad \textbf{Equation 2}$$

Where:

PMP is the probable maximum precipitation in mm.
D_x is the initial depth for either summer or autumn respectively.
TAF is the Topographic Adjustment Factor.
MAF_x is the Moisture Adjustment Factor foe either summer or autumn.

The peak PMP rainfall estimates were determined to be from the summer calculation and are summarised in Table 4.16.

Table 4.16 – PMP rainfall estimates using the GSAM

Duration (hours)	PMP_{summer}	PMP_{autumn}
24	660 mm	420 mm
36	730 mm	510 mm
48	780 mm	610 mm
72	820 mm	770 mm
96	850 mm	820 mm

RORB was then used to generate the PMF hydrographs. The PMP totals were distributed for input into RORB using the coastal rainfall patterns for catchments of less than 100 km2. The resulting flow hydrographs were then run through the hydraulic model to determine the PMF extents for Portland. The peak flow for each duration for Wattle Hill Creek, Finn Creek and Portland are summarised in Table 4.17.The highest peak was observed for the 24 hour duration and the peak volume was from the 36 hour duration event.

Table 4.17 – PMF peak flows and overall hydrograph volumes

Duration (hours)	Peak Flow Rate (m^3/s)			Volume (m^3)		
	Wattle Hill	Finn Ck	Portland	Wattle Hill	Finn Ck	Portland
24	433	51	481	2,239	242	2,479
36	320	39	344	2,258	244	2,500
48	329	38	359	2,249	243	2,490
72	276	33	302	1,930	208	2,137
96	170	20	184	1,783	192	1,974

4.8 Conclusion

The process undertaken to develop and validate the inflows for the Portland model followed the following key steps in Table 4.18 The table summarises the key steps and outcomes from these processes.

Table 4.18 – Key steps and outcomes for the development of the hydrology for Portland

No.	Item	Description	Key Outcome
1	Regional rainfall assessment	Assessing the recurrence interval of the rainfall during the 1946 event	1946 event was approximately a 500 year ARI rainfall event.
2	RORB model		
2.1	k_c	Comparing the k_c parameters from a range of sources (ARR98, regional assessment, previous modelling)	k_c range was between 11 and 16 from similar catchments, ARR98 value was 13.5. Adopted value was 12.5.
2.2	Losses	Deriving the regional loss rates and developing the initial loss and continuing losses.	From the regional assessment the loss rates were set at initial loss of 10 – 20 mm and a continual loss from 1.5 mm/hr to 2.5 mm/hr.
3	RORB Sensitivity		
3.1	Sensitivity of RORB parameters	Assessing scenarios of k_c, IL and CL (with an ‘m’ of 0.8)	The most appropriate parameters were k_c = 12.5, m = 0.8, IL = 15 mm, CL = 2.0 mm/hr.
3.2	Regional streamflow comparison	Assessing the selected parameters against the regional methods of developing peak design flows.	The 100 year ARI estimate of 103 m^3/s was within the range of area translated flows from the region of 85 – 123 m^3/s.
4	Daily rainfall runoff models	Developing daily rainfall runoff models to predict a stream flow time series for the ungauged Portland catchment.	No model could be calibrated from regional flow gauges.
5	Hydraulic model of the 500 year 1946 event	Running the generated 500 year ARI 1946 event (72 hour event) through the hydraulic model to test output levels.	Great calibration for the k_c = 12.5, m = 0.8, IL = 15 mm, CL = 2.0 mm/hr design flows. Levels in Fawthrop Lagoon at + 3 mm over observed.

The process undertaken as part of the hydrologic assessment aimed to specify the likely recurrence interval for the 1946 event and reproduce the 1946 flood. As there was no known gauge this process was undertaken preliminarily using the rainfall data and known regional hydrology parameters.

The rainfall assessment on the Portland rain gauge (90070) indicated that the 1946 event was in the vicinity of a 500 year ARI rainfall event. The CRC-FORGE method estimates the 1946 event as approximately equivalent to a 500-year rainfall event at Portland and this ARI was adopted for the duration of the streamflows for the calibration to this event.

The regional parameter assessment indicated that a k_c of between 11 and 18 should be utilised (12.5 was adopted) and an 'm' of 0.8 should be used in the RORB model for Portland. The selected k_c of 12.5 matched the recommended k_c of 13.5 (ARR98) for Victorian catchments with < 800 mm mean annual rainfall. The regional parameters indicated that the initial losses should be in the range of 10 to 20 mm and the continuing loss between 1.5 and 2.5 mm/hour.

Rainfall runoff models were attempted for the nearby catchment of Surry River but a reliable calibration could not be obtained that could capture the extreme 1946 event and as such this approach was not utilised.

The 1946 event was considered as a 500 year rainfall event and hence assumed to correlate to approximately a 500 year ARI flood. The 500 year ARI flood hydrographs were run through the 1946 conditions hydraulic model and the best calibration was achieved using the IL of 15 mm and the CL of 1.5 mm/hour. It should be noted that the 500 year ARI (72 hour) peak flow rate was 168.7 m^3/s which was higher than the RWC (1988) predicted peak flow of 114 m^3/s, however the calibration of the hydraulic model indicated that the selected 72 hour event was producing the levels experienced during the 1946 event accurately.

It is important to note that the sensitivity analysis indicated that the RORB model was sensitive to the selection of the kc and loss parameters and subsequently the confidence that is attributed to the final design hydrology selected is in the range of +/- 30%. This is slightly higher than typical Victorian catchments, however this is a direct result of the Portland catchment being ungauged.

Ultimately the design events have been developed using the RORB model parameter set of k_c = 12.5, m = 0.8, IL = 20 mm, CL = 2.0 mm/hr. A range of design hydrographs have been produced for the 5, 10, 20, 50, 100, 200 and 500 year ARI storm events for durations of 6, 9, 12, 18, 24, 30, 36, 48 and 72 hours. The wide range of design storm durations is required to assess the impact of storm volume on flood levels in the hydraulic model. The design hydrographs are shown in Appendix D indicating the expected peak flow, flood timing and volume that are used as inputs to the hydraulic model.

4.9 1992 Storm Event

While the study was being undertaken, a reference level for a storm event that occurred during the 1992 storm event was acquired (refer Appendix C). From analysis of the rainfall during the event it was deemed that the event was between a 10 and a 20 year flood event. Using the hydraulic model and the calibrated RORB design inflows a cross check was undertaken to ensure that the flood model was replicating the flooding experienced during this event. Analysis of the hydraulic model results shows that during the 20 year event, the area where the marker was located was inundated with depths of 69 cm present. In the 10 year event, depths of 43 cm are present. Given the limited data and the reasonable depths achieved, it was concluded that the model behaves generally in accordance with the expected flood behaviour correctly in events of this magnitude.

5 HYDRAULIC MODELLING

The WL|Delft 1D2D modelling system, SOBEK, was used to compute the channel (1D) and overland flow (2D) components of the study. SOBEK is a professional software package developed by WL|Delft Hydraulics Laboratory, which is one of the largest independent hydraulic institutes in Europe (situated in The Netherlands) and is world-renowned in the fields of hydraulic research and consulting (WL|Delft, 2005).

This combined package allows for the computation of channel and pipe flow (including structures such as culverts, weirs, gates and pumps, and pipe details such as inverts, obverts, pipe sizes and pipe material) by the 1D module, which is then dynamically linked to the 2D overland flow module. The 1D and 2D domains are automatically coupled at 1D-calculation points (such as manholes) whenever they overlap each other. The model commences with the 1D component operating as the inflow increases until such time as the pipe or channel is full and overflows, with the flow then moving to the 2D domain. The 1D network and the 2D grid hydrodynamics are solved simultaneously using the robust Delft scheme that handles steep fronts, wetting and drying processes and subcritical and supercritical flows (Stelling, 1999).

The advantages of this system are that the channel/pipe system is explicitly modelled as a sub-system within the two-dimensional overland flow computation. This means that generalised assumptions regarding the capacity of the channel/pipe system are not required. This system employs a unique implicit coupling between the one and two-dimensional hydraulic components that provides high accuracy and stability within the computation.

5.1 Hydraulic Model Development

The hydraulic models consist of two main hydraulic components:

- The channel network (1D); and
- 2D grid of the surface topography.

The establishment of these two components of the model is described in the following sections.

5.1.1 Channel System (1D)

Survey was undertaken on specific locations along Wattle Hill Creek and Finn Creek to obtain cross sections for use in the 1D channel network. In addition to the cross sections, the dimensions of the major bridges and culverts were captured and are included in the 1D network. The location of the cross sections and structures are shown in Figure 5.1.

A 1D channel network was developed for both Wattle Hill Creek and Finn Creek, with culverts and bridges included in the model as discrete elements.

5.1.2 Topography (2D)

The topography was defined using a Digital Terrain Model (DTM) of the region. Two topographic layers were established, a lower resolution grid for the calibration and a higher resolution grid of the region for the final model runs. The grid resolution was reduced to minimise run times to aid the calibration process. The full model was established in higher resolution to ensure that all possible topographical features were included in the model.

The dimensions of the grids are summarised in Table 5.1. The 2D model extent is shown in Figures 5.2 and 5.3 for the calibration and full model respectively. The grid size was set at 5 m in the design events as this was deemed sufficient to capture the topography and detail of the model while allowing run times to be reasonable.

The topography along the river in the 2D grid was flattened to the approximate top of bank level as the 1D network of cross sections represented the river storage. This removes the double counting of volume storage within the system and improves stability of the 1D to 2D interaction.

Table 5.1 – Topography grid size

Parameter	Calibration model	Full model
Cell size	10m x 10m	5m x 5m
Grid Cells (x direction)	393 columns	1137 columns
Grid Cells (y direction)	312 rows	624 rows

5.1.3 Hydraulic Roughness

The hydraulic roughness for the overland flow model was described using a two-dimensional roughness map of Manning's "n" values. This was developed by digitising different land-use zones from the digital aerial images within a GIS environment (MapInfo). The roughness values were set to the values as shown in Table 5.2. The final roughness grid is shown in Figure 5.4.

The roughness parameters are consistent with the values specified by Chow (1973), the manning's 'n' for the roads, residential and commercial are consistent with previous modelling experience and practices.

The Manning's 'n' value of 0.08 for the flood plain corresponds to medium to dense brush which is considered reasonable for the Wattle Hill Creek floodplain. The roughness parameter of 0.08 was also adopted for the lower reaches of the Finn Creek floodplain. This is because images of the creek and surrounds show medium to dense brush present in most locations.

Table 5.2 – Calibrated Roughness Parameters, Mannings 'n'

Parameter	Roughness Manning's 'n'
Roads	0.018
Main river channel	0.035
Farmland / dense floodplain	0.08
Residential	0.15
Commercial/Industrial/Port	0.5
Rail	0.022
Road Reserve	0.08

5.1.4 Boundary Conditions

Boundary conditions were established at both the upstream boundaries for Wattle Hill Creek and Finn Creek and at the downstream boundary located in the ocean

The upstream Wattle Hill Creek boundary was setup as a single 2D boundary with an inflow hydrograph representing the hydrology of the known calibration events. Where the event hydrograph was unknown this boundary was set as a steady state flow boundary, that is, it had a continuous flow entering the model until a steady state solution was reached. The topography was modified around this 2D boundary to aid the flows entering the model.

The downstream model boundary was setup as a time varying tidal boundary with a peak tidal level equivalent to the 10-year ARI sea level. This is the combination of both storm surge and tidal components of the sea level. The time series from the 1994 storm surge event (a 10-year ARI event) was used as the boundary level and the peak sea level was matched to occur at the same time as the peak flood flows. For the climate change cases, the levels in the time series were increased by 0.8 m. This approach maintains the tidal properties of the system whilst ensuring that levels are not artificially raised by extended elevated sea levels. Figure 5.1 shows the adopted tidal boundary under existing conditions.

It is considered that this tidal boundary condition is likely to be conservative; a preliminary joint probability assessment has been undertaken to assess the likelihood of an elevated sea level occurring at the same time as a flood event. This analysis is found in section 6.

6 JOINT PROBABILITY ASSESSMENT

An assessment was undertaken to examine the probability of extreme streamflow events occurring simultaneously with extreme sea levels. The purpose of this assessment was to determine if any significant correlation existed between the streamflow events and high sea levels for planning purposes for Portland. If an observable correlation was found then additional scenario modelling could be undertaken for Portland utilising extreme flood events coupled with extreme sea levels.

It should be noted that the data and work included in this report are a summary of the more detailed study.

6.1 Available Data

Wattle Hill Creek and Finns Creek are ungauged and as such streamflow data from the Surrey River at Heathmere and the Merri River at Woodford has been used in the analysis. The Surrey River catchment and the Merri River catchment are both located to the east of Portland. Although there is no streamflow data available for Wattle Hill Creek and Finn Creek, the neighbouring catchments are likely to experience similar rainfall patterns to Portland. The records for instantaneous discharge at these locations are long enough to perform reasonably accurate flood frequency analysis and were chosen due to the close proximity to Portland,

6.2 Assessment Approach

In order to assess the joint probability of extreme flood events and tides four approaches were adopted, these approaches included:

1. Assessing the average daily flow at both the Merri and Surrey Rivers against the peak daily sea level at Portland.
2. Comparing the top 50 independent daily peak sea levels at Portland to the discharge recorded at the Merri and Surrey Rivers on the same day. A lag of +/- 24 hours was applied to the data to account for timing differences and the spatial difference in gauge locations in relation to Portland.
3. The average recurrence intervals (ARI) for the Merri and Surrey River flows, and the Portland tidal data was undertaken to get the relevant ARIs for the extreme events. The sea level ARI was based on the mean higher high water (MHHW), mean higher low water (MHLW), mean lower high water (MLHW) and mean lower low water (MLLW). The peak flood ARIs were then assessed for correlation against the corresponding tidal ARIs.
4. The method extracts the top streamflow events for the Merri and Surrey Rivers and assesses these against the corresponding tidal levels. Only events where both flow data and tide data are available have been analysed and as such some ARI events will need to be interpolated from the results to obtain an appropriate tide level.

6.3 Approach 1

The first approach examines the average daily flows in the Surry River and Merri River against the daily peak tide levels for Portland. This approach aims to determine if there is any immediate correlation discernable from the direct comparison of streamflows to peak tide levels.

Figure 6.1 and 6.2 shows the results of the comparison of the Portland peak daily tide levels against daily flow rates for Surrey River and Merri River respectively. From the figures it is evident that there is no correlation identified between the peak daily tide levels at Portland and the average daily streamflow in either river. Evident in both Figure 6.1 and 6.2 is the clumping of data points around the y-axis which relates to low average daily flows recorded in the rivers. This observation is expected as during the majority of the streamflow record the Surrey and Merri Rivers are in low flow conditions, whereas the tide oscillates each day regardless of weather conditions. The data tends to centre around the 0.47 m AHD mark, which is the MHHW for Portland.

Figures 6.1 and 6.2 show a linear relationship fitted through the data set and each of these fitted lines has a low R^2 value indicating that there is no clear relationship or correlation between the high tide level and the average daily flow rates. Although this method of fitting a line to the data is simplistic, it is evident from visual inspection that there is not clear correlation between the full data sets.

6.4 Approach 2

In order to assess only the peak events, the top 50 sea levels from the 1991 to 2008 assessment period were selected and compared to the maximum streamflows on the corresponding day. This process was undertaken to directly determine if there was a correlation with the larger tide events and streamflows. The plots of the peak 50 tide levels are plotted in Figure 6.3 and Figure 6.6 against the Surrey and Merri Rivers corresponding flows. It is evident from the figures that no correlation was exhibited from this comparison.

To ensure that the lack of correlation was not caused by timing differences in the data due to the use of the tide data from Portland and flow data from Surrey and Merri River, the peak tides were assessed against the peak flows from +/- 24 hours as compared to the peak tides. This approach also aims to check that there was no lag between extreme high tides and streamflow events. Figures 6.4, 6.5, 6.7 and 6.8 show that there was no correlation observed for these lagged comparisons.

6.5 Approach 3

The third approach to directly assess the ARIs for the respective peak streamflow events and compare these to the ARIs for the tides that occurred on the day of the peak flows. Figures 6.9 and 6.10 show the streamflow ARI's in the Surrey and Merri Rivers compared to the calculated tide level ARIs at Portland. The data shows that again no discernable correlation was observed between an extreme flow event and a corresponding elevated tide.

6.6 Approach 4

Approach 4 examined the peak 50 flood events at the Surrey and Merri Rivers and compared these to the tidal levels on the day of the flow peak. These plots are shown in Figures 6.11 and 6.12. The figures are plotted with the peak flow ARI against the observed sea level. Like the previous approaches, no correlation is discernable from this method.

Table 6.1 and Table 6.2 summarise the 6 peak flow events present in the Merri and Surrey Rivers against the recorded tide level in Portland at the time of the peak flows. The data indicates that the rainfall events that caused the flows in the river did not have any discernable impact on tide levels.

Table 6.1 – Merri River discharge ARI and Portland sea level ARI on same day (6 peak events)

Merri River			Portland		
Time	Discharge (m^3/s)	ARI	Time	Sea Level (m)	ARI
27/08/2001 15:00	242.3	27.2	27/08/2001 15:00	-0.085	0.028
14/09/1996 08:03	160.3	6.2	14/09/1996 08:00	0.075	0.028
10/10/1992 03:14	114.9	4.0	10/10/1992 03:00	0.307	0.028
18/09/1991 01:52	101.0	3.2	18/09/1991 01:00	0.502	0.144
23/09/1992 00:48	96.0	2.9	23/09/1992 00:00	0.222	0.028
31/08/2004 14:00	95.8	2.9	31/08/2004 14:00	0.093	0.028

Table 6.2 – Surrey River discharge ARI and Portland sea level ARI on same day (6 peak events)

Surrey River			Portland		
Time	Discharge (m^3/s)	ARI	Time	Sea Level (m)	ARI
10/10/1992 03:59	34.9	13.1	10/10/1992 03:00	0.307	0.028
31/08/2004 23:19	29.2	5.7	31/08/2004 23:00	-0.294	0.028
27/08/2001 14:13	25.9	4.8	27/08/2001 14:00	-0.163	0.028
14/08/1992 02:15	25.8	4.6	14/08/1992 02:00	0.574	0.144
01/10/1996 07:12	24.5	4.1	01/10/1996 07:00	-0.023	0.028
26/07/2000 23:29	20.3	2.7	26/07/2000 23:00	0.329	0.028

6.7 Additional studies

The Portland Floodplain Management Study (RWC, 1988) provides further evidence that riverine flooding and extreme sea levels are unlikely to occur simultaneously. During the 1946 event Portland tide records show that the peak sea level during the flood was 0.5 m. A similar analysis of the 1946 data was undertaken to determine the ARI of both the streamflow and the tide level during the 1946 event. Table 6.3 indicates that there was no correlation between the two, confirming the Portland Floodplain Management Studies findings.

Table 6.3 – 1946 event correlation with Tide Levels

Event	Estimated Flow in Wattle Hill Creek (m^3/s)	Streamflow ARI (Years)	Tide Level (m AHD)	Tide ARI (Years)
March 1946	97	500	0.529	0.225

A frequency analysis of the recorded tidal information from 2006 and 2007 was undertaken to examine the exceedance of various sea levels. This type of analysis has the potential to provide guidance to an appropriate downstream boundary condition. An appropriate level may be the level which is exceeded 1% of the time. In Portland, this level is approximately 0.7 mAHD and is roughly equivalent to the 1 year ARI.

6.8 Conclusion

Each of the four methods used to assess the Portland tidal data and the streamflow data from Surrey and Merri Rivers indicate that there is no discernable correlation between extreme streamflow events and extreme tidal levels. Therefore, it is concluded that these two variables are independent of each other. This implies that the probability of a 100 year ARI streamflow event occurring in conjunction to a 100 year ARI tidal event in any year would be approximately 1 in 10,000. Therefore the assumption of a 100 year ARI riverine flooding event being coupled with a 100 year ARI tidal event is highly conservative for flood planning and mitigation.

Due to this, it is considered unnecessary that the downstream tide boundary for the design flood events is set to the 100 year ARI storm tide event when the full range of riverine flooding ARIs are being run through the hydraulic model. Rather, a 10 year ARI storm tide would be a more likely and suitable level while still being very conservative and it is this approach which is recommended for the Portland flood study.

7 RESULTS

The calibrated SOBEK model for Wattle Hill Creek and Finn Creek was used to analyse the extent, location and depths for the durations of 10, 20, 50 and 100 year ARIs. For each ARI a range of event durations (6hr, 9hr, 12hr, 24hr, 30hr, 36hr, 48hr and 72hr) were examined to find the maximum flood extents and depths.

The peak flood extents and depths for the 10, 20, 50 and 100 year ARI events are shown in Figures 7.1 – 7.8 respectively including plots of the maximum depths and maximum water surface elevations. The maximum flood extent is derived from the peak depths and water surface elevations taken from all of the durations modelled. This ensures that the maximum flood extent that is likely to be experienced is captured. The different durations produce different flood depths and extents due to the differences in the hydrographs and volumes of flood waters entering the system (see Figure 7.13 for the peak duration for the 100 year ARI event).

7.1 Existing Conditions Results

Figure 7.1 and 7.2 shows the 10 year ARI flood depths and water surface elevations. Along Wattle Hill Creek the 10 Year ARI flood is generally well contained within the flood channel for the creek. Minor overtopping of Kerrs Road is present with depths of up to 30 cm recorded. No other roads along Wattle Hill Creek were affected. Along Finn Creek no breakouts are present upstream of the railway however some very minor flooding is present along Smith Street south of the railway. Some flooding is present along the rear of properties on Otway Street and Clarke Street. Henty Street also becomes overtopped, with depths up to 20 cm present along the road. The levee just upstream of Fawthrop Lagoon was significantly overtopped in this event.

Figure 7.3 and 7.4 shows the 20 year ARI flood depths and water surface elevations. Generally flooding present within both systems is as expected, with increases in depth throughout the catchment. No additional roads are impacted in the 20 year ARI, however several more properties are affected. Also to the south of the catchment minor flooding is present along the railway, with depths of less than 10 cm apparent. The most noticeable increase in flooding is present on Finn Creek where the flow has broken out of the channel prior to the railway.

Figure 7.5 and 7.6 shows the 50 year ARI flood event overtops Anderson road and overtops the railway. The area to the north of Julia Street experiences significantly more flooding, with more properties and roads near Clarke and Otway Street are impacted. The area immediately north of Fawthrop Lagoon also experiences large areas of inundation with properties along Glenelg Street now affected.

Figure 7.7 and 7.8 shows the 100 year ARI flood event. During this event, the flood is large enough to inundate Julia Street. In addition the area immediately north of Fawthrop Lagoon is inundated. Several properties in this area previously unaffected by the 50 year ARI event become inundated in this event. Generally the extent of the flooding in the 100 year isn't significantly larger than the 50 year ARI which is due to the topography of the area.

Flood hazard is defined by combining the flood depth and flow speed to form a hazard category for a given design event. An alternate definition of flood hazard (or safety risk) is provided by Melbourne Water based on both the velocity-depth product and the total flood depth. Melbourne Water defines 5 classes of safety risk as shown in Table 7.1. A draft floodway overlay has been developed where areas with a safety risk of greater than 2 (in the 100-year ARI event) are included. Figure 7.9 shows the flood hazard for the 100 year ARI event. Figure 7.10 shows these as the Land Subject to

Inundation Overlay (LSIO) and Urban Floodway Zone (UFZ) where the LSIO is where the hazard class is 1 or 2, and the UFZ is where the hazard is greater than 2.

Table 7.1 – Melbourne Water Safety Risk Definition

Safety Risk Category		Definition		
		V*D		Depth
High	5	> 0.84 m^2/s	or	> 0.84 m
Moderate to High	4	0.6 - 0.84 m^2/s		0.6 - 0.84 m
Moderate	3	0.4 - 0.6 m^2/s		0.4 - 0.6 m
Low to Moderate	2	0.2 - 0.4 m^2/s		0.2 - 0.4 m
Low	1	< 0.2 m^2/s		< 0.2 m

In order to determine the impact on flooding due to higher storm surge events, the 100 year ARI existing conditions using the 100 year tidal levels was run. Figure 7.11 and 7.12 shows the maximum depths and water surface elevations for the results of this analysis with Figure 7.13 showing the difference between the 100 year ARI existing conditions using the 100 year tidal levels and the 100 year ARI conditions using the 10 year tidal levels. By increasing the tide level, only slightly more flooding is experienced within the Portland floodplain. Depths in Fawthrop Lagoon are increased by approximately 5 cm. Upstream of Fawthrop Lagoon the increase in flood depths is less noticeable with increases of 0 to 2 cm observed up until Julia St. The critical duration for the 100 year ARI event is shown in Figure 7.14 and the figure shows the peak flood depths are caused primarily by the 9 hour duration storm event.

7.1.1 100 Year ARI Existing Storm Surge Conditions Results

In order to analyse the impact of a 100 year ARI storm surge on the flood levels within Portland the existing scenario was run with a 100 year ARI sea level event. No flow was run through Wattle Hill Creek and Finn Creek to isolate the impacts of the storm surge

The results for this scenario are shown in Figures 7.15 and 7.16 for the maximum depths and water surface elevation. The figures show that a storm surge of this magnitude would be likely to cause increases in flooding up to the upstream end of Fawthrop Lagoon. Flood depths within Fawthrop Lagoon averaged 1.1 m from storm surge. The 100 year storm surge operating over current sea level conditions was insufficient to compromise any infrastructure in the Portland Catchment.

7.2 Impact of Climate Change

In order to assess the impact of predicted climate change three sets of model runs were proposed and run, these include:

- Current 100 year ARI flood event coupled with the predicted 2100 climate change 10 year and 100 year ARI tidal levels.
- Storm surge as a result of the 2100 climate change predicted 100 year ARI storm surge (with no inflows on Wattle Hill Creek and Finn Creek).
- Climate change adjusted 100 year ARI inflows coupled with the predicted 2100 climate change 100 year ARI tidal levels.

The three sets of climate change runs were examined to determine the individual increases in flood depths of the inflows, storm surge and increased tide levels. The most extreme climate change scenario is the combination of the climate change adjusted 100 year ARI inflows coupled with the 2100 climate change 100 year tide level. This scenario should provide the extreme upper limit of the expected impacts of climate change for Portland.

7.2.1 100 Year ARI Flows with the 2100 Climate Change Tide Results

In order to identify the impact of sea level rise on the flood levels in the Portland catchment, the 100 year ARI event was run with the predicted 2100 10 year and 100 year ARI tidal levels. The 2100 10 year and 100 year ARI tidal levels were set at 1.89 m and 2.02 m respectively (*The Effect of Climate Change on Extreme Sea Levels along Victoria's Coast*, McInnes Et al, 2009. Table 6).

Figures 7.17 and 7.18 show the maximum depths and water surface elevations for the results of the 100 year ARI flood event coupled with the 2100 10 year ARI tide level analysis. The predicted climate change tide level was 0.82 m higher than the existing 10 year tide level and this is reflected in the increase in flood depths downstream of Fawthrop Lagoon. The flood depths increased through Fawthrop Lagoon by approximately 21 cm. Upstream of Fawthrop Lagoon a slight increase in flood depth can be observed up to the Henty Highway. A difference plot between the 100 year ARI results and the 100 year ARI results with the 2100 10 year ARI tide boundary is shown in Figure 7.19. This figure shows the areas of increased impact.

Figures 7.20 and Figure 7.21 show the maximum depths and water surface elevations for the results of the 100 year ARI coupled with the 2100 100 year ARI tide level. The additional depth of the tide increased depths around Fawthrop Lagoon.

7.2.2 2100 Climate Change 100 Year ARI Tide Storm Surge Results

In order to analyse the impact of a 100 year ARI storm surge on the flood levels within Portland, the climate change scenario was run with the 2100 predicted 100 year ARI sea level event. No flow was run through Wattle Hill Creek and Finn Creek to isolate the impacts of the storm surge.

The results for this scenario are shown in Figures 7.22 and 7.23 for the maximum depths and water surface elevations respectively. The inundation progresses further upstream under the 2100 climate change tide conditions with the effect reaching almost as far upstream as Finn Street on Finn Creek and the Henty Highway on Wattle Hill Creek. The water surface elevation predicted within Fawthrop Lagoon was approximately 2.0 m. This level impacts infrastructure in the Portland Area by overtopping Henty St and Tyers St.

7.2.3 100 Year ARI with Climate Change with the 2100 Climate Change Tide Results

The final climate change run was the climate change adjusted 100 year ARI event coupled with the 2100 climate change 100 year ARI tide levels. This is the worst case climate change scenario and the flood depths and water surface elevations are shown in Figures 7.24 and 7.25. A difference plot between the 100 year ARI with 100 year ARI tide and the climate change 100 year ARI inflows and 2100 climate change 100 year ARI tide is shown in Figure 26.

The Figures show an increased flood extent under the 2100 climate change conditions for the 100 year ARI flow and sea levels. The depths in Fawthrop Lagoon increased by approximately 1 m in depth. This is mainly caused by the increase in the downstream ocean level of 0.8 m under the climate change tide conditions. All of the floodplain had some increase in depth.

7.3 Probable Maximum Flood (PMF)

The probable maximum flood (PMF) was run through the hydraulic model and the maximum flood depths and water surface levels are shown in Figures 7.24 and 7.25. As expected, significant flooding is experienced during the PMF with significant depths present in the majority of inundated areas. The critical duration was the 36 hour duration PMF.

8 SENSITIVITY ANALYSIS

The sensitivity analysis was undertaken as part of this flood study predominantly due to the lack of available data for developing the inflows for the hydraulic model. The uncertainty of the hydrology was required to be clarified so that the confidence in the flood study could be well understood by all using the results. The sensitivity was undertaken first on the hydrology and then this was assessed by runs through the hydraulic model.

8.1 Hydrological Sensitivity

The sensitivity was firstly undertaken on the hydrological model through the variation of the parameters used in the rainfall-runoff modelling utilising RORB. The range of parameters that were used included:

- k_c of 11, 12.5 and 16.
- Initial losses of 10 mm, 15 mm and 20 mm.
- Continuing losses of 1.5, 2.0 and 2.5 mm/hour.

The full description of the sensitivity assessment is described in Section 4.3. For the 100 year ARI, the flows at Portland ranged from a minimum of 56.2 m^3/s up to 152.1 m^3/s, with the mid range peak flow corresponding to 103.4 m^3/s. This suggests that the range of parameters utilised for the RORB model can cause a range of flows that vary by +/- 45 % around the mean peak flow of 103.4 m^3/s. This range of uncertainty was expected due to the lack of flows available for the calibration and the use of regional parameters in the rainfall-runoff model.

8.2 Hydraulic Sensitivity

In order to analyse the hydraulic models sensitivity to the selected inflows, the 100 Year ARI storm events were run using +/- 20% of the design inflows. Figures 8.1 to 8.3 show the maximum depths from the hydraulic runs and Figures 8.2 and 8.4 show the difference plots between the 20% increase and the 20% decrease against the 100 year ARI storm event respectively.

On average the 20% increase in flow increased the flood depths present by 0.15 m, which equates to an 8.6 % increase in depths across the study area floodplain. A similar result is obtained for the 20 % reduction in inflow with a reduction of floodplain depths of approximately 0.15 m. The 20 % change in flows led to a change in flood depths by approximately +/- 9 %. In addition, this variation in depth did not significantly change the flood extent boundaries.

Specifically the increase in flows by 20 % had the following impacts:

- Increased flood depths adjacent to Finn Street by approximately 0.12 m
- Increased flood depths adjacent to Henty Street by approximately 0.12 m
- Increased flood depths adjacent to Glenelg Street by approximately 0.34 cm
- Introduced additional flooding near Calvert Street which was close to existing properties.

In this flood study there is considerable uncertainty with the design peak flows due to the lack of recorded data within the catchment. The sensitivity scenario examines this issue by increasing the 100 year ARI design flow by 20 % to 152.1 m^3/s. The results of this model run show that the peak depth increase adjacent to houses and properties of 0.34 m. This has an impact on the assigned freeboard that is recommended for housing being built within the LSIO. Typically a 300 mm freeboard would be employed, however given the uncertainties of the design flows and the increase in flood depths of 0.34 m under the 20 % increase in flow sensitivity scenario, it is recommended that a freeboard of 600 mm be adopted for properties at risk of flooding.

The sensitivity analysis shows that the model is not significantly sensitive to the change in inflows and this allows for additional certainty to the flood extents and depths developed as part of this study. It was also observed from the hydraulic modelling results that the flood extents of 50 and 100 year ARI events were not significantly different. This is primarily due to the topography of Wattle Hill Creek and Finn Creek catchments at Portland. This implies that the flood extents produced as part of this flood study are likely to be reasonable for the use in flood planning.

The sensitivity on the Manning's roughness was not considered as the Manning's equation is essentially a product of the flow rate and Manning's 'n'. Hence the variation of the inflows by +/- 20% effectively constitutes an assessment of the variation of the roughness by a similar. This is due to the fact that the Manning's equation demonstrates that the product of the flow rate and Manning's 'n' is a function of flow depth, through the hydraulic radius and flow cross-sectional area. It is expected that varying the roughness by +/- 20% (while maintaining the same inflows) would yield similar variations in flood extents as shown by the inflow sensitivity analysis.

9 ECONOMIC DAMAGES

The economic impact of flooding can be defined by what is commonly referred to as 'flood damages'. These flood damages can be defined as being direct, indirect or intangible as defined in Figure 9.1.

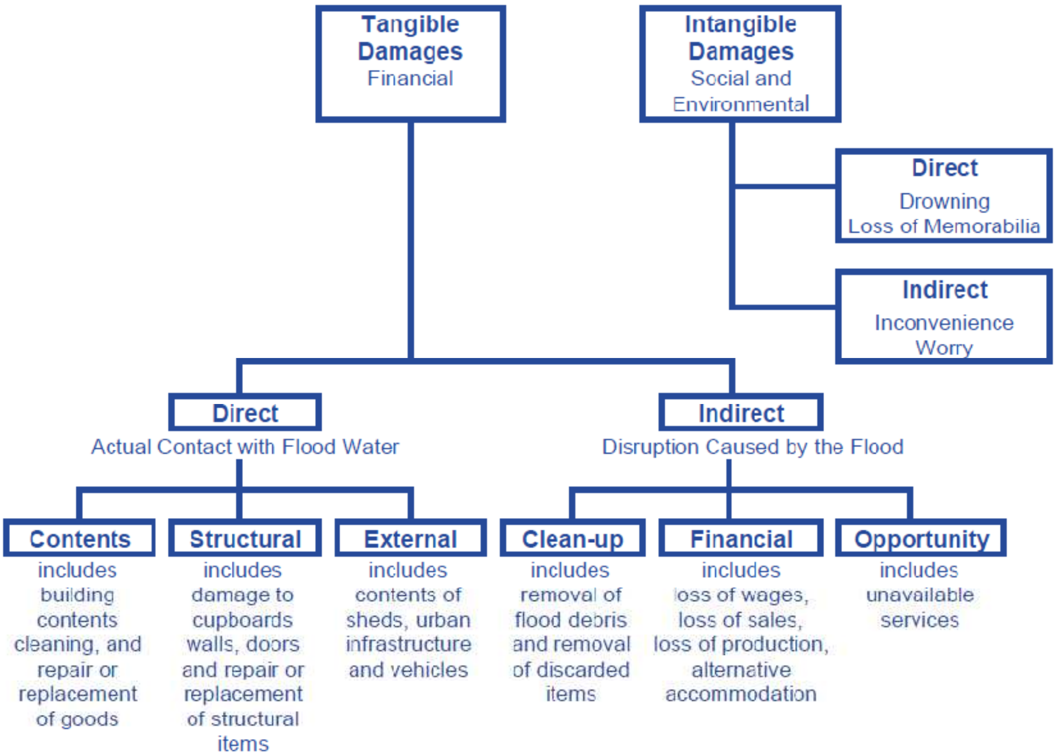

Figure 9.1 – Types of flood damage (Floodplain Development Manual (NSW Gov, 2005))

The direct damage costs are just one part of the flood damage overall cost. The flood damages are broken down into two distinct groups, tangible and intangible damages. The damage assessment in this report is restricted to the tangible damages and makes no estimate of the costs associated with the 'intangible' costs, such as social distress and loss of memorabilia.

The 'tangible' damages are further divided into direct and indirect damages. The indirect damages are damages caused by the disruptions of the flooding (such as clean up costs and accommodation costs), whereas the direct damages are caused by contact with the flood waters directly (such as damage to carpets and household contents).

For Portland it has been assumed that the residents will have no warning time and hence no allowance has been made for the residents protecting or removing their valuables. This assumption has been made as it gives a more conservative estimate of flood damages as the maximum 'potential' damage is assessed.

Flood damages can be assessed by a number of methods including the use of computer programs such as FLDAMAGE, ANUFLOOD or via more generic methods such using spreadsheets. For the purposes of this project, generic spreadsheets have been used based on experience by Cardno in this area. The use of both the Floodplain Management Manual (NSW Gov, 2005) and The Rapid Appraisal Method for floodplain Management (NRE, 2000) were utilised in this flood damage assessment.

9.1 Damage Analysis

A flood damage assessment has been undertaken for the existing catchment and floodplain as part of the current study. The assessment is based on damage curves that relate to the depth of flooding on a property to the likely damage to a property.

Ideally, the damage curves would be calibrated to the specific catchment for which the study was undertaken, however, damage data in most catchments is not available and as a result damage curves from other catchments are utilised. The Department of Environment, Climate Change and Water NSW (DECCW) has carried out research and prepared a methodology (draft) to develop damage curves based on state-wide historical data. This methodology is only for residential properties and does not cover industrial or commercial properties.

The DECCW methodology is only a recommendation and there are currently no strict guidelines regarding the use of damage curves in Victoria. The Rapid Appraisal Method (RAMS) suggests specific damage values for residential, commercial and industrial buildings, however, these values are not specific to Victoria and the flood damage curves developed by DECCW are based on a more robust methodology.

The following sections provide an overview of the methodology applied for the determination of damages within the Finn Creek and Wattle Hill Creek floodplains.

9.1.1 Residential Damage Curves

The *Floodplain Management Guideline No. 4 Residential Flood Damage Calculation* prepared by DIPNR (now DECCW) (DIPNR, 2004) has been used in this damage assessment. These guidelines include a template spreadsheet program that determines damage curves for three types of residential buildings;

- Single storey, slab on ground,
- Two storey, slab on ground, and
- Single storey, high-set.

No floor level data was available and a generalised method of assuming that the floor level was 300 mm above the average ground level of the delineated house was assumed. It has been assumed that all residential properties are slab on ground.

Damages are generally incurred on a property prior to any over floor flooding. There are two possibilities:

- The flooding overtops the representative ground level but does not necessarily reach the base of the house. When this representative ground level is exceeded by a depth of 10 cm, a nominal damage value of $3,301 (January 2011 dollars) has been adopted to represent garden damage.
- The flooding overtops the garden and also reaches the base of the house. The DECCW curves allow for a damage of $9,802 (January 2011 dollars) to be incurred when the water level reaches the base of the house (the base of the house is determined by the floor level less 0.5 m for slab on ground houses). This accounts for the garden damage as specified in the point above, but also includes some damage to cars and structures.

In summary, a cost of $3,301 (January 2011 dollars) was applied when only the property was overtopped by greater than 10 cm of depth of flood water. When the flooding reaches 0.5 m below the floor level of the house the DECCW damage curves (adjusted to current dollar values) have been adopted. This equates to $9,802 (January 2011 dollars) for flooding depths between 0.5 m below the floor height, when the flood water overtop the floor level the DECCW damage curves are used to determine the economic damage.

9.1.2 Other Parameters

There are a number of input parameters required for the DECCW curves, such as the area of the floor of houses in the floodplain and level of flood awareness. The damage assessment adopted values within the recommended range specified by the DECCW guidelines. The average house size for Portland was unknown and was estimated at the average of 200 m^2. This area reflects the ground floor only.

As no floor survey has been undertaken, the average floor height was estimated by first digitising the properties affected by flooding using aerial photography and cadastre information. The average topography level was then extracted using GIS for each house and the floor level was set at 300 mm above this level. This is a reasonable approximation of floor height as most houses are constructed with a 300 mm freeboard. This calculation uses the assumption of the 300 mm freeboard (as compared to the assumed residential damages starting at 0.5 m below the floor level of the house) because the conservative average topography level for the site has been used.

Floor levels from the Portland Floodplain Management Study (RWC, 1988) were included in Appendix B of this report however the addresses of the properties were not recorded for the majority of the floor levels. The floor levels stated for 3 Bentick Street did not match the current property. Aerial imagery suggests that the property at 3 Bentick Street has been redeveloped since 1988 and hence the floor level was set from the assumptions as stated above. The properties at 7 and 9 Bentick Street have the floor levels set at the level surveyed in the RWC Report (1988).

Table 9.1 – Portland Floodplain Management Study (RWC, 1988) floor levels

LOCATION	FLOOR LEVEL mAHD	FLOOD LEVEL mAHD	FREEBOARD (m)
Bentinck Street			
No. 3	1.85	2.2	-0.35
No. 7	2.19	2.2	-0.01
No. 9	2.23	2.2	+0.03
Henty Street	2.66	2.6	+0.06
Portland Court	2.65	2.6	+0.05
Clarke Street	2.64	2.6	+0.04
Wyatt Street	2.60	2.6	0

Conservatively, the Effective Warning Time has been assumed to be zero as Portland has no flow gauge. A long Effective Warning Time allows residents to prepare for flooding by moving valuable household contents (e.g. the placement of valuables on top of tables and benches).

The Wattle Hill Creek and Finn Creek catchments, while rural, have access to Warrnambool, Hamilton and Mt Gambier via multiple highways and as a result it is assumed that there are no post-flood inflation costs. These inflation costs are generally experienced in regional areas where re-construction resources are limited and large floods can cause a strain on these resources. For the local flooding assessed in this study it is unlikely that there would be large regional impacts.

