

Geraldton CBD and Port Flood and Inundation Study

Project Modelling Report

CW1096100



Prepared for
City of Greater Geraldton and Midwest Port
Authority – Port of Geraldton

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Executive Summary

Summary

This Project Modelling Report outlines the technical foundation of the assessment of the flood and inundation hazards in the City of Greater Geraldton coastal zone, including the Port of Geraldton and the suburbs of West End, Geraldton and Beresford.

The flood and inundation hazards assessment considered a range of processes including rain, storm tide (including wave set-up and overtopping) and tsunamis. Three design events (i.e. 20yr ARI or 5%AEP, 100yr ARI or 1%AEP and 500yr ARI or 0.2%AEP) and three timeframes (i.e. 2030, 2070 and 2110) with adequate allowance for mean sea level rise.

A comprehensive high-resolution hydraulic model was developed that integrates detailed coastal boundary conditions and hydrological conditions modelled specifically for each scenario. The Port and the City provided local survey information used to inform the digital elevation model and develop the drainage infrastructure. The design rainfalls were obtained from the Bureau of Meteorology (BoM) and temporal pattern from the regional Australian Rainfall and Runoff (ARR) Data Hub. The design storm tide levels were based on the City's Coastal Hazard Risk Management and Adaptation Plan (CHRMAP).

Modelling results indicate that:

- > The tsunami cases modelled present minimal risk to the Geraldton area. This is demonstrated in Figure 8-5 and Appendix B with minimal flooding shown. Some areas of inundation on terrestrial locations are a result of the initial tailwater level being higher than the existing ground level. This may indicate that there is backflow through the stormwater network to some of these locations.
- > During the rain events, isolated locations within the study area are inundated by flood waters up to 2m deep. These are generally confined to drainage features and inlet locations. Some locations shown are where basement carpark entrances have been captured in the terrain survey or where terrain filling has introduced localised ponding. Typically, depths on land during the 500yr ARI rain event are less than 0.5m with flow paths typically confined to road reserves and existing drainage reserves/park areas. Furthermore, of those residential properties that are impacted by flooding, depths are generally less than 300mm.
- > The most significant coastal inundation hazards are attributed to storm tide & rainfall events and wave overtopping.
 - Residential areas in Zone C become inundated during the 20yr ARI event, with depths increasing to approximately 2m in the 500yr ARI. Foreshore street in Zones A and B are inundated by flood depths of up to 1.8 m during the 500yr ARI event. During this event, a significant number of residential and commercial properties are inundated.
 - In the Port area, the two reclaimed land areas are expected to experience significant overtopping under all scenarios to 2110, with the fishing boat harbor northern reclaim being submerged under the 100yr ARI event.
 - Wave overtopping rates at the extraction locations during storm tide exceed limits of tolerable discharges provided in overtopping guidelines for pedestrian, and vehicles safety and damage to building and defence structures (Table 5-12).

This Project Modelling Report provides the basis for subsequent mapping and planning considerations. It is accompanied by:

- > Inundation and flood maps (water level and water depth) for each scenario;
- > Inundation and flood animations for each scenario;
- > Data package containing all GIS layers created and model output files;
- > Stormwater Infrastructure Plan for the City, which rationalises the existing drainage network and considers future drainage upgrades; and
- > Emergency Response Plan for the City, which further characterises high flood hazard areas and outlines appropriate response.

Recommendations

Development level and coastal protection

Typically, buildings and structures should, as a minimum, be designed with a constructed floor level that provides adequate inundation immunity for the life of the project. Depending on the governing policy, finished floor level should be above the 1 in 100yr inundation level plus a freeboard allowance of 0.5 m (SPP2.9), alternatively development level should be above the 1 in 500yr coastal inundation level (SPP2.6).

Finished floor level are provided in the accompanying Emergency Response Plan.

A risk-based approach is recommended to be followed that considers the changing inundation hazard events frequency overtime (as sea levels rise) and the cumulative probability of such events to be exceeded during the life of the project.

Where coastal protections are likely to get inundated and overtopped by waves, there is a need to:

- > Consider the adequacy of existing rock works, adjacent pavement and neighbouring property development level going forward; and
- > Ensure adequate mitigation measures that meet standard personal safety and infrastructure integrity requirements are in place to address storm tide inundation and wave overtopping risk.

Design water level review

Considering the recent extreme water level review findings and the significance of the storm tide level in the coastal inundation extent, it would be beneficial to ascertain the extreme water level in the study area independently from previous studies. A more detailed assessment of the design water levels that would consider synthetic cyclone climatology and synoptic storm tide modelling is recommended to establish these extreme water levels. This could be part of the next CHRMAP periodic review - a process whereby the most recent and reliable information (including the design water level) can be examined and the CHRMAP updated as required.

1 Introduction

Cardno was commissioned through a collaboration between the City of Greater Geraldton and the Midwest Port Authority – Port of Geraldton have, to undertake an inundation study for the Geraldton CBD and Port area shown in Figure 1-1. Coastal, hydrologic and hydraulic processes modelling was undertaken to determine inundation depths and durations across the study area. This involved integrated modelling of rainfall across the catchment and coastal boundary conditions associated with cyclonic and non-cyclonic storm tide and tsunami events for 3 design events covering a range of Average Recurrence Interval (ARI) or Annual exceedance probability (AEP), including 20yr ARI (5% AEP), 100yr ARI (1% AEP) and 500yr ARI (0.2% AEP). The effect of sea level rise (SLR) was also assessed for 3 timeframes including 2030, 2070 and 2110. The outputs of this modelling form the basis of a range of inundation maps and informs the City's Emergency Response Plan and Stormwater Infrastructure Plan accompanying this Project Modelling Report.

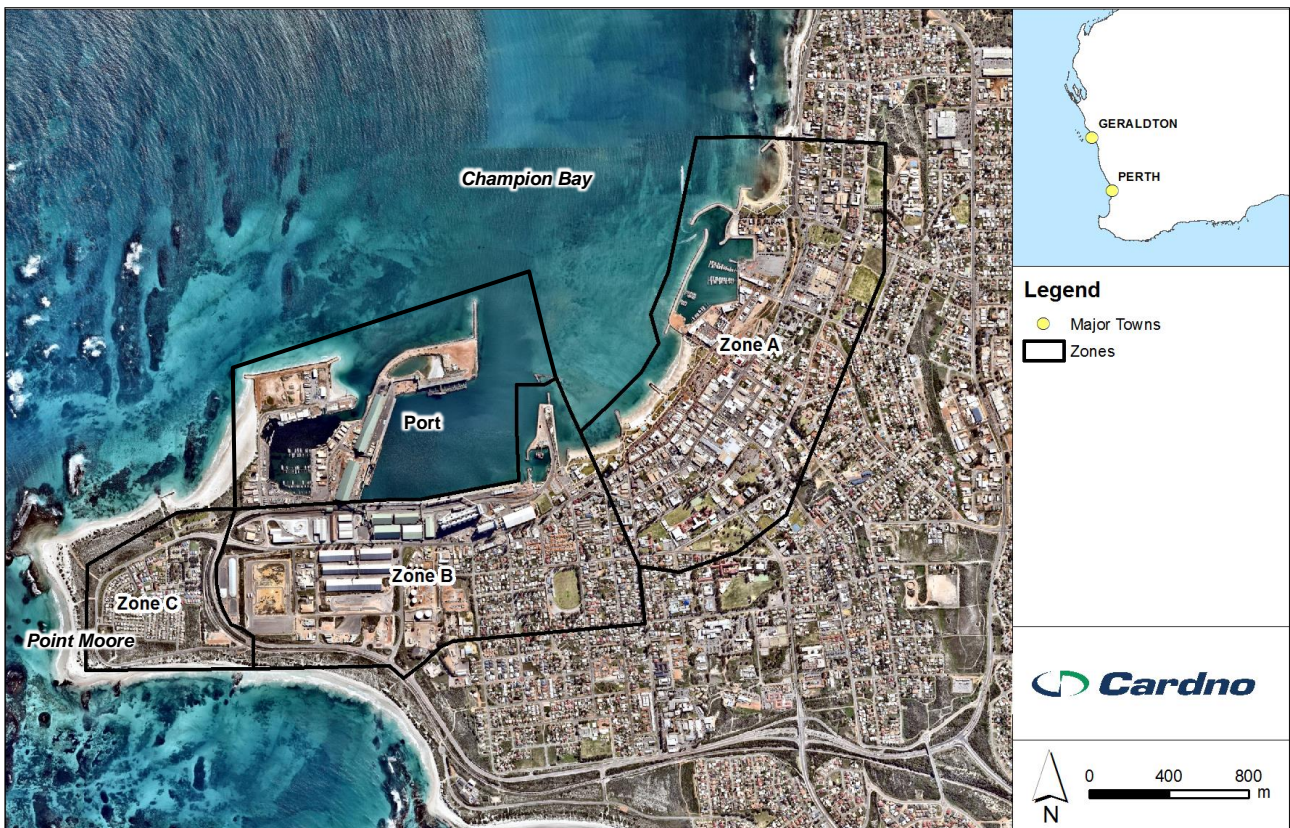


Figure 1-1 Locality plan.

2 Study area

2.1 Geographic location

Geraldton is a coastal city located in the Mid West region of Western Australia, approximately 400 km north of Perth CBD. The City can be divided into four locations: Zone A (Central Business District or CBD), Zone B (Beachlands), Zone C (West End) and the Port area, as shown in Figure 1-1. The topography can be characterised as flat and low-lying and therefore susceptible to coastal inundation. The coastal regions are lined with dunes, offering a barrier against storm tide and coastal inundation, however these dunes are susceptible to erosion and overtopping during large events.

The City is separated from external hydrological catchments by a 3 km long ridge in a north-south orientation as can be seen in Figure 3-1. In addition, there are no watercourses that drain through the Study Area, with the closest river being the Chapman River, located approximately 4 km north of the Study Area. The surface geology of the City can be characterised as sandy, with aeolian, beach and coastal sediments.

2.2 Climate and rainfall

Geraldton's climate can be characterised as Mediterranean with a semi-arid influence. The region has hot dry summers and warm winters. Annual precipitation is approximately 442 mm, with the majority occurring in winter months. The main sources of rainfall are winter fronts, troughs and cut-off lows from the South-West.

2.3 Coastal processes

2.3.1 Water level

Geraldton's tidal regime is microtidal and mainly diurnal with an astronomical tidal range of about 1.2m. The water level can be affected by other processes such as weather systems (storm tide), local wave conditions on the beach (wave set-up), remote earthquakes (tsunami) and global warming (global mean sea level rise).

2.3.2 Wind

Geraldton experiences seasonal extremes in weather, from hot summer days characterised by north-easterly winds from the interior of the state, to cold, wet, windy winter days as cold fronts from the Southern Ocean move through the region. On rare occasions tropical cyclones can move rapidly southwards and travel along the Western Australia (WA) coast while dissipating. AS/NZS1170.2:2011 (Standards Australia, 2011) places Geraldton in the non-cyclonic Wind Region B.

2.3.3 Wave

Offshore coastal waters west of Champion Bay and Point Moore experience high wave energy (MWPA, 2014). The four main elements of the offshore wave climate in Geraldton are:

- > Locally generated seas resulting from sea breeze conditions (waves 0.5–1.5m high from the south to south-west);
- > Seas generated locally by the passage of cold fronts during winter (waves 1-3m high from the west to south-west);
- > Swell waves generated by storms in the southern Indian Ocean (waves 1-3m high from the south-west); and
- > Severe waves caused by dissipating tropical cyclones.

2.3.4 Tsunami

Western Australia is susceptible to tsunamis from seismic sources that originate from distant sources including the Sunda Arc (sunda2 in Figure 2-1). The Indian Ocean tsunami ("Boxing Day 2004" tsunami) revealed the destructive effects of tsunamis across the Indian Ocean with a maximum run-up exceeding 30m in Banda Aceh (Indonesia). In Western Australia the impact was greatly reduced as the tsunami wave height tend to decrease with the distance travelled (Pattiaratchi, 2020). The Geraldton tide gage recorded a maximum water level of 1.8mAHD during that event (DoT, 2019). A tsunami in Western Australia is more likely to cause dangerous rips and currents and travel up river systems and estuaries rather than flood low lying land (DFES, 2020).

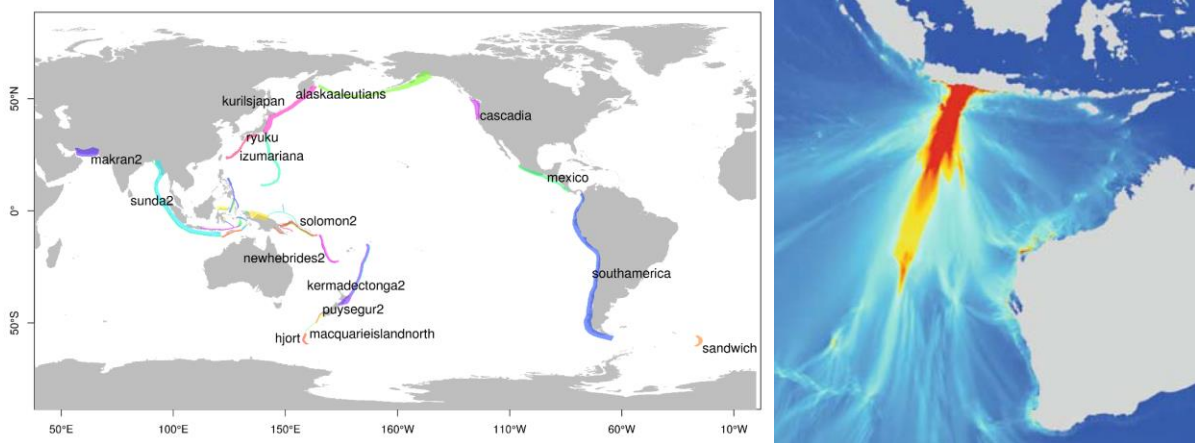


Figure 2-1 Tsunami source zones (left, GA 2018) and example of tsunami wave generation (right, Burbidge et al. 2009).

2.3.5 Global mean sea level rise

According to the latest Intergovernmental Panel on Climate Change (IPCC) assessment report, global mean sea level is rising and accelerating. The sum of glacier and ice sheet contributions is now the dominant source of global mean sea level rise. Future rise in global mean sea level caused by thermal expansion, melting of glaciers and ice sheets, and land water storage changes, is strongly dependent on which Representative Concentration Pathway (RCP) emission scenario is followed. Sea level rise at the end of the century is projected to be faster under all scenarios, including those compatible with achieving the long-term temperature goal set out in the Paris Agreement. Global mean sea level will rise between 0.43m and 0.84m by 2100 relative to 1986–2005 (Oppenheimer et al. 2019). These updated results are consistent with earlier reviews of mean sea level variations along the Western Australian (WA) coastline, and current recommendations on appropriate allowances for mean sea level change to be used in coastal planning (DoT, 2010).

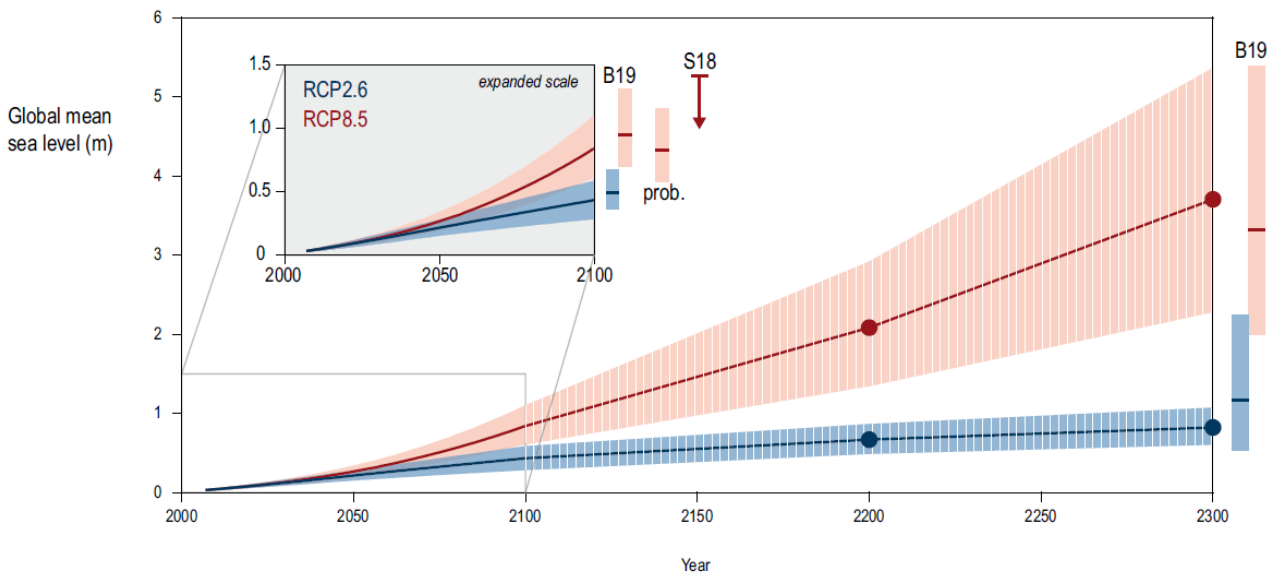


Figure 2-2 IPCC projected sea level rise SLR until 2300 (Oppenheimer et al. 2019).

3 Data review

3.1 Elevation data

3.1.1 Topography

Elevation data was provided by the City in the form of Light Detection and Ranging (LiDAR) point cloud data. This was used to generate a digital elevation model (DEM) of the Study Area, shown in Figure 3-1. Buildings were filtered from the datasets and elevations at the footprint were interpolated using elevations at the boundaries. This dataset covers the Study Area in its entirety.

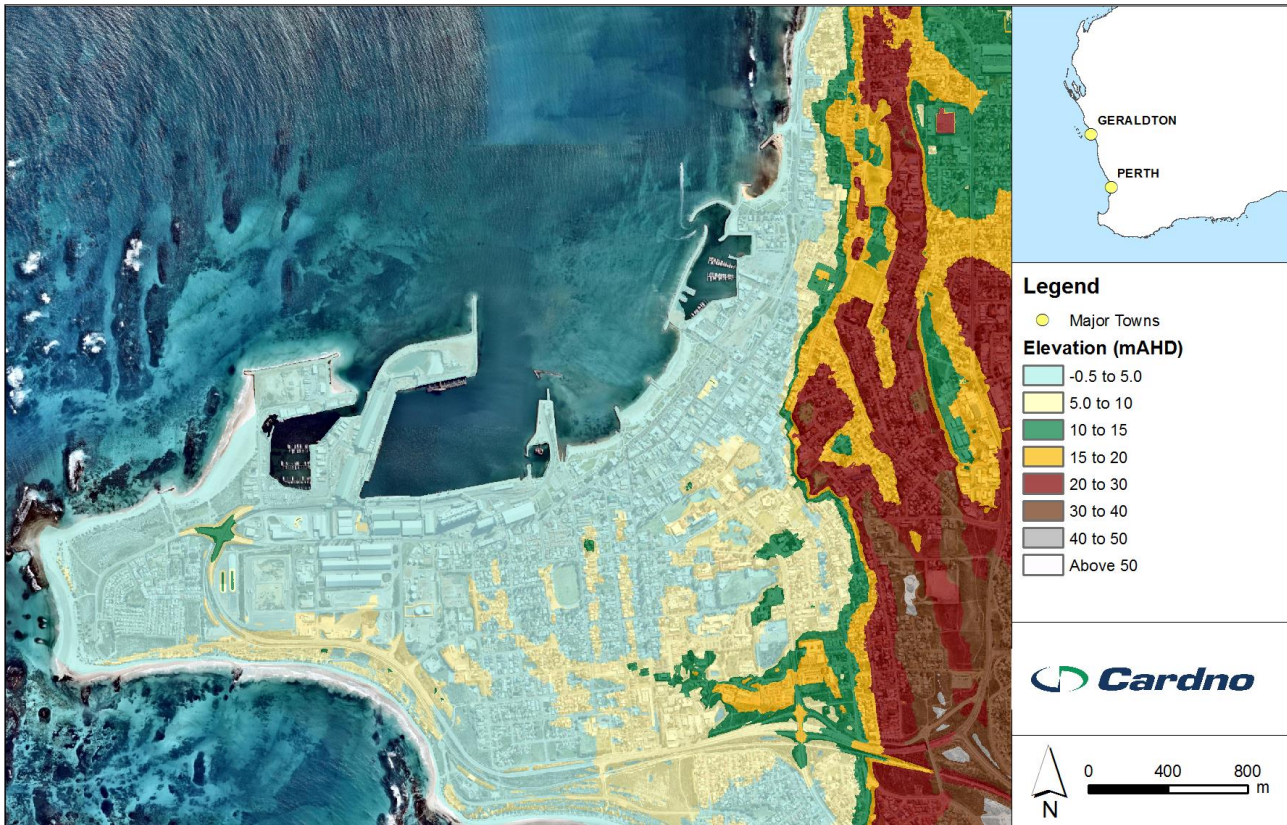


Figure 3-1 DEM of the study area derived from LiDAR data.

3.1.2 Bathymetry

Bathymetry data was based on a combination of national bathymetric data for the deep water from Geoscience Australia, navigation charts for coastal regions from the Australian Hydrographic Service, and nearshore bathymetric LiDAR data from the Department of Transport, as shown in Figure 3-2.

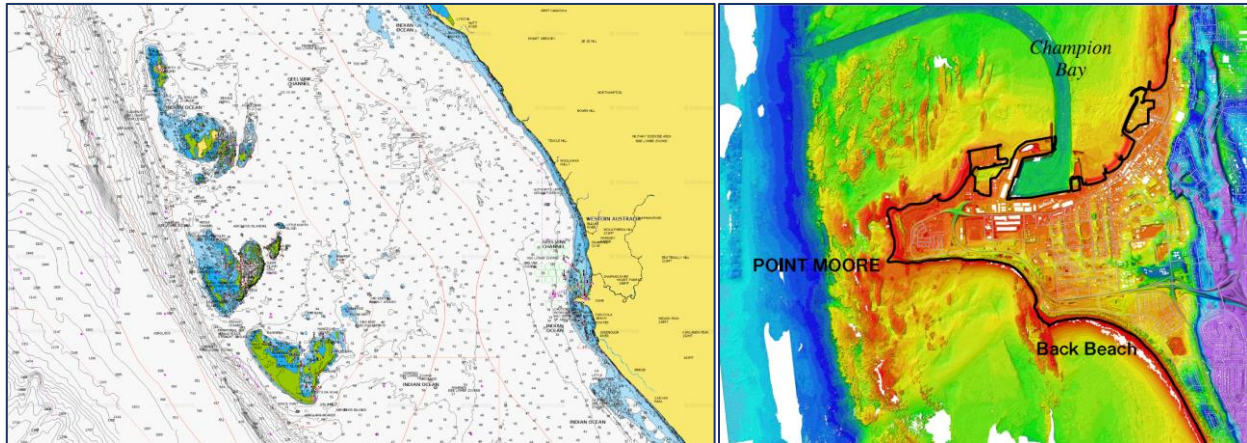


Figure 3-2 Example of bathymetric data from the Australian Hydrographic Service (left) and the Department of Transport (right).

3.2 Land use

Land use data was provided by the City as a Shapefile. The distribution of land use categories across the Study Area is shown in Figure 3-3. These categories were used to determine effective impervious areas and hydraulic roughness values.

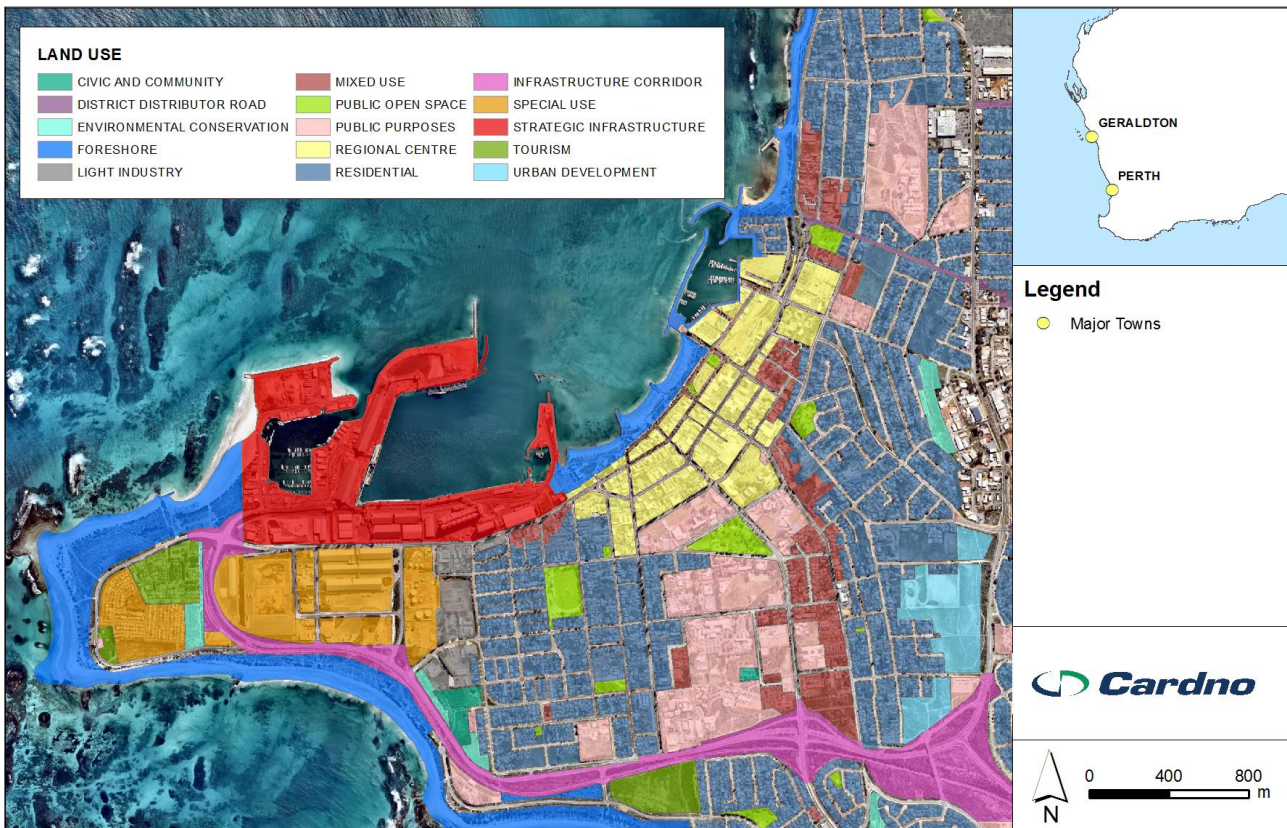


Figure 3-3 Land use categories (provided by the City of Greater Geraldton Jan. 2019).

3.3 Drainage data

Drainage data was provided by the City and the Port in the form of GIS files. However, a vast quantity of the provided data contained information gaps, such as missing pipe diameters and invert levels. Cardno undertook tasks to ensure the dataset was appropriate for hydraulic modelling. These tasks and assumptions are detailed below:

- Pipe sizes smaller than 300mm diameter were filtered out of the dataset;
- Missing pipe diameters were infilled using diameters of neighbouring pipe sizes;
- A size of 300mm was adopted at locations where there was no neighbouring diameter data;
- Where a pipe was seen to be missing from the network, additional pipes were added;
- Pipes which were disconnected from the network and were seen not to drain anywhere based on aerial imagery were removed from the dataset;
- Pipes were checked for connectivity;
- Pipes were checked to be flowing in the correct direction and if not, forced to do so;
- Pits were automatically placed at pipe junctions;
- Pipe invert levels were calculated using the DEM, pipe diameter and an assumed cover of 600mm.

It was noted that there are several infiltration basins (or sumps) across the Study Area which parts of the network drain to. These have been assumed to have a design infiltration rate of 5m/day in accordance with common design practice across WA for similar soils as those found in the Study Area.

3.4 Ocean water levels

Background information on ocean water levels is provided below.

3.4.1 Tidal planes

The tidal planes at Geraldton, taken from DoT's submergence curve (DoT, 2019), are shown in Table 3-1.

Table 3-1 Water levels at Geraldton Port (DoT, 2019).

Water levels	Level (mCD)	Level (mAHD)
Highest recorded (26/12/2004)*	2.36	1.81
HAT - Highest Astronomical Tide	1.26	0.71
MHHW – Mean High High Water	1.01	0.46
MHLW – Mean High Low Water	0.86	0.31
MSL – Mean Sea Level	0.64	0.09
AHD – Australian Height Datum	0.55	0
MLHW – Mean Low High Water	0.41	-0.14
MLLW – Mean Low Low Water	0.27	-0.28
LAT – Lowest Astronomical Tide	0.07	-0.48
Lowest recorded (27/12/2004)*	-0.48	-1.03

(*) related to 2004 Boxing Day Tsunami.

3.4.2 Sea level rise allowance

Sea level rise allowances as a result of climate change have been estimated in accordance with the WA Planning Commission State Planning Policy No. 2.6: State Coastal Planning Policy (SPP2.6) and are provided in Figure 3-4 and summarised in Table 3-2.