9.1.3 Average Weekly Earnings

The DECCW curves are derived for late 2001 and have been adjusted to represent January 2011 dollars.

General recommendations by DECCW are to adjust values in residential damage curves by the increase in Average Weekly Earnings (AWE), rather than by the inflation rate as measured by the Consumer Price Index (CPI). DECCW proposes that AWE is a better representation of societal wealth, and hence an indirect measure of the building and contents value of the home. The most recent data for AWE from the Australian Bureau of Statistics (ABS) was in February 2010. Therefore all ordinates in the residential flood damage curves were updated to the February 2010 dollars. In additional, all damage curves include GST as per the DECCW recommendations.

While not specified, it was assumed that these curves were derived in November 2001, which therefore assumes the use of the November 2001 AWE (issued quarterly) would be appropriate. November 2001 and February 2010 AWE statistics were obtained from the ABS website (www.abs.gov.au). The AWE figures and percentage adjustment factor is summarised in Table 9.2.

Table 9.2 – Residential damage curve adjustment factor

Month	Year	AWE
November	2001	$ 898.50
August	2010	$ 1,301.70
Change	44.9 %	

Consequently, all ordinates on the damage curves were increased by 44.9 %. It has been assumed that August 2010 values are representative of January 2011 dollars.

9.1.4 Commercial Damage Curves

Commercial damage curves are determined based on those included in the *FLDamage Manual* (Water Studies, 1992). FLDamage allows for three types of commercial properties;

- Low Value Commercial,
- Medium Value Commercial,
- High Value Commercial.

In Portland all commercial has been assumed to be low value commercial based on FLDamage. In determining these damage curves, it has been assumed that the effective warning time is approximately zero, and the loss of trading days has been approximated at 10.

The commercial damage curve is linked to the floor area of the property and the floor level survey has estimates of the floor area of the individual properties. These areas will be used to factor these curves, the curves have been determined for a standardised 100 m^2.

The CPI was used to bring the 1990 data to September 2010 (CPI was obtained from the ABS www.abs.com.au). It was assumed that the Water Studies (1992) data was in June 1990 dollars. The CPI adjustment factor is shown in Table 9.3.

Table 9.3 – Commercial damage curve adjustment factor

Month	Year	CPI
June	1990	102.5
September	2010	173.3
Change	69.1%	

Consequently, damages have been increased by 69.1% and GST has been included.

9.1.5 Industrial Damage Curves

Industrial damage curves are determined based on those included in the *FLDamage Manual* (Water Studies, 1992). The industrial damage curves were set using the same principles as the commercial damage and used the same adjustment factor as stated in Table 9.3. The industrial damage curve was based on the low value industrial damage curve which is shown in Figure 9.1.

9.1.6 Road damages

Road damage was assessed based on the Rapid Appraisal Method (RAM) which assigns a damage value for major roads, minor roads and unsealed roads. The RAM was developed in May 2000 and the damages are quoted in May 2000 dollars. To convert these to March 2010 dollars, the CPI was used to adjust for inflation. The adjustment factor is shown in Table 9.4.

Table 9.4 – Roads damage adjustment factor

Month	Year	CPI
May	2000	126.2
September	2010	173.3
Change	37.3 %	

The RAM uses a single estimate cost per km for roads which are inundated and includes:

- Initial repairs to roads
- Subsequent additional maintenance to roads
- Initial repairs to bridges (based on 1/3 of road damages)
- Subsequent additional maintenance to bridges.

The RAM estimates of the costs per km of inundated road are shown in Table 9.5. These unit damages were adjusted using the CPI adjustment factor. The RAM also states that the damages to roads and bridges generally outweighs the costs associated with other infrastructure such as water, electricity, gas and sewerage services and is a good approximation for the overall damage to the regional infrastructure.

Table 9.5 – Unit damages for roads and bridges (dollars per km inundated)

	Initial road repair	*Subsequent accelerated deterioration of roads*	*Initial bridge repair and increased maintenance*	Total cost applied per km to inundated roads (May 2000 $)	Total cost applied per km to inundated roads (Dec 2009 $)
Major sealed roads	*$ 32,000*	*$ 16,000*	*$ 11,000*	$ 59,000	**$ 81,007**
Minor sealed roads	*$ 10,000*	*$ 5,000*	*$ 3,500*	$ 18,500	**$ 25,401**
Unsealed roads	*$ 4,500*	*$ 2,250*	*$ 1,600*	$ 8,350	**$ 11,465**

9.1.7 Adopted Damage Curves

The adopted damage curves are shown in Figure 9.1. As described above, the commercial and industrial damage curves are standardised for a property of 100 m^2.

9.2 Annual Average Damage

Annual Average Damage (AAD) is calculated on a probability approach, using the flood damages calculated for each design event.

Flood damages (for a design event) are calculated using the ‘damage curves’ described in the sections above. These damage curves approximate the damage occurring on a property for varying depths of flooding. The total damages in the summation of the damage to all houses and properties within the flood extent for that design event.

The AAD attempts to quantify flood damage that a floodplain would receive on average during a single year. It does this by using a probability approach. A probability curve is drawn, based on the flood damages calculated for each design event. This is shown in Figure 9.2. For the example, the 100 year ARI design event has a 1% chance of occurring in any given year, and as such the 100 year ARI flood damage is plotted at this point on the AAD curve. AAD is then calculated by determining the area under the curve.

Further information on the calculation of AAD can be found in the *Floodplain Development Manual* (NSW Government, 2005).

9.3 Results

The results of the flood damage assessment are shown in. Based on the analysis as described in the above section the annual average damages (AAD) for the floodplain under existing conditions is approximately **$ 40,322**.

Table 9.6 – Summary of Economic Flood Damages

Site location	Properties with over floor flooding	Properties with over ground flooding	Total Damages ($ Sept 2009)
10 year ARI			
Residential	0	0	$0
Commercial	0	0	$0
Industrial	0	1	$10,950
Road and infrastructure damage			$13,059
10 year ARI total			**$24,009**
20 year ARI			
Residential	0	0	$0
Commercial	0	0	$0
Industrial	1	1	$41,100
Road and infrastructure damage			$25,580
20 year ARI total			**$66,679**
50 year ARI			
Residential	0	9	$220,347
Commercial	0	0	$0
Industrial	1	2	$79,399
Road and infrastructure damage			$49,257
50 year ARI total			**$349,003**
100 year ARI			
Residential	10	23	$1,315,766
Commercial	0	1	$0
Industrial	2	3	$224,369
Road and infrastructure damage			$73,614
100 year ARI total			**$1,613,750**

9.4 Assumption and Qualifications

A significant assumption in the calculation of the AAD was the assumption that the damages below the 5 year ARI were extrapolated with the assumption that there are no damages at the 2 year ARI event. Assuming a different slope for this line or a different ARI for zero damages will result in a change in the AAD calculated value. A paper was presented at the 2006 Floodplain Management Conference (Thomson et al, 2006) highlighting the issues associated with this assumption. In addition the AAD was calculated up to the 100 year ARI event rather than the PMF and this may impact on the AAD.

10 MITIGATION MEASURES

Following the economic damage assessment mitigation measures were considered for Portland. Mitigation of flooding is a long-term and ongoing process that is developed in conjunction with the relevant flood studies in order to mitigate the risks to the community for future flood events. This mitigation assessment will examine both structural and non-structural measures in order to determine the most appropriate mitigation for the Portland community.

The flood management structure has been outlined in the Emergency Management Manual Victoria (1997) and mitigation falls under the prevention activities. These activities include planning, legislation, regulation, land use controls, enforcement and structural works. Leading on from preparedness is the lead into response and the need for adequate warnings leading up to flood events occurring.

For Portland the AAD was low due to the limited impact that the flood events had on the township. Due to the relatively low AAD it is unlikely that structural measures would be economically viable form of mitigation for the township. Rather than explore the structural measures which are economically not feasible, the focus for the mitigation of Portland is on the non-structural planning controls that ensure that future buildings and works are adequately protected from flood damage. In addition to these non-structural works, the development of an adequate warning system and increasing the community awareness around being prepared for floods may also lead to a reduction in the expected AAD for the community.

Non-structural mitigation measures include developing planning controls in the Glenelg Shire Planning Scheme including Land Subject to Inundation Overlay (LSIO) and Urban Floodway Zone (UFZ) to control future works within the floodplain.

Typically the LSIO is defined as the extent of the 100 year ARI flood extent and any proposed development within this overlay would be specifically required to provide buildings with at least 300 mm of freeboard over the predicted peak 100 year ARI flood depth and specific approval from the local Catchment Management Authority, in this case the Glenelg Hopkins CMA (GBCMA). The proposed LSIO has been defined in Figure 7.10.

The UFZ is an area that is designated as having a high hazard risk and typically no development is allowed within this zone. The UFZ is often defined as either the 10 year ARI flood extents or by an area where the hazard class exceeds 2 (see Table 7.1 for hazard classes). The UFZ for Portland has been defined in Figure 7.10 and in this case has been developed from the 10 year ARI flood extent. It should be noted that the difference in flood extent from the 10 year to the 100 year ARI event is quite small so it may be prudent to adopt the 100 year ARI as the UFZ.

The planning controls reduce flood damage over time by applying their conditions at the time of development of a flood affected property. The use of the UFZ also clearly indicates where development would not be allowed due the impacts on the floodplain. This provides a strong signal to the development community. The overlay and zone controls do not preclude development per say, and detailed analysis of the impacts of specific proposals enable the extents and types of these overlays to be modified and removed as development occurs.

Overall, a major requirement for the improvement of the flood response and planning of Portland is the gauging of Wattle Hill Creek and/or Finn Creek to allow for both forewarning of rising flood waters and to improve the estimates of the various peak flood depths.

11 RECOMMENDATIONS

This project has provided base flooding information for the Portland township. The following actions are recommended:

- Incorporate the results of the study into the Glenelg Planning Scheme and create appropriate Land Subject to Inundation and Floodway Overlays
- Utilise the data set to inform the flood planning provisions of the Municipal Emergency Response Flood Sub Plan for Portland
- Utilise the model to assess the impact of proposed developments in and around Fawthrop Lagoon.
- Implement non-structural planning controls to reduce flood damage over time in the Portland region
- Implement LSIO and UFZ overlays to control development in the floodplains of Wattle Hill Creek and Finn Creek
- Investigate the potential for flood warning and prediction at Portland.

The study found that the annual average damages (AAD) were $40,332 per annum. This relatively low amount in dollar terms indicates that planning controls are likely to be the most appropriate way to reduce flood risk in Portland.

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Figures

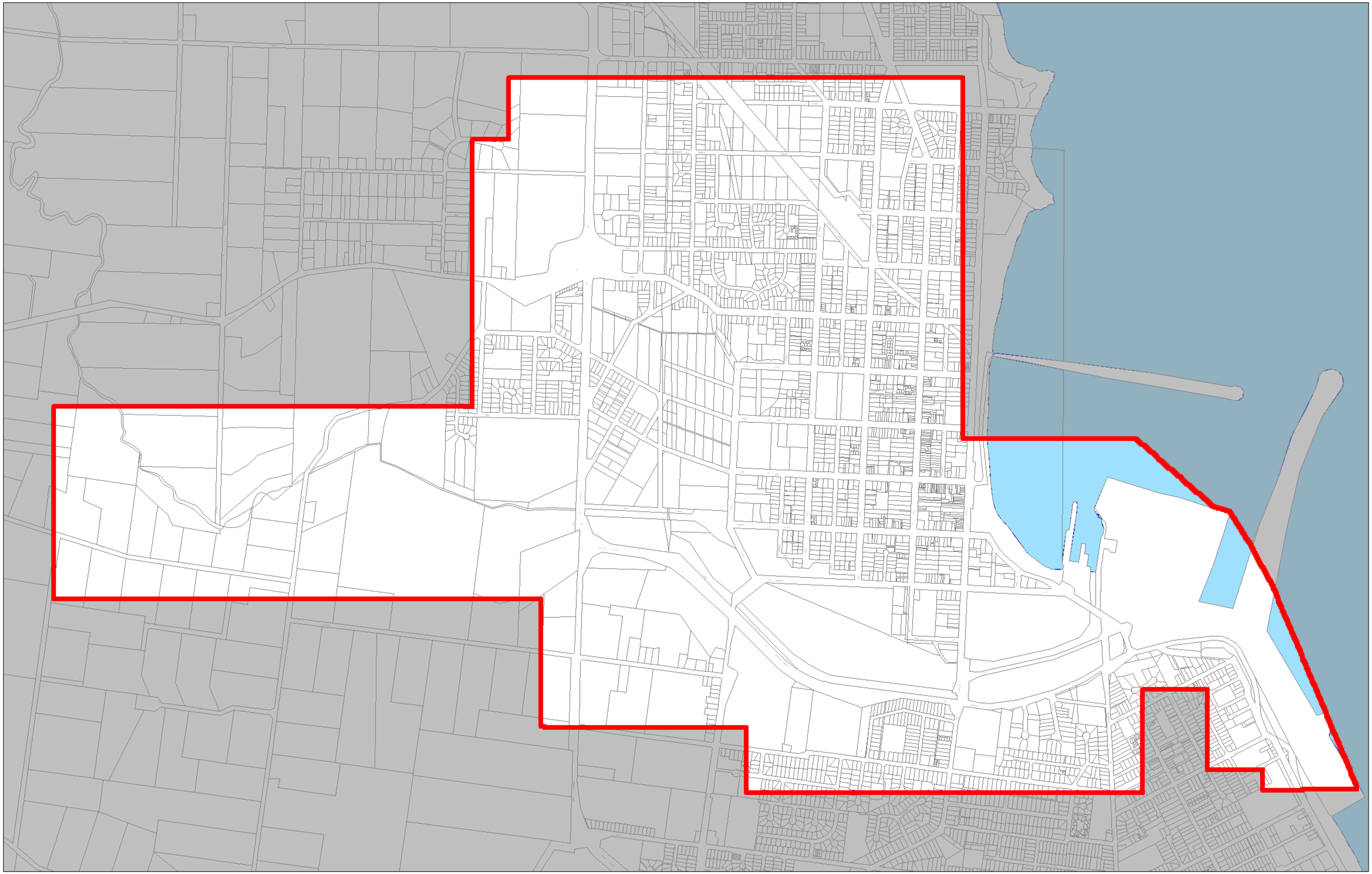


Figure 1.1 - Hydraulic Model Extents

Figure 3.1 - Aerial and ground survey comparison

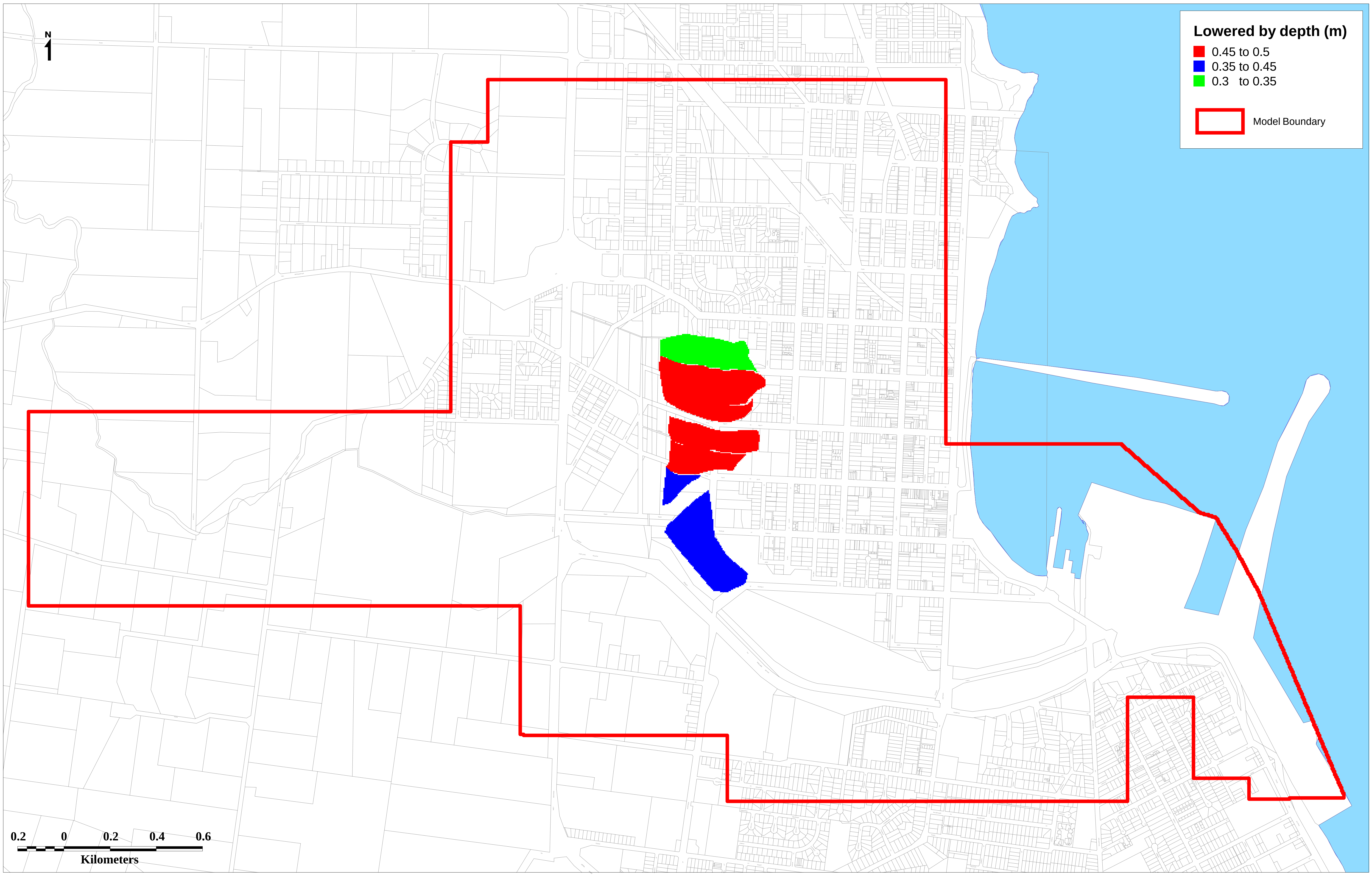


Figure 3.2 - Lowered 2D grid cells from Future Coasts LiDAR

Figure 4.1 - Rainfall gauge locations in the Portland area

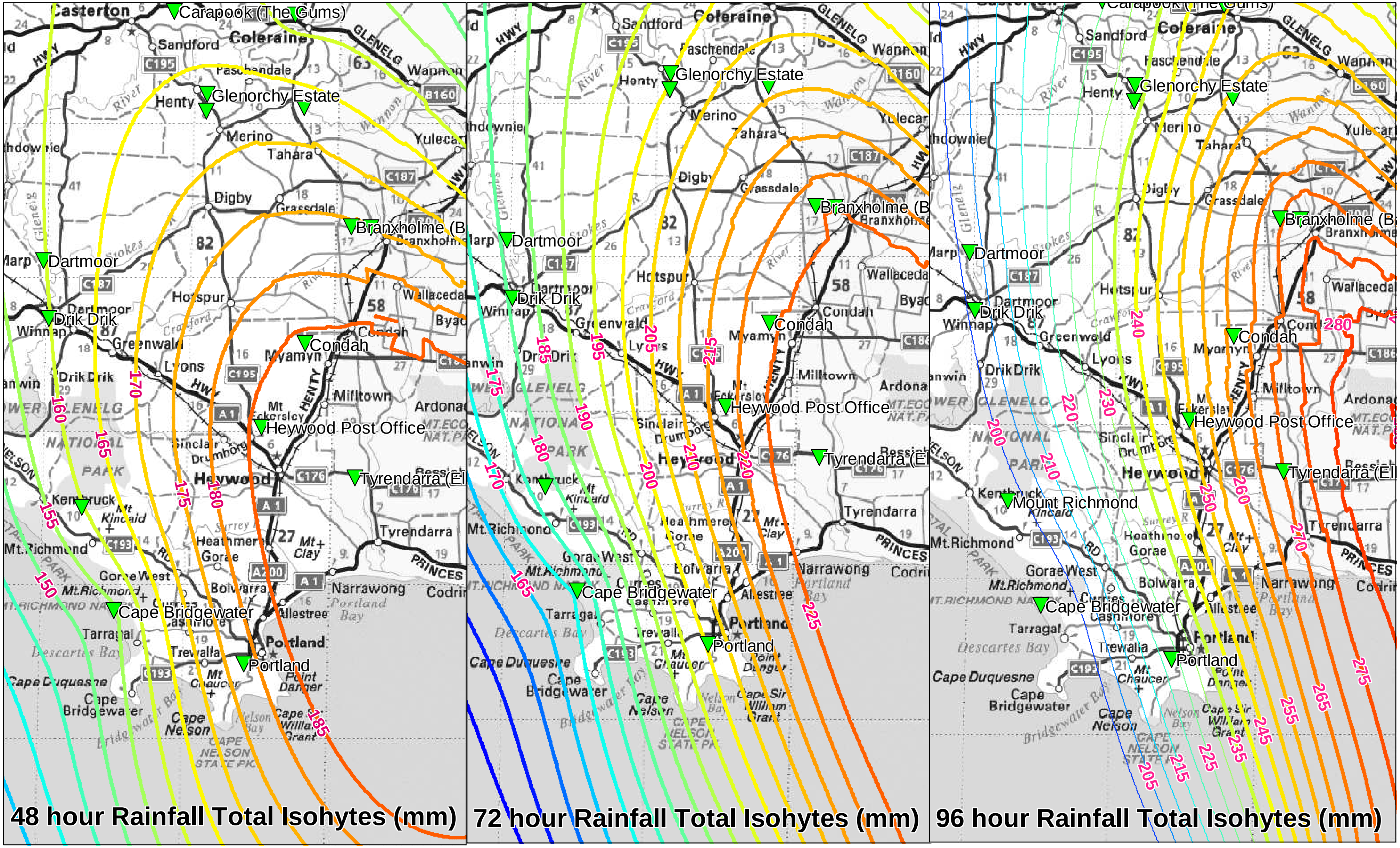


Figure 4.2 - Rainfall distribution for the 48, 72 and 96 hour durations for the March 1946 event

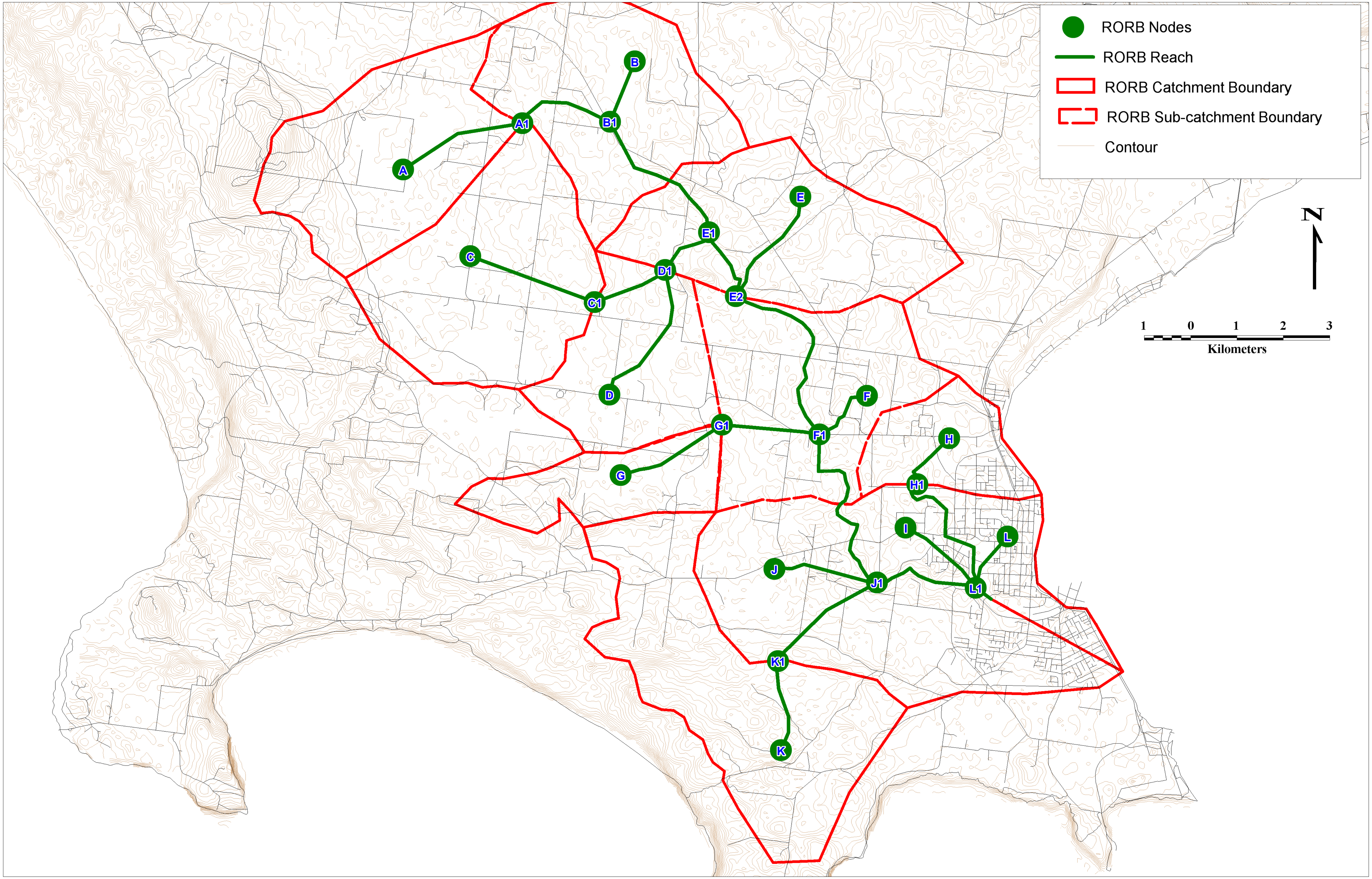


Figure 4.3 - Portland RORB Catchment Model

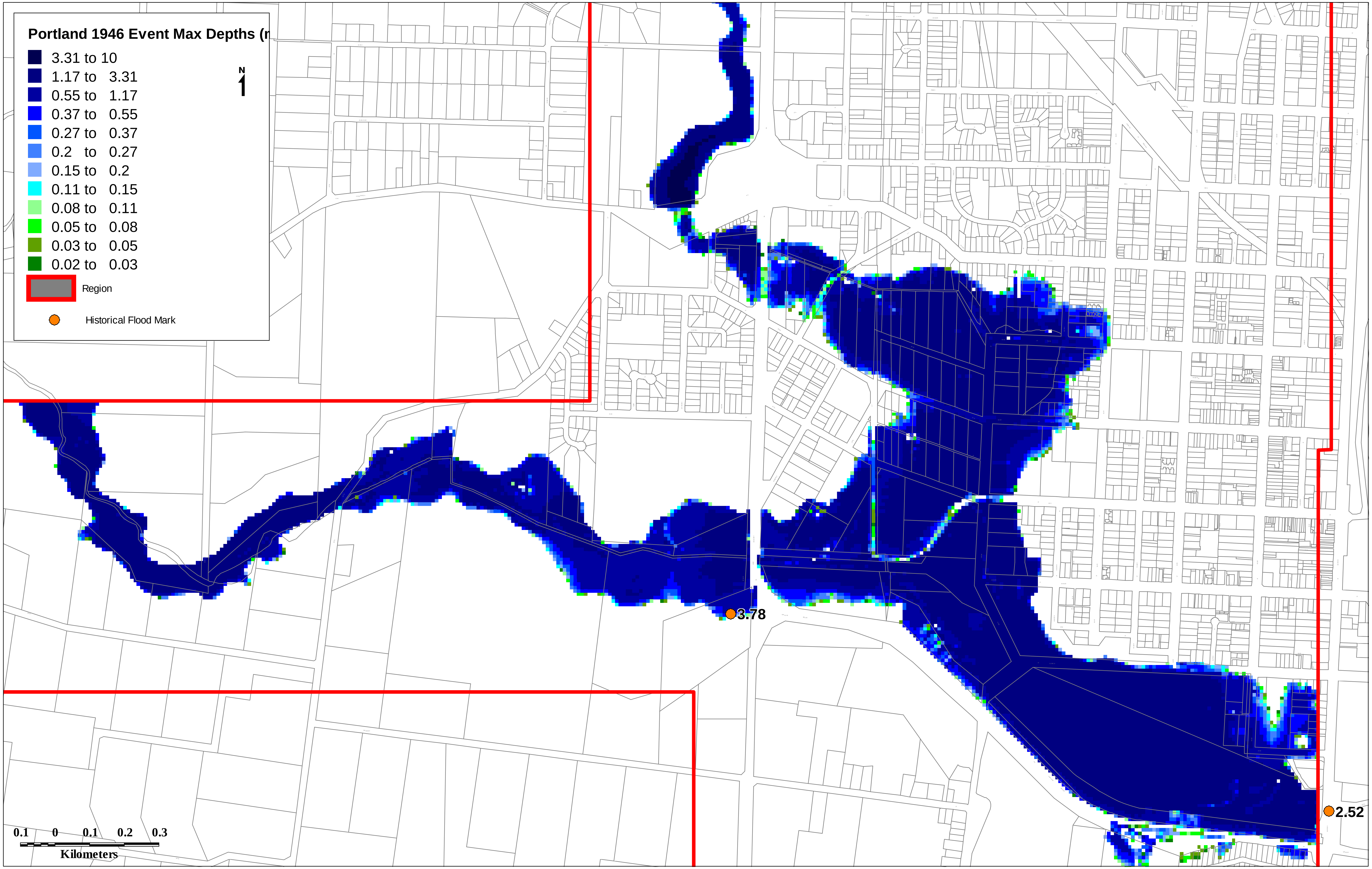


Figure 4.4 - 1946 Calibration Event Hydraulic Results

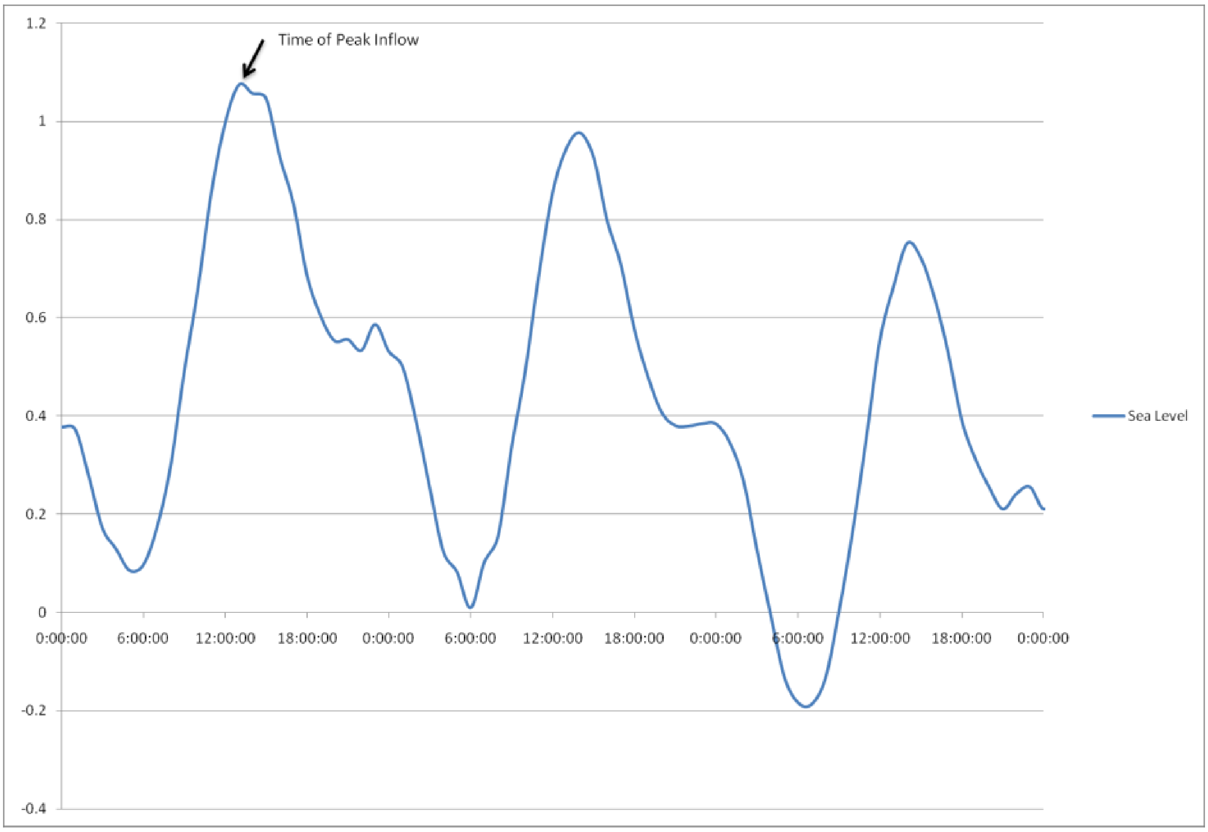


Figure 5.1 - Downstream Tidal Boundary, Existing Conditions

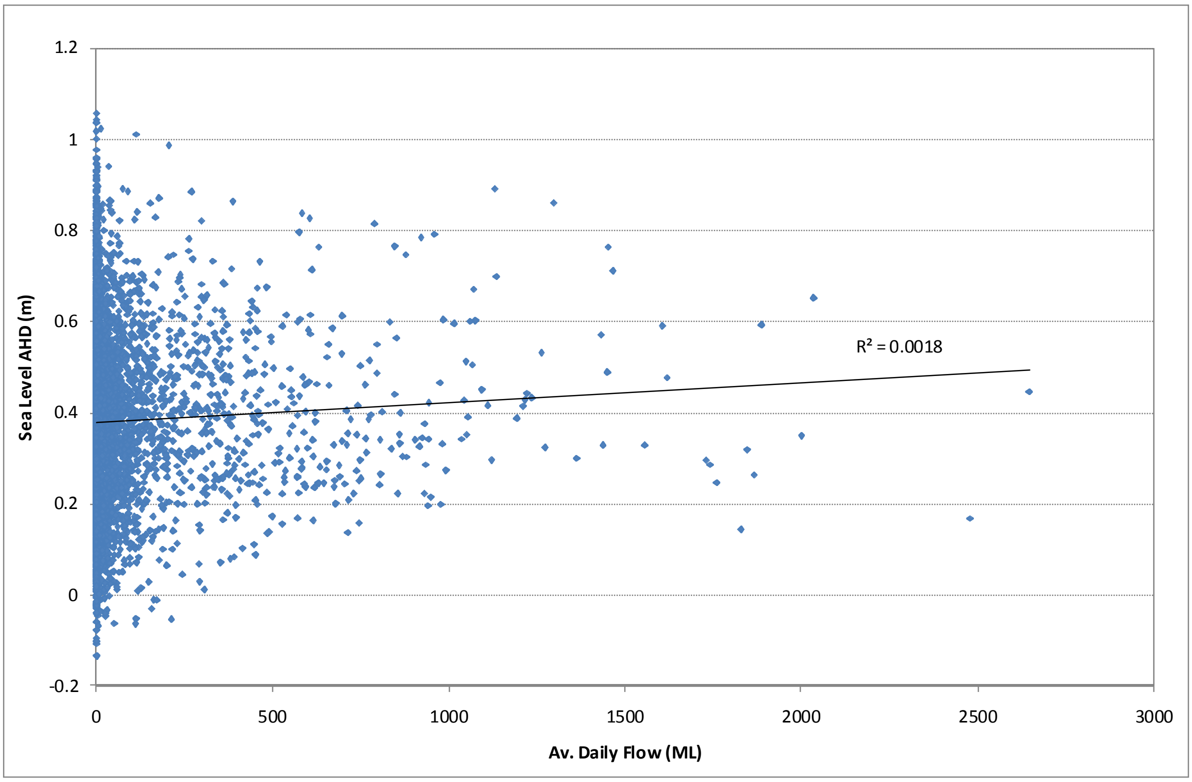


Figure 6.1 – Average daily streamflow at Surry River vs. peak daily sea level at Portland

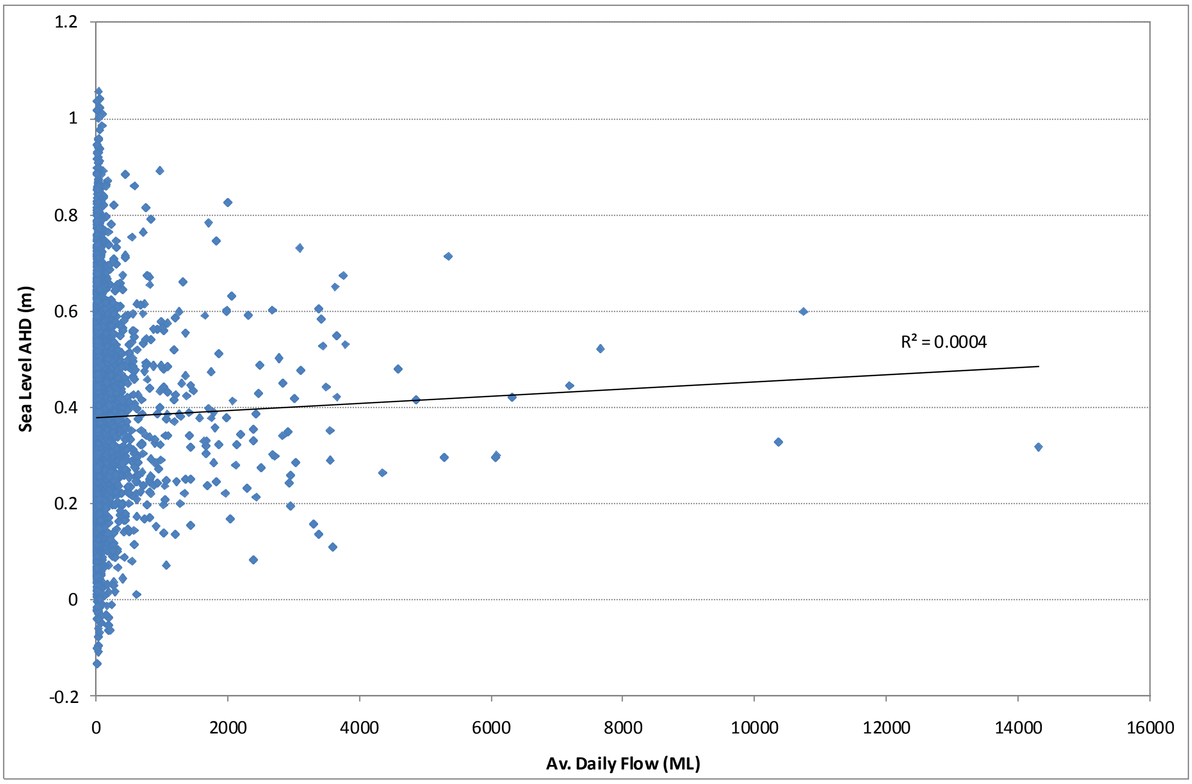


Figure 6.2 – Average daily streamflow at Merri River vs. peak daily sea level at Portland

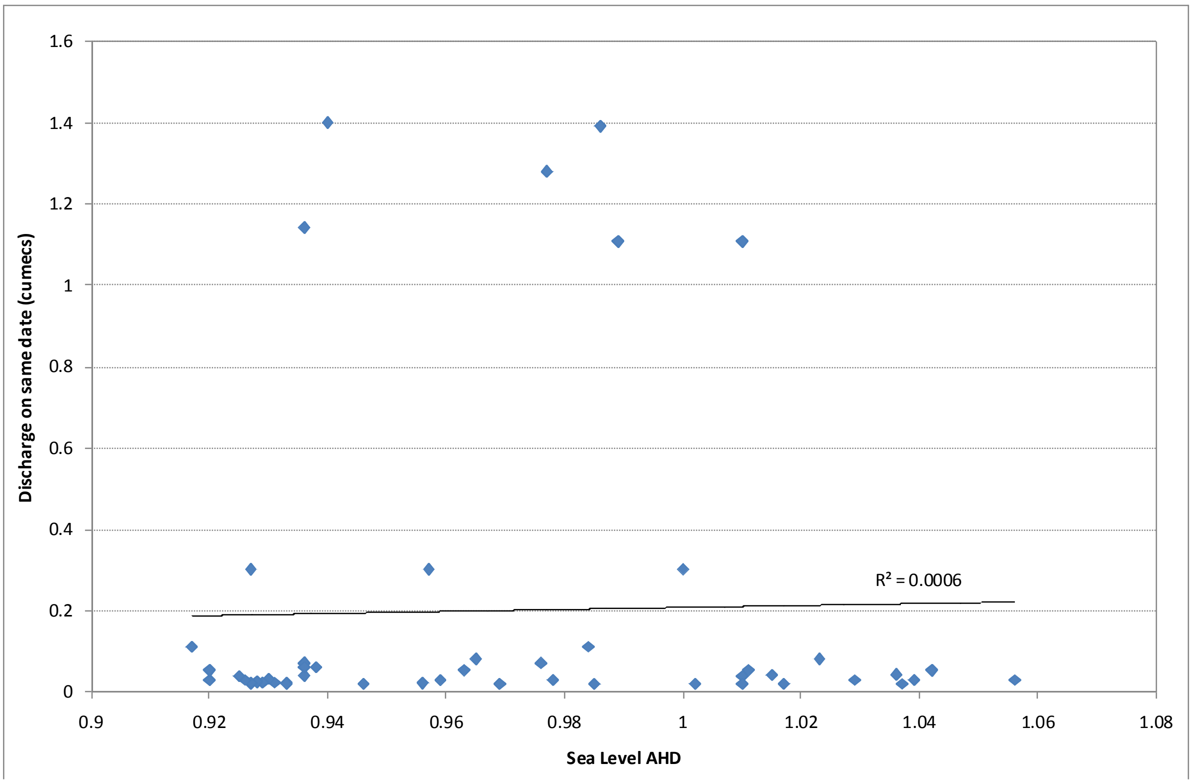


Figure 6.3 – Top 50 peak daily sea levels at Portland vs. the flows in the Surrey River