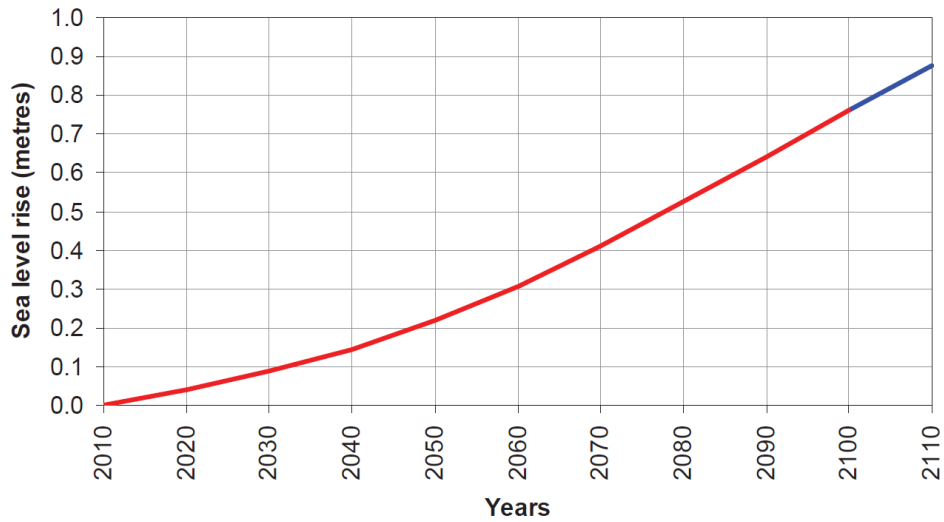


Figure 3-4 Recommended allowance for sea level rise in coastal planning for WA (DoT, 2010).

Table 3-2 Sea level rise allowances after the City's Coastal Hazard Risk Management and Adaptation Plan – CHRMAP (Baird, 2019), which is consistent with SPP2.6 sea level rise scenario.

Timeframe	2030	2070	2110
Sea level rise allowance (m)	+0.07	+0.39	+0.90

3.4.3 Extreme water levels

Further information and analysis of extreme and adopted design water levels are presented in Section 5 - Coastal and nearshore processes modelling.

4 Hydrology

A hydrology model of the CBD was constructed using the runoff-routing software RORB (Laurenson et al., 2010). This model gives an indication of the critical storm durations across the CBD. A Monte-Carlo analysis was undertaken to determine the critical flows and durations at sub-catchments across the Study Area. The Monte Carlo process involves stochastically sampling from a distribution of design rainfall depths for a range of AEPs and a random selection of temporal patterns, resulting in thousands of model runs. The durations which presented the median peak flows at chosen sub-catchments were adopted as the critical durations for each AEP being analysed. The ensemble of ten design temporal patterns for the critical durations were then run to determine the temporal pattern that corresponded with the peak flow from the Monte Carlo analysis.

4.1 Catchment delineation

The DEM derived from the LiDAR elevation data was used to define the main drainage channels throughout the catchment, which mainly constituted of overland flow through kerbed roads. Sub-catchments were then delineated using the topography. The sub-catchment plan for the CBD is shown in Figure 4-1.

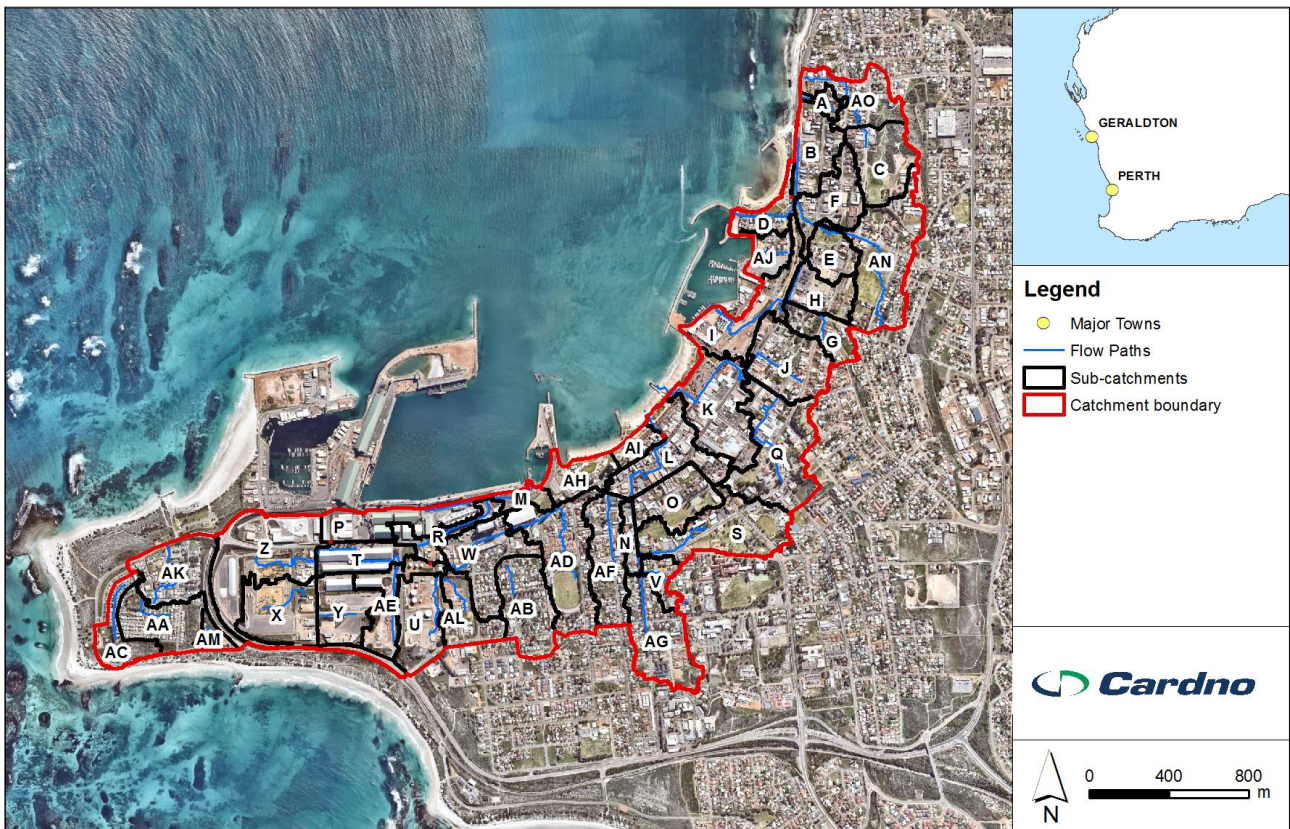


Figure 4-1 Sub-catchment plan.

4.2 Model setup

The ArcRORB (v1.7) extension for ArcGIS was used to setup the RORB model as follows:

- Reaches were assigned as Type 3: lined channel or pipe;
- Slopes were derived from the elevation data;
- Junction nodes were placed at channel confluences and locations where a hydrograph output was required;
- Multiple ocean outfalls, which were converged to a single outlet using dummy reaches. These have no influence on hydrologic calculations.

- Infiltration basins were modelled as outflows from the model at a constant rate of 5m/day¹. No groundwater interaction has been modelled.

The model setup is shown in Figure 4-2.

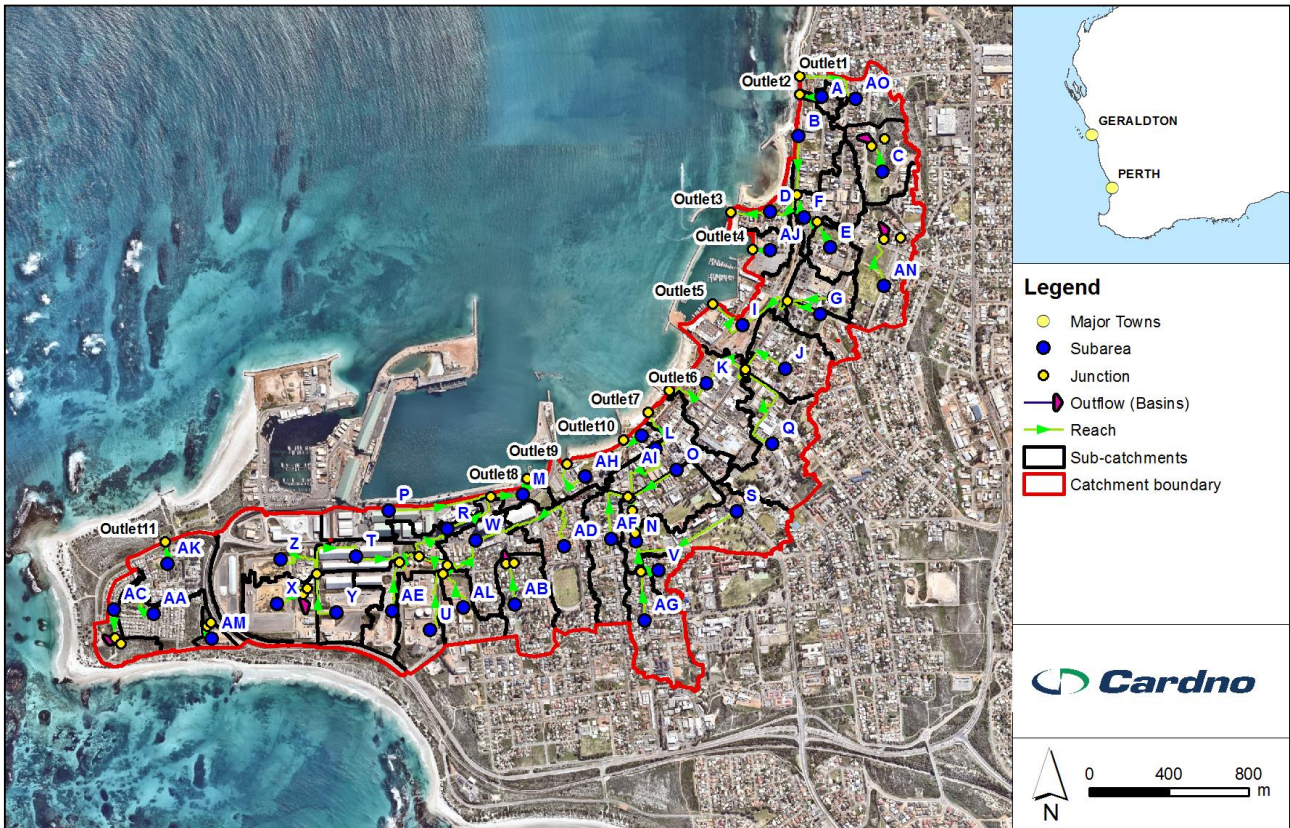


Figure 4-2 RORB model setup.

4.3 Model parameters

RORB uses an empirical coefficient ‘ k_c ’ applicable to the catchment and drainage network and is the principal storage and routing parameter of the model. In addition, a non-linearity factor ‘ m ’ is used in the RORB storage equations. In the absence of flow gauge data in the CBD, a regional estimate of the design k_c value was adopted from Australian Rainfall and Runoff 2019 based on Equation 1 for the Wheatbelt, North-West and Kimberley Region. The variable ‘ L ’ represents the length (km) of the main stream length and S_e represents the equal area slope of the main stream. In this case, the main stream was taken as the longest kerbed flow path through the Study Area.

$$k_c = 1.06L^{0.87}S_e^{-0.46} \quad \text{Equation 1}$$

This resulted in a k_c value of 1.29 and the recommended default value of 0.8 for ‘ m ’ was adopted to represent the non-linearity in catchment storage.

¹ The 1:250,000 Geraldton – Houtman Abrolhos geological map sheet describes the ground conditions at Point Moore and surrounds as ‘Dune and beach sands - white calcereous and quartzose sands.’ Considering this classification and experience in similar ground conditions it was assumed that the soils in the study area basins are well drained sands, and a hydraulic conductivity of 5m/day was assumed. Often materials such as these could have hydraulic conductivities in excess of 10m/day, however 5m/day represents a conservative assumption in the absence of at-site data, and potential clogging at the top of layer of basins. It is noted that an infiltration rate of 5m/day may not be appropriate for some climate change scenarios where the base of the basins will be permanently inundated due to sea level rise. Sensitivity testing was conducted on some basins to assess their ability to manage inflows in a high groundwater scenario and it was determined they still function in those conditions to manage water volume.

4.4 Impervious fractions

The supplied land use data was simplified into twelve different land use categories as shown in Table 4-1 and used to determine the impervious fraction for each sub-catchment. The approximate fraction impervious was estimated using aerial photography and GIS. Effective impervious areas for each land use category was then calculated using the methods outlined in Book 5 of ARR 2019, and are presented in Table 4-1. The effective impervious fraction for each sub-catchment is shown in Figure 4-3.

Table 4-1 Effective Impervious Fractions for Land Use Categories

Land use	Fraction impervious (%)	Effective Impervious Fraction (%)
POS	5	2
Low veg	0	0
High Veg	0	0
Commercial (RAC-3)	80	60
Industrial	95	74
Foreshore	90	69
Public purposes	60	42
Residential - Low density	50	33
Residential - Medium density (R60)	70	51
Residential - High density	80	60
Roads (including reserve)	90	69

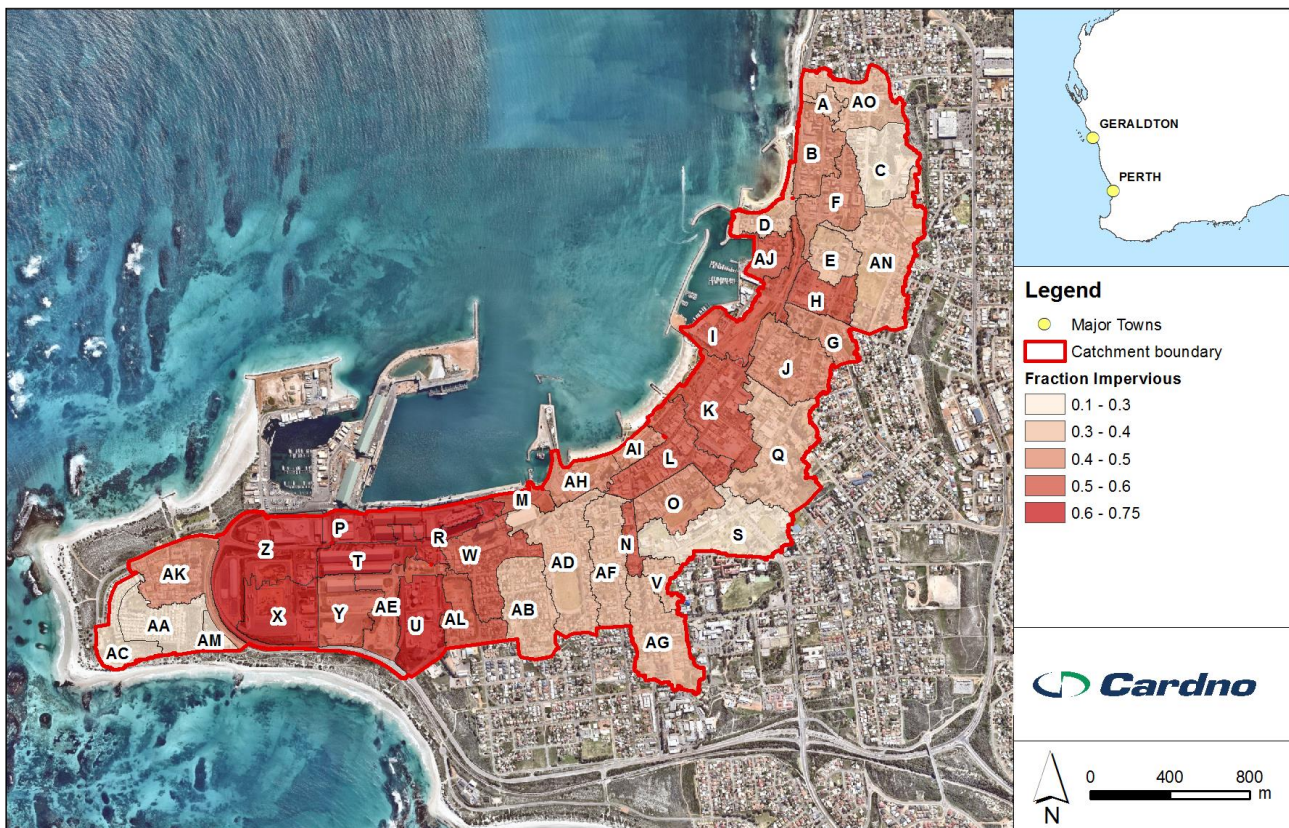


Figure 4-3 Sub-catchment impervious fractions.

4.5 Rainfall losses

An initial loss (IL) and continuing loss (CL) model was used to determine rainfall losses for the catchment. Loss data was obtained from the ARR Data Hub and values of 45 mm IL and 3 mm/hour CL were applied in the RORB model. The fraction impervious for each sub-catchment was calculated through ArcRORB using the effective impervious fractions in Table 4-1 for the different land use categories. The initial and continuing losses for each sub-catchment was then calculated by RORB using the following equations:

$$IL_i = (1 - F_i)IL_{pervious} \quad \text{Equation 2}$$

$$CL_i = (1 - F_i)CL_{pervious} \quad \text{Equation 3}$$

Where:

IL_i : Initial loss of sub-catchment i

CL_i : Continuing loss of sub-catchment i

F_i : Effective impervious fraction for sub-catchment i

$IL_{pervious}$: Pervious initial loss for the catchment

$CL_{pervious}$: Pervious continuing loss for the catchment

4.6 Design rainfall depths

Design rainfall depths were obtained from Intensity Frequency Duration (IFD) curves available from the Bureau of Meteorology (BoM) for 28.78° S, 114.61° E. This IFD data is shown in Appendix A.

4.7 Storm temporal pattern

Point temporal patterns were obtained from the ARR Data Hub for the Southern and South Western Flatlands (West) region.

4.8 Model verification

In the absence of gauged flow data for the CBD area, the Rational Method was chosen to check the validity of the RORB model setup. The Rational Method was used to estimate peak flows for specific sub-catchments in the model, which were compared to the modelled peak flow in RORB. The sub-catchments chosen were B, O and U to represent spatial variation of flows throughout the Study Area catchment as they are located in Zones A, B and C respectively. The time of concentration was determined using Izzard's kerb flow travel time equation:

$$t = 0.025 \frac{L}{S^{0.5}} \quad \text{Equation 4}$$

Where:

t : time of gutter flow (min)

L : length of gutter flow (m)

S : slope of gutter (%)

The results of this verification for the 1% AEP event are shown in Table 4-2. It can be seen that the differences in peak flow between the Rational Method and RORB model are within approximately 5%², indicating that flows generated by the RORB model are within an acceptable level of accuracy.

Table 4-2 Rational Method Comparison with RORB Model Flows – 1% AEP

Sub-catchment	Q (m ³ /s) – Rational Method	Q (m ³ /s) – RORB	Critical Duration – RORB (hrs)	Difference
B	2.20	2.10	0.5	4.97 %
U	2.78	2.72	0.5	2.43 %
O	2.18	2.06	0.5	5.89 %

4.9 Model results

A number of durations and temporal patterns were modelled. The modelled RORB peak flow results, critical durations and ARR temporal patterns at critical locations in the Study Area for the 5%, 1% and 0.2% AEP design events are presented in Table 4-3, Table 4-4 and Table 4-5 respectively. These locations include the major ocean outlets and several sub-catchments spread across the Study Area.

Table 4-3 RORB Model Results – 5% AEP

Location	Peak Flow (m ³ /s)	Critical Duration (hours)	Temporal Pattern
AF	1.25	2	TP20
AG	1.37	2	TP13
B	1.47	0.5	TP16
J	1.88	0.5	TP16
O	1.42	0.5	TP16
U	2.00	0.5	TP15
Outlet 3	3.95	2	TP20
Outlet 5	2.54	2	TP20
Outlet 6	3.10	2	TP18
Outlet 7	3.85	2	TP13
Outlet 8	6.09	2	TP16

² Within 10% is accepted industry practice.

Table 4-4 RORB Model Results – 1% AEP.

Location	Peak Flow (m ³ /s)	Critical Duration (hours)	Temporal Pattern
AF	2.01	1.5	TP28
AG	2.30	1.5	TP29
B	2.10	0.5	TP25
J	2.75	1.5	TP28
O	2.06	0.5	TP29
U	2.72	0.5	TP25
Outlet 3	5.98	1.5	TP22
Outlet 5	3.95	1.5	TP29
Outlet 6	5.10	1.5	TP22
Outlet 7	7.24	2	TP28
Outlet 8	10.21	2	TP21

Table 4-5 RORB Model Results – 0.2% AEP.

Location	Peak Flow (m ³ /s)	Critical Duration (hours)	Temporal Pattern
AF	2.94	2	TP24
AG	3.40	1.5	TP28
B	3.08	1	TP29
J	4.19	1	TP29
O	3.17	1	TP29
U	4.16	1	TP29
Outlet 3	9.17	1	TP29
Outlet 5	5.57	1	TP30
Outlet 6	7.41	1.5	TP28
Outlet 7	10.72	1.5	TP28
Outlet 8	14.44	2	TP22

5 Coastal and nearshore processes modelling

5.1 Background

5.1.1 Key coastal processes

The flood and inundation model required coastal boundary conditions to represent the effects of elevated water level and wave overtopping flow during the specified design events. Key coastal processes terms and associated water levels are schematically illustrated in Figure 5-1.

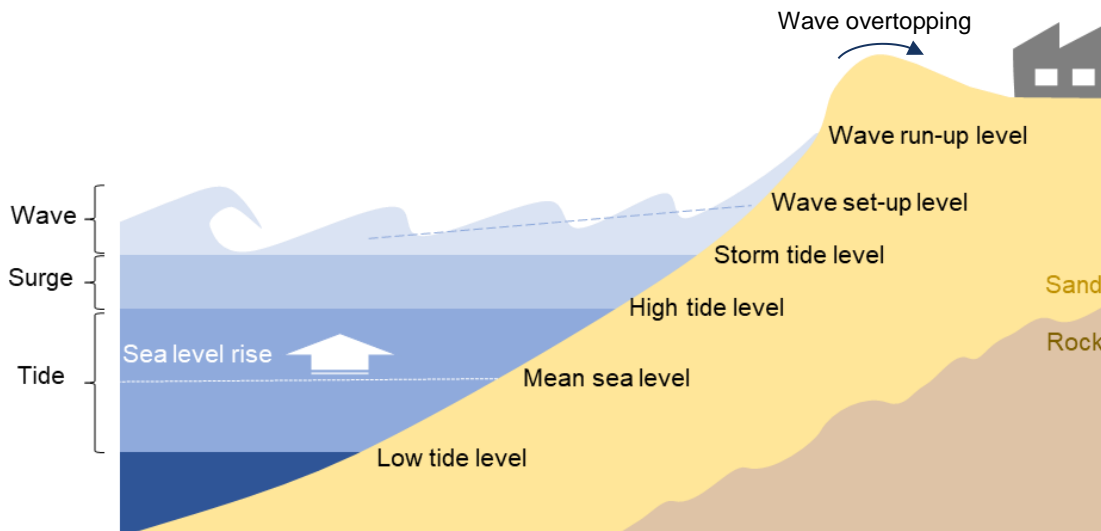


Figure 5-1 Illustration of key coastal processes terms and associated water level.

5.1.2 Design scenarios considered

The design scenarios considered in the flood and inundation model included storm tide conditions (due to cyclonic and non-cyclonic events), tsunami conditions and the effects of sea level rise for 3 timeframes and 3 return periods, as illustrate in Table 5-1.

Table 5-1 Design scenarios.

Source	Storm Tide						Tsunami		
	Synoptic storms			Tropical cyclones			Remote earthquakes		
Timeframe	2030	2070	2110	2030	2070	2110	2030	2070	2110
20yr ARI (5% AEP)	✓	✓	✓	✓	✓	✓	✓	✓	✓
100yr ARI (1% AEP)	✓	✓	✓	✓	✓	✓	✓	✓	✓
500yr ARI (0.2% AEP)	✓	✓	✓	✓	✓	✓	✓	✓	✓

5.1.3 Design scenarios adopted

The design storm events include consideration for storm tides generated under both cyclonic and non-cyclonic conditions, as well as the effects of sea level rise, for a range of return periods. These storm tide conditions were simplified in accordance with the City's Coastal Hazard Risk Management and Adaptation Plan (CHRMAP) findings (Baird, 2019), by taking the maximum water level from cyclonic conditions and non-cyclonic conditions. This resulted in the adoption of 9 storm tide scenarios and 9 tsunami scenarios, as shown in Table 5-2.

Table 5-2 Design scenarios adopted.

Timeframe	Strom Tide			Tsunami		
	2030	2070	2110	2030	2070	2110
20yr ARI	✓	✓	✓	✓	✓	✓
100yr ARI	✓	✓	✓	✓	✓	✓
500yr ARI	✓	✓	✓	✓	✓	✓

5.2 Coastal and nearshore modelling framework

The coastal modelling framework comprised two stages. Each stage involves the use of different numerical models capable of representing coastal and nearshore processes at an appropriate scale, ranging from the continental shelf to the beach. An overview of these two stages is provided below:

- > Stage 1 – Coastal Model - A 2D horizontal coupled hydrodynamic and wave model of the area was developed, including four nested grids ranging from coarse 2,500m offshore grid to fine 20m grid on the coast, to estimate water level and wave conditions in coastal waters under the action of selected offshore metocean conditions, including:
 - Tsunami conditions defined by the 2018 Australian Probabilistic Tsunami Hazard Assessment Database developed by Geoscience Australia;
 - Storm tide conditions defined by the extreme design water level established in the City's CHRMAP completed in 2019 and extreme design wave conditions established from the statistical analysis of the Port of Geraldton Wave Rider Buoy data; and
 - Tropical cyclone storm tide inundation defined by the Department of Transport design storm conditions in the Seashore, 2018 Report - Design Storms for Western Australian Coastal Planning: Tropical Cyclones.
- > Stage 2 - Nearshore Model - A complementary 2D vertical nearshore wave model was developed, including 17 high resolution transects along the coast, to address wave overtopping and inundation processes along the sandy shoreline and coastal protection structures under the action of coastal metocean conditions established in Stage 1 above.

These two stages are further outlined in the following sections.

5.2.1 Stage 1 – Coastal Model

The coupled hydrodynamic and wave modelling framework included:

- > Delft3D-Flow - a multi-dimensional (2D or 3D) hydrodynamic (and transport) simulation program which calculates non-steady flow and transport phenomena that result from tidal and meteorological forcing on a rectilinear or a curvilinear, boundary fitted grid (Delft3D-Flow, 2019);
- > Delft3D-Wave – a spectral wave model which computes the evolution of random, short-crested waves in coastal regions with deep, intermediate and shallow water and ambient currents. The model accounts for (refractive) propagation due to current and depth, and represents the processes of wave generation by wind, dissipation due to white capping, bottom friction and depth-induced wave breaking and non-linear wave-wave interactions (both quadruplets and triads) explicitly with state-of-the-art formulations (Delft3D-Wave, 2019).
- > Cardno's modified Holland cyclone wind and pressure field model including first-order wind-field asymmetry due to cyclone forward motion. The model was applied to selected tropical cyclone tracks defined by

timeseries of key parameters (e.g. date, geographical coordinates, central pressure and radius to maximum wind).

- > Water levels along the offshore boundaries were provided as inputs representative of sea level rise allowance, tidal conditions and tsunami stage as required.

Cardno’s validated Beresford Foreshore model (Cardno, 2012) was required to be updated to address the requirements of this study. This included the inclusion of additional hydrodynamic, wind and wave forcing, as well as multiple nested grid extensions (Figure 5-2, Figure 5-3). The other parameterisation factors (e.g. friction coefficients) were kept the same as in the original model.

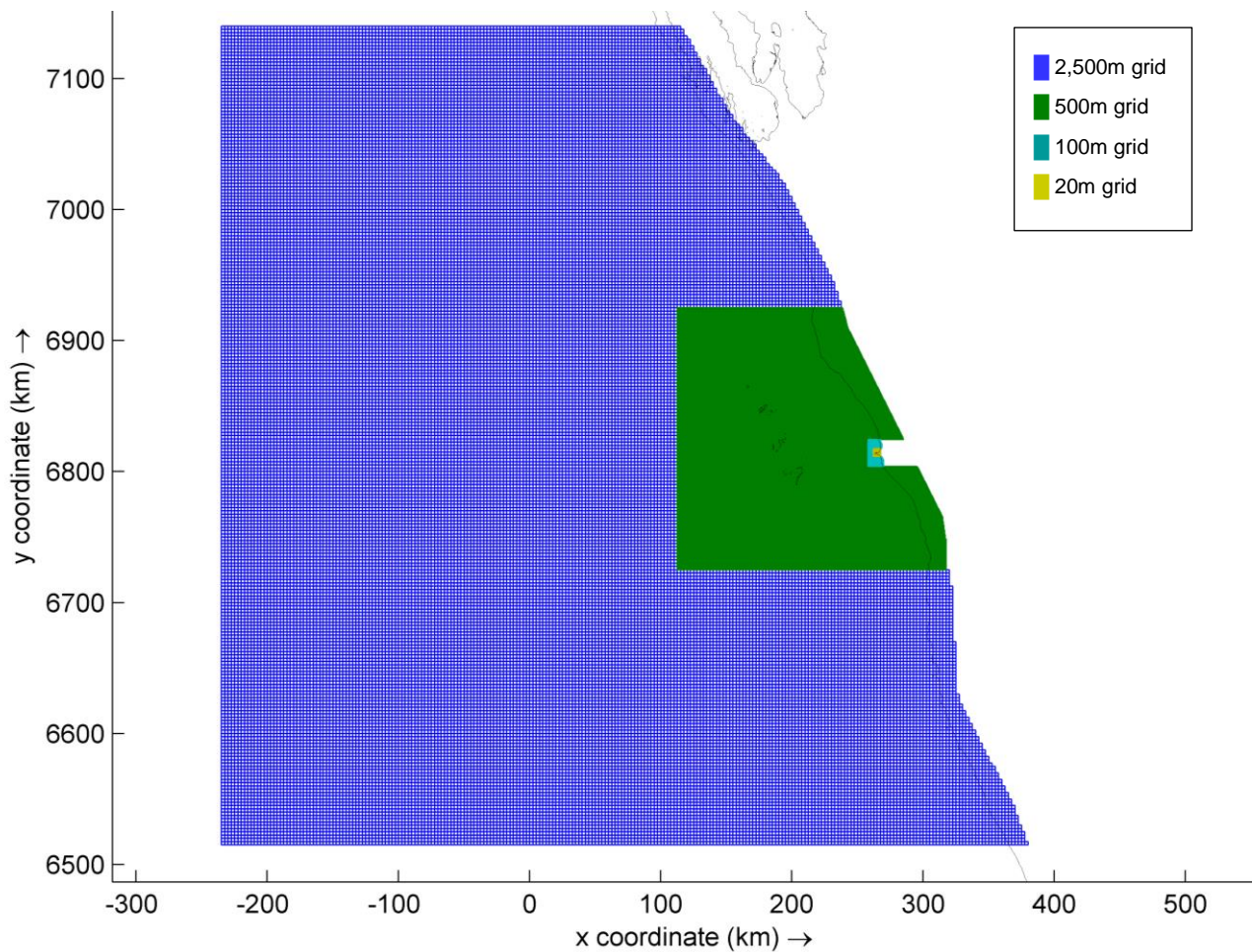


Figure 5-2 Stage 1 – Coastal Model four nested grids.