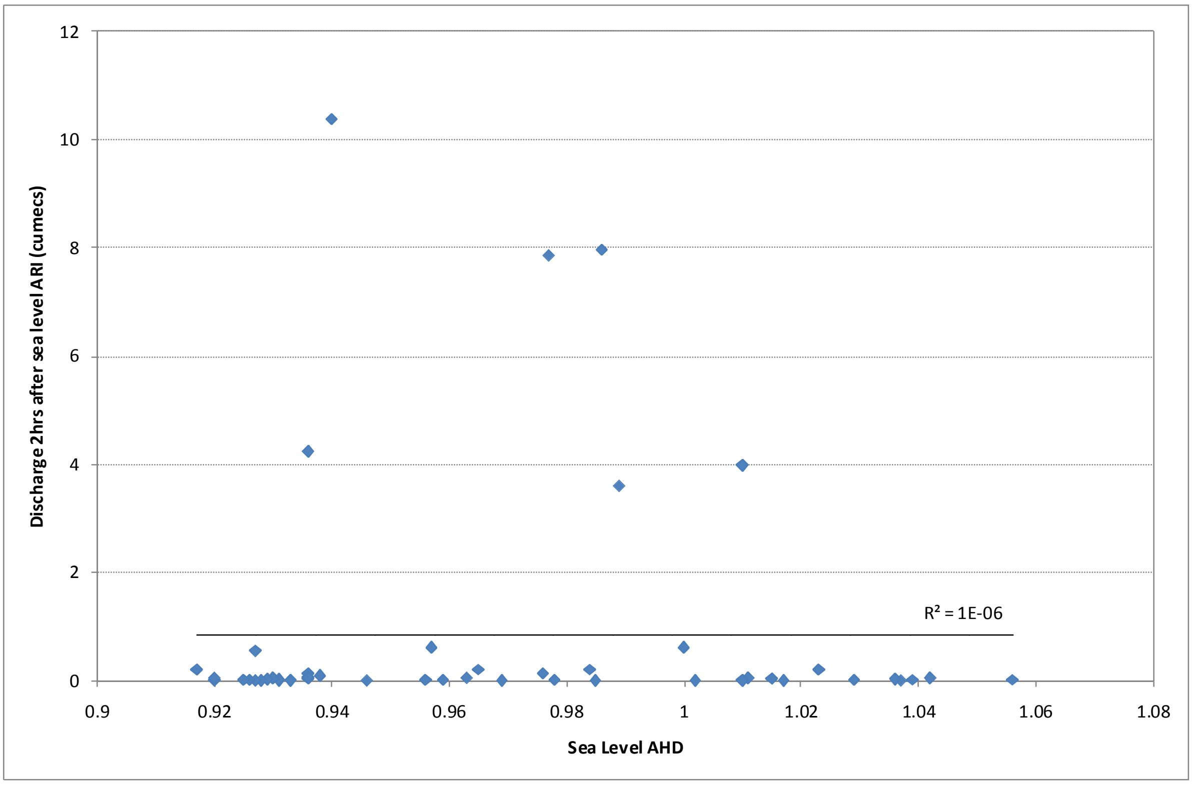


Figure 6.4 – Top 50 peak daily sea levels at Portland vs. the flows in the Surry River (+24hr)

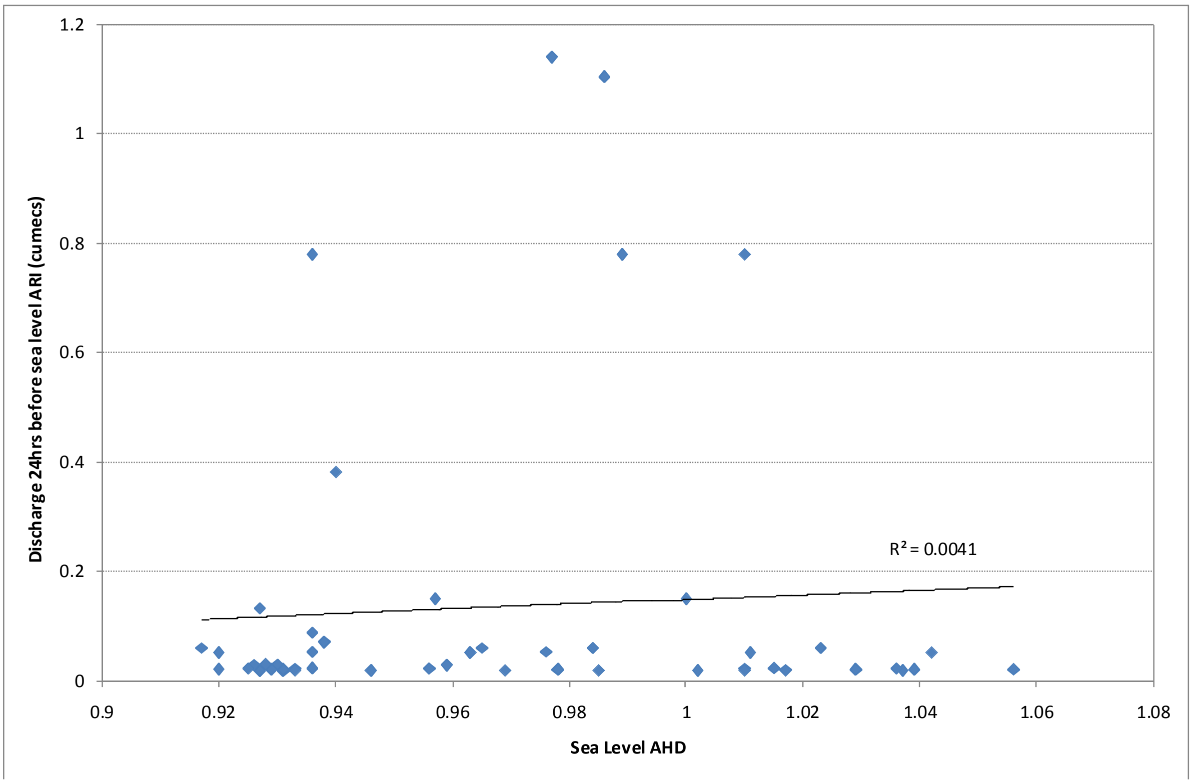


Figure 6.5 – Top 50 peak daily sea levels at Portland vs. the flows in the Surry River (-24hr)

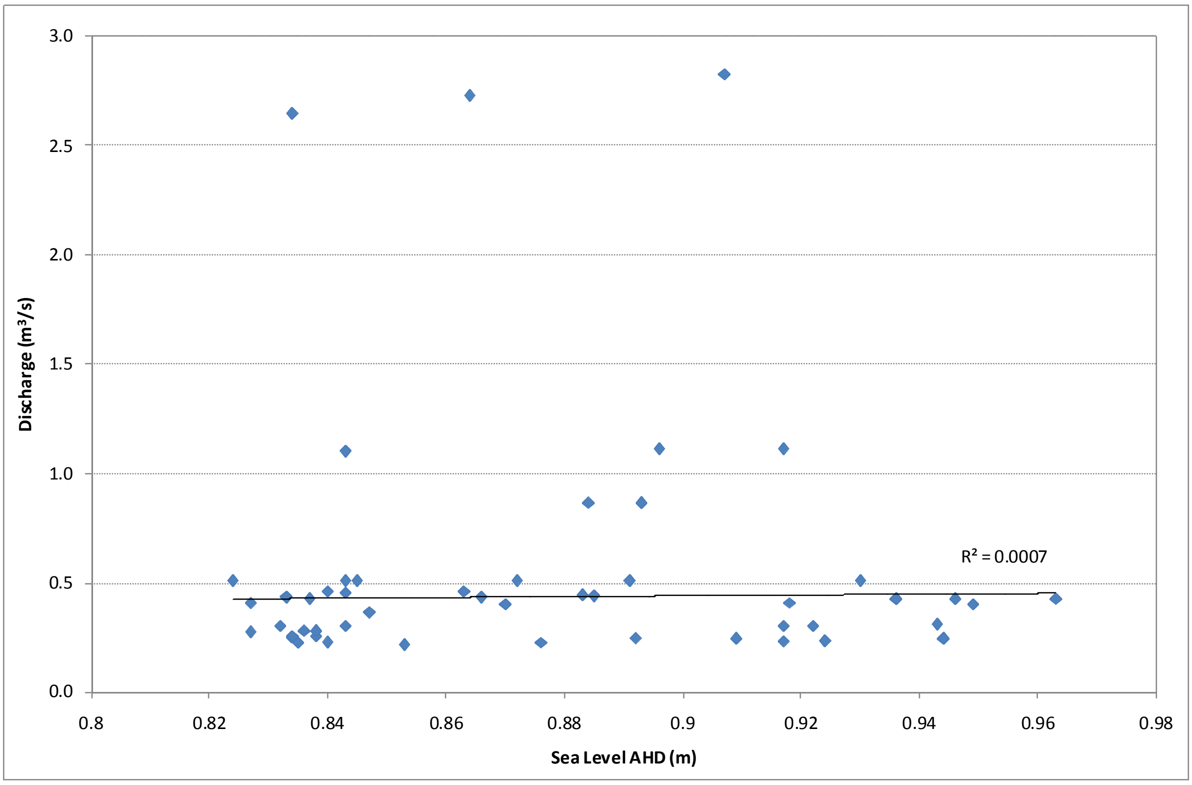


Figure 6.6 – Top 50 peak daily sea levels at Portland vs. the flows in the Merri River

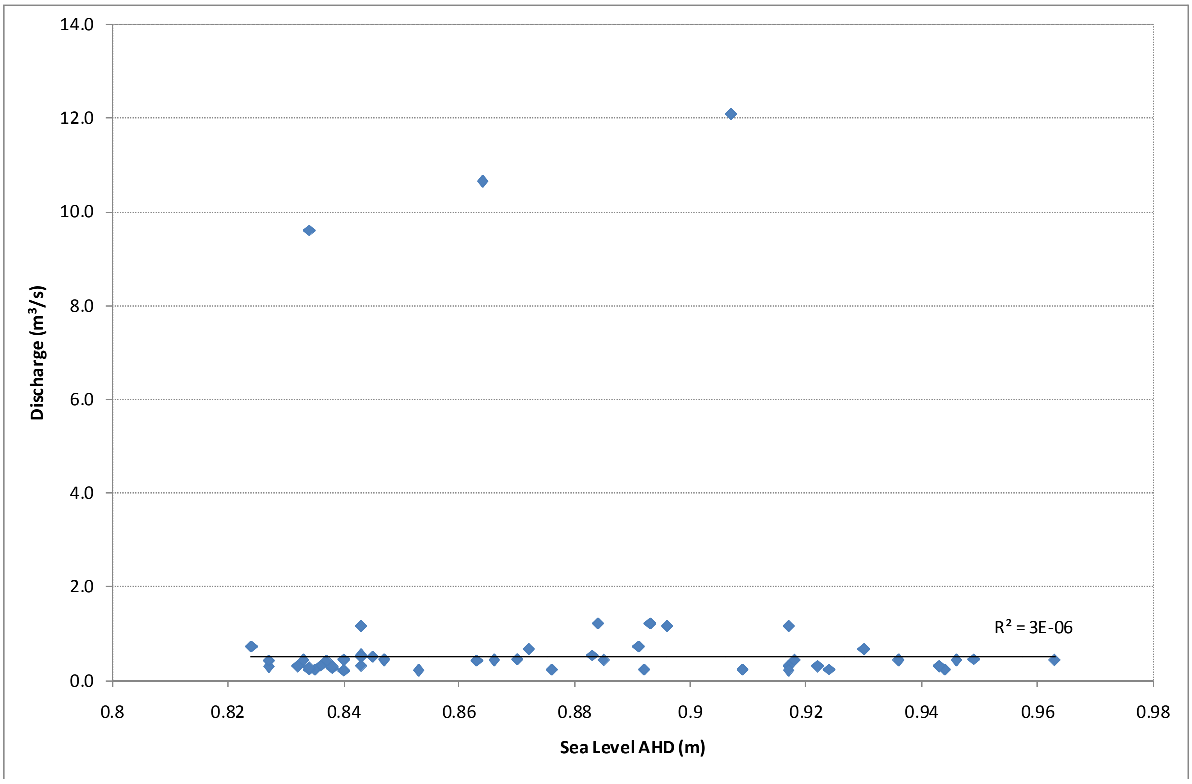


Figure 6.7 – Top 50 peak daily sea levels at Portland vs. the flows in the Merri River (+24hr)

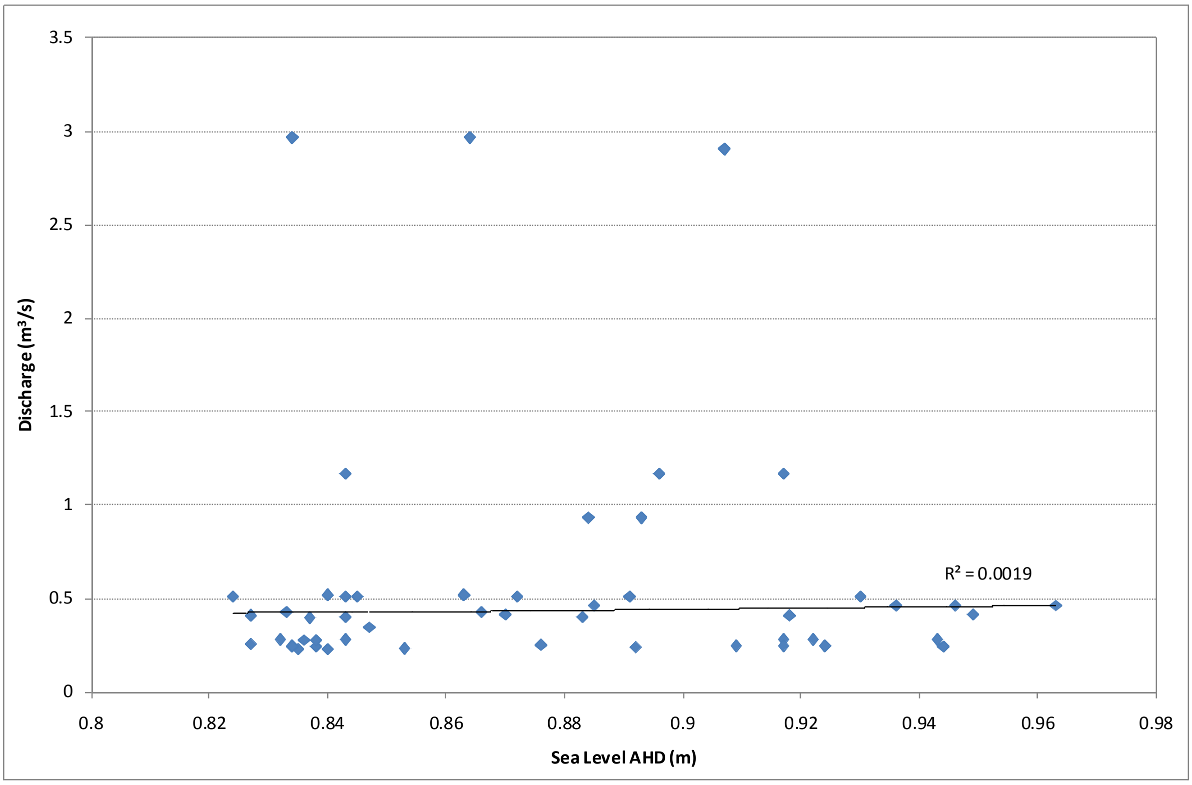


Figure 6.8 – Top 50 peak daily sea levels at Portland vs. the flows in the Merri River (-24hr)