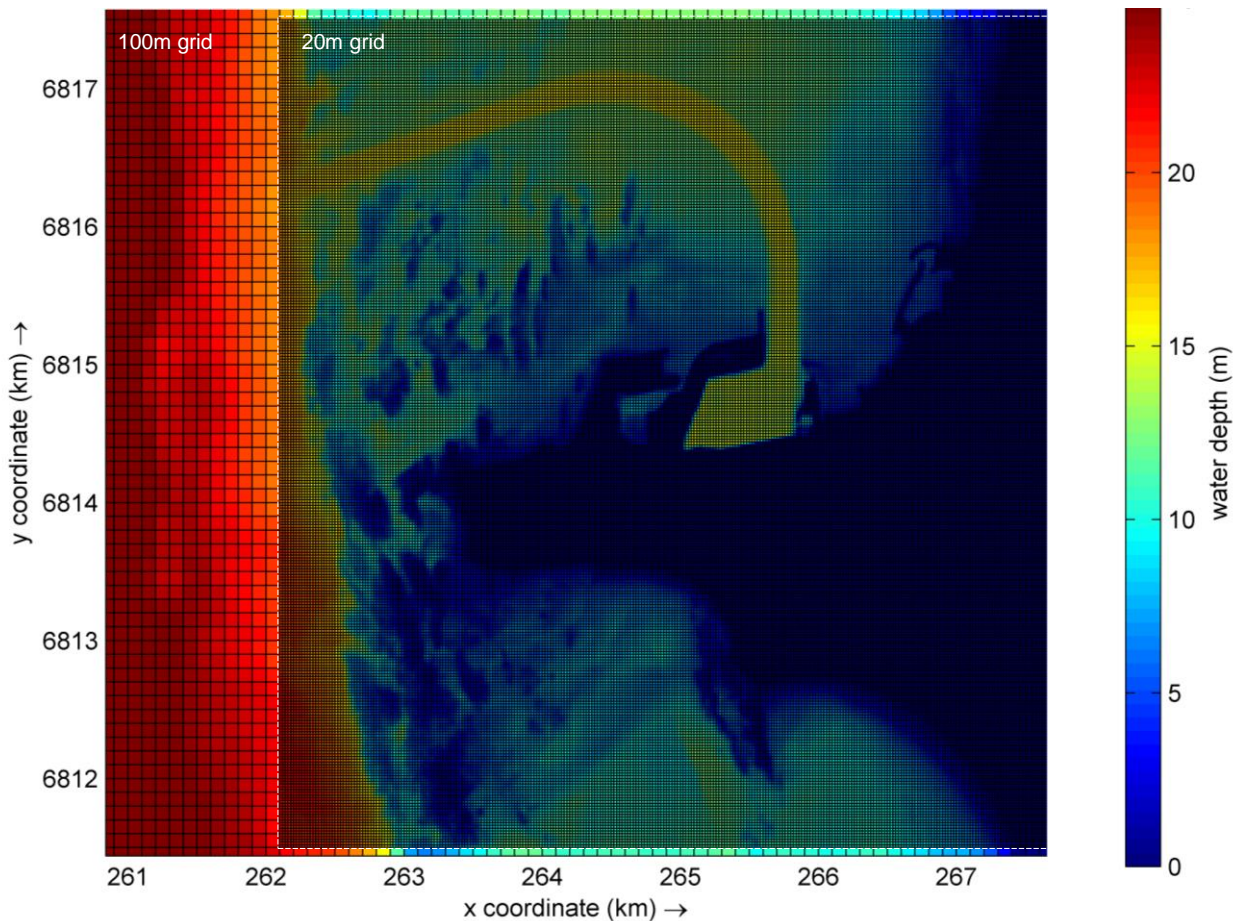


Figure 5-3 Stage 1 – Coastal Model high-resolution bathymetry.

5.2.2 Stage 2 - Nearshore Model

Complementary detailed cross-shore wave modelling was undertaken at 13 representative locations along the coast (Figure 5-4). The model included the use of:

- > Simulating Waves til SHore (SWASH, 2020) – a robust, non-hydrostatic phase resolving model capable of simulating refraction, shoaling, diffraction, bed friction, wave breaking and frequency-direction spectral input. It includes partial and full wave reflectivity off adjacent features (e.g. beach, rock revetment, wall). An illustration of the cross-sectional wave conditions and statistics is provided in Figure 5-10;
- > Parametric reconstruction of storm tide events developing over a 6-day period consistent with historical events duration. The continuous 6-day storm tide event was reconstructed parametrically based on SWASH run results obtained for 7 “steady states” of wave and water level conditions ranging from ambient conditions to design criteria conditions. Each SWASH run simulated a period of 40 minutes, which provided sufficient data to obtain statistics such as average wave overtopping and peak steady water level (including wave set-up). An illustration of the parametric reconstruction of storm tide events is provided in Figure 5-11; and
- > Dune breaching analysis in accordance with SPP2.6, which provides guidance to assess the capacity of a dune to provide protection from inundation based on its cross-sectional area above the design water level.

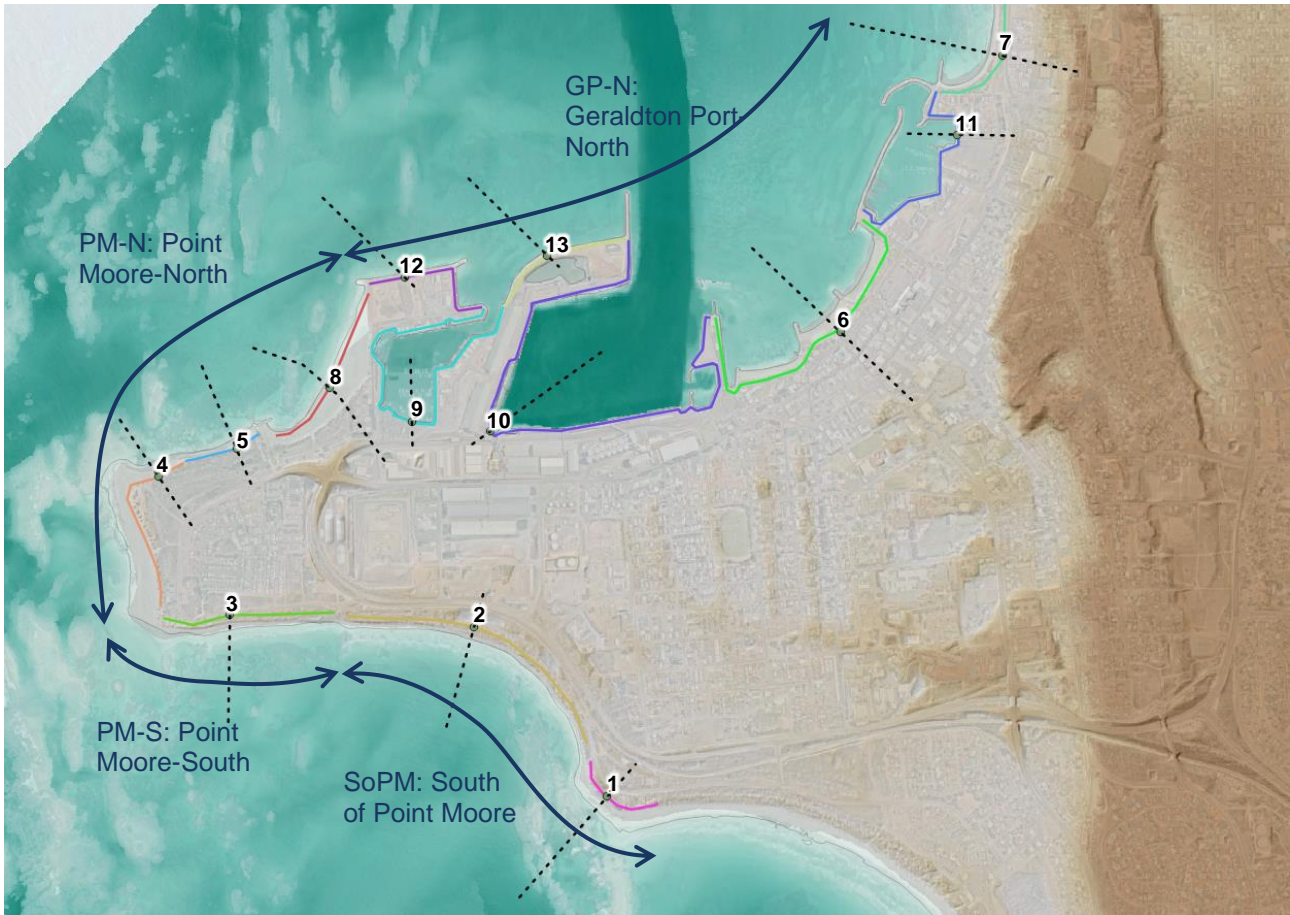


Figure 5-4 Stage 2 - Nearshore Model transects locations (dashed line), applicable shoreline section (coloured lines) and City's CHRMAP areas (solid navy lines).

5.3 Extreme design water level conditions

5.3.1 Review of extreme water level estimates

A review of extreme water levels was conducted, including:

- > Desktop review of existing extreme water levels studies readily available, including
 - CHRMAP (Baird 2019 and MRA 2015, 2016, 2017);
 - UWA (2018, Extreme sea level study); and
 - MWPA (2014, Geraldton Port Guideline).
- > Statistical extreme analysis of the longest water level record provided by the Port to estimate design water level driven by frequent synoptic conditions; and
- > Direct simulation of 500yr ARI tropical cyclone design storm event from DoT's design cyclone study to estimate design water level driven by infrequent tropical cyclone conditions.

5.3.1.1 CHRMAP design water level

The City's CHRMAP design water levels were obtained from (Baird 2019 and MRA 2015, 2016, 2017). The shoreline water levels including wave set-up were obtained for 4 regions along the coast, maintaining consistency with a number of reports covering the study area. The nearshore water level was inferred for these 4 regions by removing an allowance for wave set-up in the range 0.8-1.1m as reported in the supporting references. The resulting nearshore design water levels are summarised in Table 5-3.

Table 5-3 Extreme water level (mAHD, excluding wave set-up) at Geraldton provided by the City's CHRMAP for Geraldton Port - North area (GP-N), Point Moore - North area (PM-N), Point Moore - South area (PM-S) and South of Point Moore area (SoPM), as illustrated in Figure 5-4.

Timeframe	2030				2070				2110			
Area	GP-N	PM-N	PM-S	SoPM	GP-N	PM-N	PM-S	SoPM	GP-N	PM-N	PM-S	SoPM
20yr ARI	1.35	1.35	1.35	1.35	1.67	1.67	1.67	1.67	2.18	2.18	2.18	2.18
100yr ARI	2.30	2.00	1.60	1.60	2.62	2.32	1.92	1.92	3.13	2.83	2.43	2.43
500yr ARI	2.70	2.40	1.90	2.10	3.02	2.72	2.22	2.42	3.53	3.23	2.83	2.93

5.3.1.2 UWA Extreme sea level study

Present day extreme sea level statistics available on (<http://sealevelx.ems.uwa.edu.au>) were calculated from a 59 year (1958-2016) hindcast of sea levels around Australia. The high-resolution numerical model included the effects of astronomical tides, storm surges due to wind and pressure, and seasonal and interannual mean sea level variability (UWA, 2018). The project was undertaken by the Coastal Oceanography Group at the University of Western Australia, funded by the Bushfire and Natural Hazard CRC.

The extreme still water level for this study in Geraldton is shown in Figure 5-5 and summarised in Table 5-4.

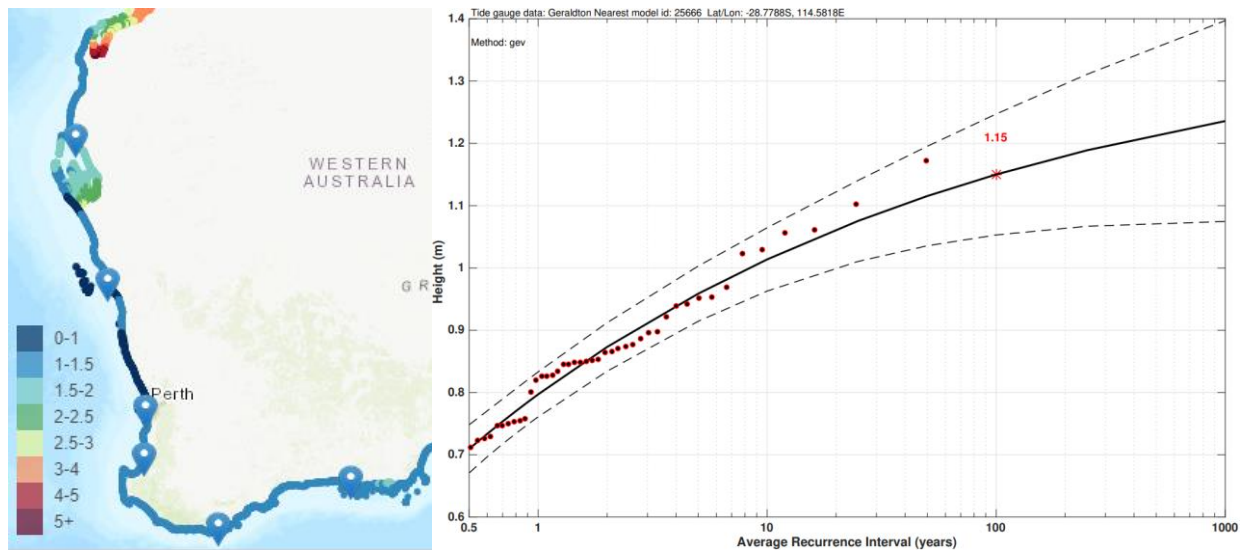


Figure 5-5 Present day extreme still water level (mAHD): 100-year return period (left), design curve (right) [<http://sealevelx.ems.uwa.edu.au/index.php> accessed 6/09/2019].

Table 5-4 Present day extreme water level (mAHD, excluding wave set-up) at Geraldton obtained from UWA extreme water level study.

Storm event	Water Level (95% confidence)
20yr ARI	1.05 (1.12)
100yr ARI	1.15 (1.25)
500yr ARI	1.21 (1.35)

5.3.1.3 MWPA Geraldton Port Guideline

MWPA Geraldton Port Guideline design water levels refer to the levels provided in MWPA (2014) which cited Cardno (2012) as supporting reference. The 1 in 500yr ARI event was derived by extrapolation of the lower design water level estimates as shown in Figure 5-6.

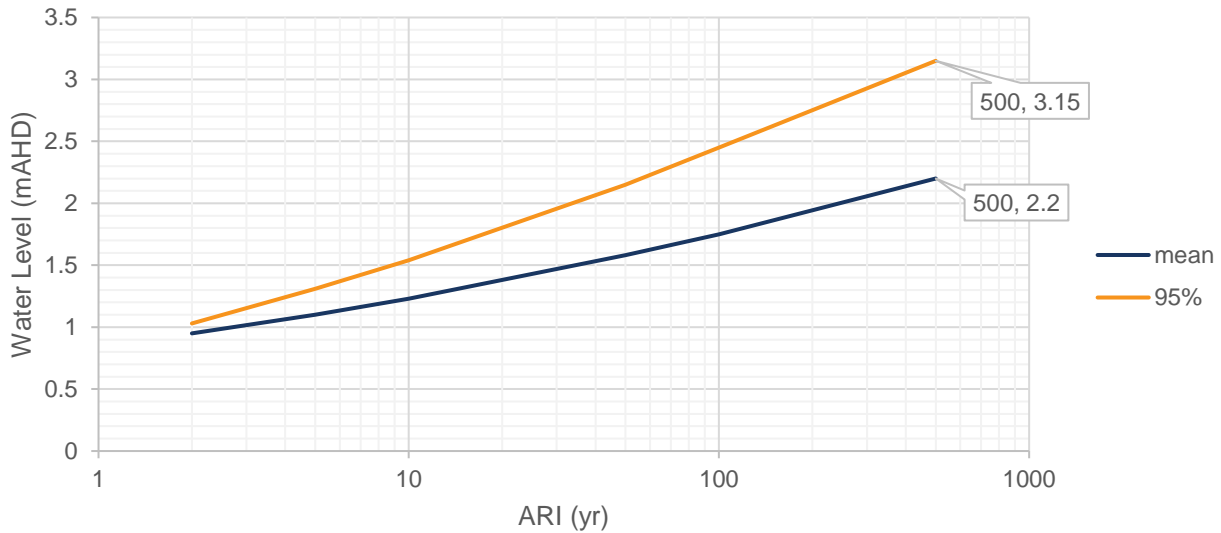


Figure 5-6 Design water level from MPWA (2011) and Cardno (2012), including extrapolation to 1 in 500yr ARI.

Table 5-5 Present day extreme water level (mAHD, excluding wave set-up) at Geraldton obtained from MPWA (2011) and Cardno (2012).

Storm event	Water Level (95% confidence)
20yr ARI	1.40 (1.80)
100yr ARI	1.70 (2.50)
500yr ARI	2.20 (3.15)

5.3.1.4 Extreme value analysis of Geraldton Port tide gage

An extreme value analysis (EVA) of Geraldton Port tide gage was conducted as part of the extreme water level review. 30 years of data (1986/02-2018/03) was analysed using the univariate Peak Over Threshold method and a 7-day period storm clustering threshold (Figure 5-7). Two noteworthy events were identified in this analysis, including the 2004 Boxing Day Tsunami and the July 2010 Strong Cold Front. This analysis enabled the determination of the probabilities of water level for extreme events, as summarised in Table 5-6.

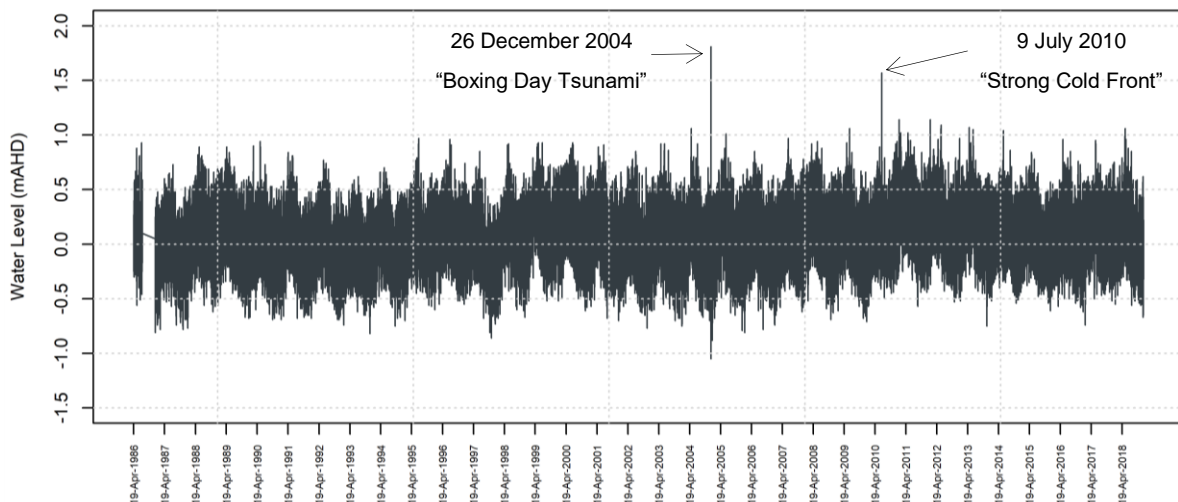


Figure 5-7 Tide gage record at Geraldton Port obtained from the Port of Geraldton.

Table 5-6 Present day extreme water level (mAHD, excluding wave set-up) at Geraldton obtained from the EVA of the port data.

Period (1986/02-2018/03)	Water Level (95% confidence)
20yr ARI	1.35 (1.57)
100yr ARI	1.59 (2.15)
500yr ARI	1.85 (3.03)

5.3.1.5 Direct simulation of 500yr ARI tropical cyclone design storm event

A tropical cyclone storm surge modelling assessment was conducted as part of the review of the above metocean design criteria estimates. The direct simulation of DoT’s 500yr ARI tropical cyclone design storm event for Geraldton was undertaken. The modelling was based on:

- > Tropical cyclone track specifications (i.e. location, central pressure, forward speed, heading) provided by the Department of Transport (Seashore, 2018 Design Storms for Western Australian Coastal Planning: Tropical Cyclones, prepared for the Department of Transport).

This enabled the determination of the extreme water level at the site and wave conditions under this specified event, as shown in Figure 5-8. The peak water level at Geraldton Port tide gage is given in Table 5-7.

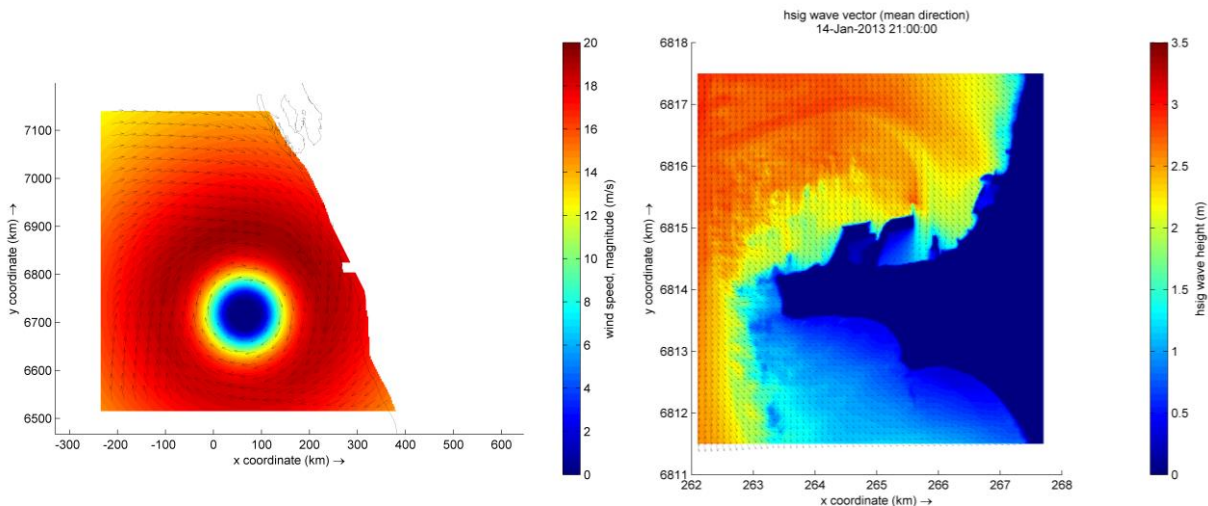


Figure 5-8 Snapshots of tropical cyclone wind field (left) and associated wave conditions (right).

Table 5-7 Extreme water level (mAHD, excluding wave set-up) at Geraldton obtained from the simulation of DoT’s 500yr ARI tropical cyclone design storm event.

Storm event	Water Level
500yr ARI	1.2

5.3.1.6 Summary of the extreme water level review

A summary of extreme water level study results is shown in Table 5-8. The design water level adopted for the CHRMAP are above the other extreme water level estimates, but within the 95% confidence interval of the tide gage record extreme value analysis.

Discrepancies in design water level are common, as every study has their own limitations and assumptions. However, design water level is an important parameter affecting inundation and wave overtopping hazard and subsequent risk to the exposed assets and their functions.

In accordance with the precautionary principle, and in the context of the inundation study, it is probably conservative to adopt the highest design water level estimates identified in the extreme water level review. Accordingly, the design water level established in the CHRMAP and summarised in summarised in Table 5-3 have been subsequently adopted for the Geraldton CBD and Port Flood and Inundation Study.

Table 5-8 Summary of extreme water level (mAHD) for the 500yr ARI “offshore” design water level for present day conditions.

Water level (95% confidence)	Region	Type of analysis	Comment / source of information
2.70 (not provided)	“Geraldton Port – North”	Synthetic cyclone modelling	Based on CHRMAP corrected to subtract wave set-up
2.40 (not provided)	“Point Moore – North”	Synthetic cyclone modelling	Based on CHRMAP corrected to subtract wave set-up
2.2 (3.15)	Port area	10yr water level record extreme value analysis	Interpolated from design water level provided up to 100yr ARI in Port guidelines MWPA (2014) and cited Cardno (2012) report
1.85 (3.03)	Port area	30yr water level record extreme value analysis	Inundation hazard study (this study).
1.21 (1.35)	Geraldton	Synthetic cyclone modelling + 60 year hindcast	UWA extreme sea level rise study (2018)
1.2 (n/a)	“Geraldton Port – North”	Single track simulation at a fixed background tide level (MHHW).	DoT tropical cyclone design storm (2018). The track parameters are being revised by the authors as the intensity of this cyclone is considered to be too low (personal communication, May 2020).

5.3.2 Adopted design water level

As indicated above, the design water level established in the CHRMAP have been adopted for the Geraldton CBD and Port Flood and Inundation Study. The nearshore water levels were derived from storm tide levels modelling results undertaken by MP Rogers (and utilised in CHRMAP), excluding wave set-up. These nearshore water levels are summarised in Table 5-3. These nearshore water levels were suitable to be applied directly as boundary conditions to the high-resolution Stage 1 - Coastal Model so adequate water level is provided for the simulation of the wave transformation over key bathymetric features in the area, including shallow reefs and deep navigation channel.

Similarly, these nearshore water levels were suitable to be applied directly as boundary conditions to the Stage 2 - Nearshore Model so adequate water level is provided for the simulation of the detailed wave transformation over the beach and coastal protections to estimate wave set-up, wave run-up and wave overtopping.

5.4 Extreme design wave conditions

An extreme value analysis (EVA) of Geraldton Port wave data located at the entrance of the outer channel was conducted. 20 years of data (1999/03 to 2018/12) was analysed using a bivariate Peak Over Threshold method and a 7-day period storm clustering threshold. This enabled the determination of joint probabilities of wave height and period for extreme events, as summarised in Table 5-9.

Stage 1 wave modelling was undertaken using the offshore design wave criteria at Beacon 1 along the model boundary condition for 2 wave directions (north-westerly and south-westerly) and the adopted nearshore design water level to enable the representation of the sheltering effect of the study area by Point Moore and estimate adequate design wave conditions for the Stage 2 wave modelling.

Table 5-9 Offshore extreme wave conditions at Beacon 1 (outer channel entrance) obtained from the EVA of the Port data.

Period (1999/03-2018/12)	Hs (m)	Tp (s)
20yr ARI	4.8	17.0
100yr ARI	5.1	18.1
500yr ARI	5.4	18.9

5.5 Storm tide scenarios

A total of 18 Stage 1 – Coastal Modelling simulations were undertaken to characterise the 9 storms tide design scenarios adopted under both the south-westerly and north-westerly conditions, as illustrated in Figure 5-9. The worst-case scenario of both conditions was kept as adopted design conditions.

A total of 819 Stage 2 – Nearshore Modelling simulations were undertaken to characterise the 9 storms tide design scenario adopted for 7 stages of the storm tide in 13 locations, as illustrated in Figure 5-10.

Each storm tide inundation scenario identified in the adopted design scenarios (Table 5-2) were defined by nearshore water level time series and mean overtopping discharge, as shown in Figure 5-11.

The maximum mean overtopping discharges and maximum overtopping volumes are reported in Table 5-10 and Table 5-11 respectively.