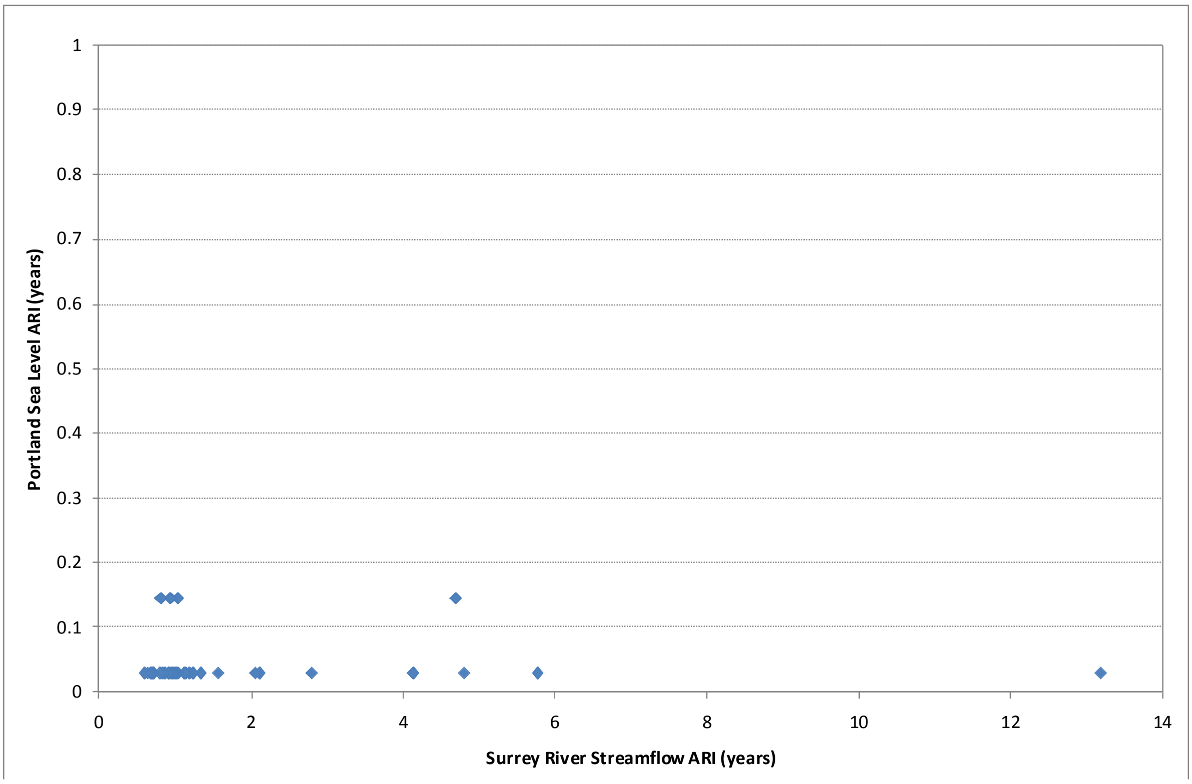


Figure 6.9 – Surrey River Discharge ARI vs. Sea Level ARI at Portland

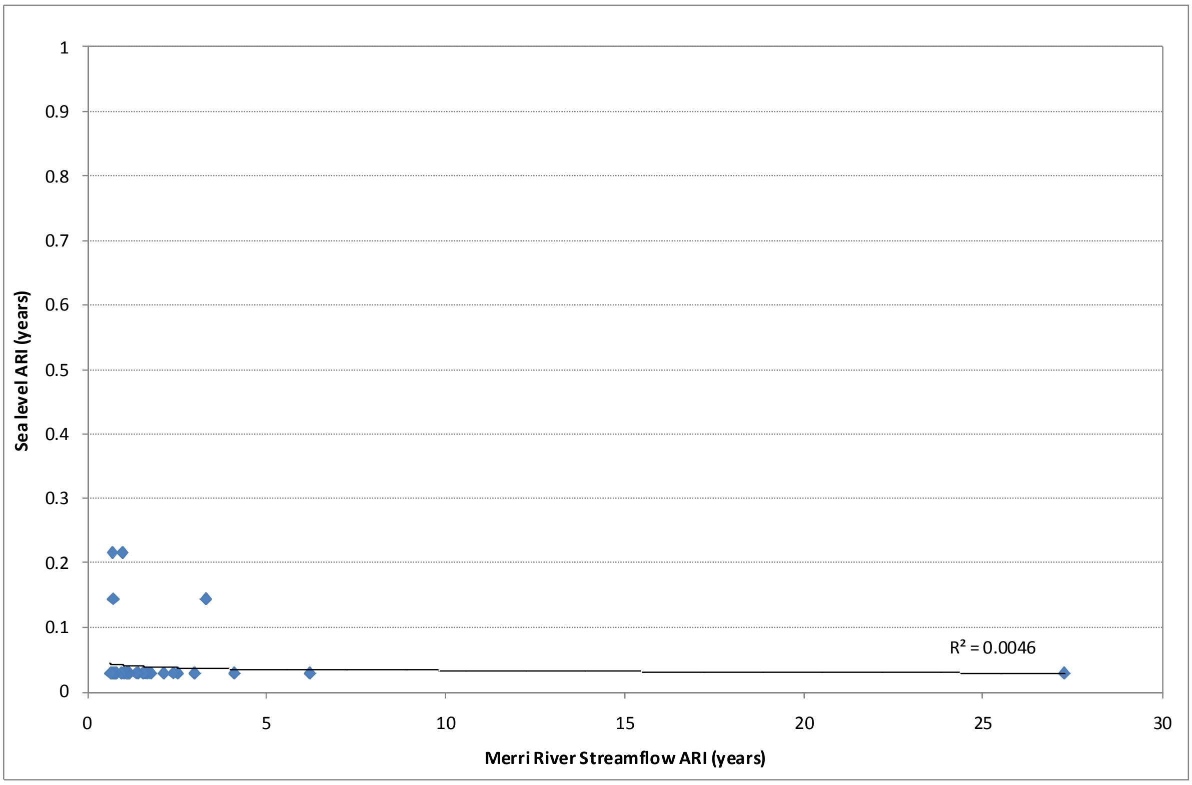


Figure 6.10 – Merri River Discharge ARI vs. Sea Level ARI at Portland

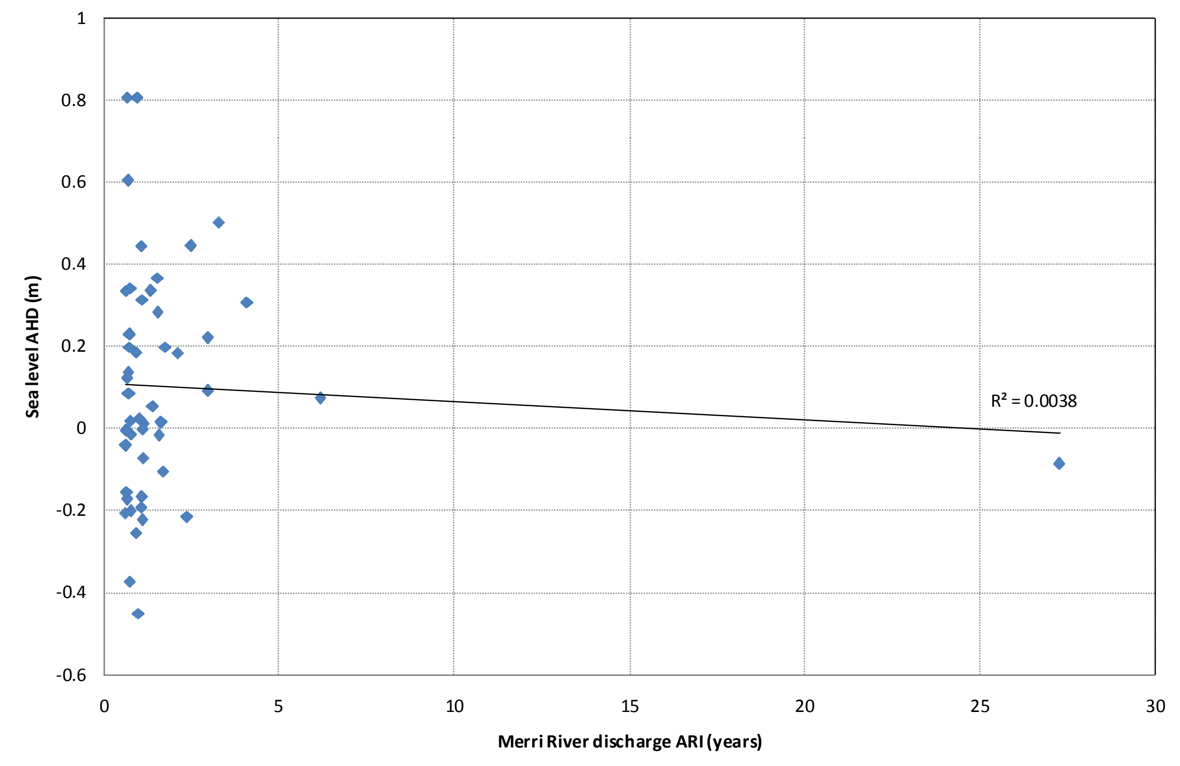


Figure 6.11 – Merri River Discharge ARI vs. Sea Level Portland

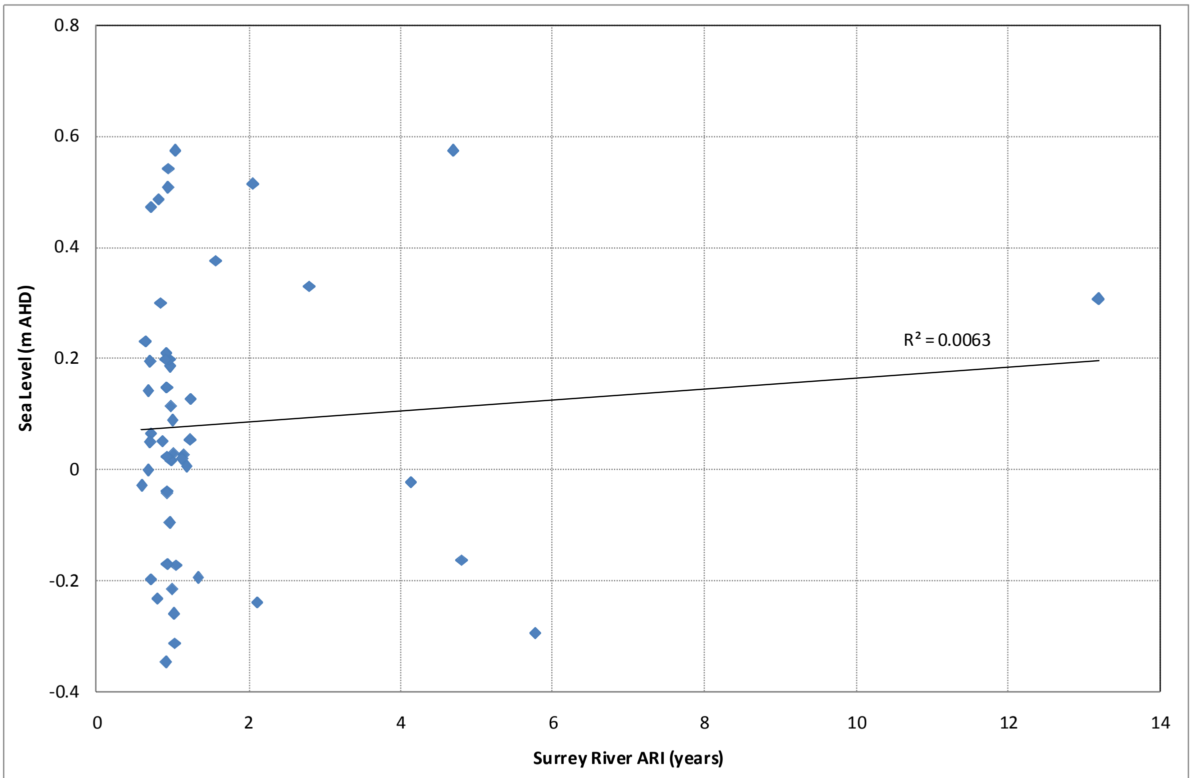


Figure 6.12 – Surrey River Discharge ARI vs. Sea Level Portland

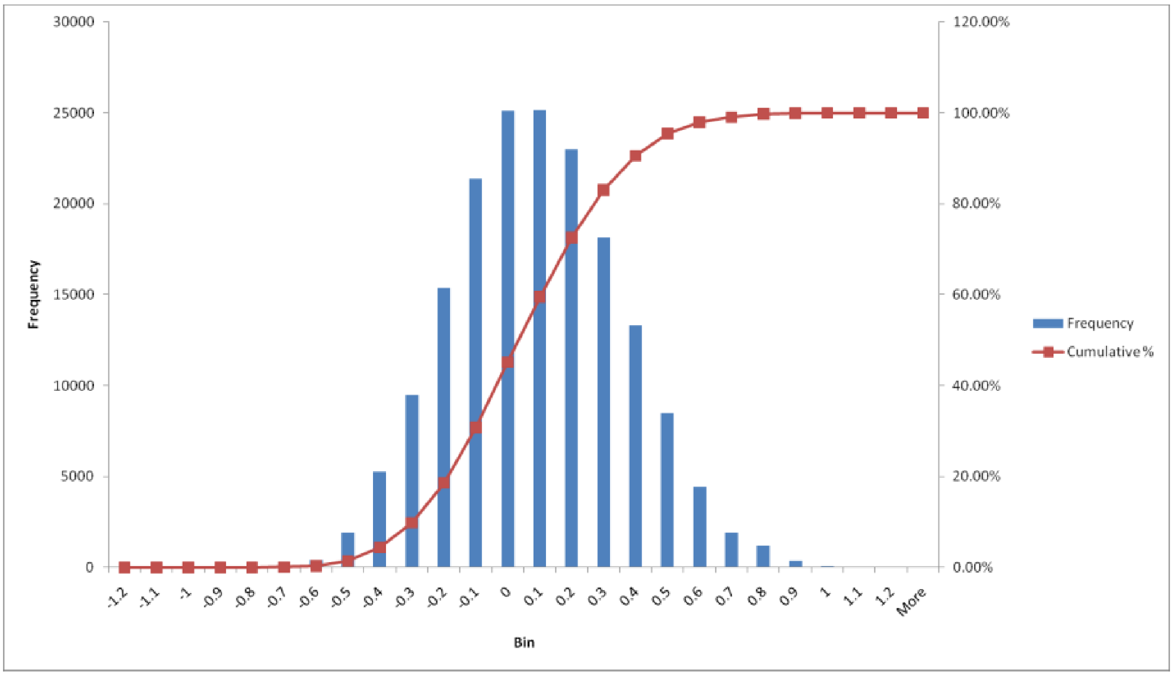


Figure 6.13 – Portland Tidal Level Histogram

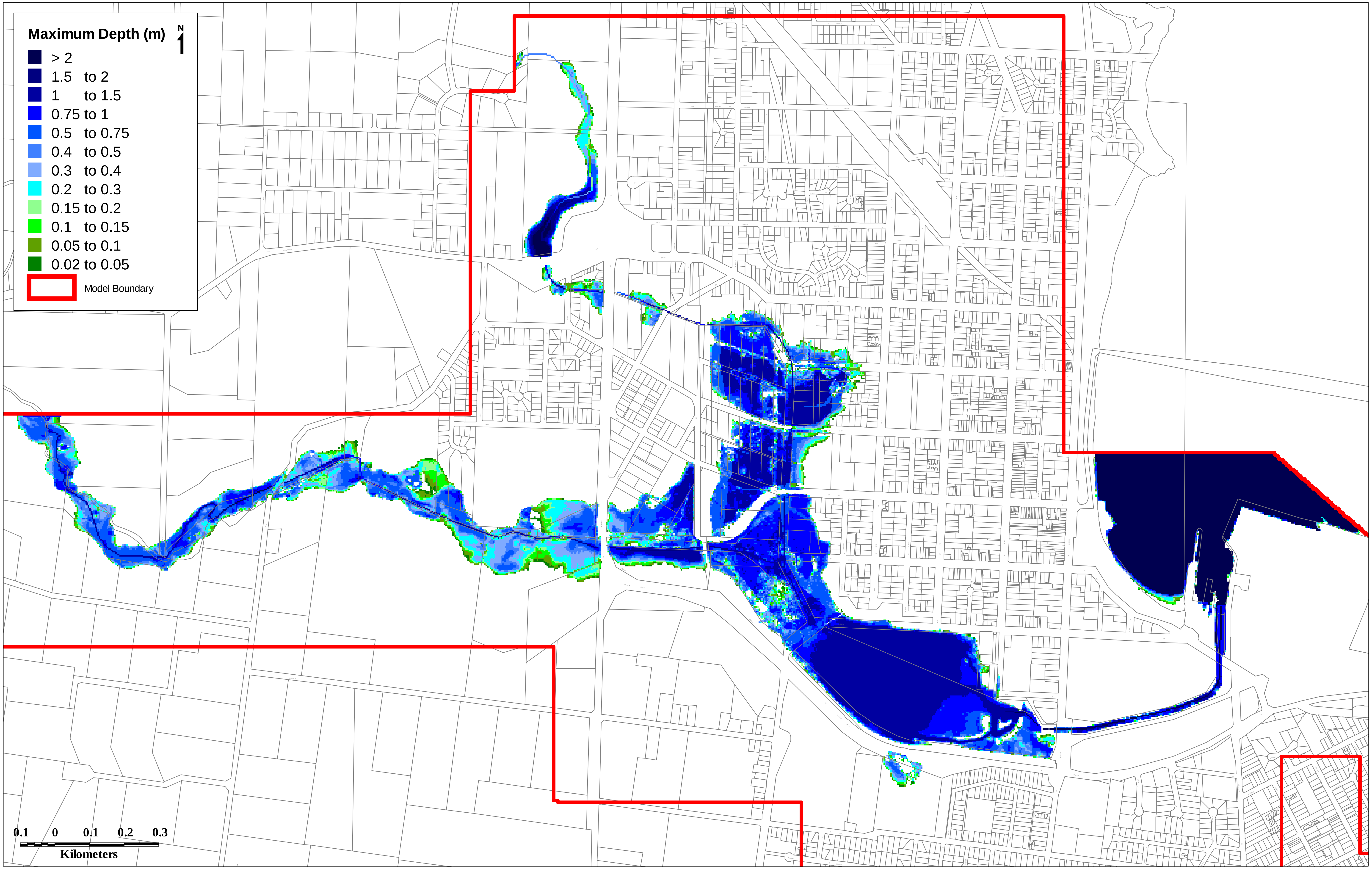


Figure 7.1 - Existing 10 year ARI Storm Event Depth

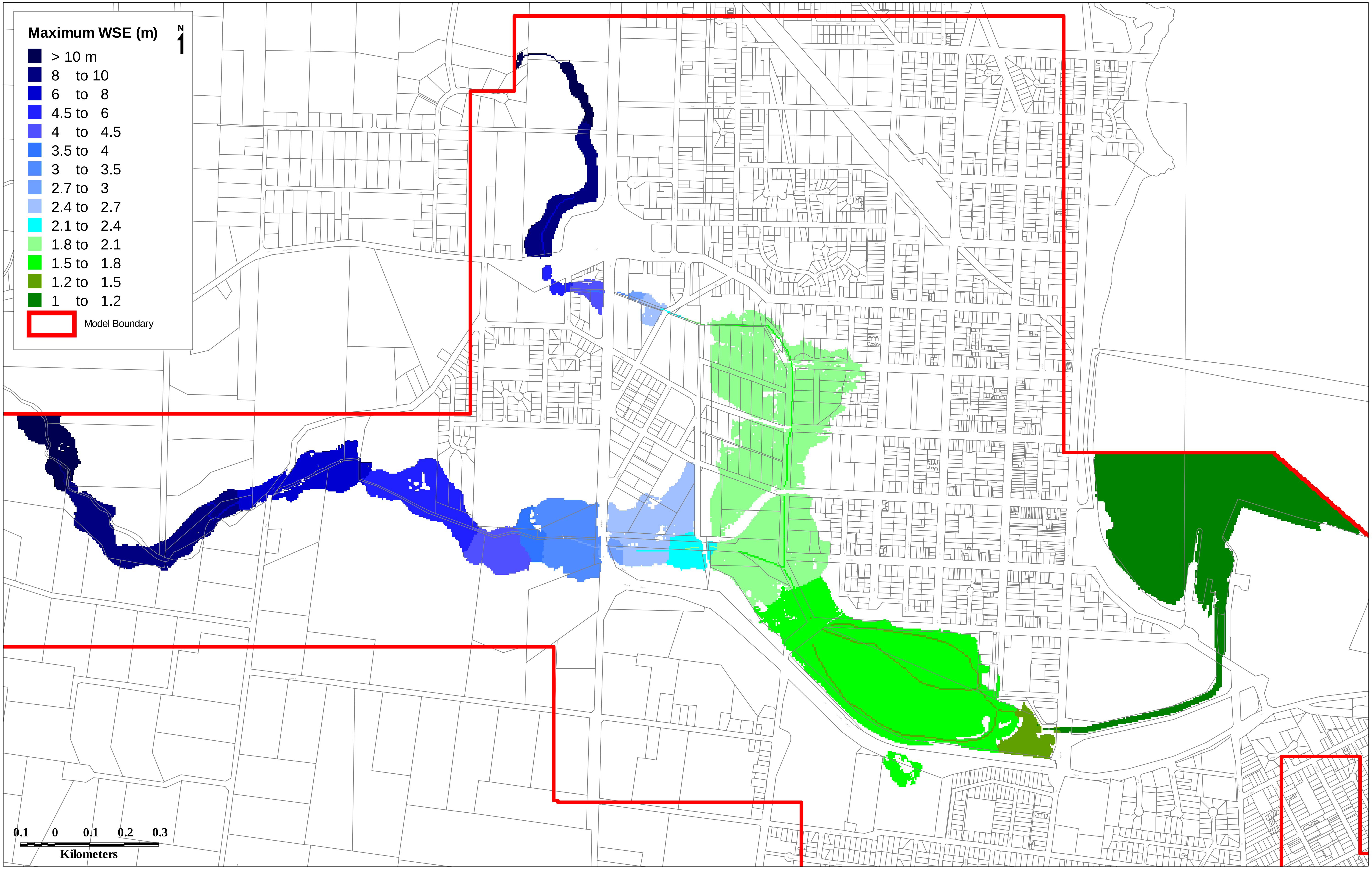


Figure 7.2 - Existing 10 year ARI Storm Event Water Surface Elevation

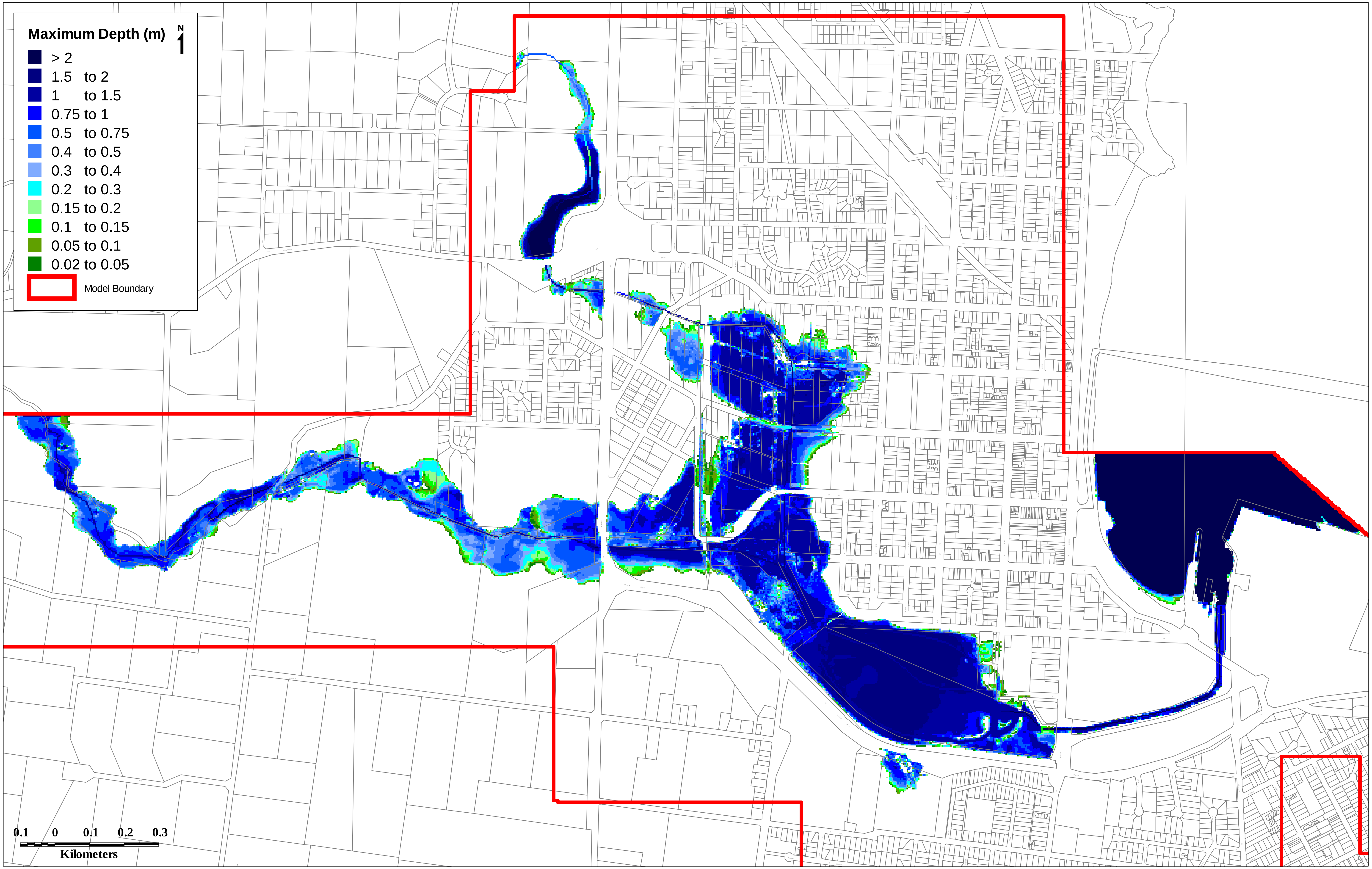


Figure 7.3 - Existing 20 year ARI Storm Event Depth

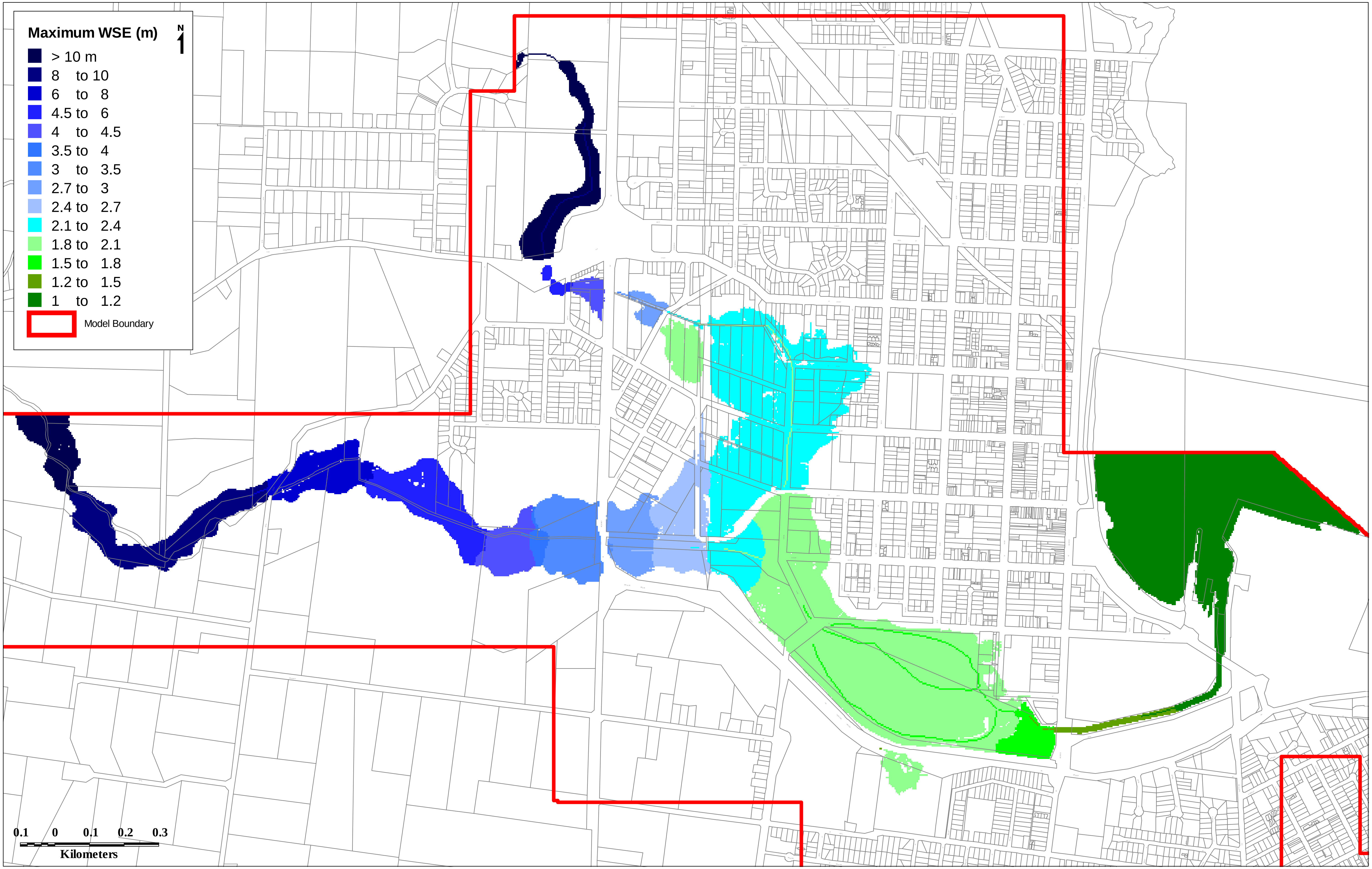


Figure 7.4 - Existing 20 year ARI Storm Event Water Surface Elevation

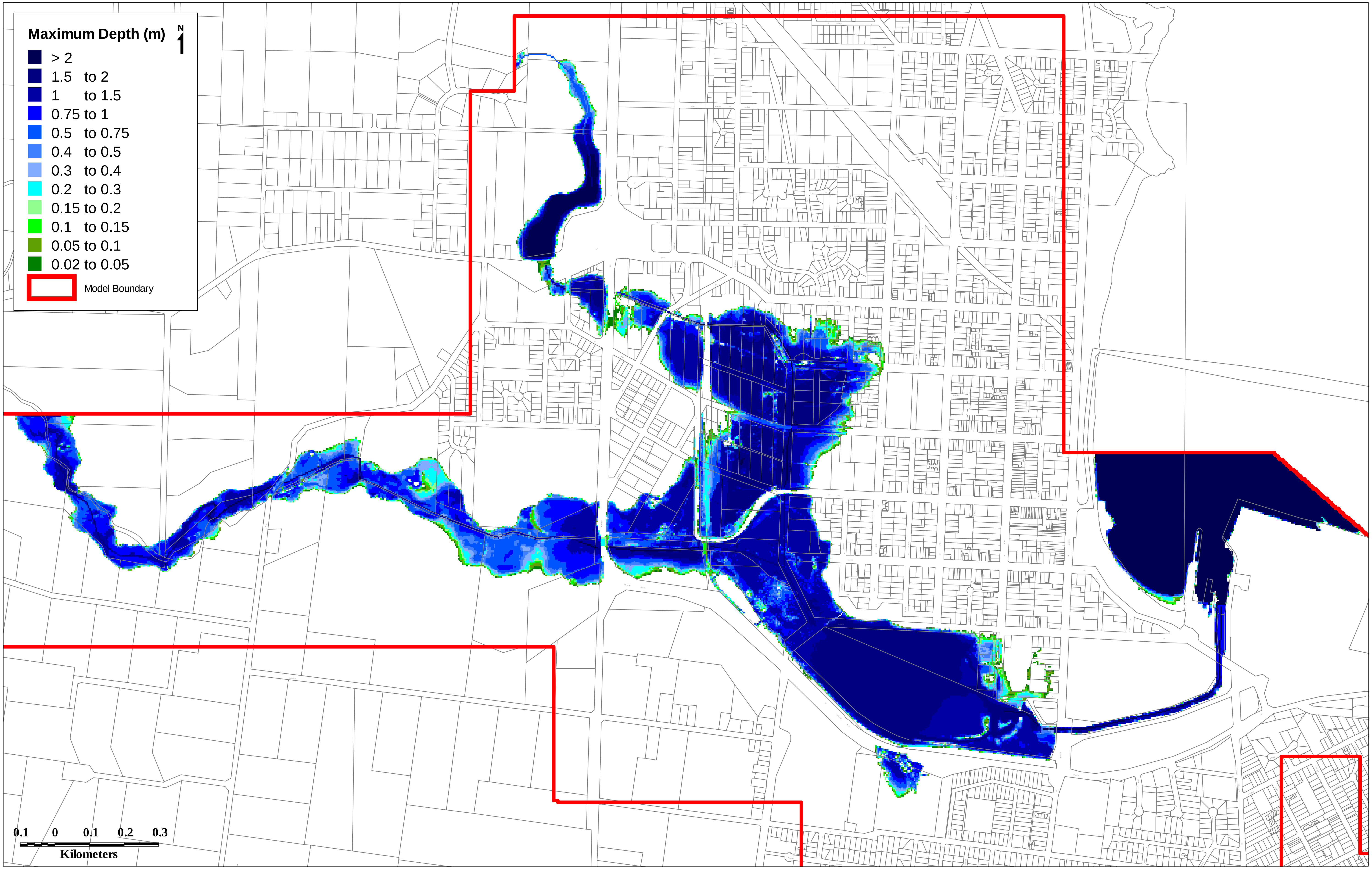


Figure 7.5 - Existing 50 year ARI Storm Event Depth

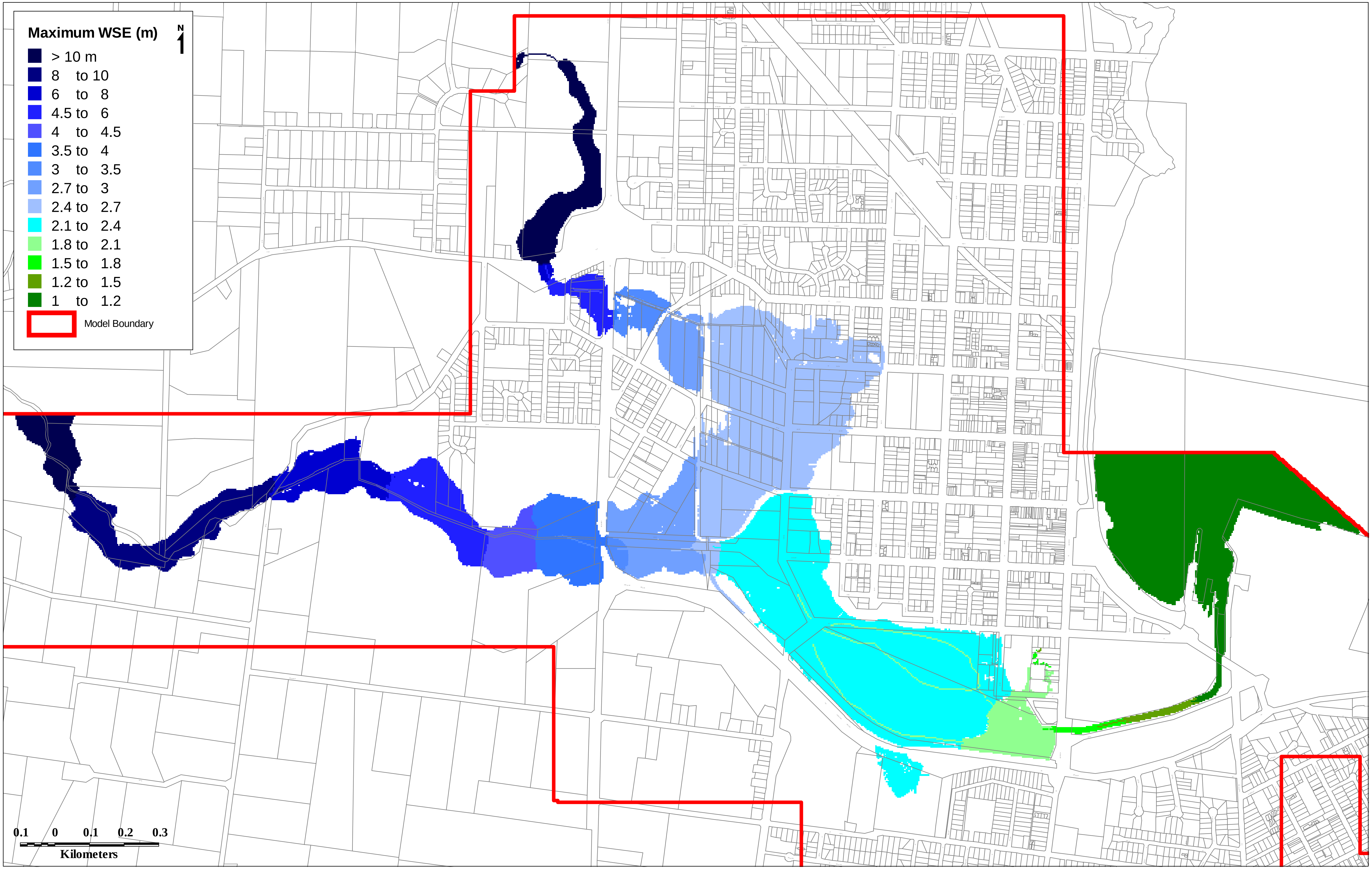


Figure 7.6 - Existing 50 year ARI Storm Event Water Surface Elevation

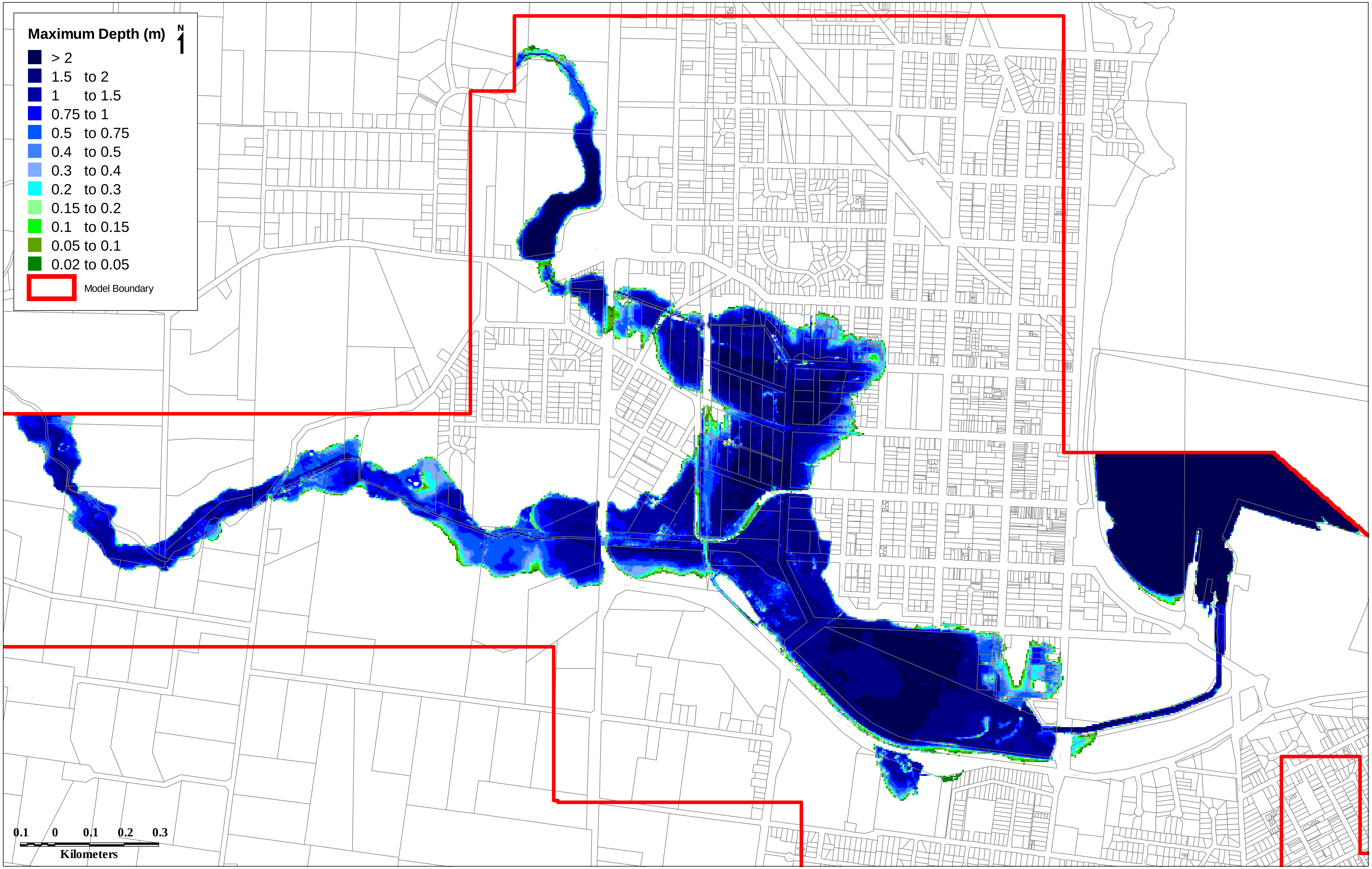


Figure 7.7 - Existing 100 year ARI Storm Event Depth

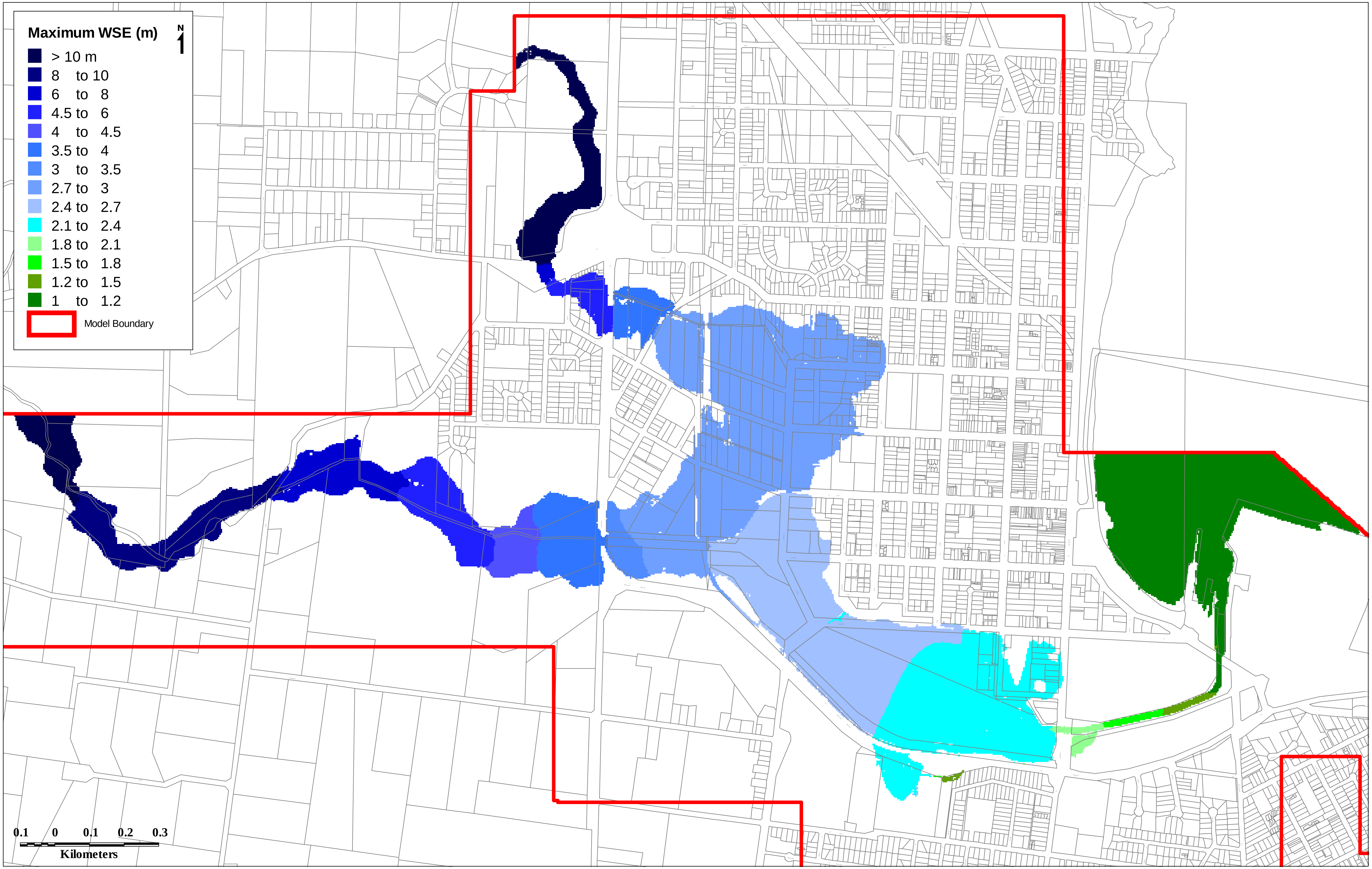


Figure 7.8 - Existing 100 year ARI Storm Event Water Surface Elevation

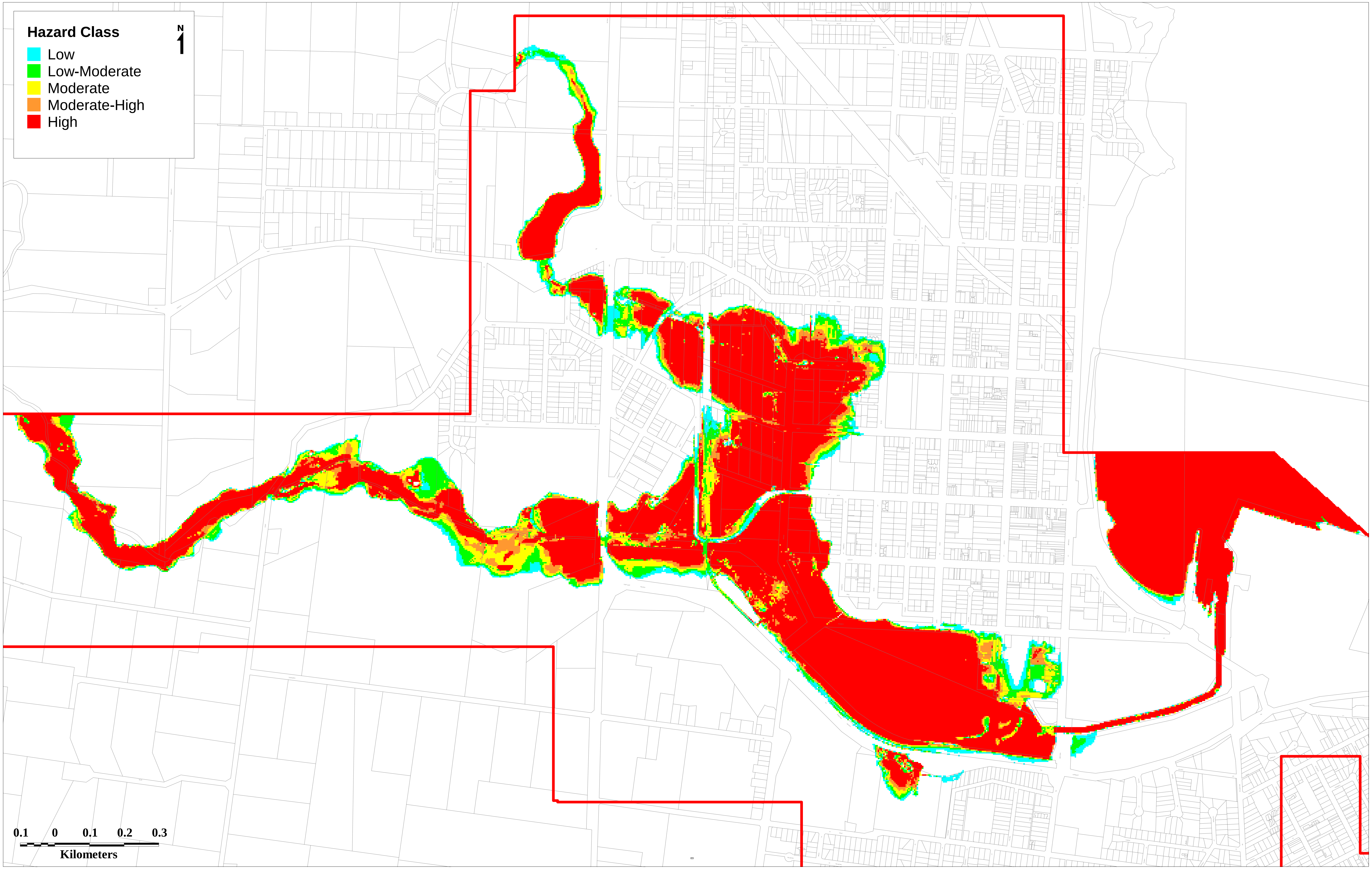


Figure 7.9 - Existing 100 year ARI Storm Event Hazard Class

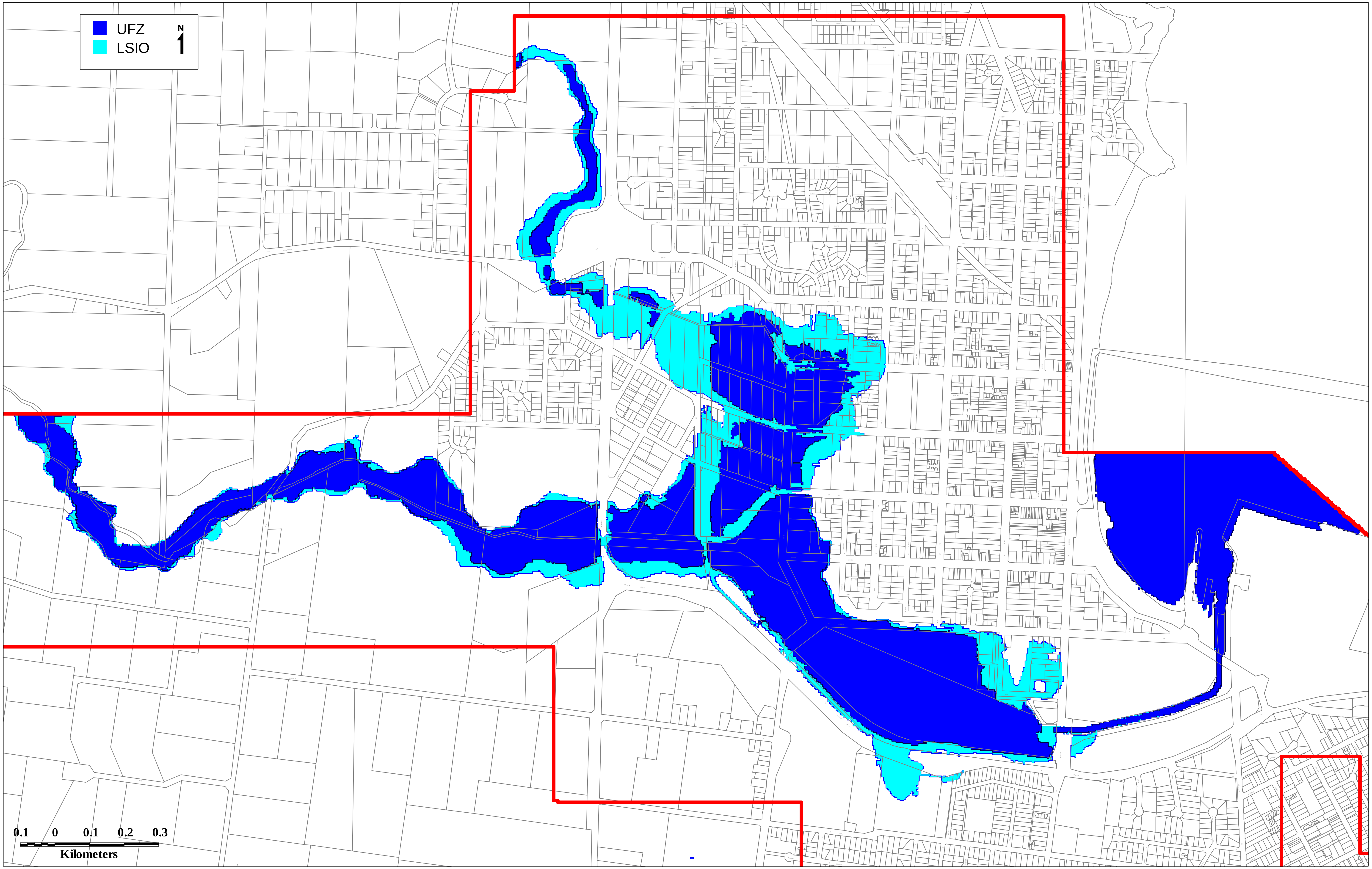


Figure 7.10 - Existing 100 year ARI Storm Event LSIO and UFZ Classification

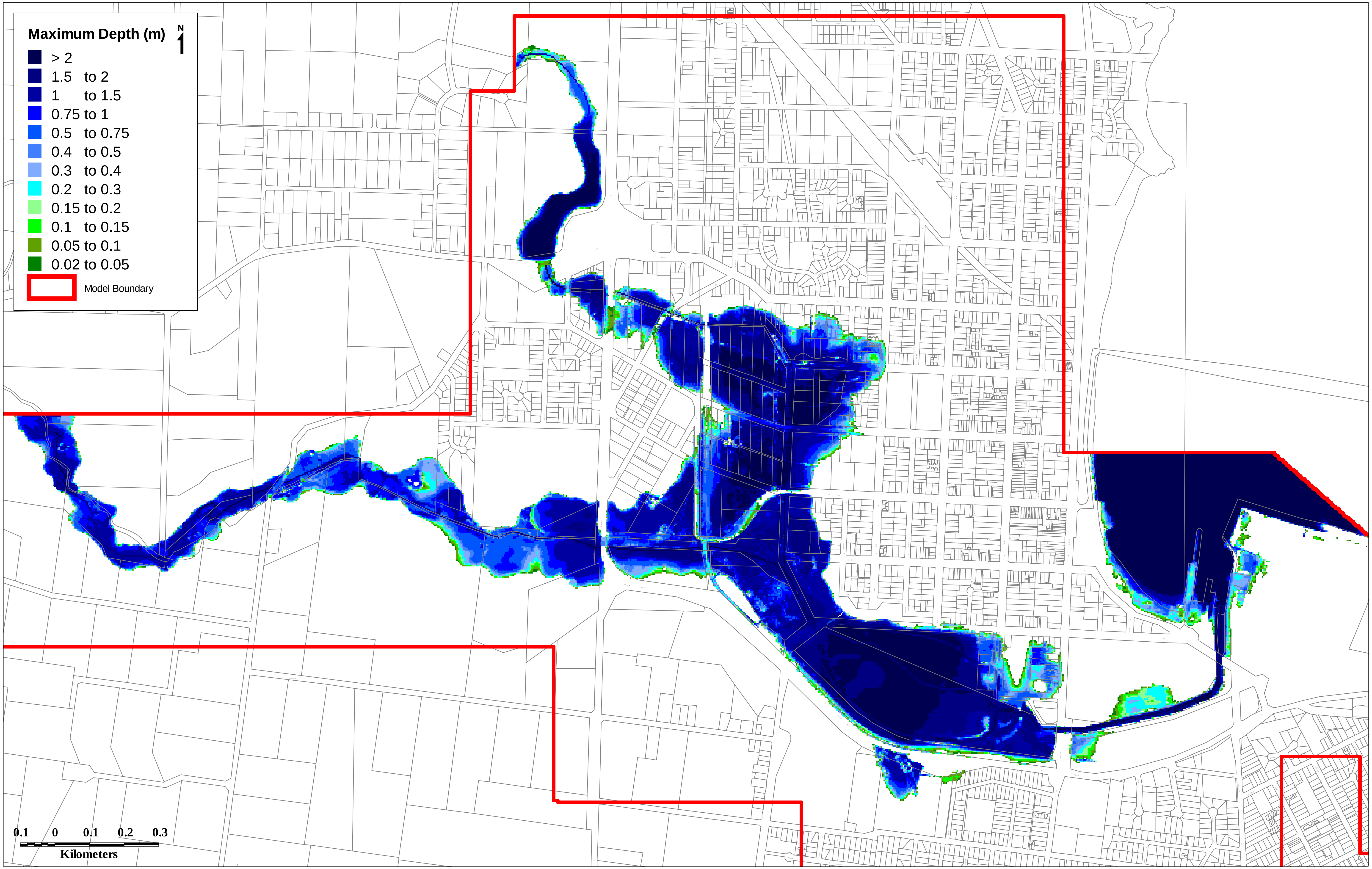


Figure 7.11 - Existing 100 year ARI Storm Event, 100 year Sea Level, Depth

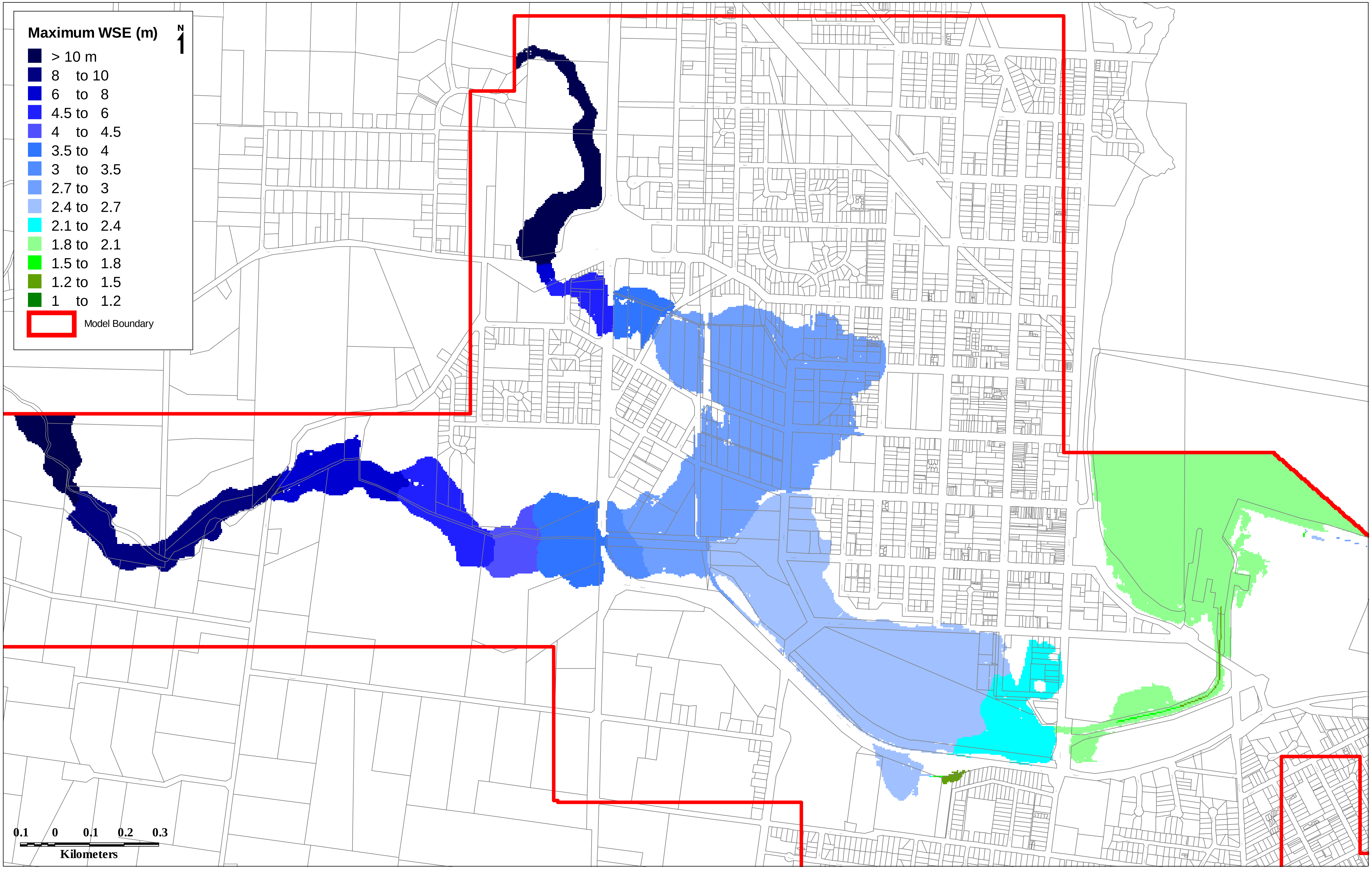


Figure 7.12 - Existing 100 year ARI Storm Event, 100 year Sea Level, Water Surface Elevation

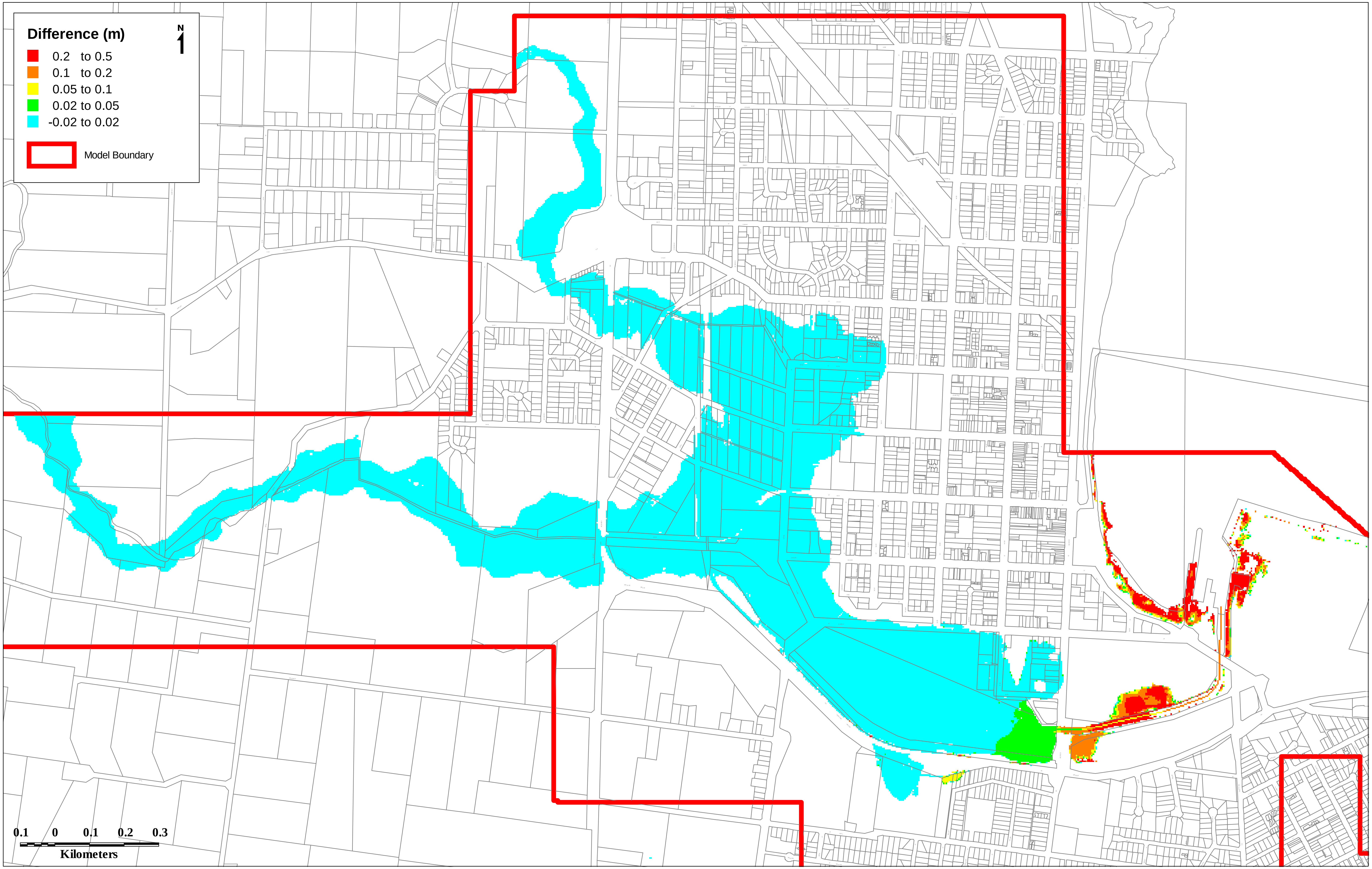


Figure 7.13 - Difference Plot, Existing 100 year ARI - 100 year sea level minus 10 year sea level