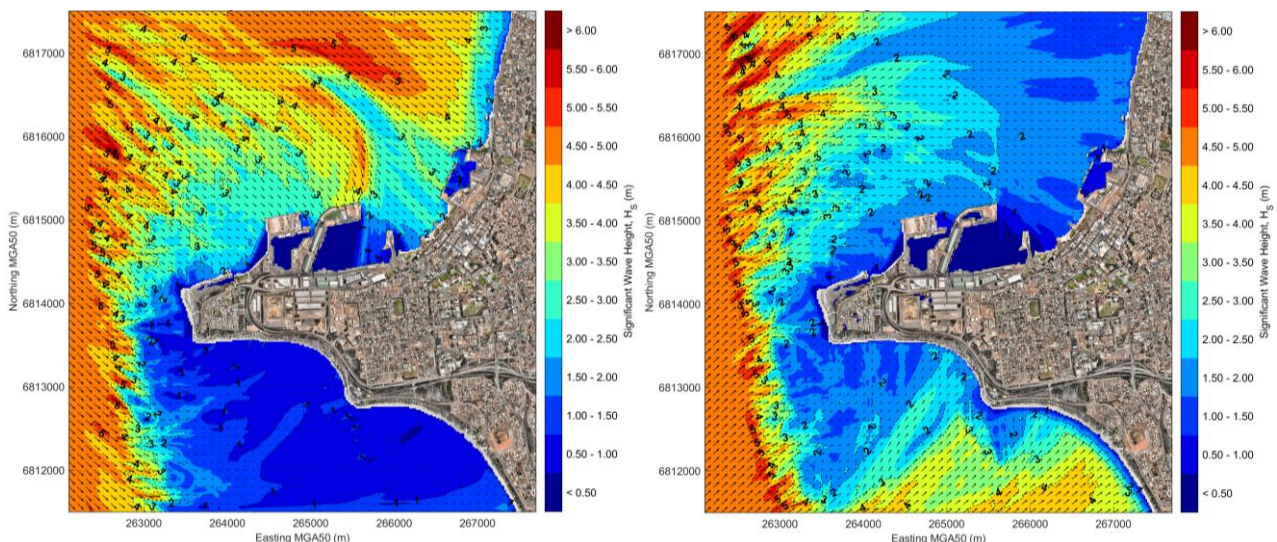


Figure 5-9 Examples of extreme wave conditions for south-westerly conditions (left) and north-westerly conditions (right).

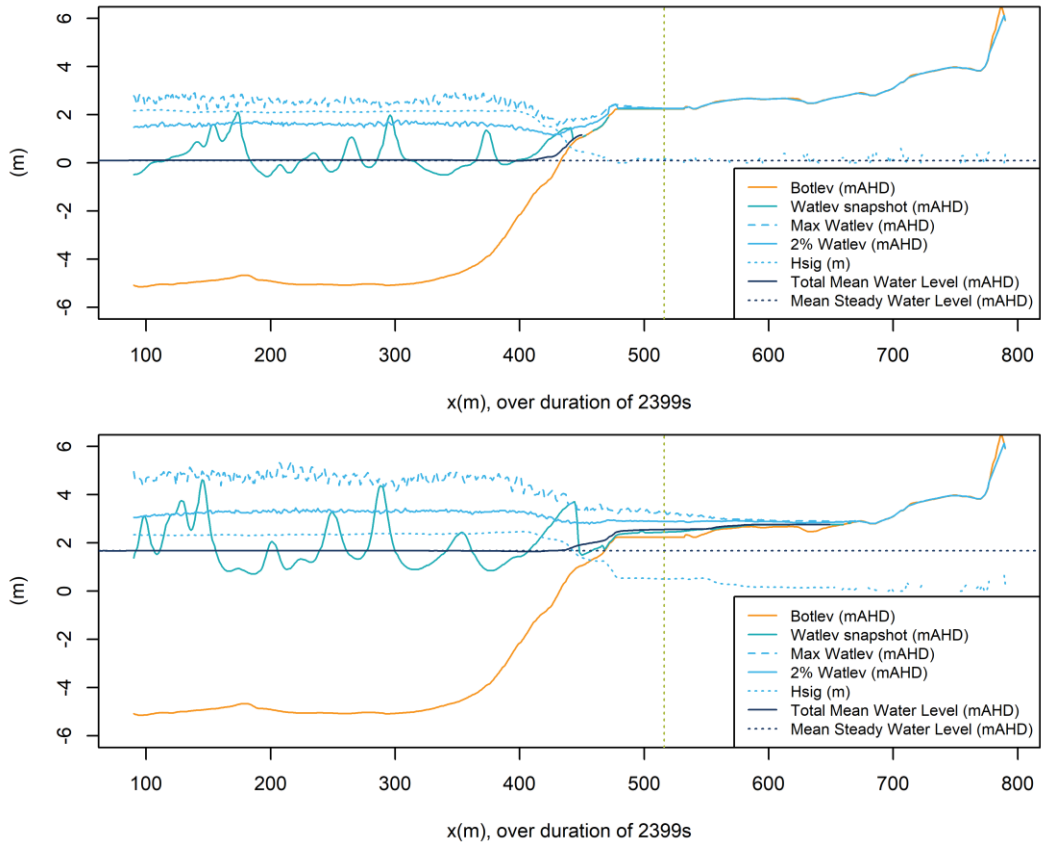


Figure 5-10 Examples cross-sectional wave conditions and statistics for 2 different water level inputs.

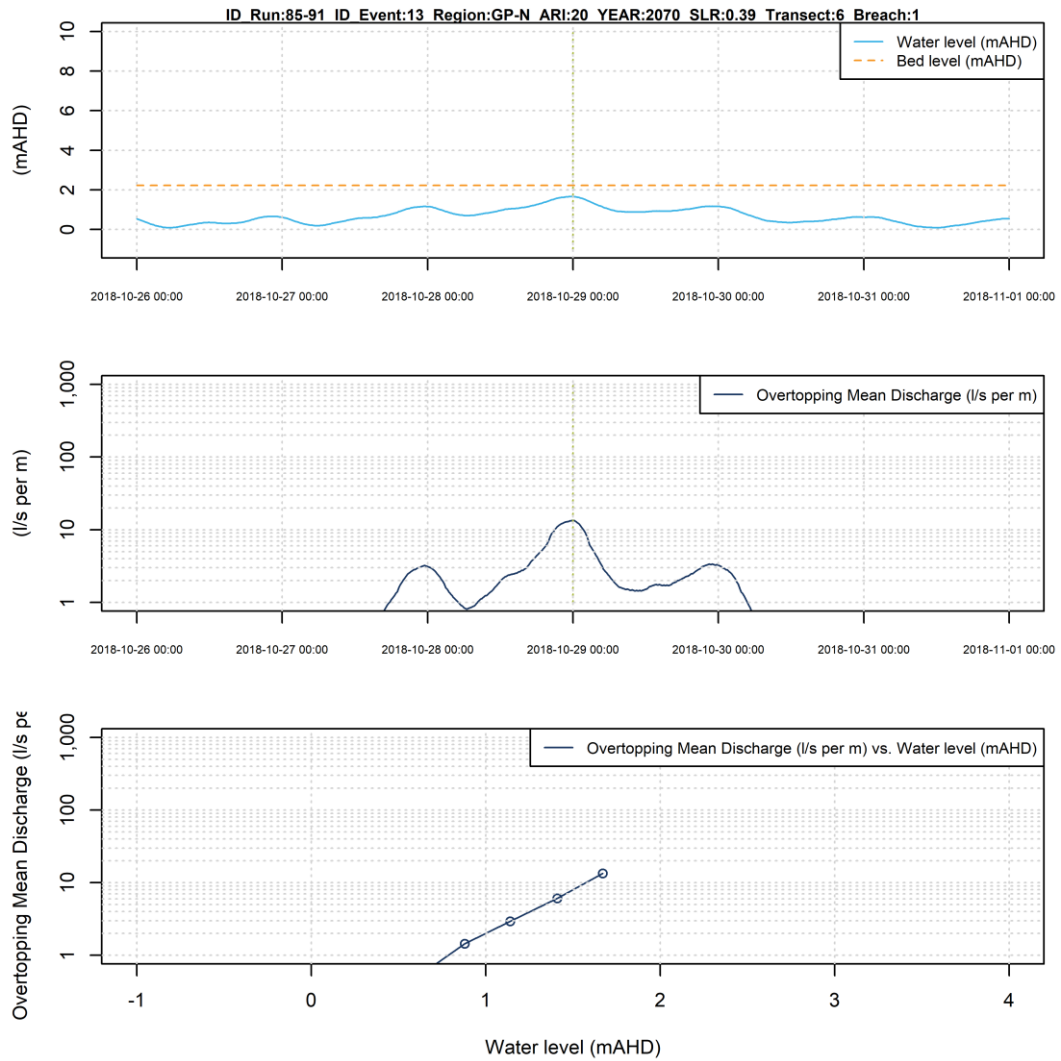


Figure 5-11 Example of parametric reconstruction of storm tide events.

Table 5-10 Maximum mean overtopping discharge (l/s per m) for each transect and scenario.

ARI Transect / Year	20yr			100yr			500yr		
	2030	2070	2110	2030	2070	2110	2030	2070	2110
1	0	0	0	0	0	0	0	0	17
2	0	0	0	0	0	0	0	0	0
3	0	0	6	0	3	16	4	9	74
4	32	32	49	42	86	110	104	96	263
5	39	57	116	117	124	125	129	135	141
6	9	13	30	35	50	59	55	59	64
7	7	24	24	30	40	59	46	56	55
8	30	54	56	58	61	61	65	66	81
9	3	3	3	3	3	3	3	3	3
10	0	0	1	1	2	5	2	4	4
11	2	2	2	3	4	4	4	4	4
12	0	0	7	14	57	261	78	207	528
13	0	0	2	3	8	45	11	33	141

Table 5-11 Maximum maximum overtopping discharge (l per m) for each transect and scenario.

ARI	20yr			100yr			500yr		
	2030	2070	2110	2030	2070	2110	2030	2070	2110
1	0	0	0	0	0	0	0	0	6,503
2	0	0	0	0	0	0	0	0	0
3	0	0	1,670	0	778	3,151	1,510	2,519	4,110
4	3,913	4,652	5,792	5,344	6,241	8,377	7,317	8,163	9,812
5	5,093	6,704	8,781	7,446	9,815	12,495	10,617	12,330	16,252
6	1,094	2,957	4,773	6,090	8,195	11,278	9,628	11,384	12,773
7	390	849	3,761	4,786	5,708	9,085	5,838	5,928	8,954
8	5,084	7,177	9,963	9,120	8,451	12,653	10,406	11,432	15,725
9	1,641	2,090	2,622	2,992	3,362	3,855	3,671	3,933	4,011
10	0	0	657	922	957	1,812	1,259	1,652	2,252
11	296	834	1,537	1,957	2,185	2,346	2,181	2,562	3,083
12	0	0	883	1,639	3,147	3,942	3,239	4,006	6,072
13	0	0	1	180	2,158	3,509	1,242	2,035	5,074

Table 5-12 Acceptable limits of overtopping (Eurotop, 2018)

Category	Hazard type and reason	Mean discharge (l/s per m)	Maximum volume (l per m)
Structural design of breakwaters, seawalls, dikes and dams	Rubble mound breakwaters; $H_{m0} > 5$ m; no damage	1	2,000-3,000
	Rubble mound breakwaters; $H_{m0} > 5$ m; rear side designed for wave overtopping	5-10	10,000-20,000
	Grass covered crest and landward slope; maintained and closed grass cover; $H_{m0} = 1 - 3$ m	5	2,000-3,000
	Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; $H_{m0} = 0.5 - 3$ m	0.1	500
	Grass covered crest and landward slope; $H_{m0} < 1$ m	5-10	500
	Grass covered crest and landward slope; $H_{m0} < 0.3$ m	No limit	No limit
Property behind the coastal defence	Significant damage or sinking of larger yachts; $H_{m0} > 5$ m	>10	>5,000 – 30,000
	Significant damage or sinking of larger yachts; $H_{m0} = 3-5$ m	>20	>5,000 – 30,000
	Sinking small boats set 5-10 m from wall; $H_{m0} = 3-5$ m Damage to larger yachts	>5	>3,000-5,000
	Safe for larger yachts; $H_{m0} > 5$ m	<5	<5,000
	Safe for smaller boats set 5-10 m from wall; $H_{m0} = 3-5$ m	<1	<2,000
	Building structure elements; $H_{m0} = 1-3$ m	≤ 1	<1,000
	Damage to equipment set back 5-10m	≤ 1	<1,000
People and vehicle	People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
	People at seawall / dike crest. Clear view of the sea. $H_{m0} = 3$ m	0.3	600
	$H_{m0} = 2$ m	1	600
	$H_{m0} = 1$ m	10-20	600
	$H_{m0} < 0.5$ m	No limit	No limit
Cars on seawall / dike crest, or railway close behind crest	$H_{m0} = 3$ m	<5	2000
	$H_{m0} = 2$ m	10-20	2000
	$H_{m0} = 1$ m	<75	2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous	

5.6 Tsunami inundation scenarios

Tsunami modelling was conducted using inputs from the 2018 Probabilistic Tsunami Hazard Assessment for Australia (PTHA18) dataset detailed in GA (2018). This dataset was derived from the simulation of hundreds of thousands of possible tsunami scenarios from key earthquake sources in the Pacific and Indian Oceans, and models the frequency with which these occur.

In 2018, Geoscience Australia updated and released the Probabilistic Tsunami Hazard Assessment (PTHA), which outlines the tsunami hazard for all of Australia and its offshore territories. The 2018 update reflects improvements in science and technology and includes insights gained from tsunamis that have occurred since the last update a decade ago. The PTHA includes data for more than half a million earthquake-tsunami scenarios in Australia. It reflects advances in understanding of how frequently large earthquakes that can cause tsunamis occur and the uncertainties in these frequencies. Additionally, it has data for more locations around Australia (Davies, 2017).

The PTHA18 model provided offshore (deep water) tsunami water level timeseries as input to the Delft3D used to propagate the tsunami in coastal waters and nearshore in accordance with AIDR (2018). In turn, the Delft3D model provided the nearshore boundary conditions of the local inundation model.

The PTHA18 inputs were kindly provided by GA author Gareth Davies for the Geraldton inundation study. It encompassed earthquake information, tsunami initial conditions, and offshore (deep water) tsunami time-series at every hazard point, including the exceedance-rate. PTHA18 tsunami time-series are illustrated in Figure 5-12. They were further modulated by a typical tide (MHHW) and sea level rise allowance along the Delft3D model offshore boundary as illustrated in Figure 5-12.

A total of 9 Stage 1 – Coastal Modelling simulations were undertaken to characterise the 9 tsunami scenarios adopted, as illustrated in Figure 5-13. The resulting simulated nearshore water level time series are illustrated in Figure 5-13 and peak values for each design scenario summarised in Table 5-13.

Each tsunami inundation scenario identified in the adopted design scenarios (Table 5-2) were defined by nearshore water level time series, as shown in Figure 5-13.