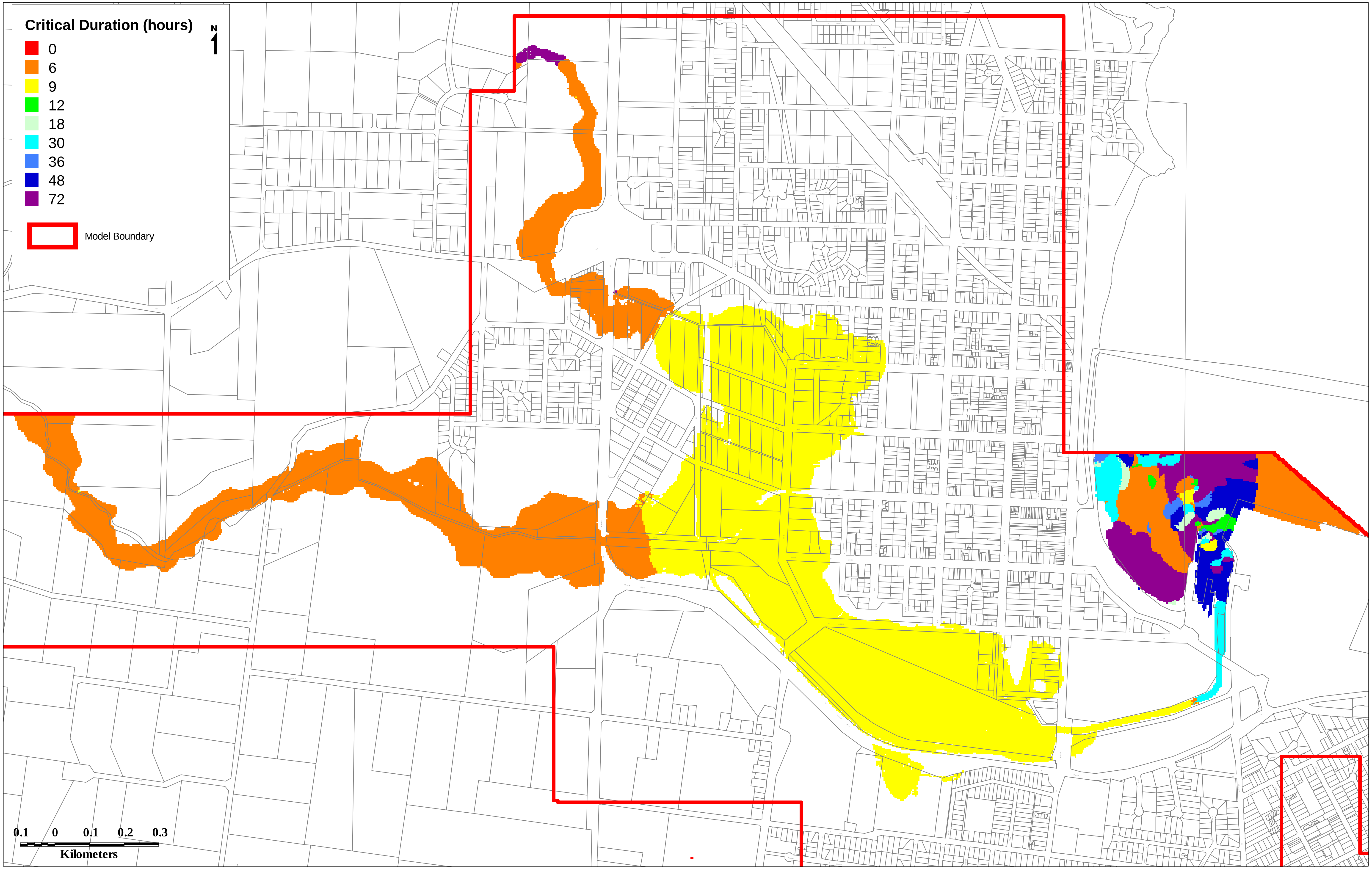


Figure 7.14 - Existing 100 year ARI critical duration

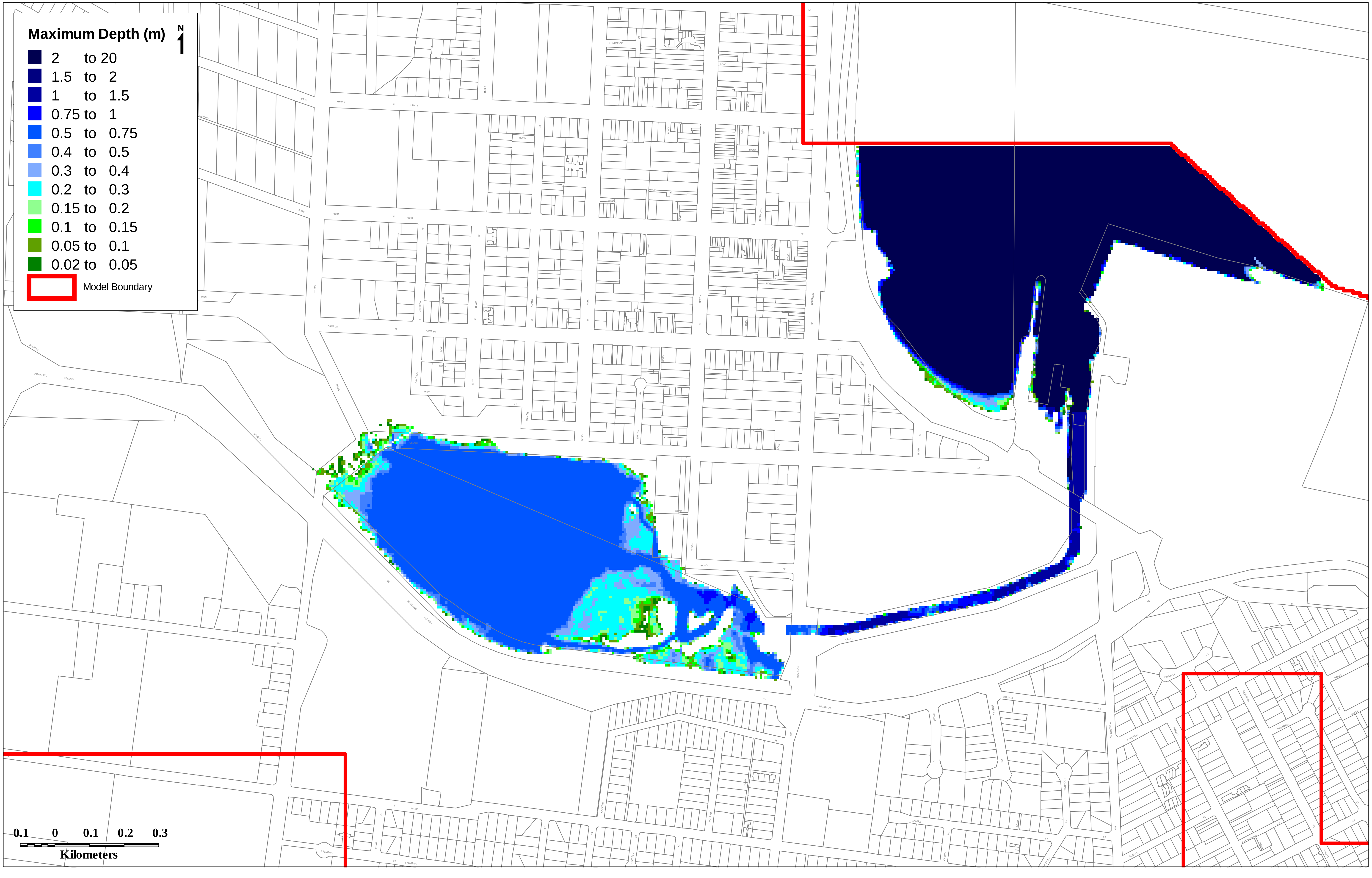


Figure 7.15 - 100 Year Storm Surge Sea Level - Depth

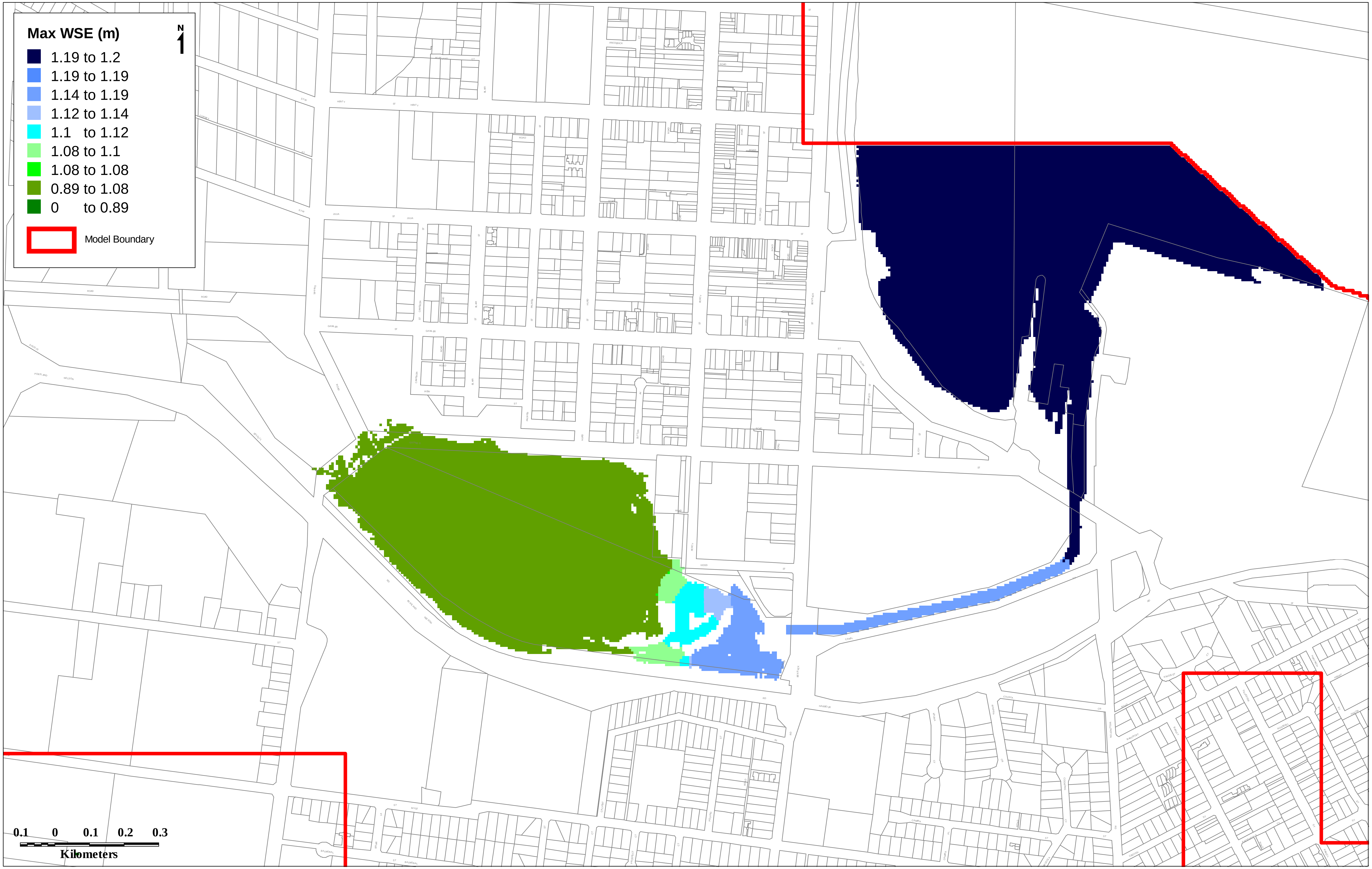


Figure 7.16 - 100 Year Storm Surge Sea Level - Water Surface Elevation

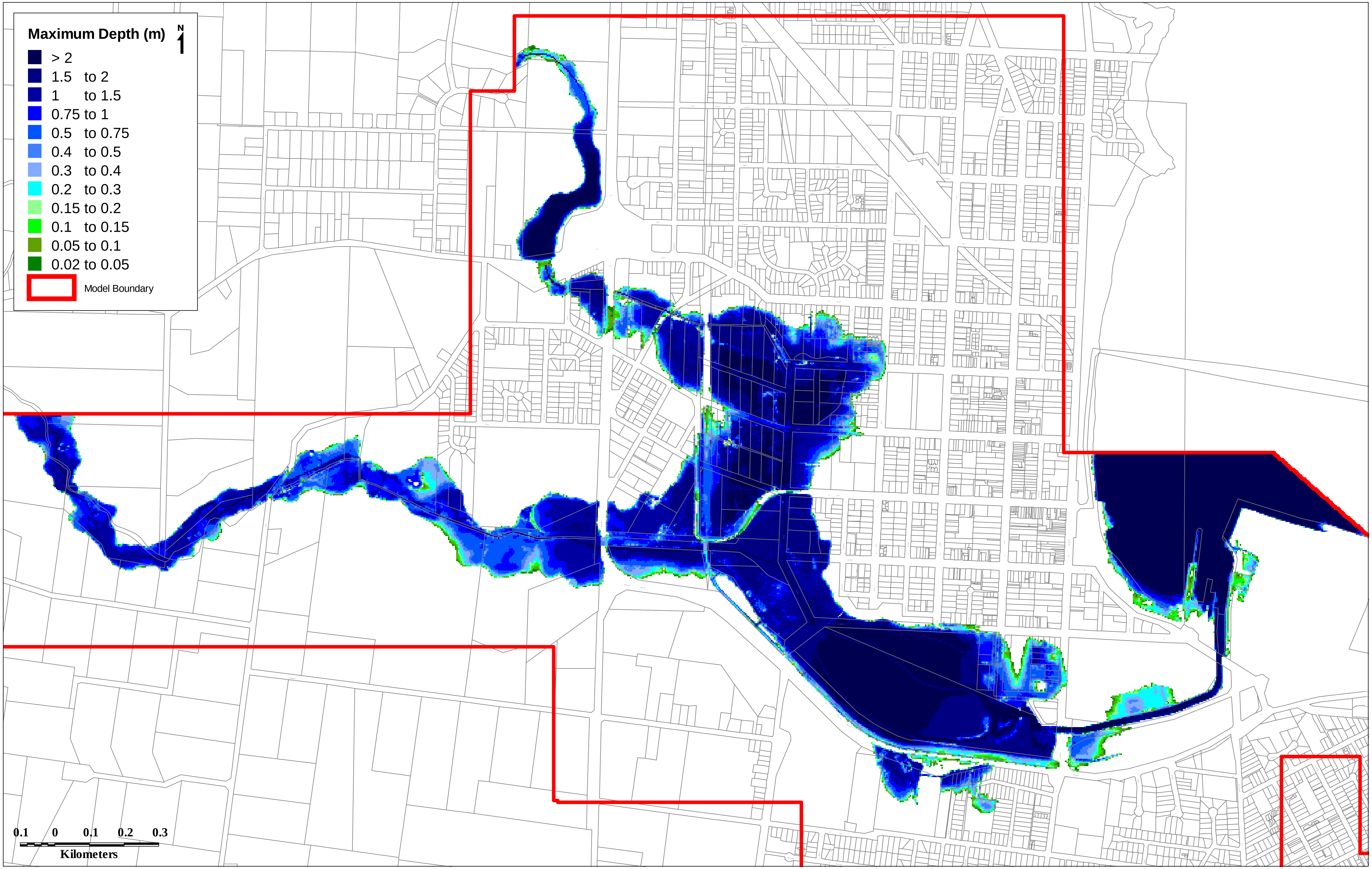


Figure 7.17 - 100 year ARI Storm Event Depth with the 2100 10 year ARI tide levels

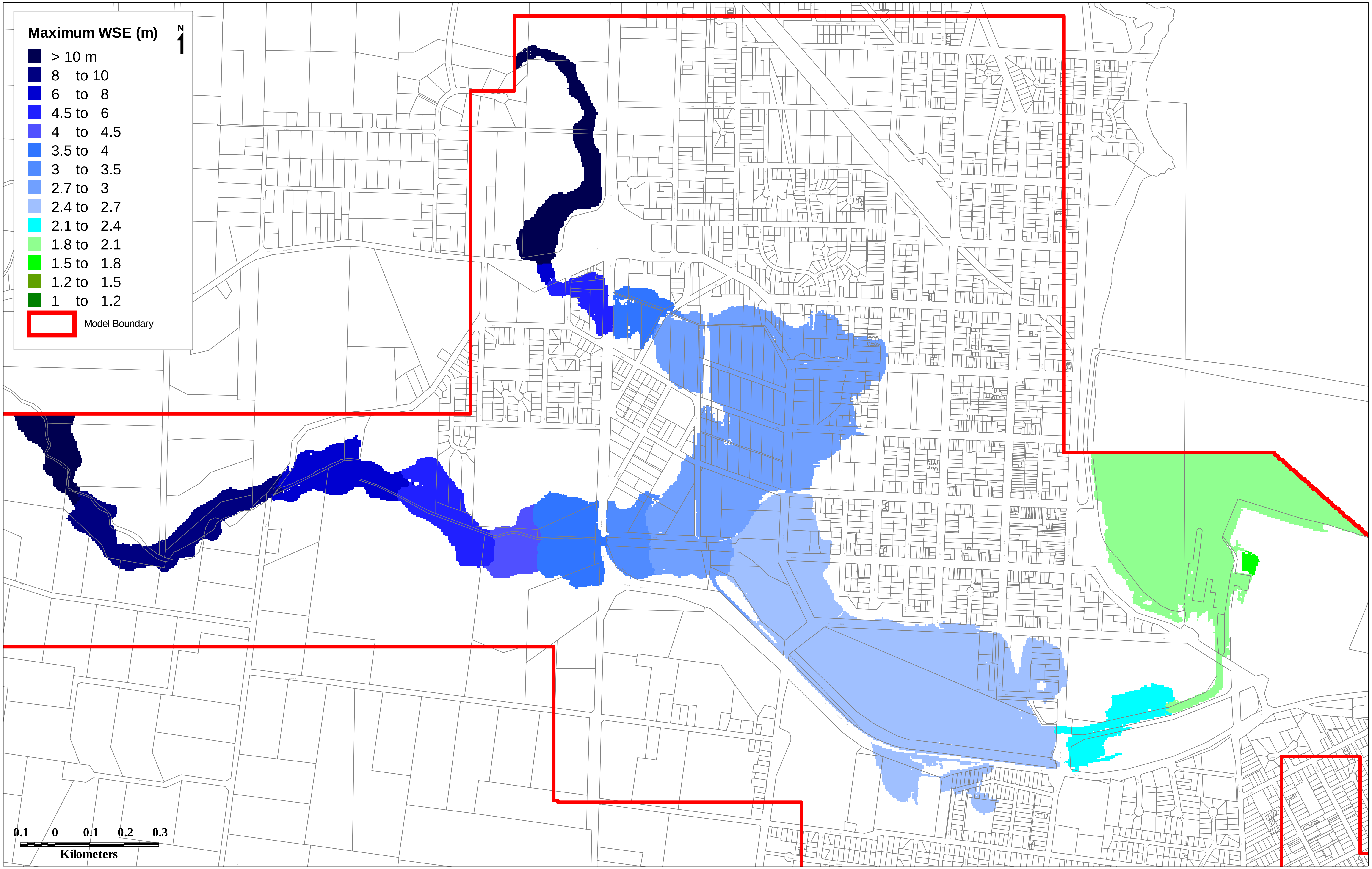


Figure 7.18 - 100 year ARI storm event water surface elevation with the 2100 10 year ARI tide levels

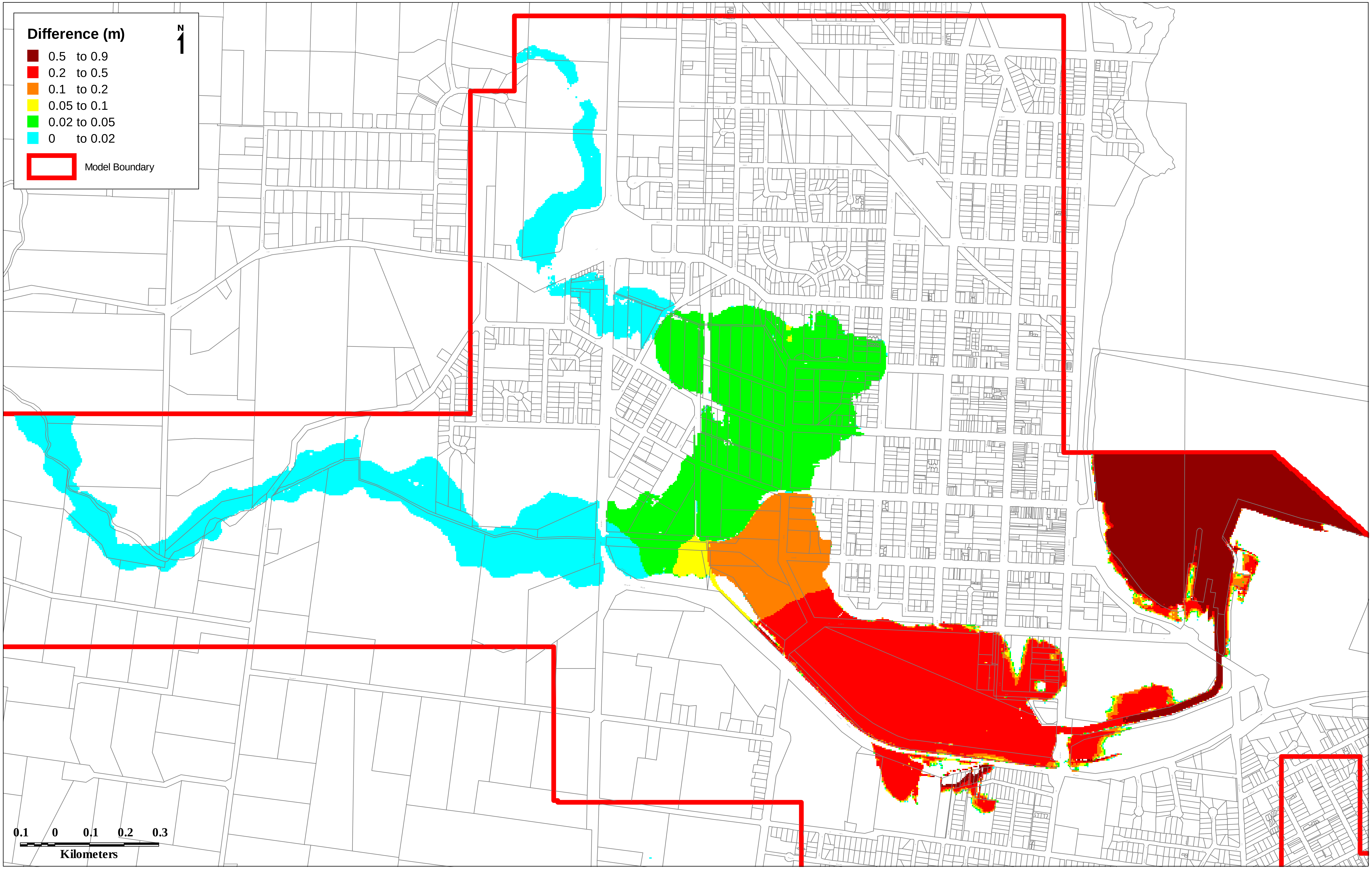


Figure 7.19 - Difference Plot, 100 year ARI Storm Event Climate Change minus Existing

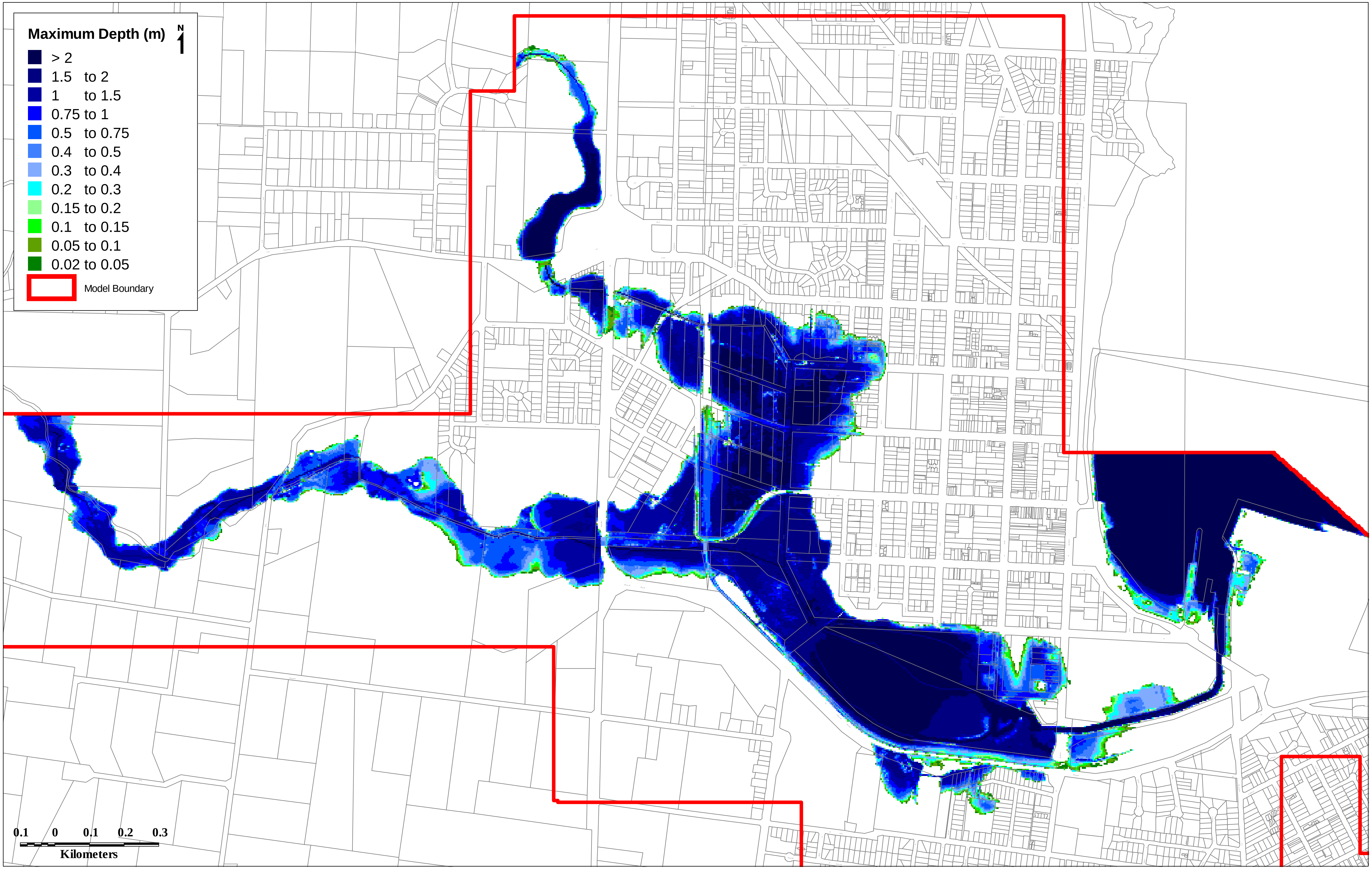


Figure 7.20 - Climate Change 100 year ARI Storm Event, 100 year Sea Level, Depth

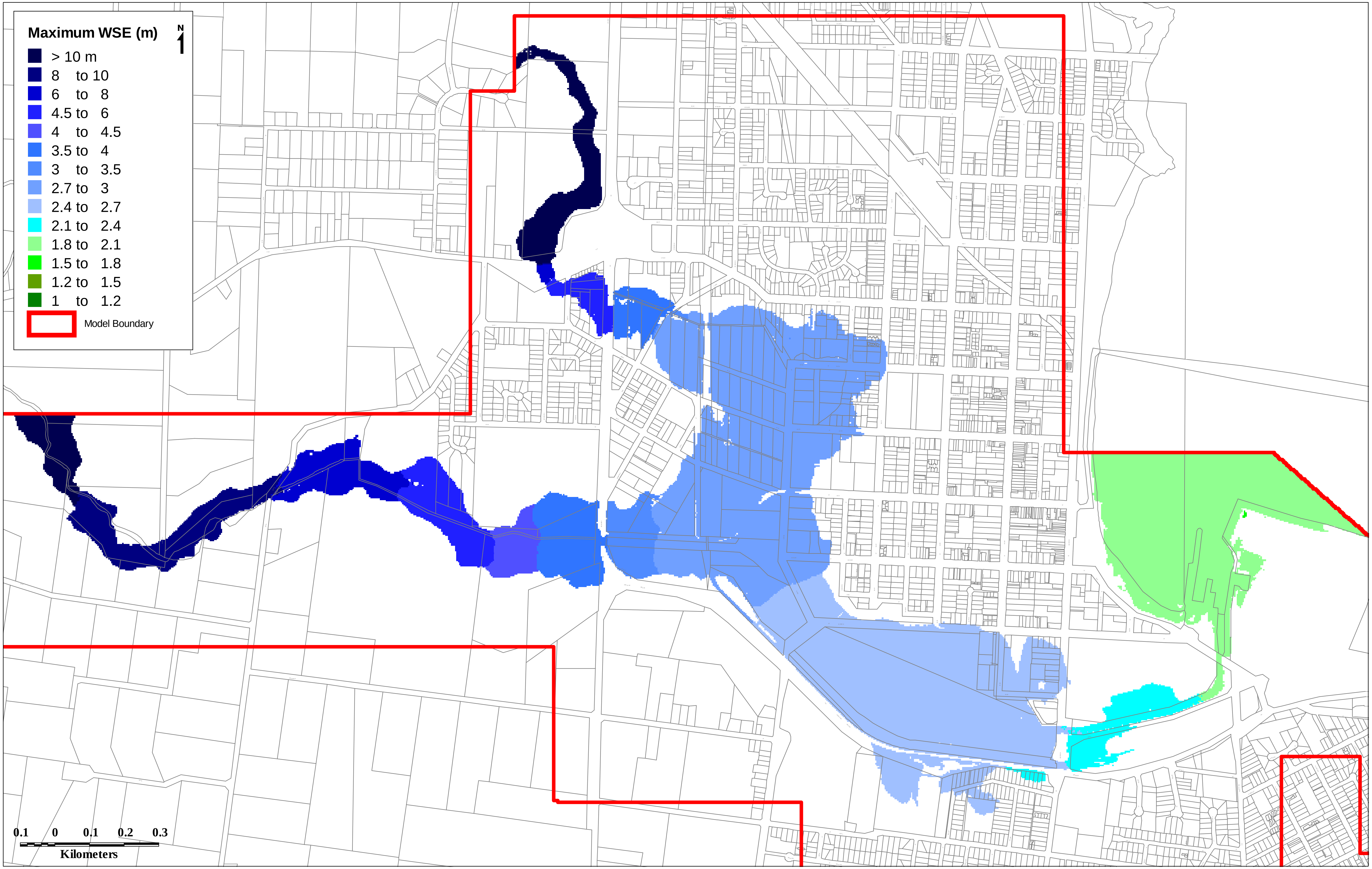


Figure 7.21 - Climate Change 100 year ARI Storm Event, 100 year Sea Level, Water Surface Elevation

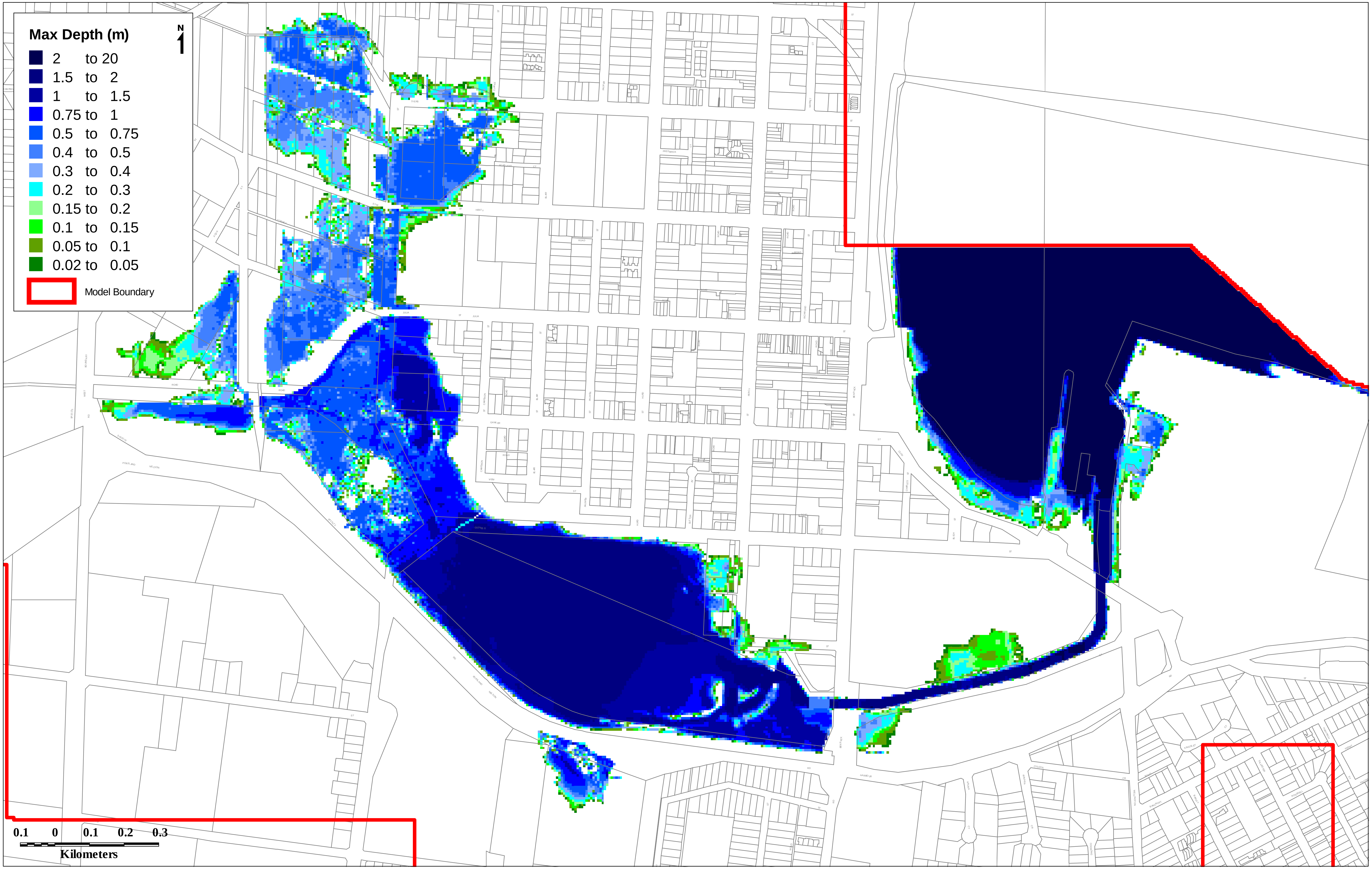


Figure 7.22 - 100 Year Storm Surge Sea Level with Climate Change - Depth

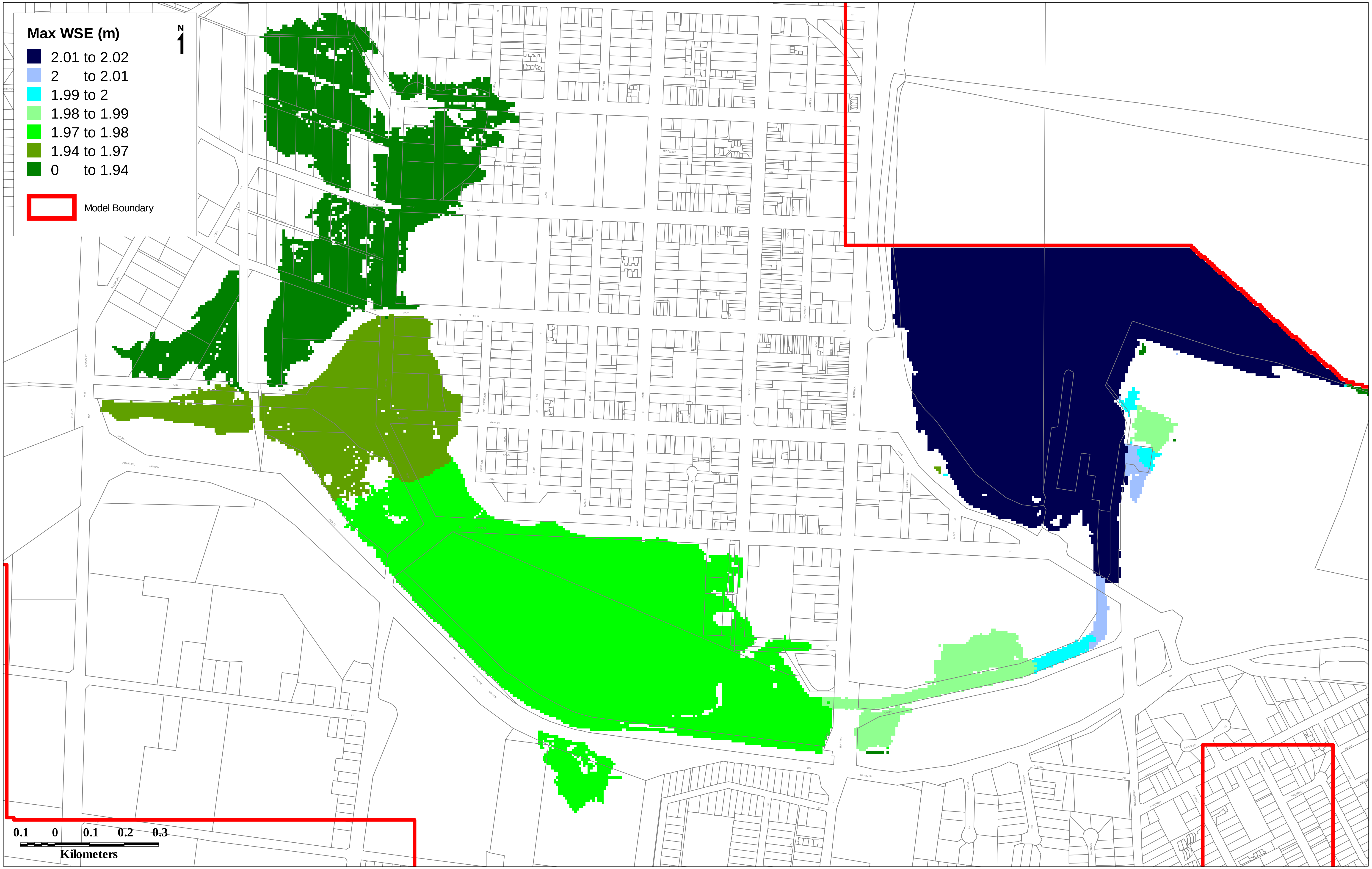


Figure 7.23 - 100 Year Storm Surge Sea Level with Climate Change - Water Surface Elevation

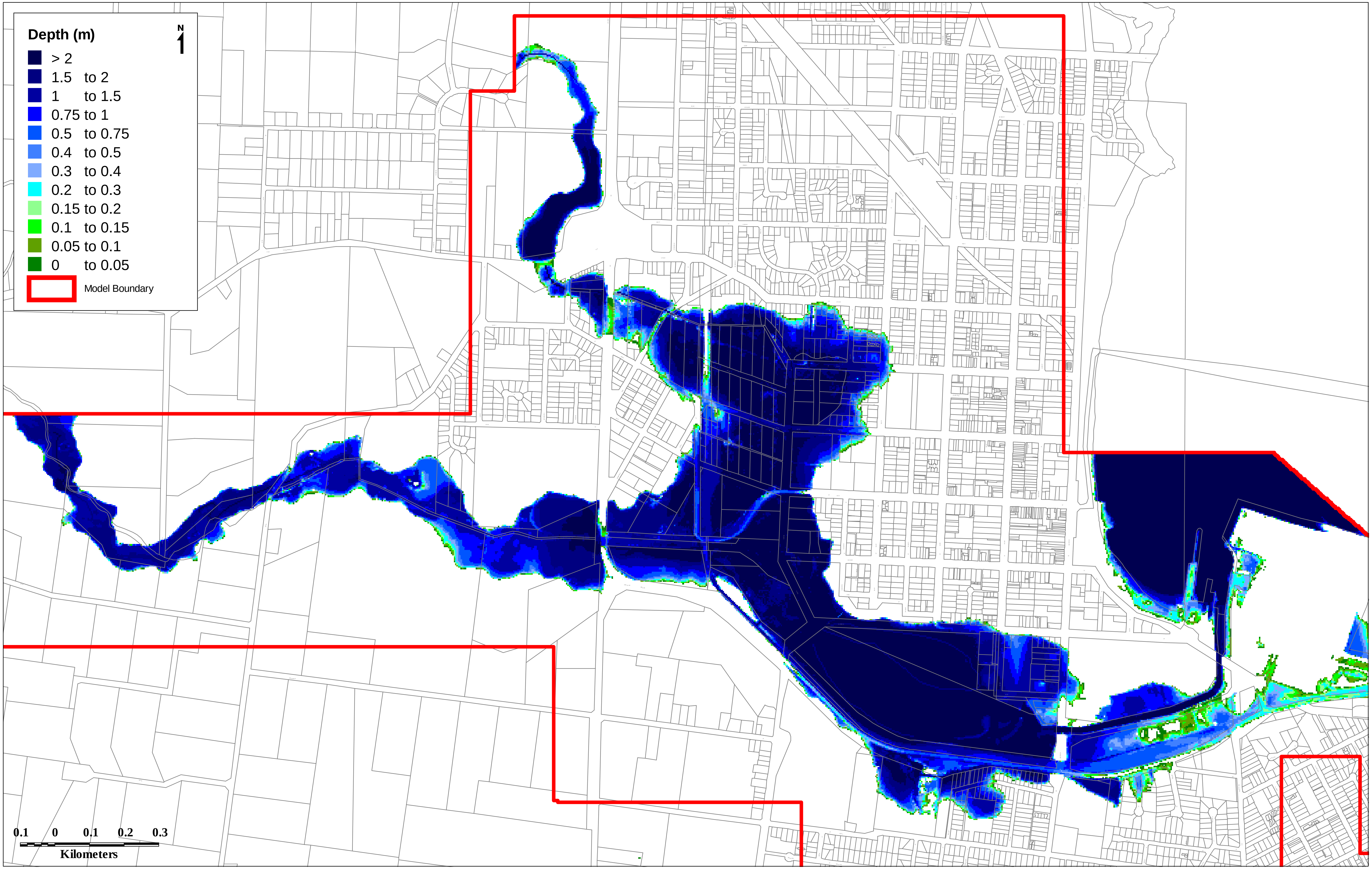


Figure 7.24 - Climate change 100 year ARI storm event depth with the 2100 climate change 100 year ARI tide levels

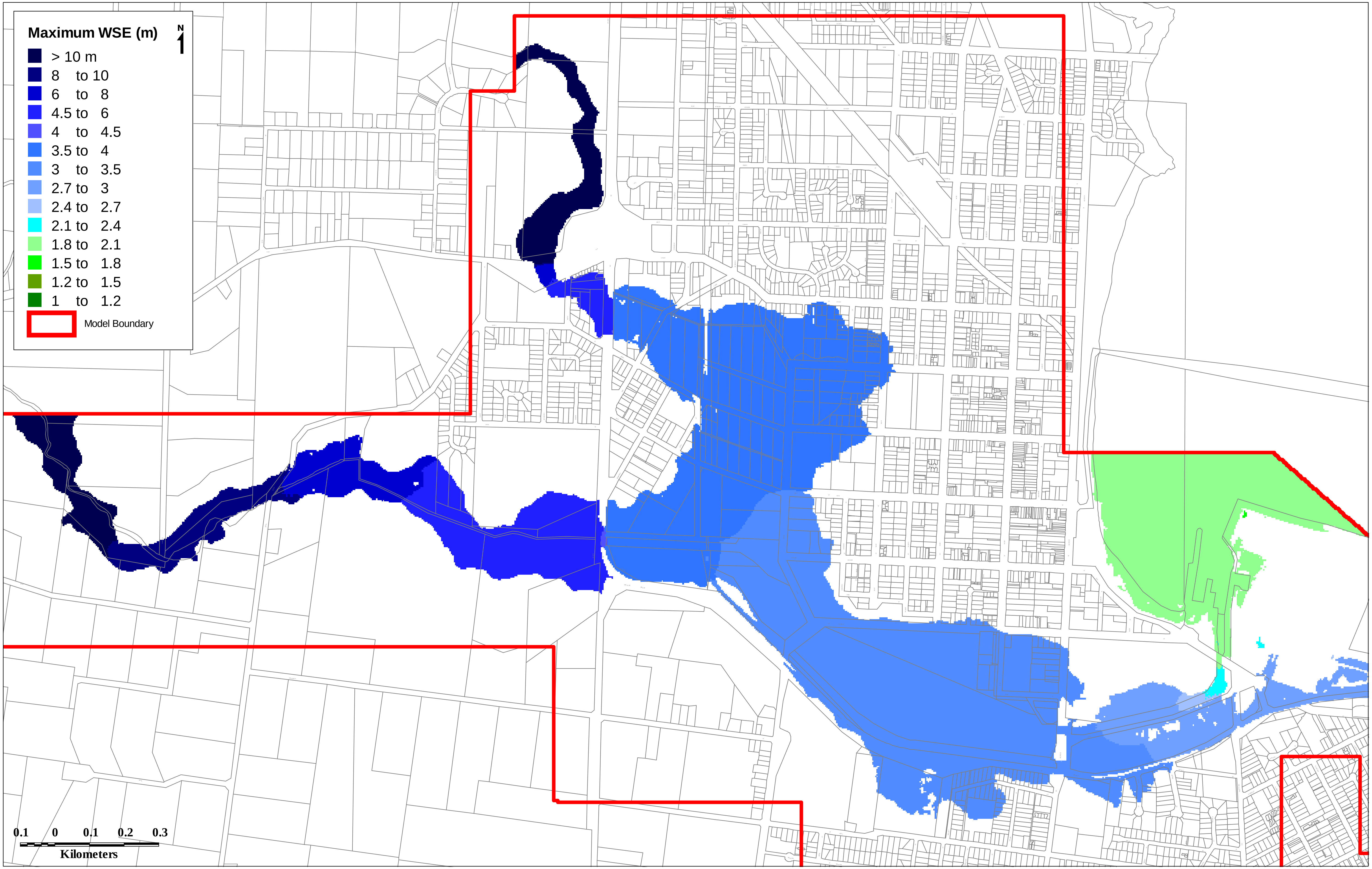


Figure 7.25 - Climate change 100 year ARI storm event water surface elevation with the 2100 climate change 100 year ARI tide levels

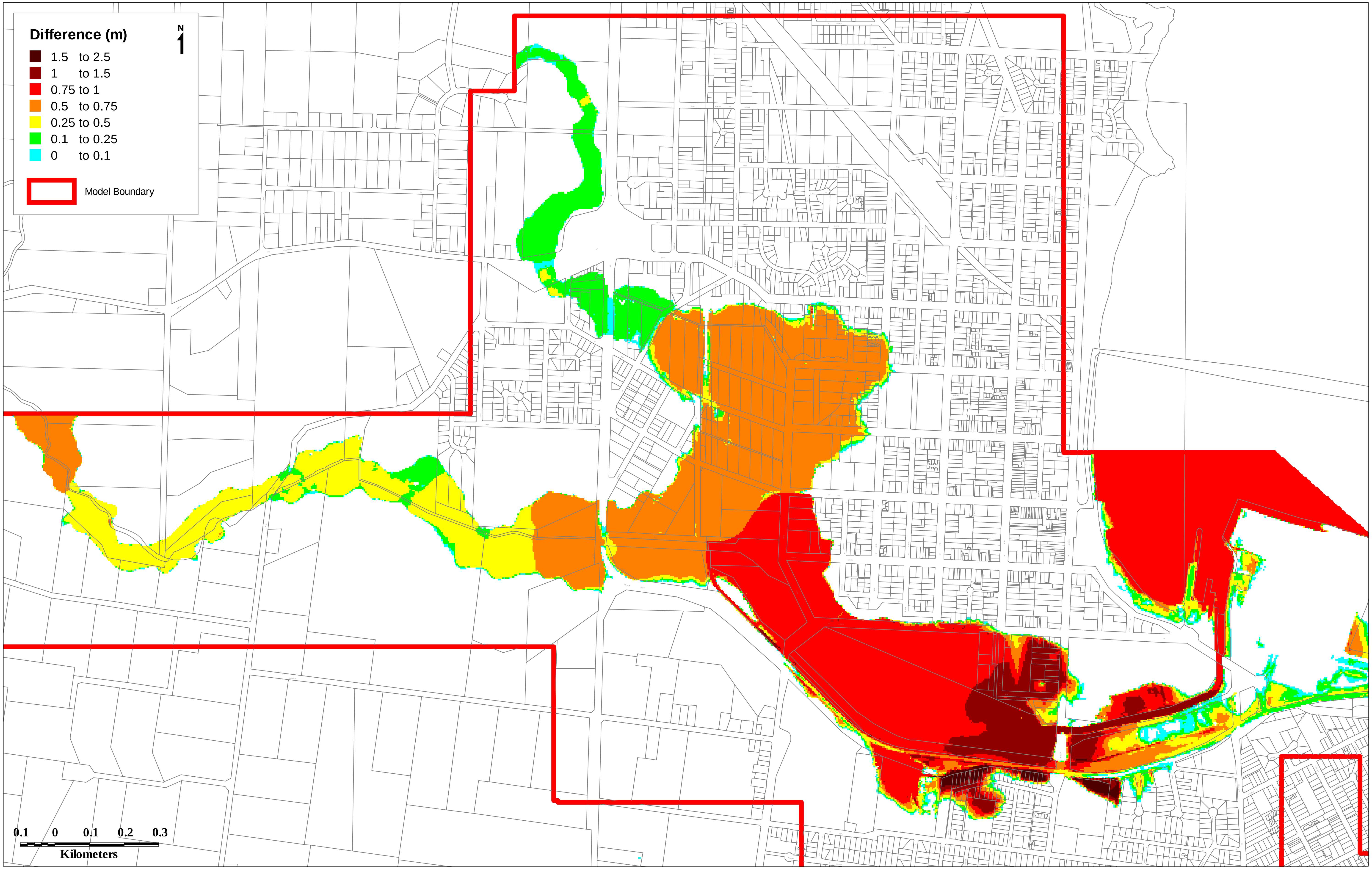


Figure 7.26 - Difference Plot, 100 year ARI climate change event with 100 year ARI climate change sea level minus Existing 100 year ARI with 100 year sea level

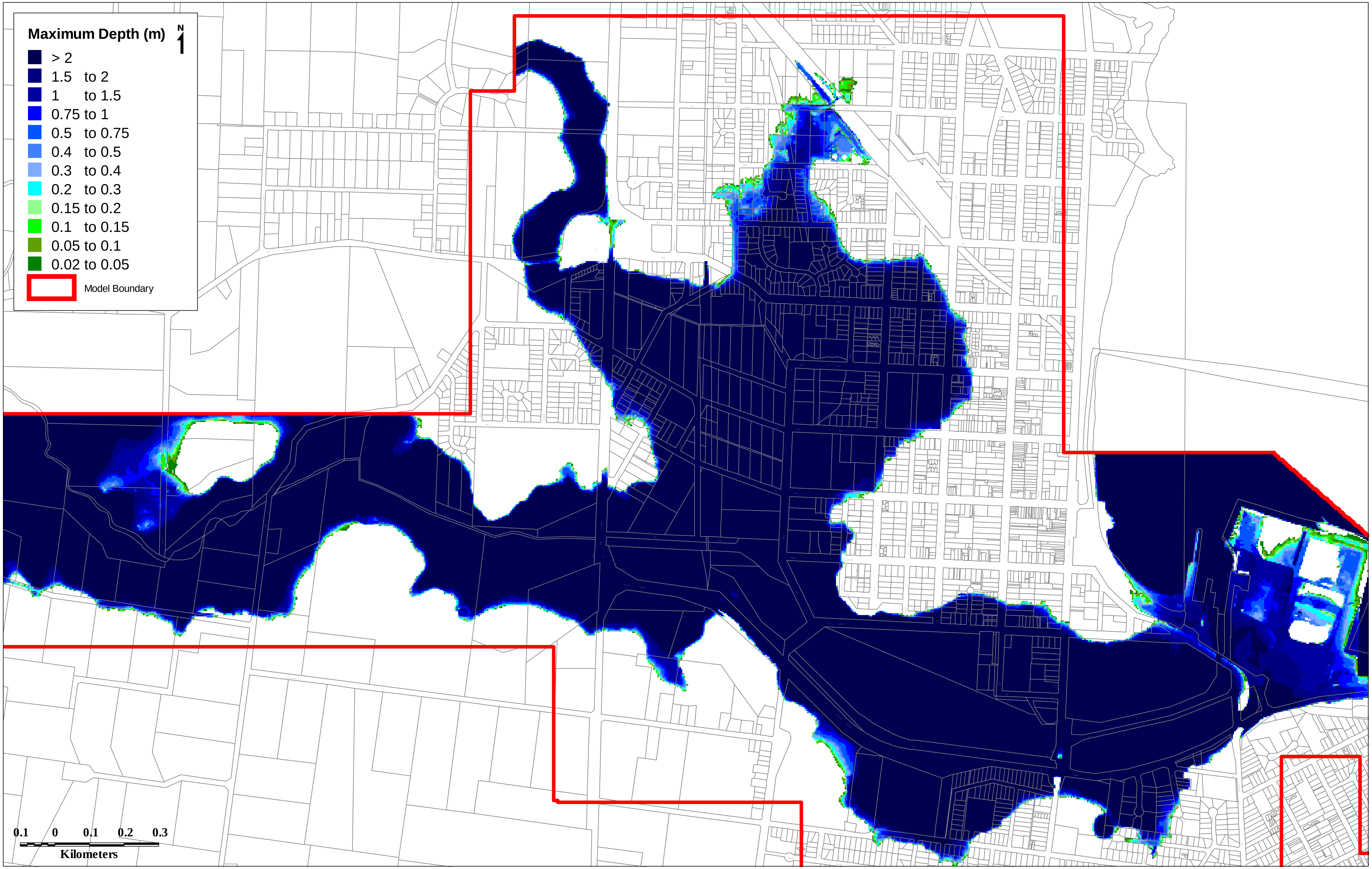


Figure 7.27 - Probable Maximum Flood Depth

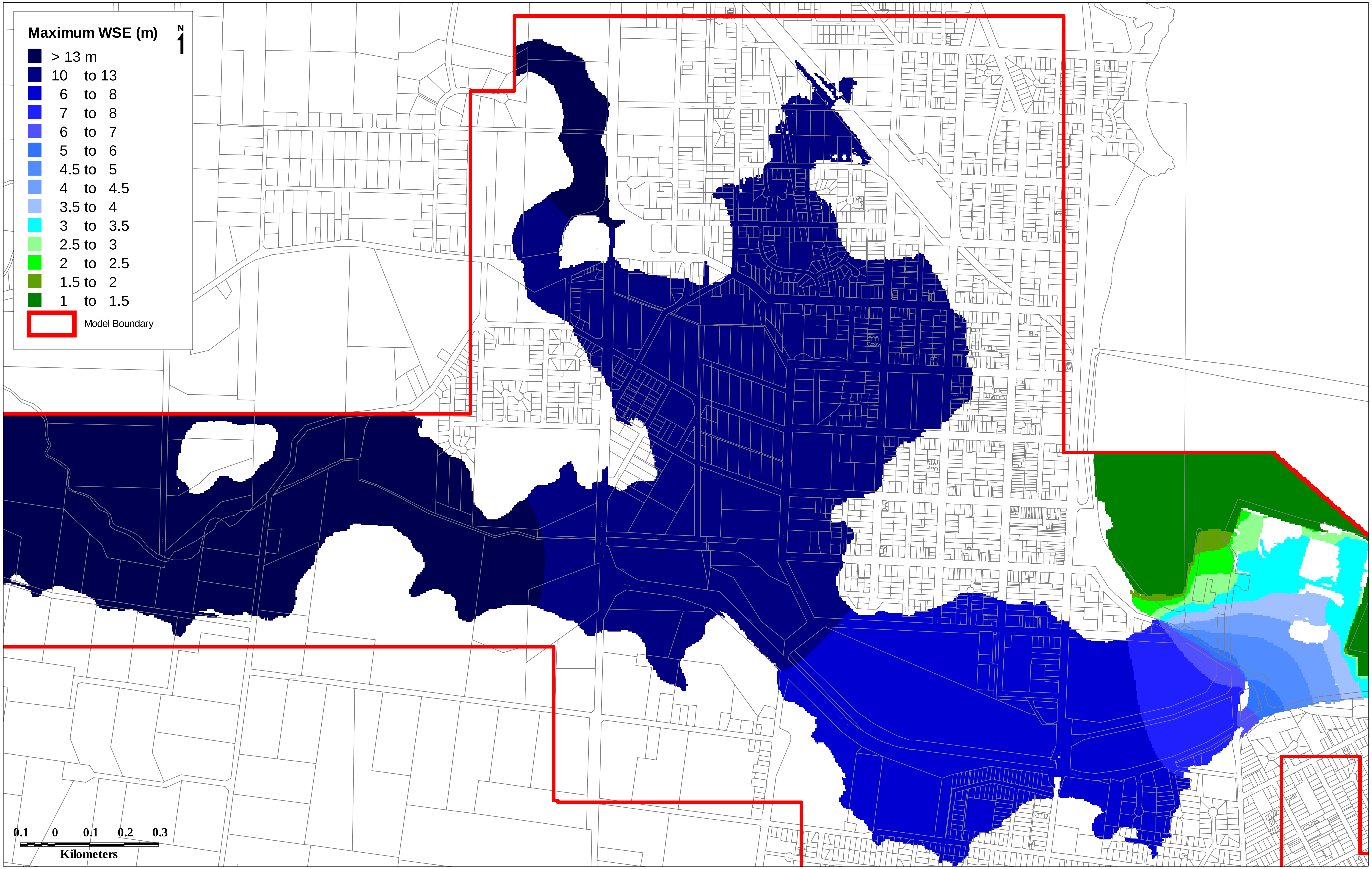


Figure 7.28 - Probable Maximum Flood Water Surface Elevation

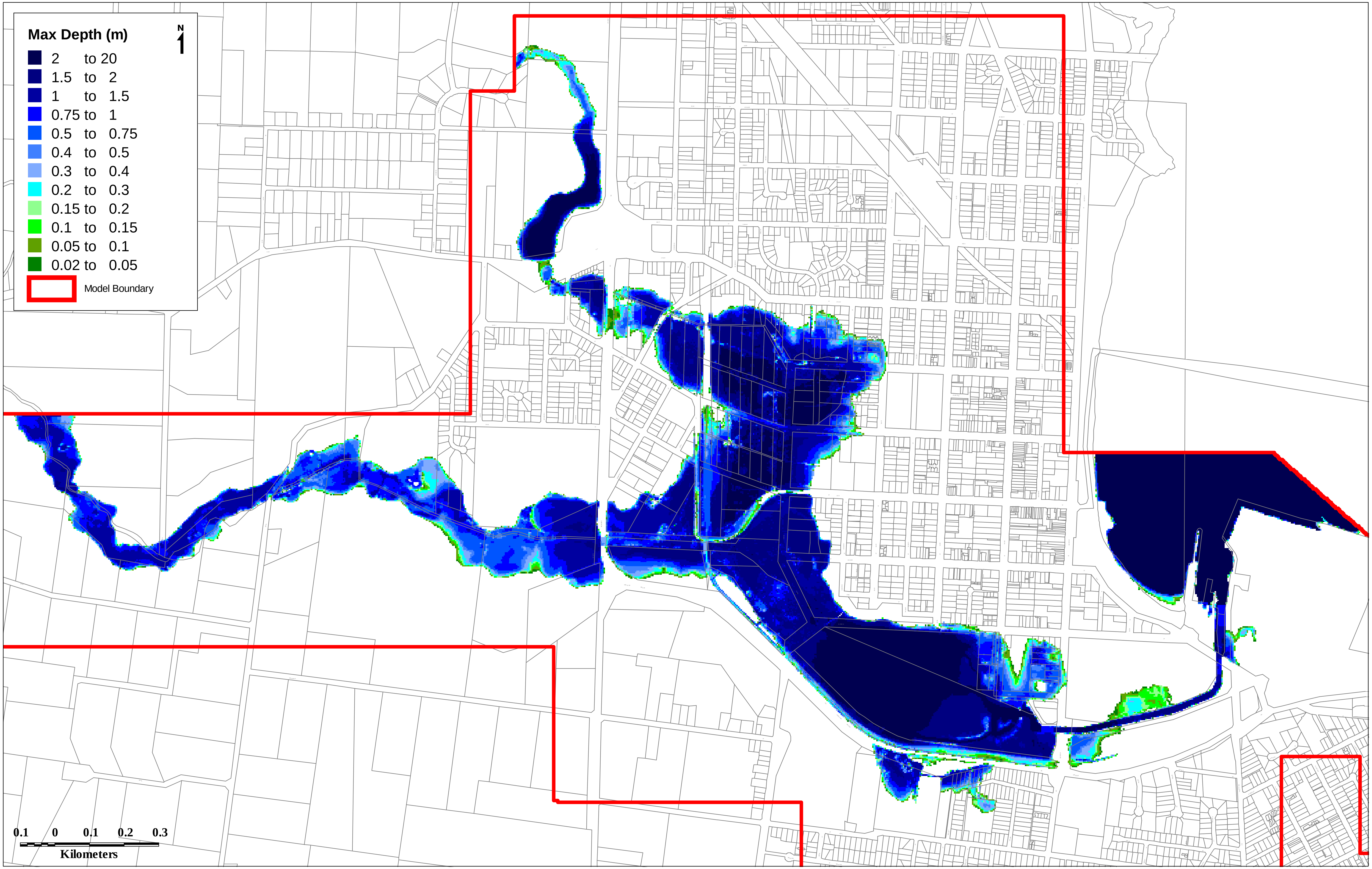


Figure 8.1 - Sensitivity Analysis - 100 Year Storm Event - Existing Conditions with additional 20% flows, 10 Year Current Sea Level

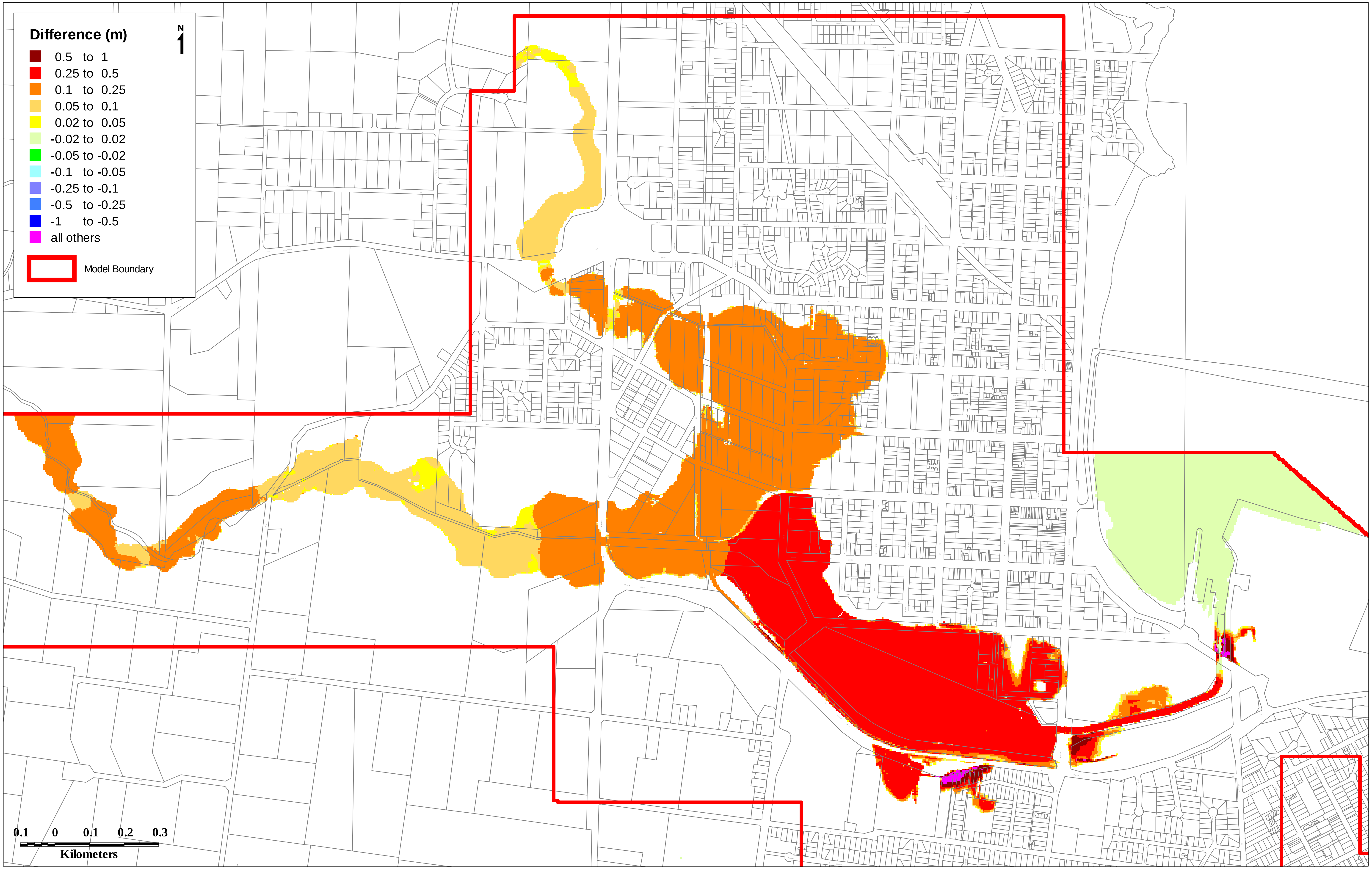


Figure 8.2 - Sensitivity Analysis - 100 Year Storm Event - 100 year plus 20% flows less Existing 100 year event

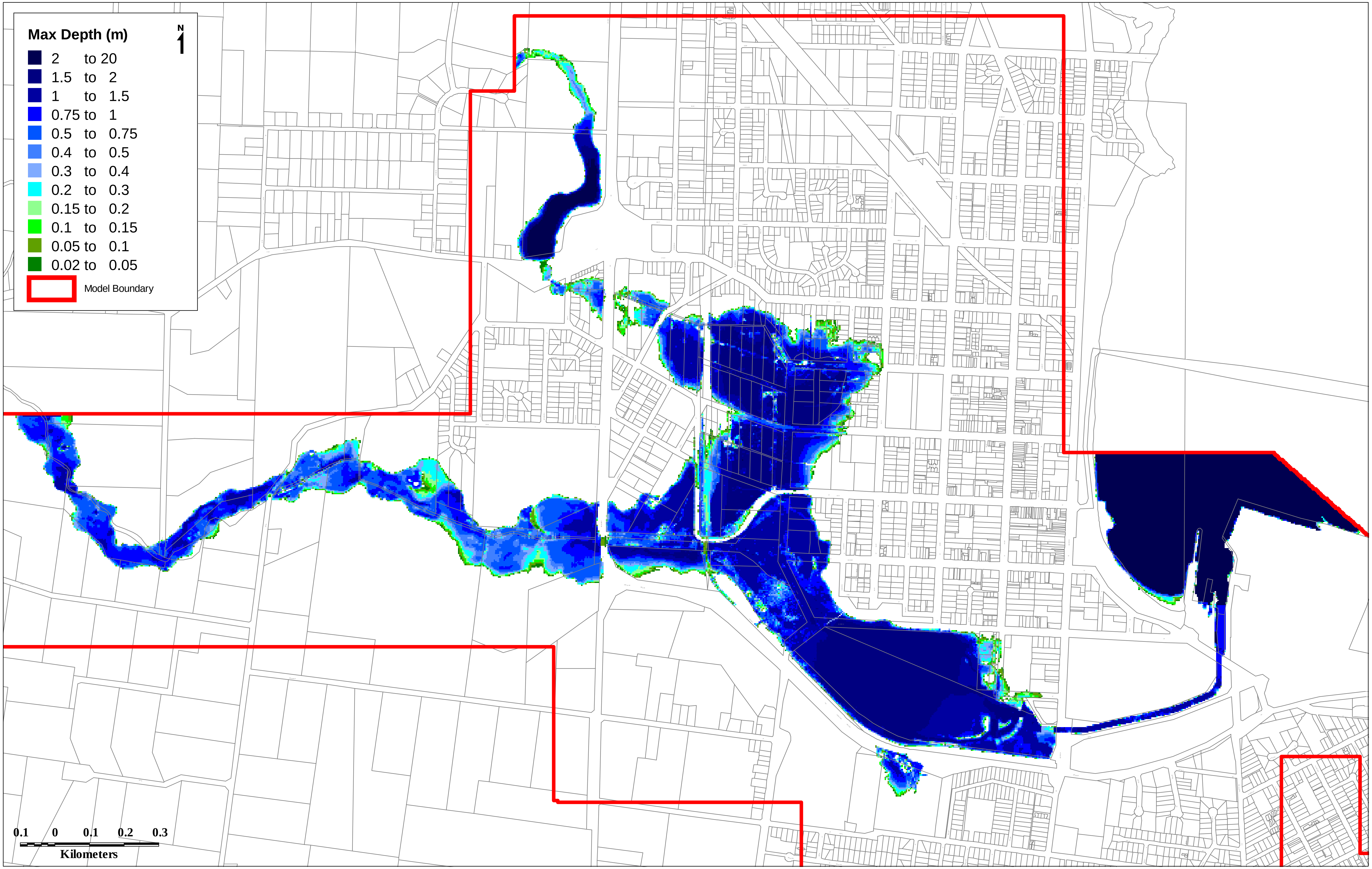


Figure 8.3 - Sensitivity Analysis - 100 Year Storm Event - Existing Conditions with 20% less flows, 10 Year Current Sea Level

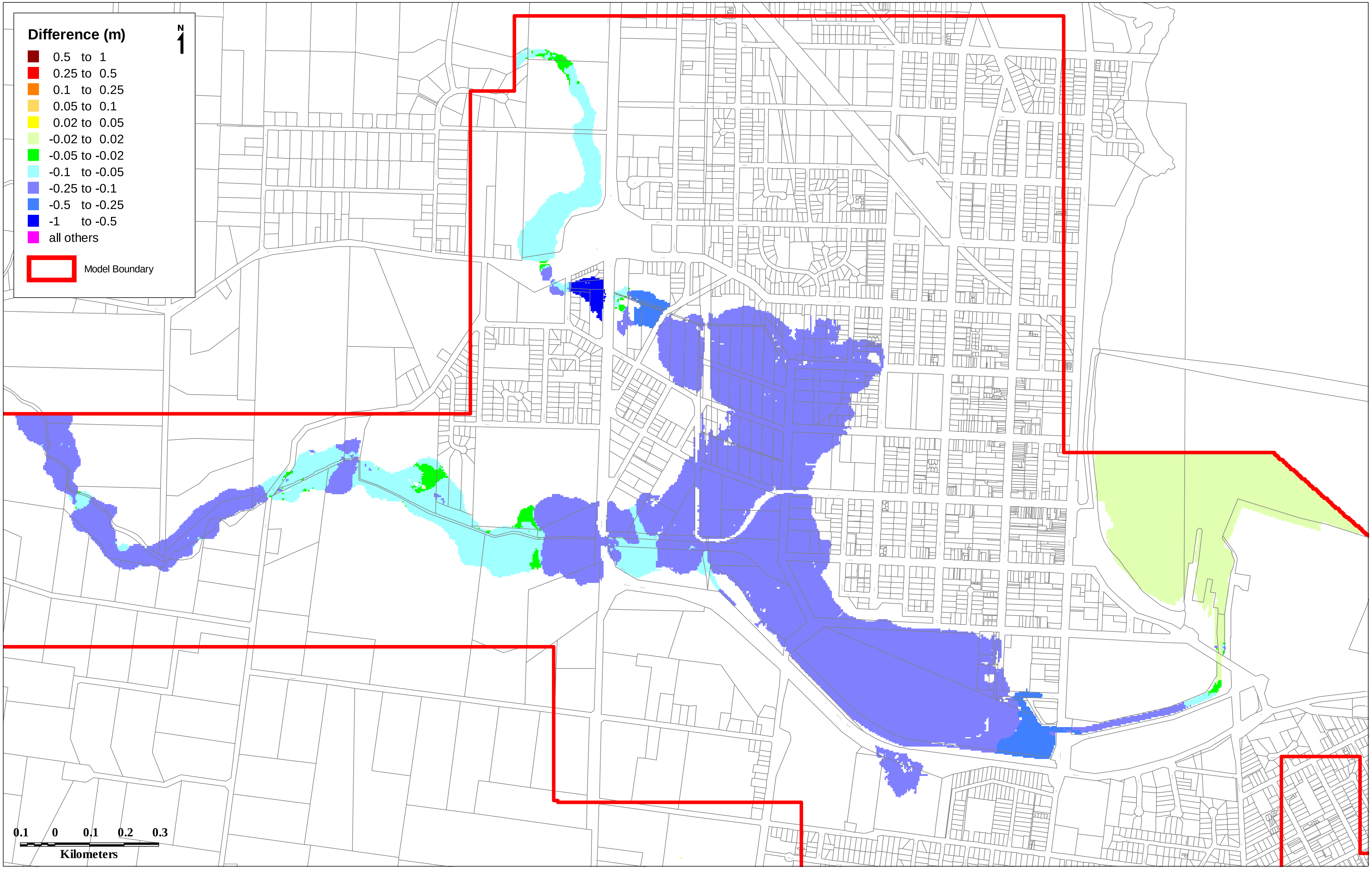


Figure 8.4 - Sensitivity Analysis - 100 Year Storm Event - 100 year with 20% less flows minus Existing 100 year event

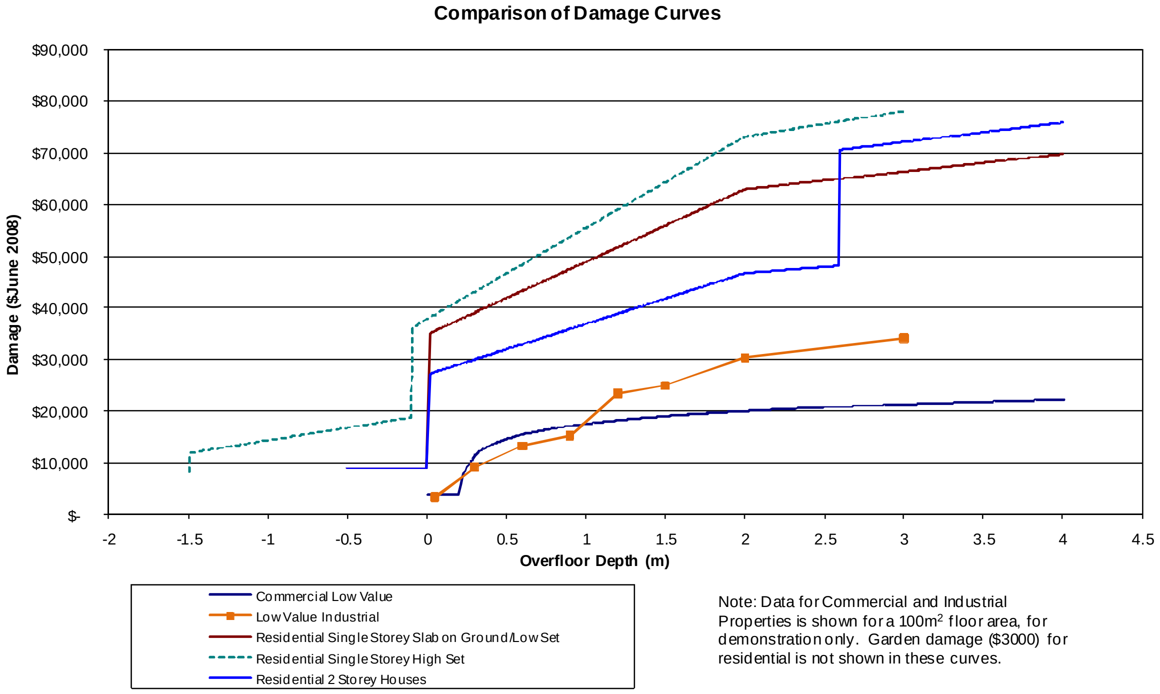


Figure 9.1 – Damage curves applied for the Annual Average Damages

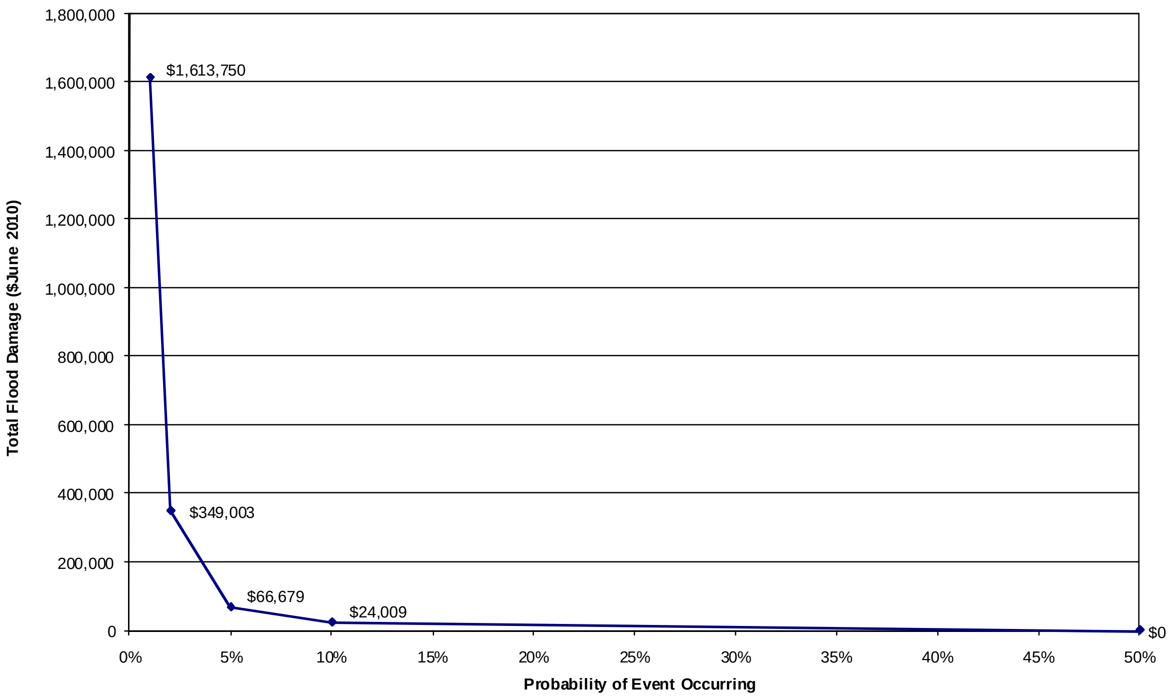


Figure 9.2 – Annual Average Damages for Portland

Appendix A

Photos of the Study Area

Figure A1 – Downstream of Finn St looking South

Figure A2 – Downstream of Finn Street looking South #2

Figure A3 – Floodplain from Finn Street

Figure A4 – Floodplain from Finn Street


Figure A5 – Floodplain from Finn Street


Figure A6 – Downstream side of Finn Street

Figure A7 – Looking downstream from Finn Street culverts

Figure A8 – Downstream side of Finn Street culverts

Figure A9 – Railway crossing downstream of Finn Street on Finn Street Creek

Figure A10 – Railway crossing upstream of Fawthrop Lagoon

Figure A11 – From the railway crossing upstream of Fawthrop Lagoon

Figure A12 - From the railway crossing upstream of Fawthrop Lagoon

Figure A13 – Pedestrian bridges at Fawthrop Lagoon

Figure A14 – Pedestrian bridges at Fawthrop Lagoon

Appendix B

Received Ground Survey Examples

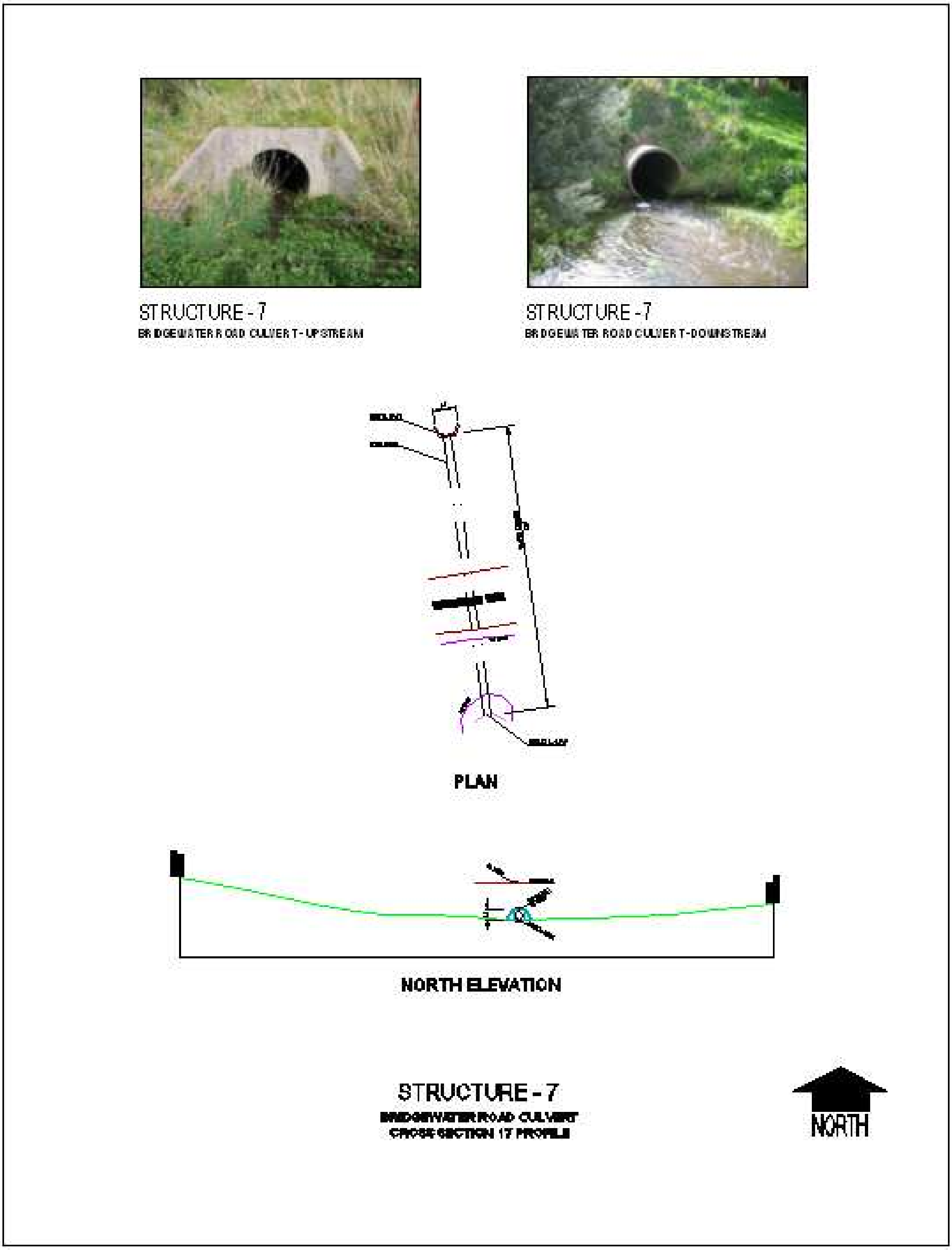


Figure B1 - Structure Survey Example

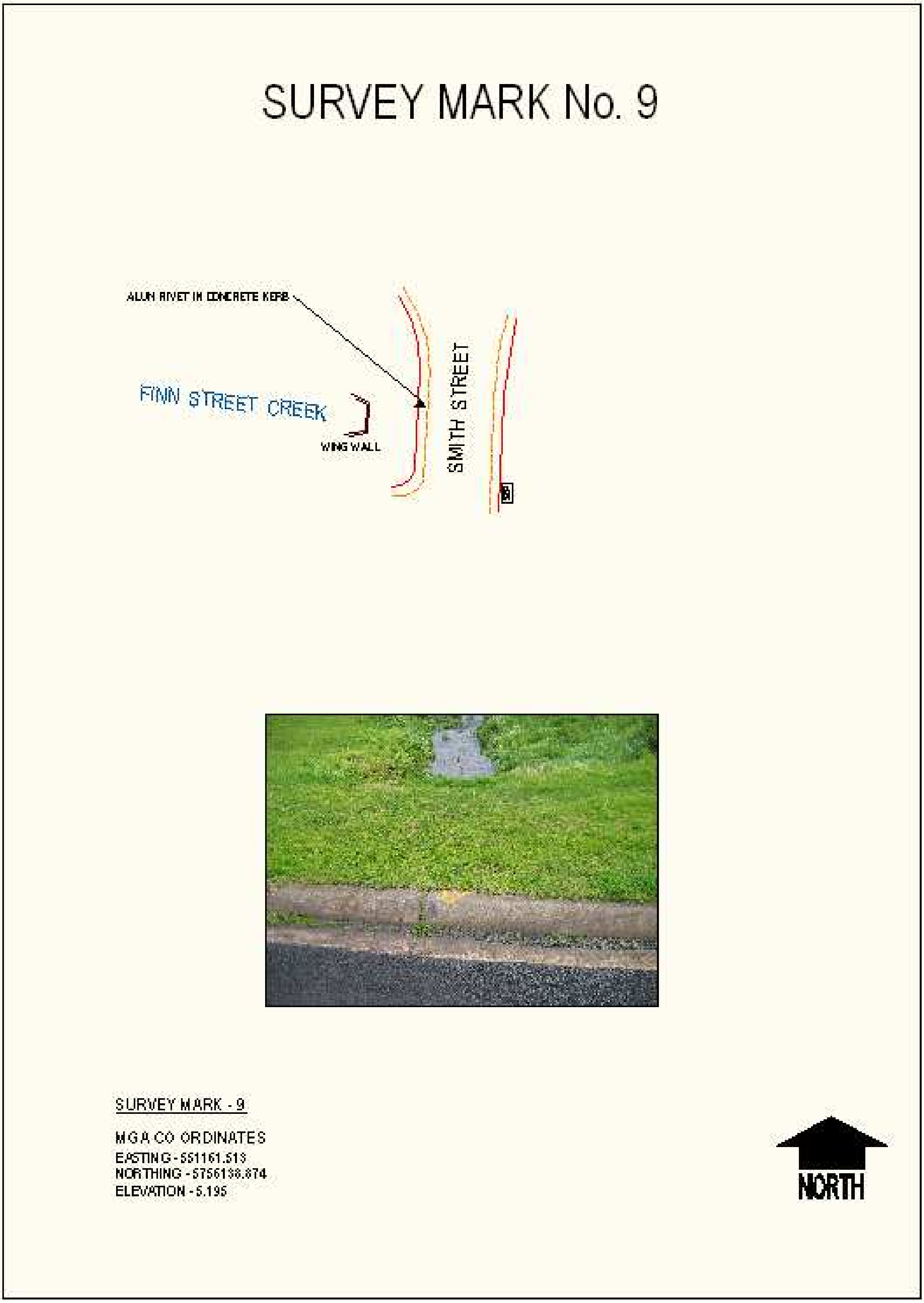
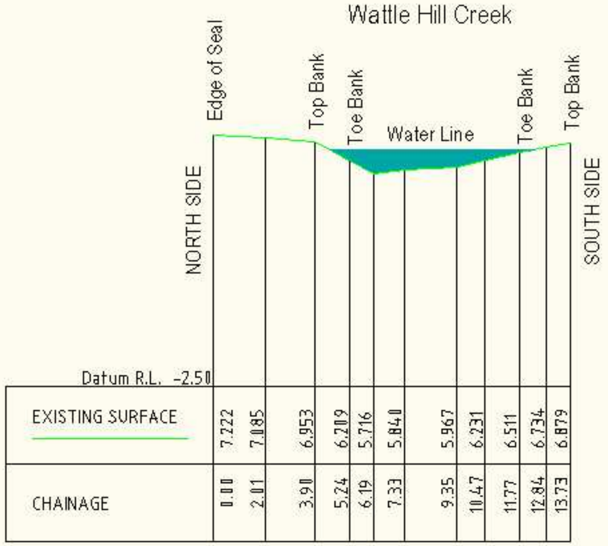


Figure B2 – Survey Mark Example



CROSS SECTION XS_5

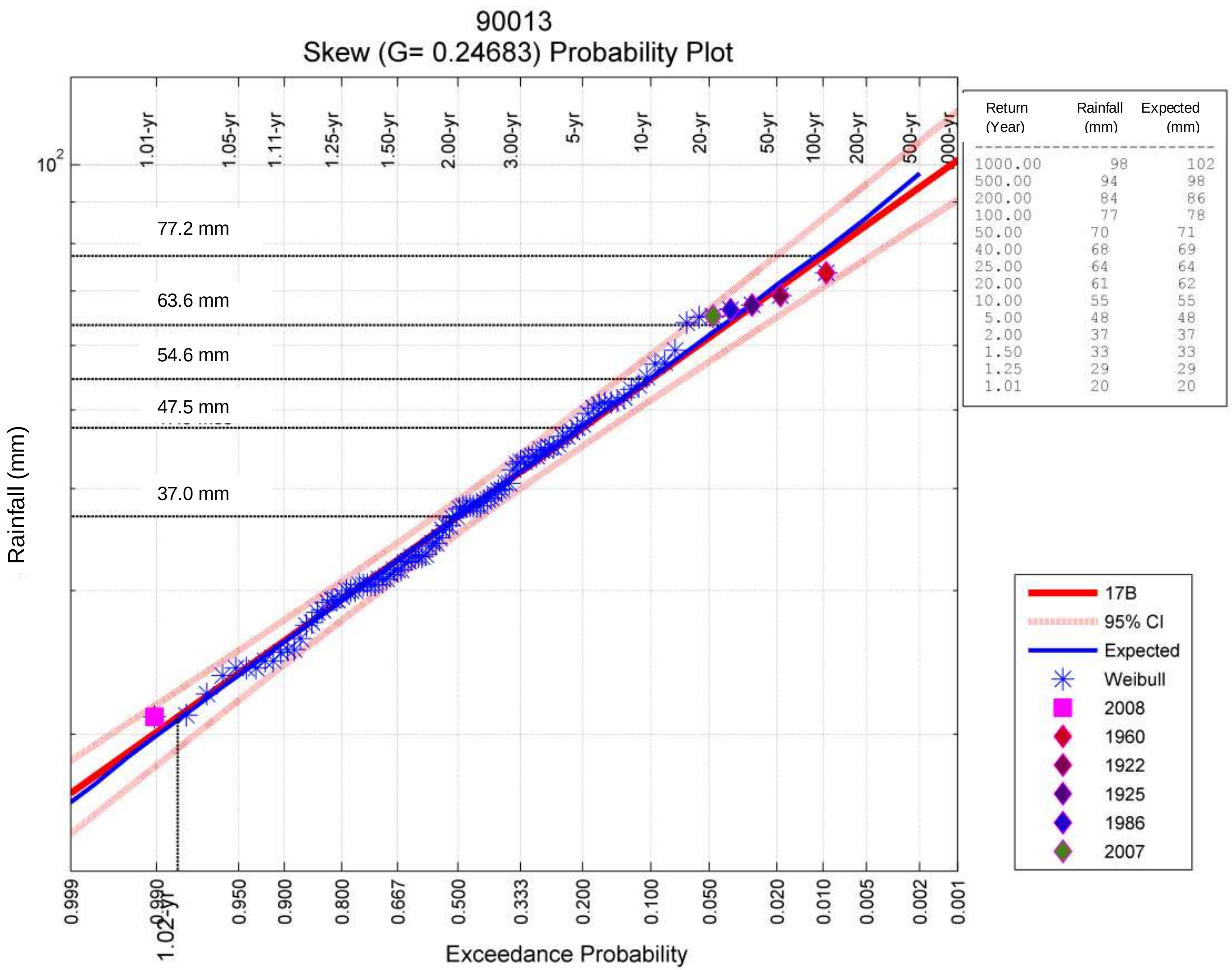
Scale Horizontal 1:200 Vertical 1:200

XS-5 DOWNSTREAM

Figure B3 – Cross Section Example

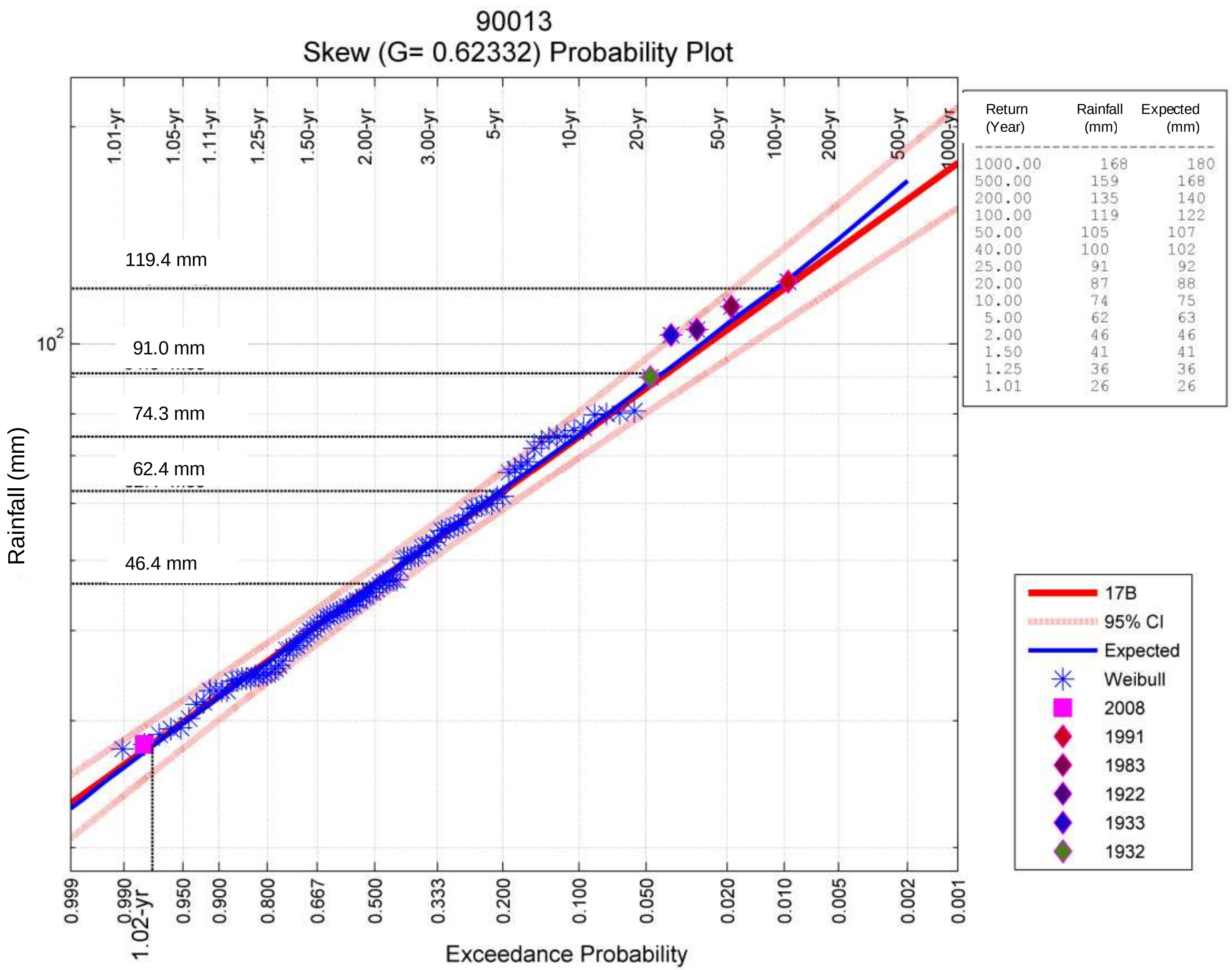
Appendix C

Rainfall Frequency Analysis

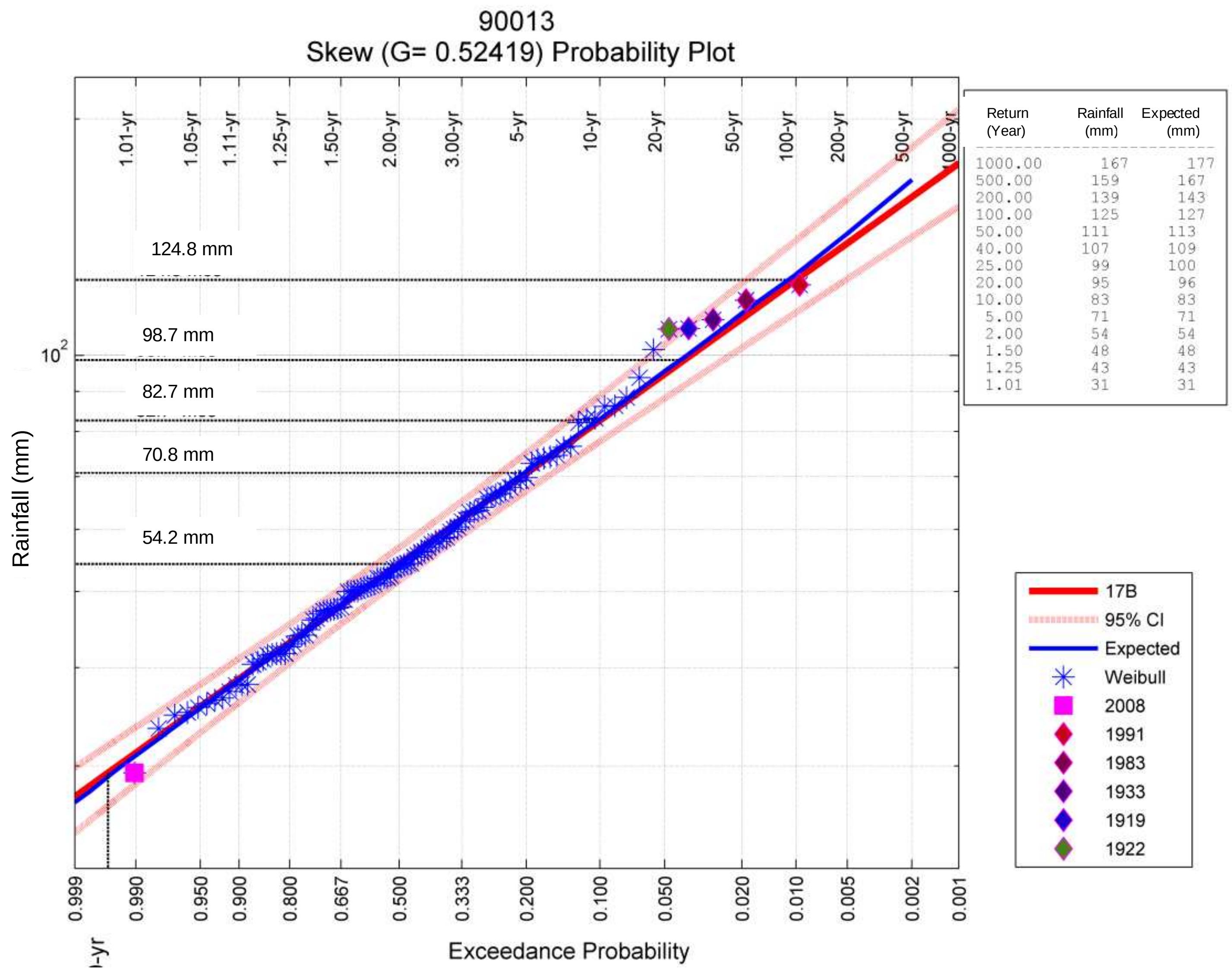


Return (Year)	Rainfall (mm)	Expected (mm)
1000.00	98	102
500.00	94	98
200.00	84	86
100.00	77	78
50.00	70	71
40.00	68	69
25.00	64	64
20.00	61	62
10.00	55	55
5.00	48	48
2.00	37	37
1.50	33	33
1.25	29	29
1.01	20	20

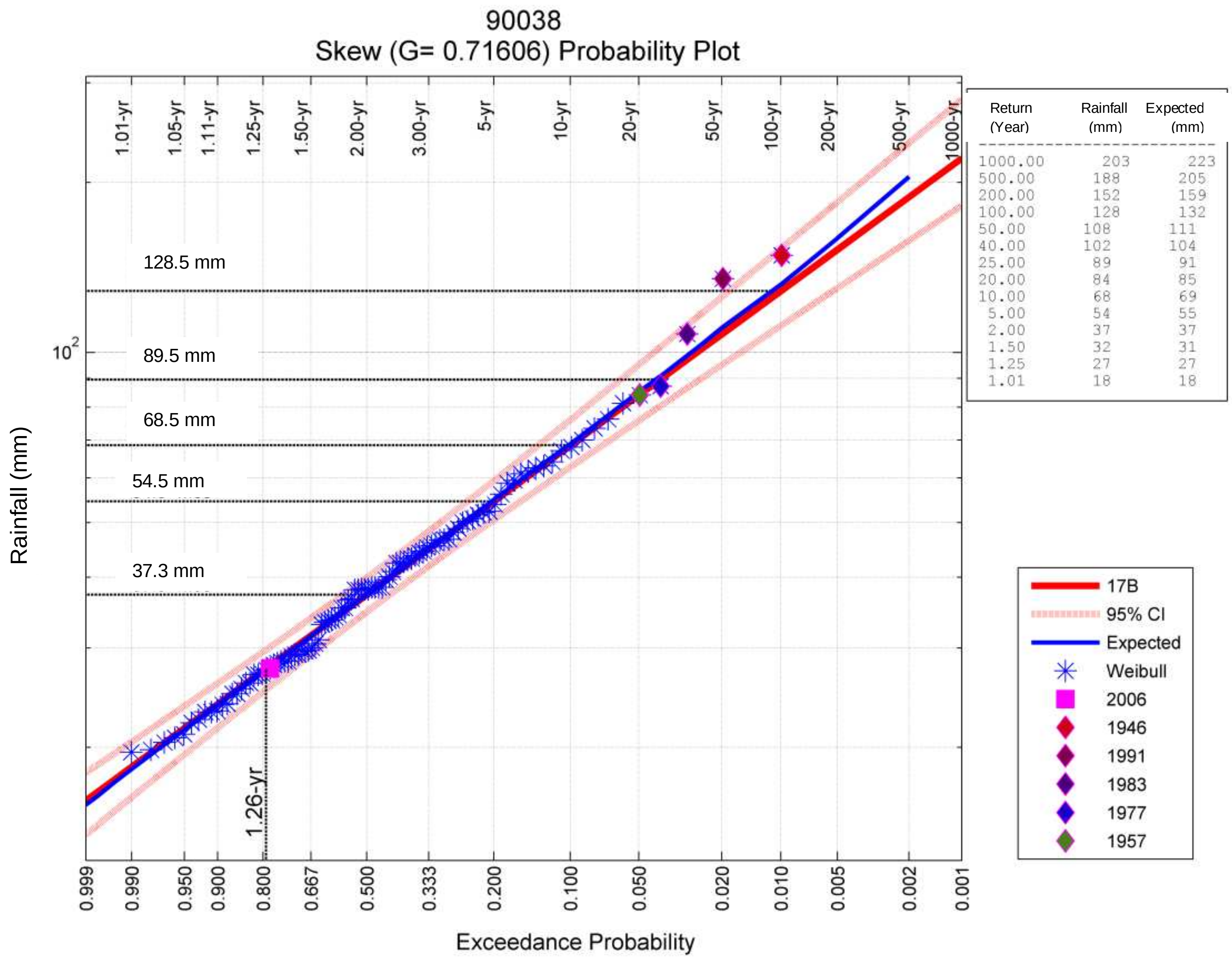
Plot C1 – Rainfall Frequency Analysis for Station 90013 – 24 hr Rainfall Event Duration



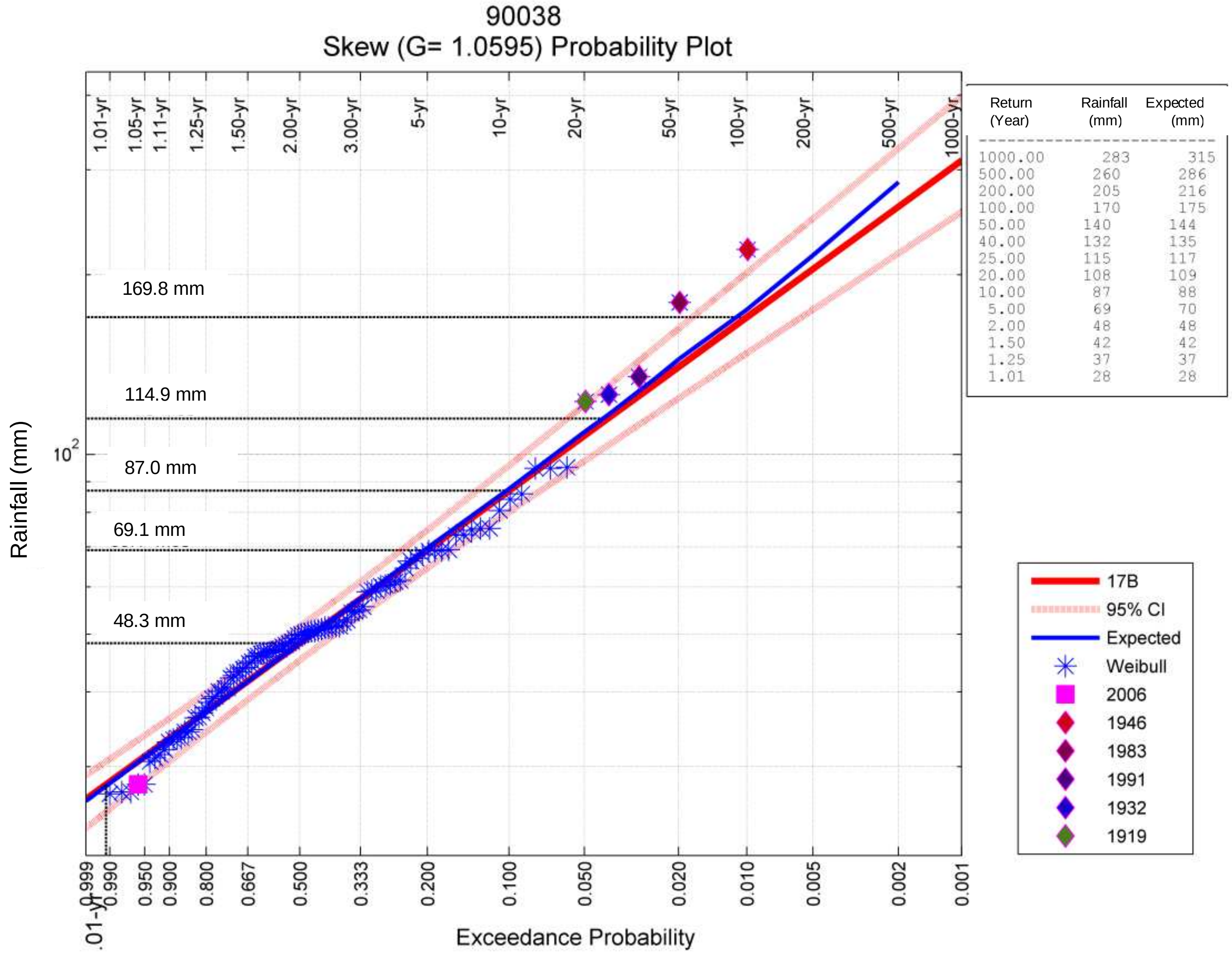
Plot C2 – Rainfall Frequency Analysis for Station 90013 – 48 hr Rainfall Event Duration



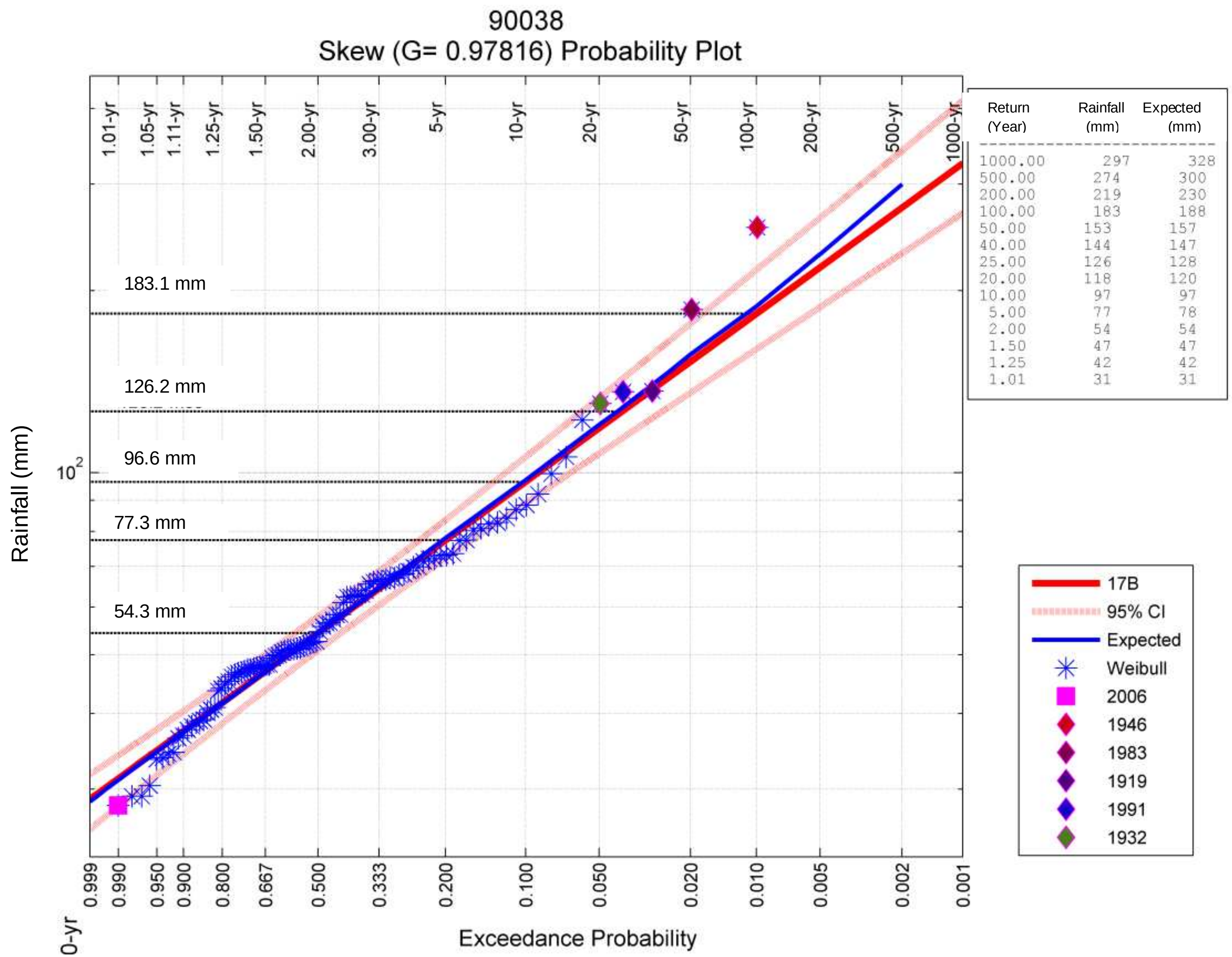
Plot C3 – Rainfall Frequency Analysis for Station 90013 – 72 hr Rainfall Event Duration



Plot C4 – Rainfall Frequency Analysis for Station 90038 – 24 hr Rainfall Event Duration

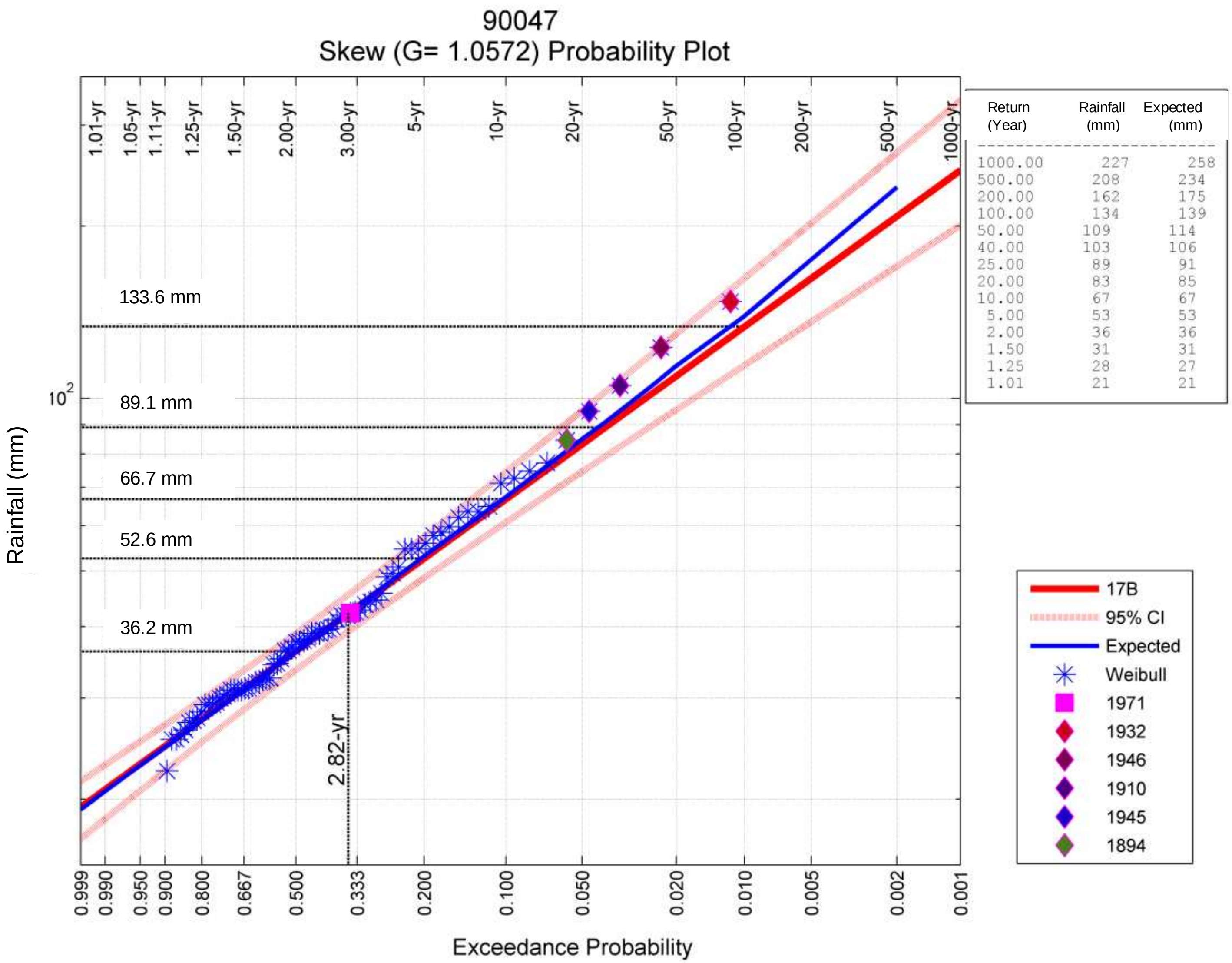


Plot C5 – Rainfall Frequency Analysis for Station 90038 – 48 hr Rainfall Event Duration



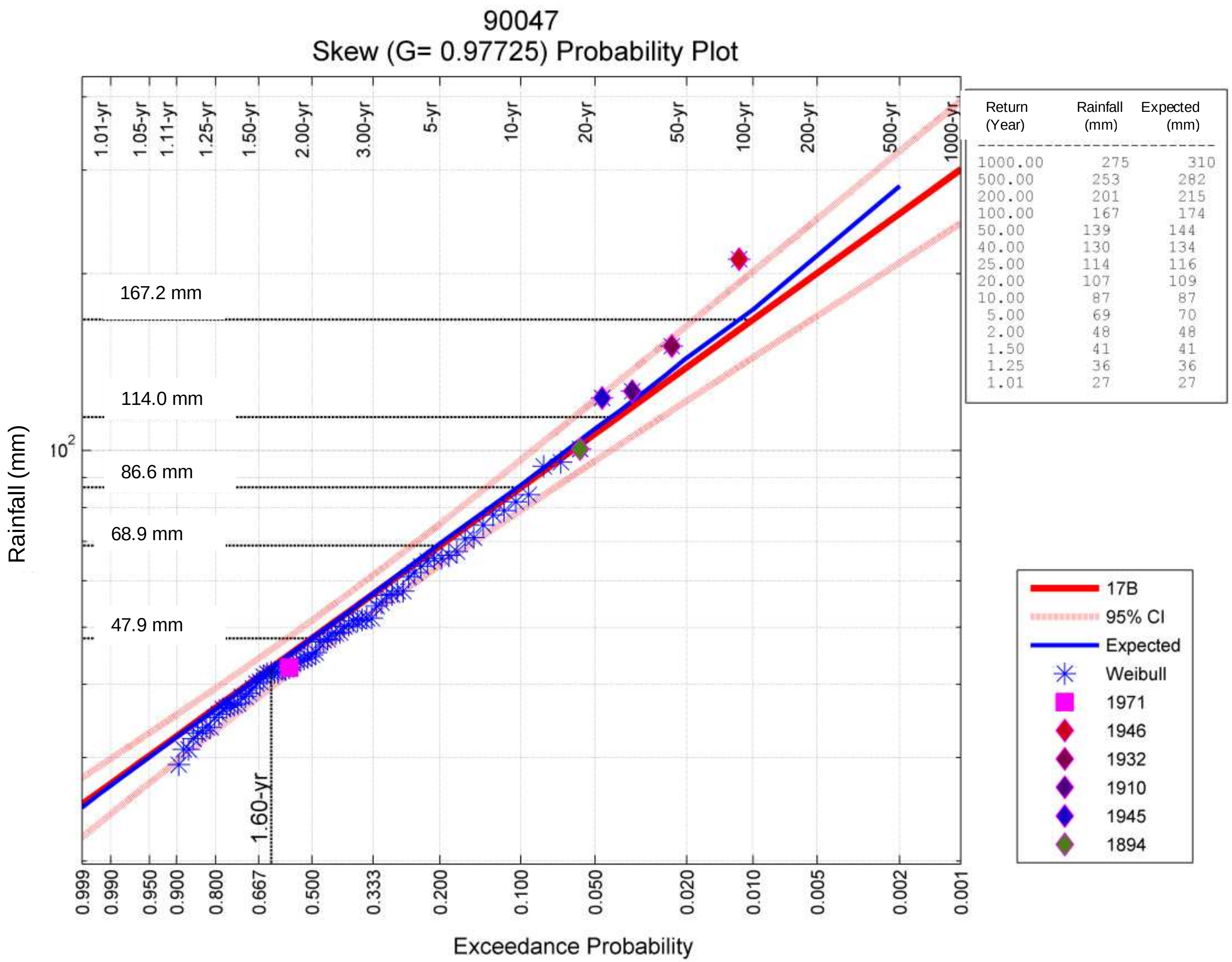
Return (Year)	Rainfall (mm)	Expected (mm)
1000.00	297	328
500.00	274	300
200.00	219	230
100.00	183	188
50.00	153	157
40.00	144	147
25.00	126	128
20.00	118	120
10.00	97	97
5.00	77	78
2.00	54	54
1.50	47	47
1.25	42	42
1.01	31	31

Plot C6 – Rainfall Frequency Analysis for Station 90038 – 72 hr Rainfall Event Duration

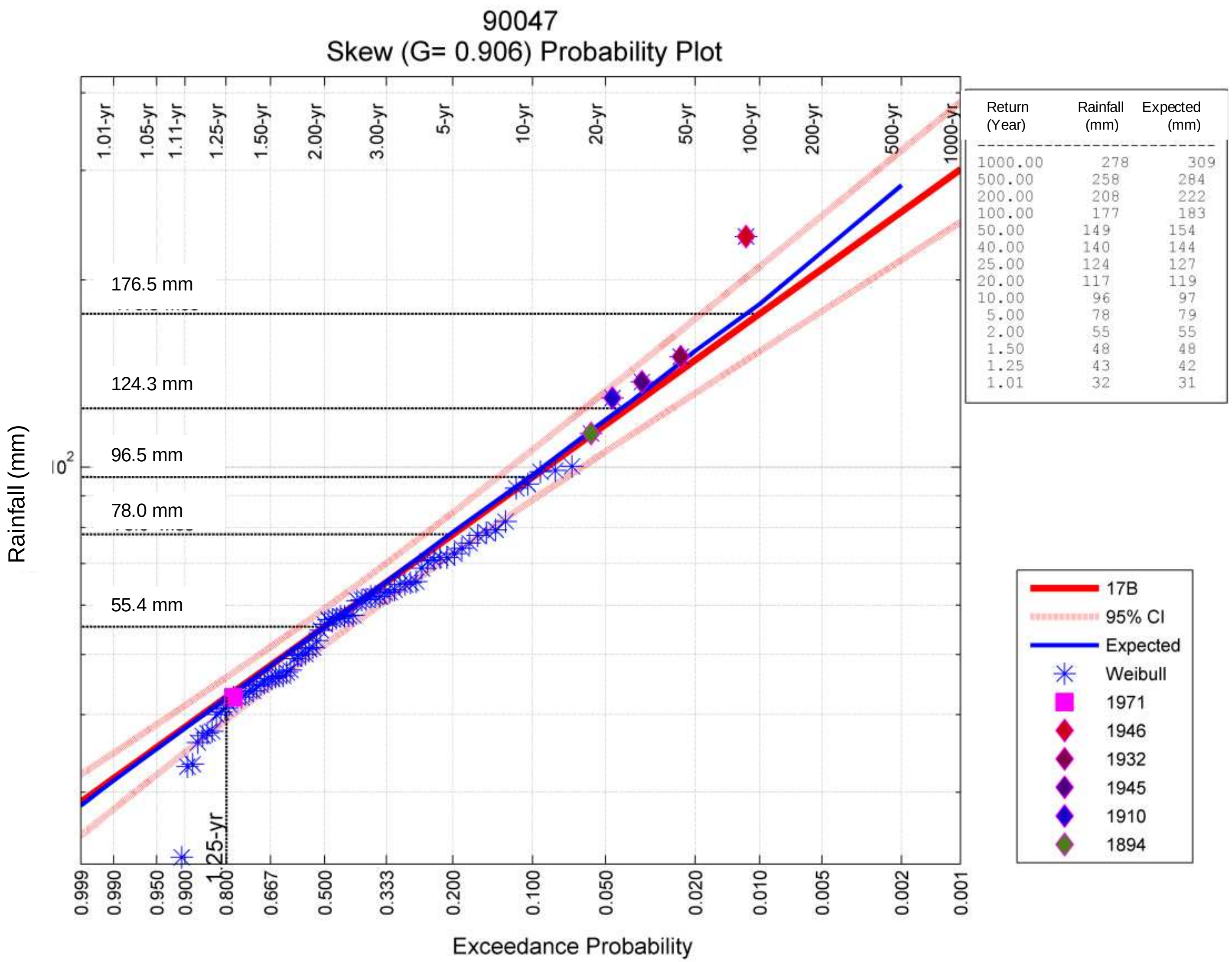


Return (Year)	Rainfall (mm)	Expected (mm)
1000.00	227	258
500.00	208	234
200.00	162	175
100.00	134	139
50.00	109	114
40.00	103	106
25.00	89	91
20.00	83	85
10.00	67	67
5.00	53	53
2.00	36	36
1.50	31	31
1.25	28	27
1.01	21	21

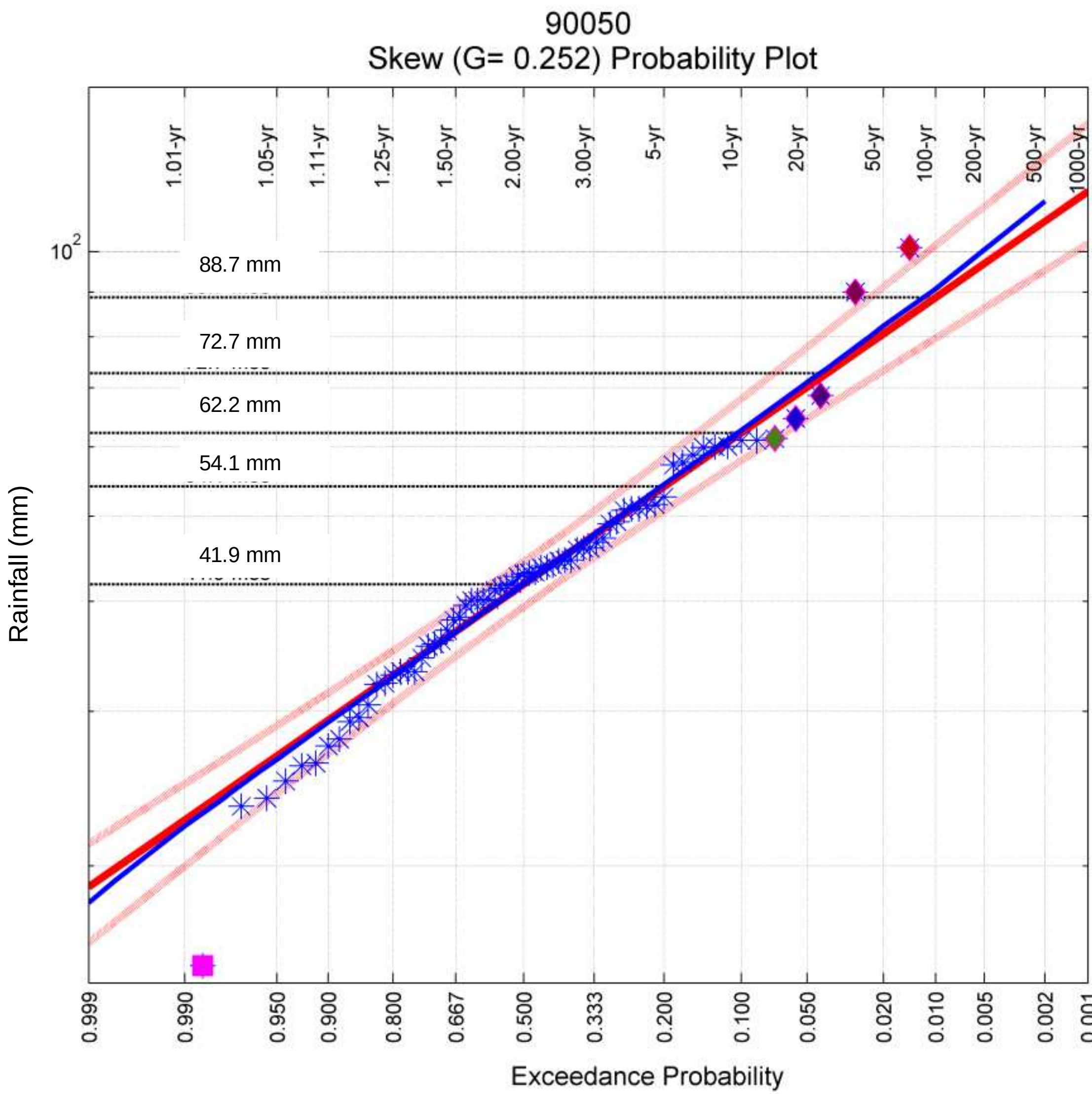
Plot C7 – Rainfall Frequency Analysis for Station 90047 – 24 hr Rainfall Event Duration



Plot C8 – Rainfall Frequency Analysis for Station 90047 – 48 hr Rainfall Event Duration

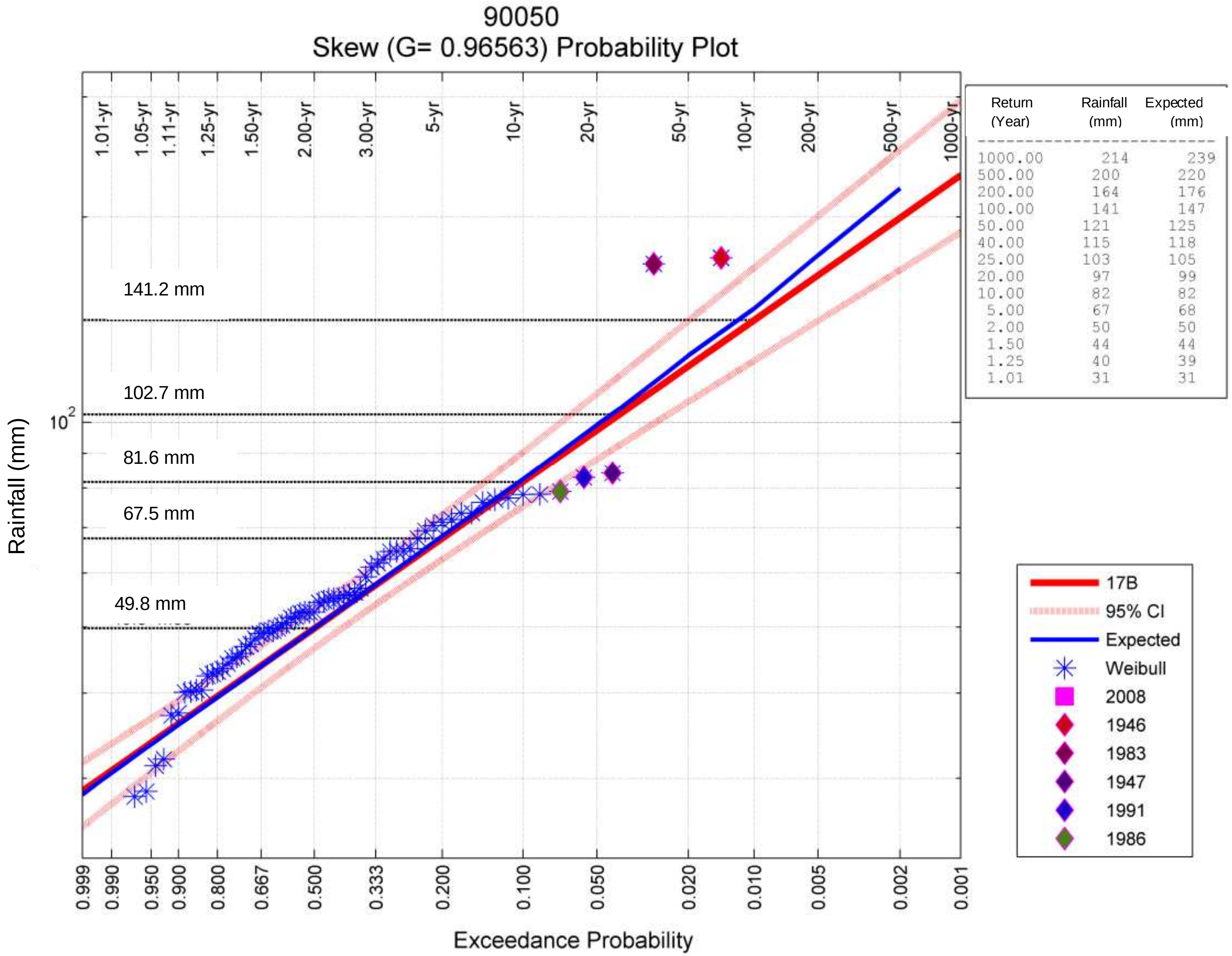


Plot C9 – Rainfall Frequency Analysis for Station 90047 – 72 hr Rainfall Event Duration



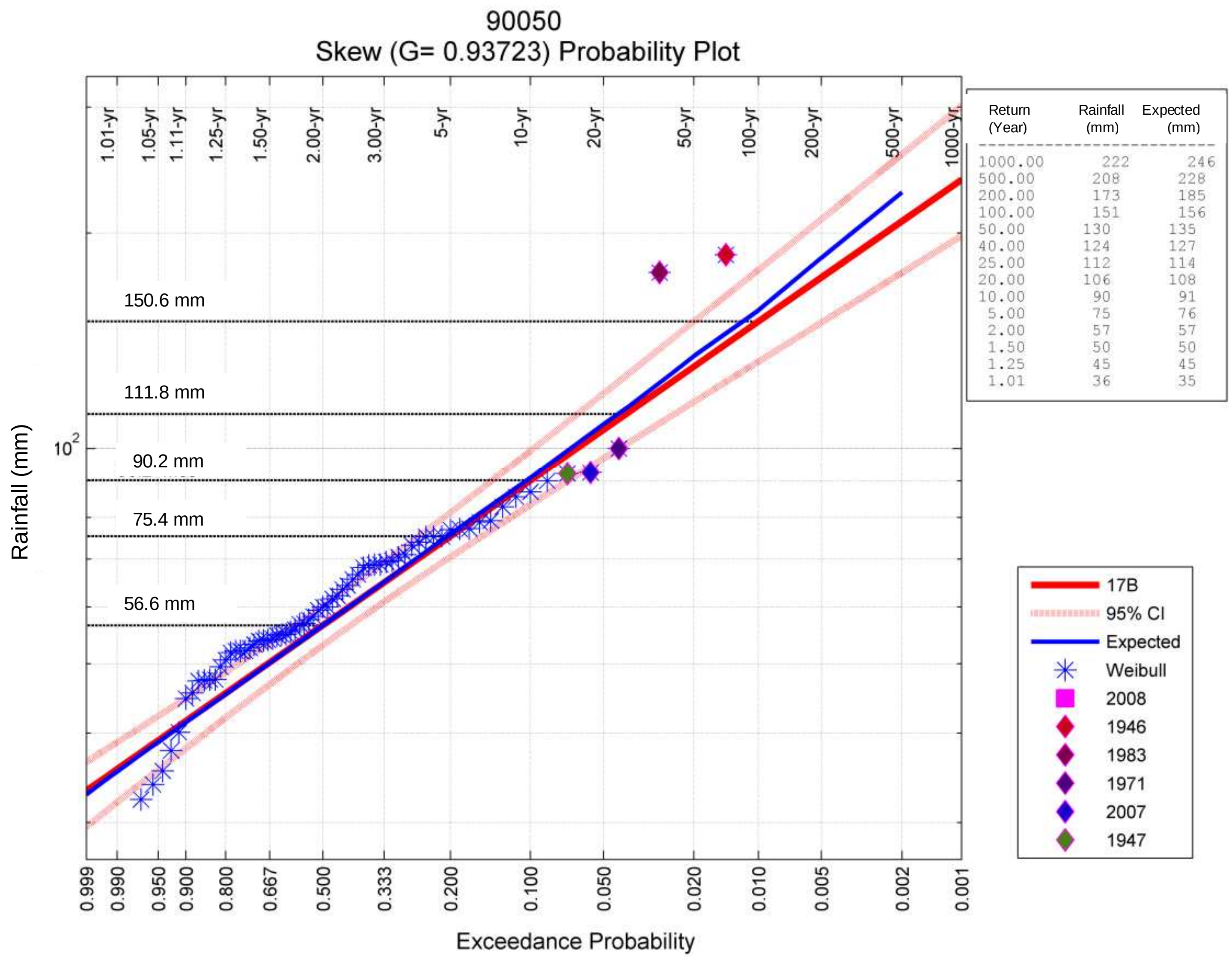
Return (Year)	Rainfall (mm)	Expected (mm)
1000.00	113	120
500.00	108	114
200.00	97	101
100.00	89	91
50.00	81	82
40.00	78	79
25.00	73	74
20.00	70	71
10.00	62	63
5.00	54	54
2.00	42	42
1.50	37	37
1.25	33	33
1.01	23	22

Plot C10 – Rainfall Frequency Analysis for Station 90050 – 24 hr Rainfall Event Duration



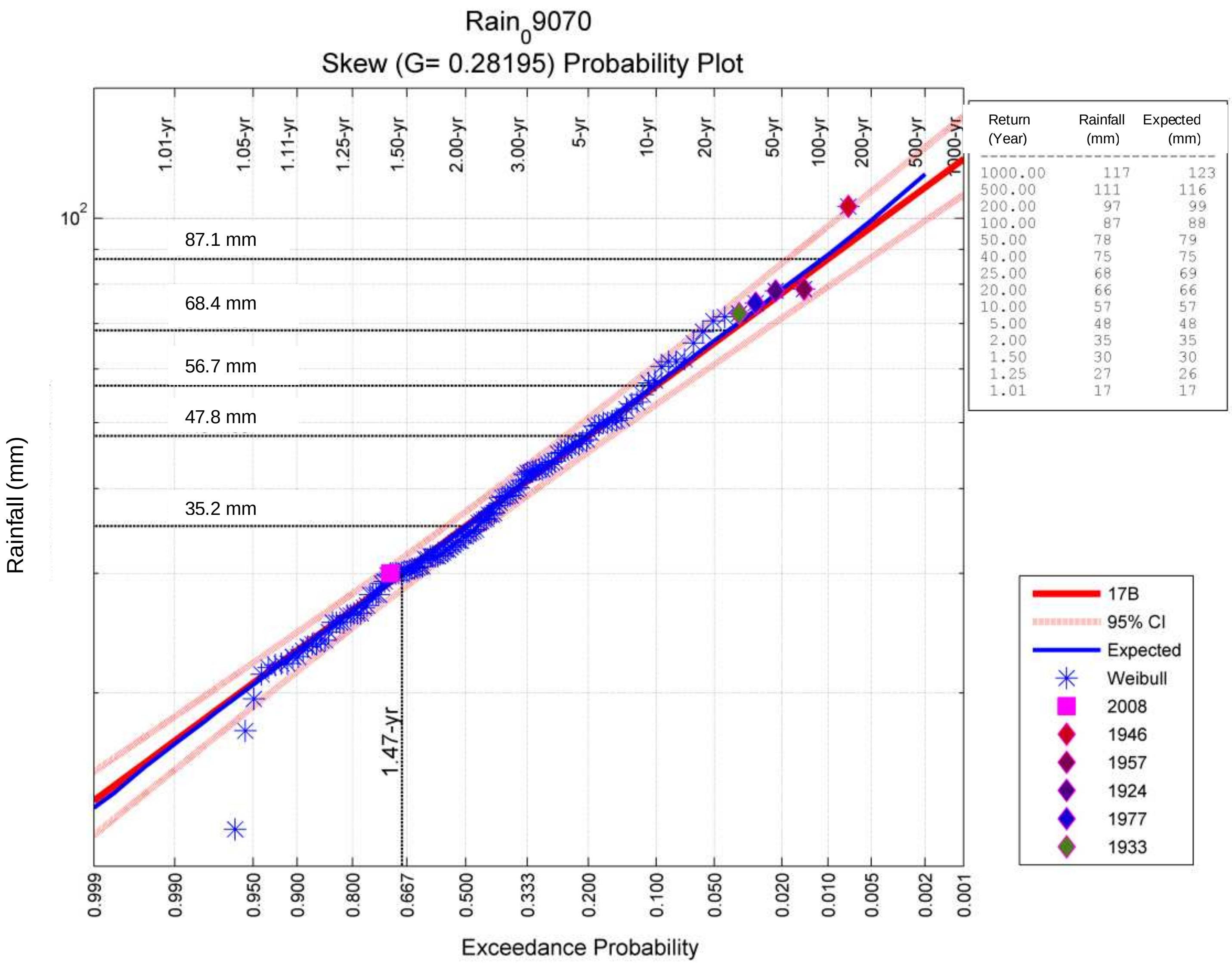
Return (Year)	Rainfall (mm)	Expected (mm)
1000.00	214	239
500.00	200	220
200.00	164	176
100.00	141	147
50.00	121	125
40.00	115	118
25.00	103	105
20.00	97	99
10.00	82	82
5.00	67	68
2.00	50	50
1.50	44	44
1.25	40	39
1.01	31	31

Plot C11 – Rainfall Frequency Analysis for Station 90050 – 48 hr Rainfall Event Duration

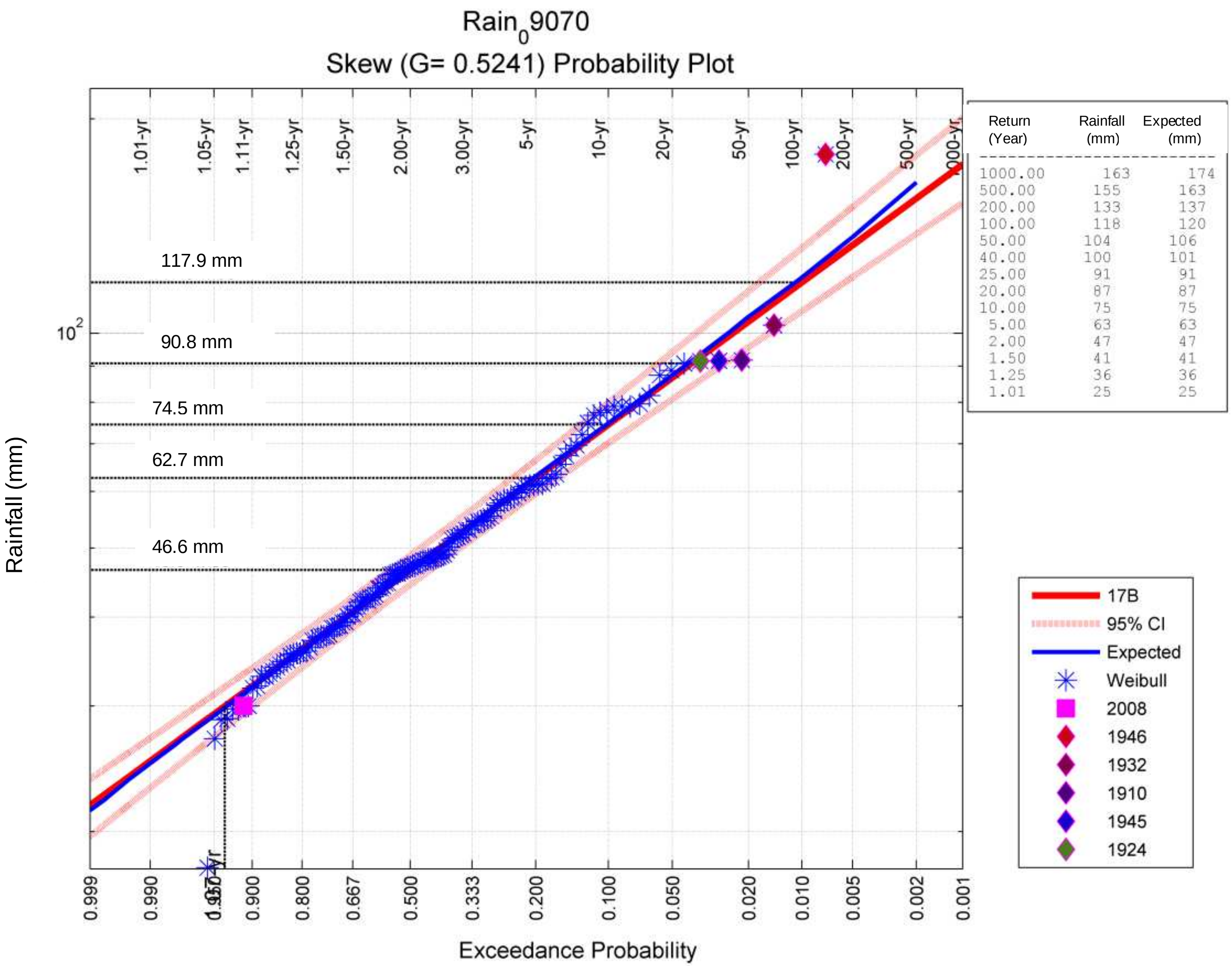


Return (Year)	Rainfall (mm)	Expected (mm)
1000.00	222	246
500.00	208	228
200.00	173	185
100.00	151	156
50.00	130	135
40.00	124	127
25.00	112	114
20.00	106	108
10.00	90	91
5.00	75	76
2.00	57	57
1.50	50	50
1.25	45	45
1.01	36	35

Plot C12 – Rainfall Frequency Analysis for Station 90050 – 72 hr Rainfall Event Duration

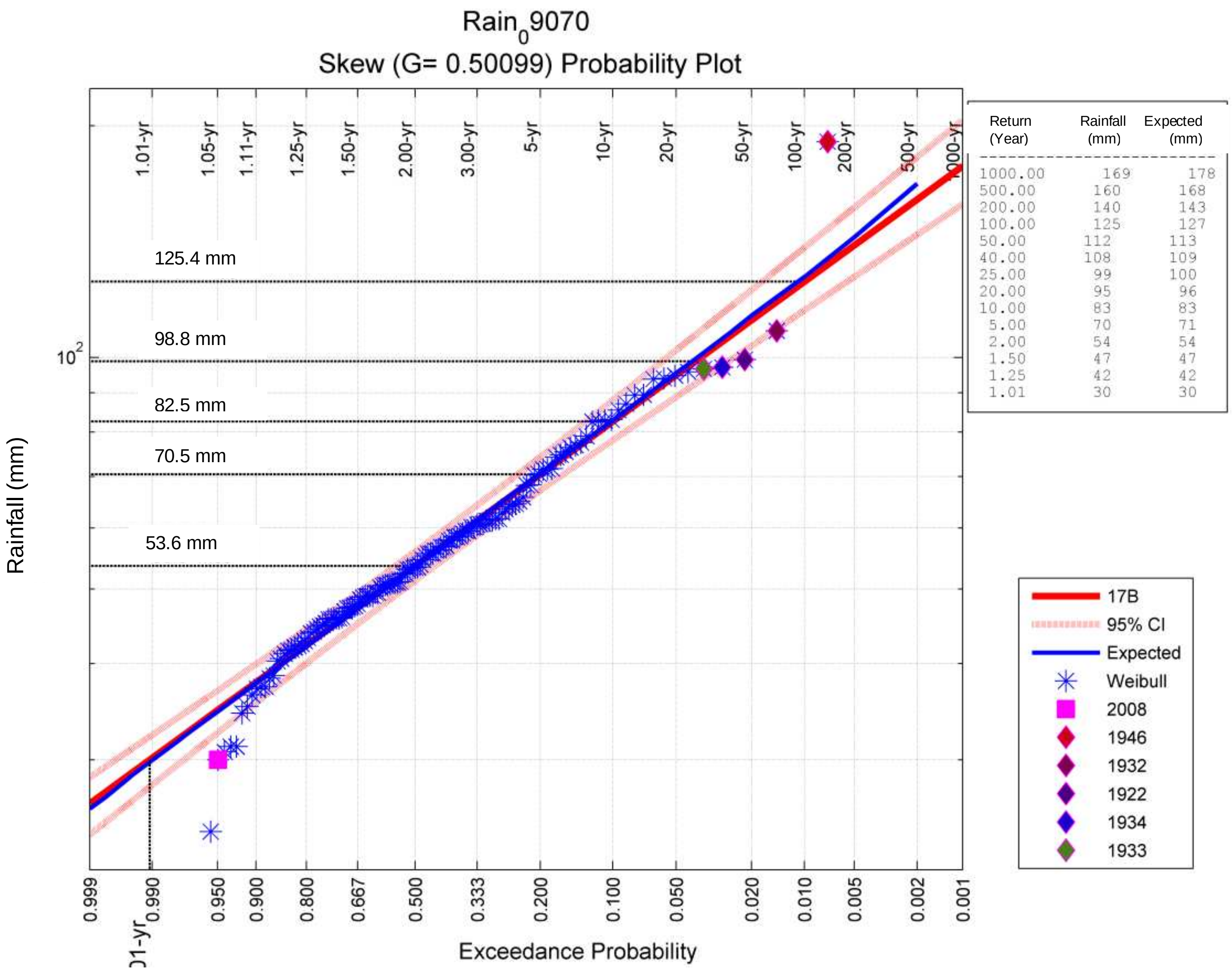


Plot C13 – Rainfall Frequency Analysis for Station 90070 – 24 hr Rainfall Event Duration



Return (Year)	Rainfall (mm)	Expected (mm)
1000.00	163	174
500.00	155	163
200.00	133	137
100.00	118	120
50.00	104	106
40.00	100	101
25.00	91	91
20.00	87	87
10.00	75	75
5.00	63	63
2.00	47	47
1.50	41	41
1.25	36	36
1.01	25	25

Plot C14 – Rainfall Frequency Analysis for Station 90070 – 48 hr Rainfall Event Duration



Plot C15 – Rainfall Frequency Analysis for Station 90070 – 72 hr Rainfall Event Duration

Appendix D

RORB & Design Hydrographs

D.1 RORB Vector Portland

Portland FS	
C Created Aug 2008, PJB, Cardno Lawson Treloar, Melbourne	
C Reach Type Flag	
1	
C The Control Vector	
1,2.83,-99,	Gen H'graph from Sub area A_A1
7	
A	
5,2.20,-99,	Route H'graph from A1_B1
3,	Store H'graph
1,1.40,-99,	Gen H'graph from Sub area B
7	
B	
4,	Add running H'graph
7	
A-B	
5,3.41,-99,	Route H'graph from B1_E1
3,	Store H'graph
1,2.87,-99,	Gen H'graph from Sub area C
7	
C	
5,1.68,-99,	Route H'graph from C1_D1
3,	Store H'graph
1,3.28,-99,	Gen H'graph from Sub area D
7	
D	
4,	Add running H'graph
7	
C-D	
5,1.32,-99,	Route H'graph from D1_E1
4,	Add running H'graph
7	
A-D	
5,1.66,-99,	Route H'graph from E1_E2
3,	Store H'graph
1,2.64,-99,	Gen H'graph from Sub area E
7	
E	
4,	Add running H'graph
7	
A-E	
5,4.29,-99,	Route H'graph from E2_F1
3,	Store H'graph
1,2.47,-99,	Gen H'graph from Sub area G
7	
G	
5,2.12,-99,	Route H'graph G1_F1
4,	Add running H'graph
3,	Store H'graph
1,1.46,-99,	Gen H'graph from Sub area F

7	
F	
4,	Add running H'graph
7	
A-G	
5,4.38,-99,	Route H'graph F1_J1
3,	Store H'graph
1,2.01,-99,	Gen H'graph from Sub area K
7	
K	
5,2.73,-99,	Route H'graph from K1_J1
4,	Add running H'graph
3,	Store H'graph
1,2.35,-99,	Gen H'graph from Sub area J
7	
J	
4,	Add running H'graph
7	
Wattlehill	
5,2.75,-99,	Route H'graph from J1_L1
3,	Store H'graph
1,1.37,-99,	Gen H'graph from Sub area H
7	
H	
5,3.30,-99,	Route H'graph from H1_L1
3,	Store H'graph
1,1.34,-99,	Gen H'graph from Sub area L
7	
L	
4,	Add running H'graph
3,	Store H'graph
1,2.00,-99,	Gen H'graph from Sub area I
7	
I	
4,	Add running H'graph
7	
Finn Creek	
4,	Add running H'graph
7	
Portland	
0	

C Subcatchment data
16.51,17.33,18.79,11.35,17.22,7.35,16.97,21.74,21.18,6.31,5.41,4.27,-99
C Subarea flag and fractions impervious
0
-99

D.2 Design Hydrographs

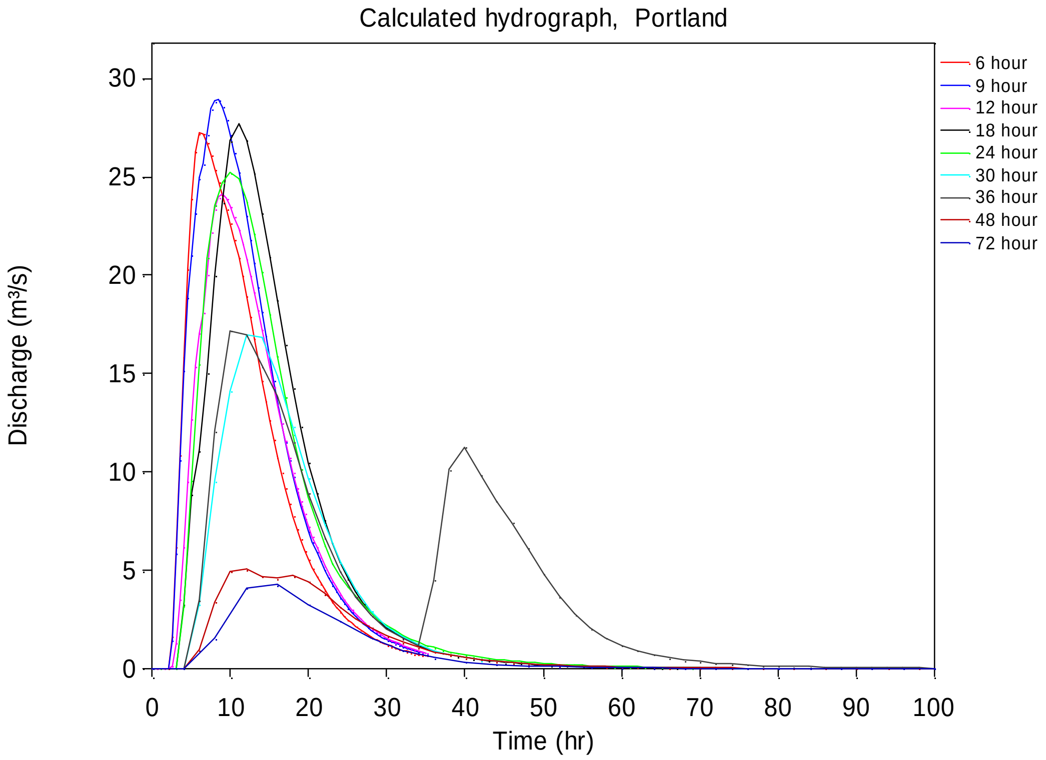


Figure D.1 – Portland 5 year ARI design hydrographs

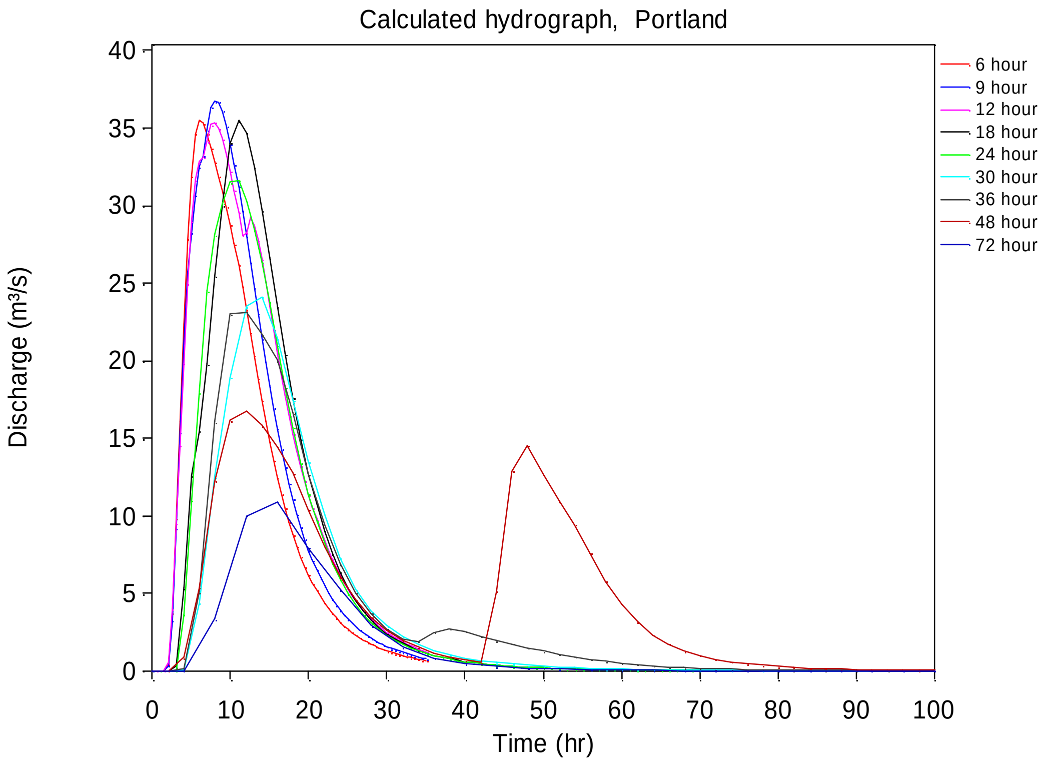


Figure D.2 – Portland 10 year ARI design hydrographs

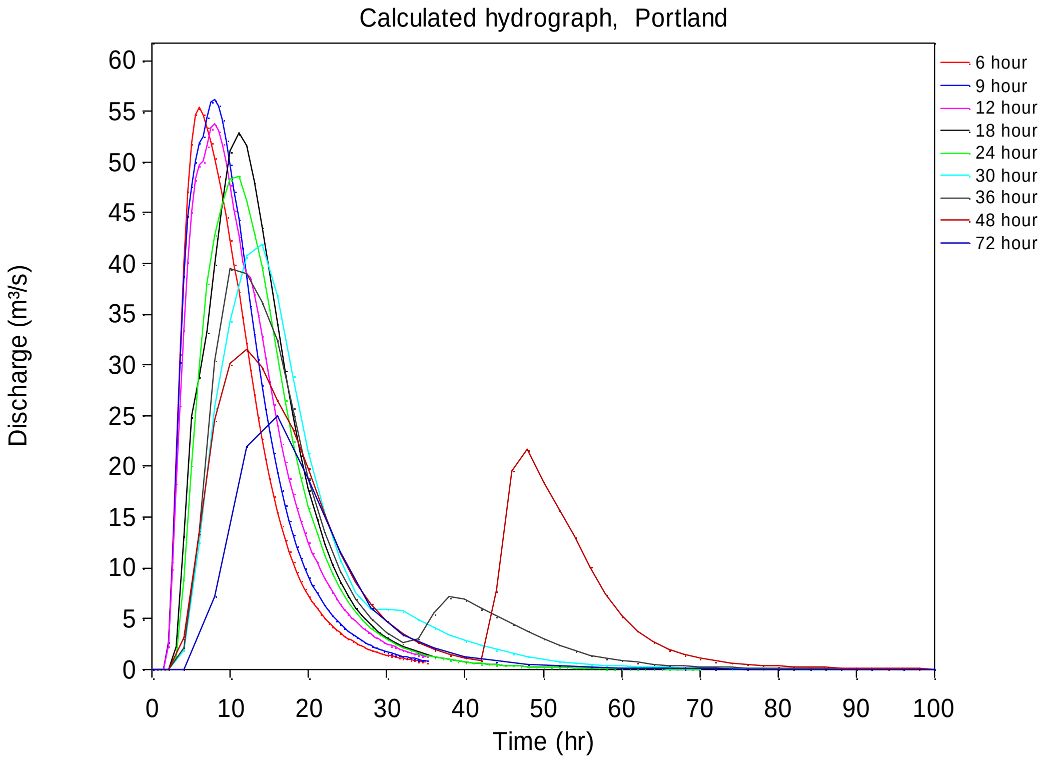


Figure D.3 – Portland 20 year ARI design hydrographs

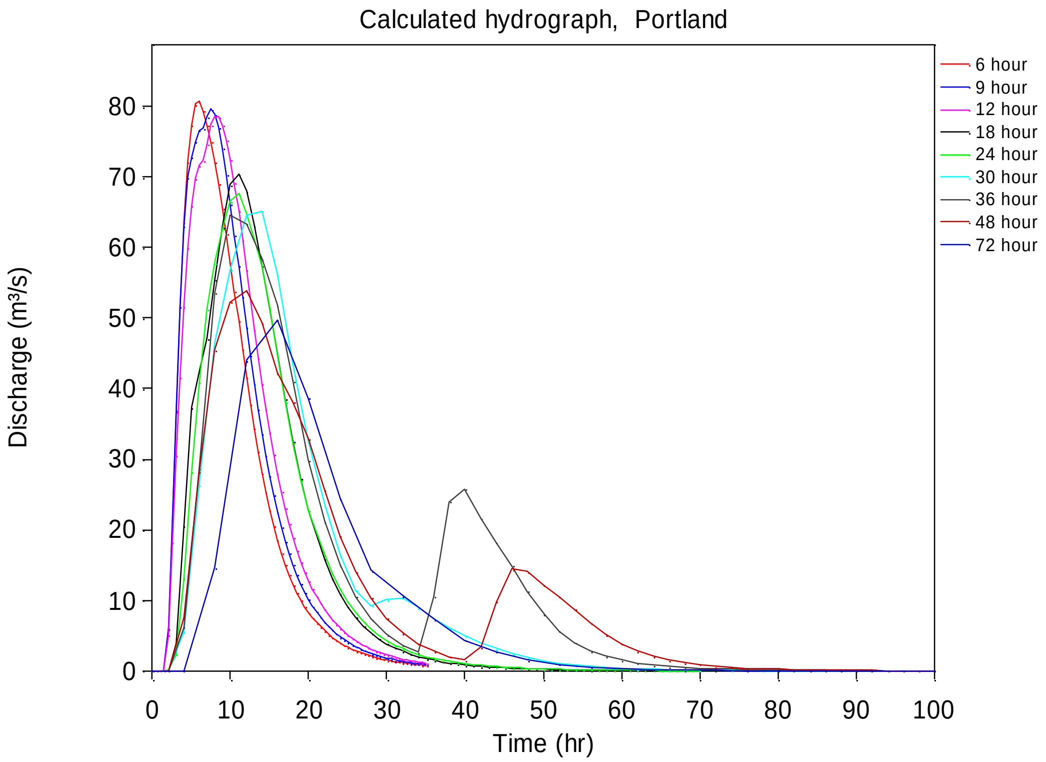


Figure D.4 – Portland 50 year ARI design hydrographs

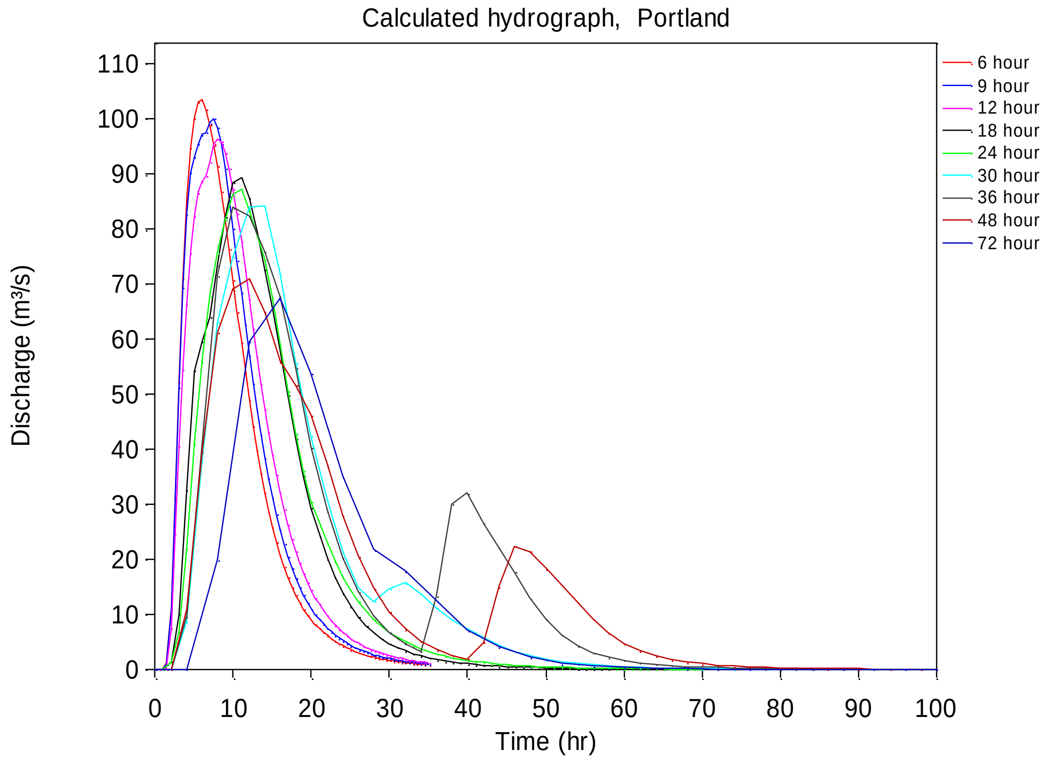


Figure D.5 – Portland 100 year ARI design hydrographs

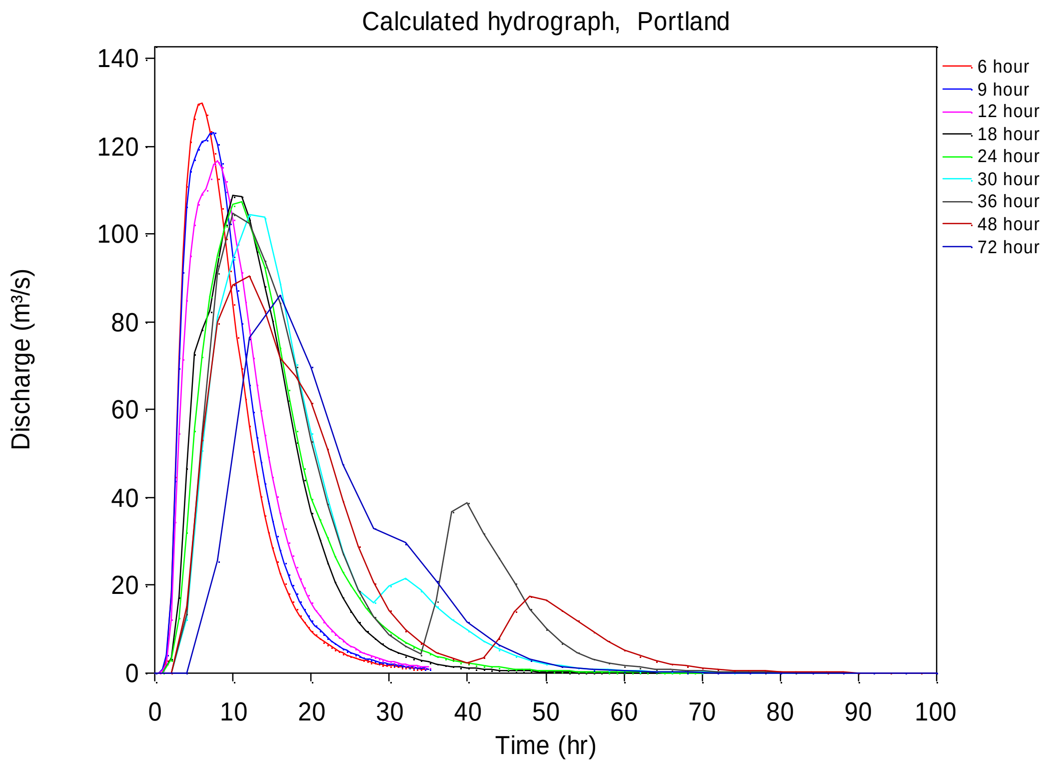


Figure D.6 – Portland 200 year ARI design hydrographs

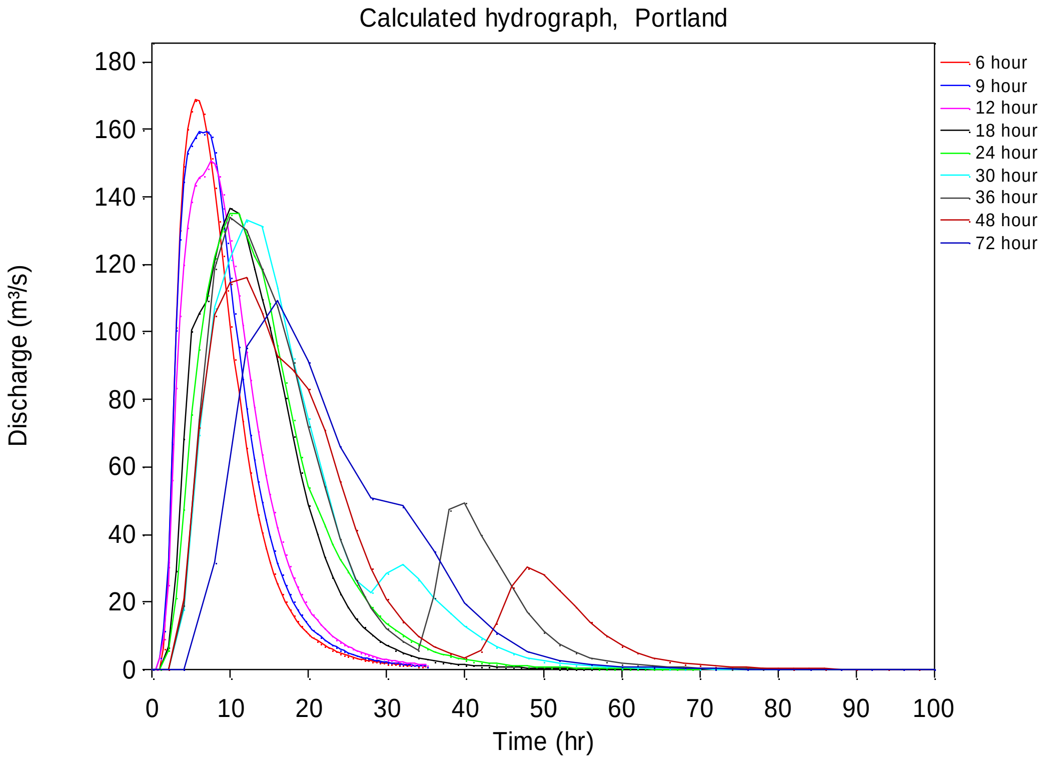


Figure D.7 – Portland 500 year ARI design hydrographs