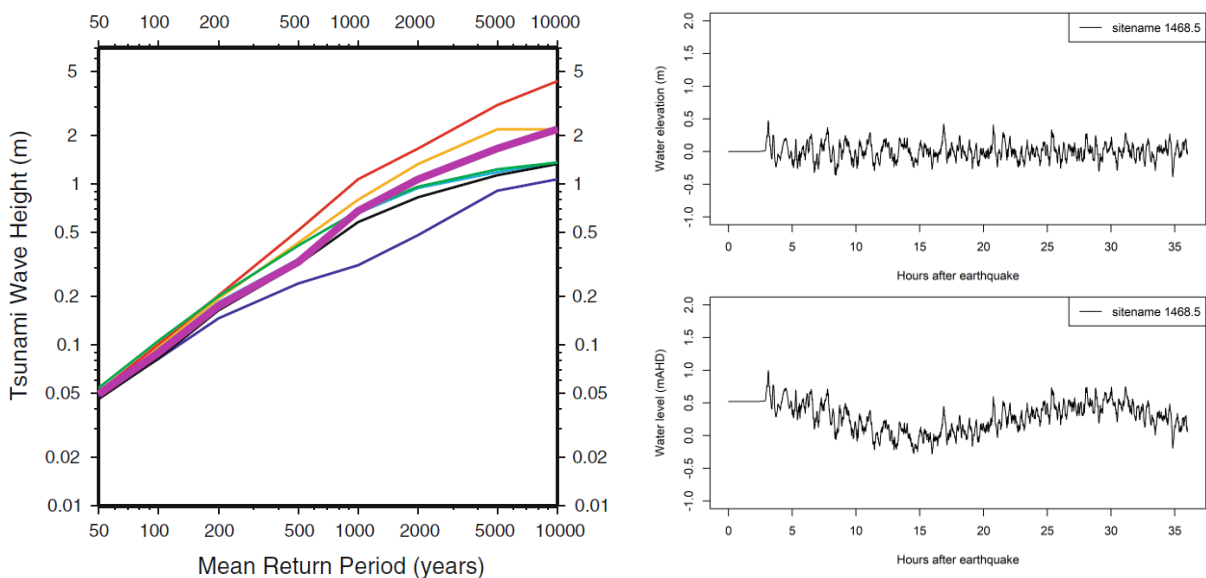


Figure 5-12 Tsunami hazard curves off the coast of Geraldton for different models after Burbidge et al. (2009) (left) and PTHA18 tsunami stage provided by GA for this project (right-top), including Delft3d tsunami stage input modulated by a typical tide (MHHW) and sea level rise allowance (right-bottom).

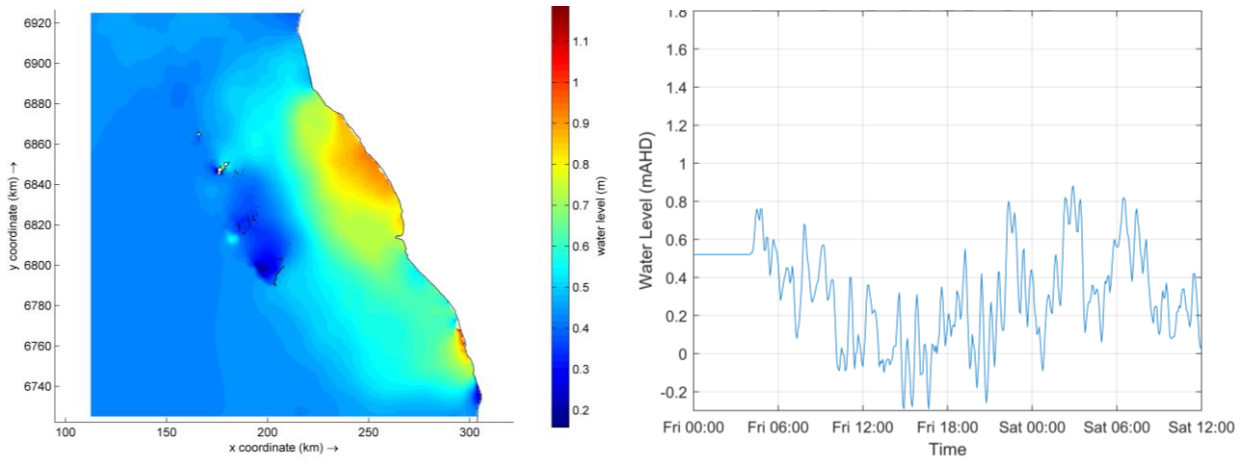


Figure 5-13 Example of Delft3D tsunami water level snapshot (left) and timeseries result at a nearshore location (right).

Table 5-13 Nearshore tsunami design water level (m) AHD).

Timeframe	2030	2070	2110
1:20 ARI	0.6	0.9	1.4
1:100 ARI	0.7	1.0	1.5
1:500 ARI	0.9	1.2	1.7

6 Preliminary hydraulic model

A preliminary 2D hydraulic model was set up using the hydraulic modelling package TUFLOW. The purpose of this model was to validate the results from the RORB model described in Section 4 and to ensure that the parameters are suitable for use in the more detailed Integrated Hydraulic Model of the Study Area.

6.1 Model boundary

The model boundary encompasses the entire catchment shown in Figure 4-1 to ensure consistency with the RORB model. The coastal boundary was extended to the west to allow for surface water outflows.

6.2 Cell size

A computational grid cell size of 4m was used to represent the topography. This cell size is small enough to represent urbanised features in the CBD such as roads, ensuring that major flow paths are adequately represented in the 2D domain.

6.3 Topography

The DEM described in Section 3.1 was used to represent the model topography for the Study Area. Building footprints derived from the LiDAR data was used to elevate the respective grid cells, allowing water to flow around building footprints.

6.4 Roughness

The roughness was applied to the model as Manning's 'n' values. The spatial distribution of roughness was prepared using the Land Use categories in Section 3.2, which was simplified into eight categories. These, along with the respective Manning's 'n' value are shown in Figure 6-1.

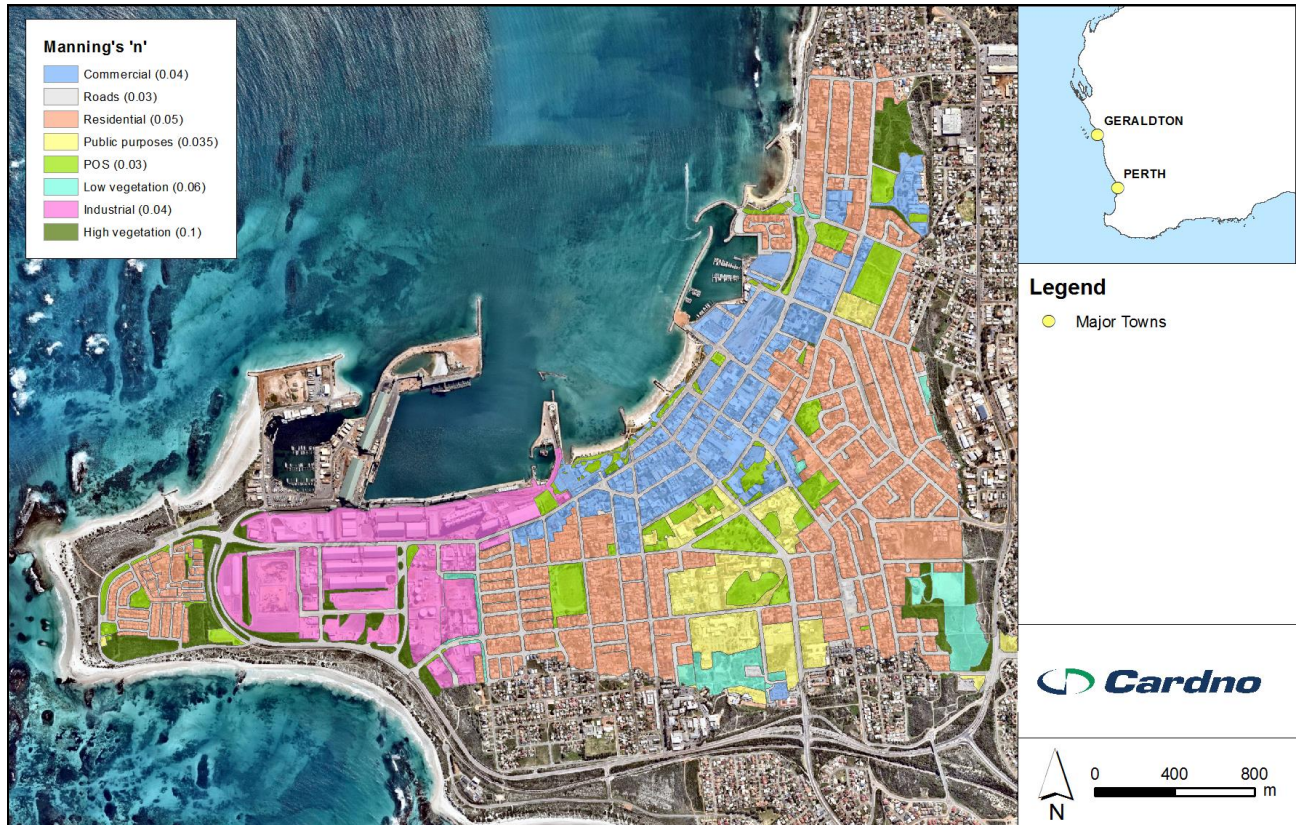


Figure 6-1 Manning's 'n' roughness values used in TUFLOW.

6.5 Boundary condition and initial water level

In the preliminary Hydraulic model, the initial water level and ocean boundary condition was set to the Mean High Low Water level (MHLW) to represent the tidal level as measured at Geraldton Port.

6.6 Rainfall

The critical storms as identified from the RORB model (discussed in Section 4.9) were simulated in the hydraulic model using a direct rainfall approach, whereby rainfall hyetographs were directly applied to the grid cells. A number of durations and temporal patterns were modelled to ensure all peak flood levels were recorded across the study area. These critical storms are presented in Table 6-1.

Table 6-1 Critical storms simulated in the hydraulic model.

AEP	Storm durations (hours)	Temporal Patterns
5%	0.5	TP16
	2	TP13, TP20, TP18
1%	0.5	TP25
	1.5	TP29, TP28, TP22
	2	TP21
0.2%	1	TP29, TP30
	1.5	TP28
	2	TP22, TP24

6.7 Rainfall losses

The rainfall losses which accounted for impervious fractions used in the RORB model were directly applied to the TUFLOW model using a 'soils layer', which was set up with an initial/continuing loss value for each RORB sub-catchment. This layer also included the infiltration basins which were set to have an initial loss of 45mm and a continuing loss of 5m/day (208mm/hr) as per common design practice for similar soil types in WA. This 'lumped' approach of applying rainfall losses was made to ensure consistency with the RORB model. Refer to Section 4.5 for a description on how these loss values were derived.

6.8 Model validation

The RORB model sub-catchments G and H (Figure 4-1) were chosen as catchments to validate against TUFLOW results. The hydrographs from the respective models for selected design storms are presented in Figure 6-2, Figure 6-3 and Figure 6-4 respectively. It can be seen that the peak flows and time of concentrations show good correlation between the two models. The RORB model slightly underestimates the flood volume, however, as flooding in the catchment is likely to be driven by peak flow and the TUFLOW model will be used to estimate flood depths across the Study Area, this does not bear significant impact for the Integrated Hydraulic Modelling phase of the Study.

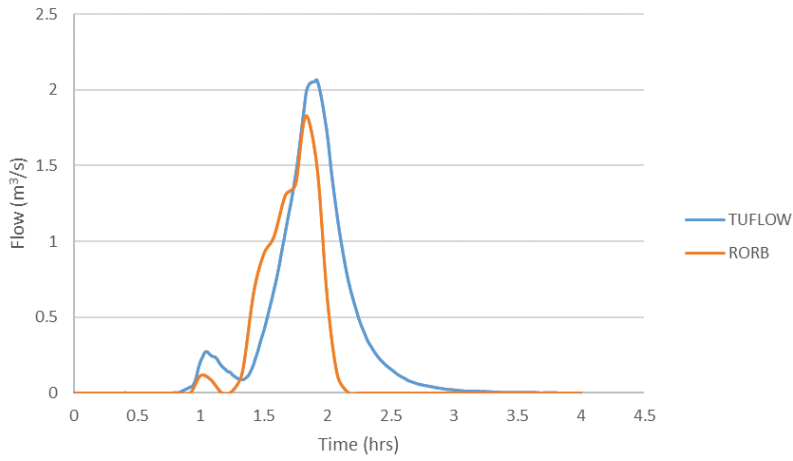


Figure 6-2 Sub-catchments G and H – 5% AEP (2hr TP20).

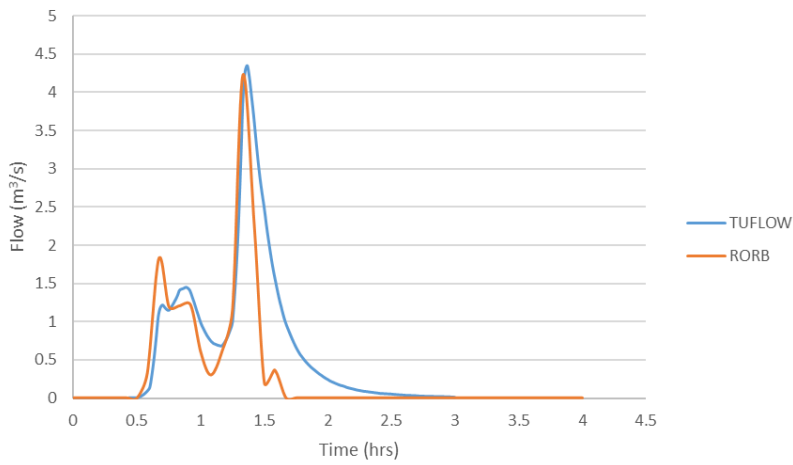


Figure 6-3 Sub-catchments G and H – 1% AEP (1.5hr TP28).

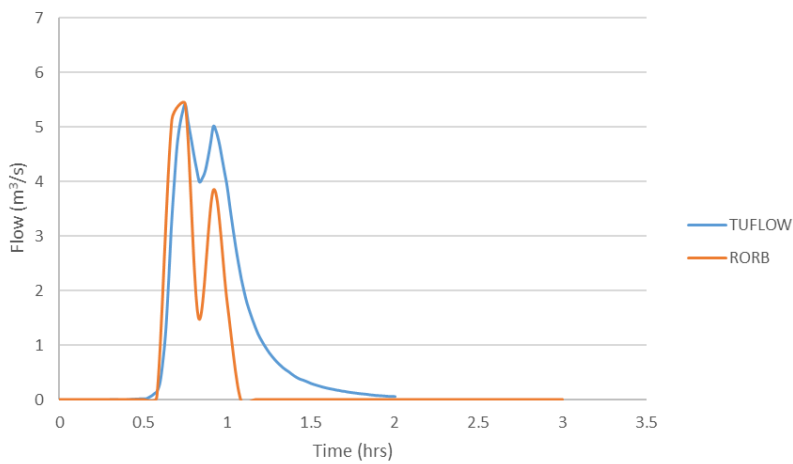


Figure 6-4 Sub-catchments G and H – 0.2% AEP (1hr TP29).

7 Integrated hydraulic model

7.1 Model setup

The preliminary hydraulic model was developed, primarily for use as a verification tool to be used in conjunction with the hydrological model and develop rainfall loss rates and material layers within TUFLOW.

The integrated hydraulic model setup involved taking all preliminary hydraulic model input layers, coastal outputs and results and building integrated hydraulic models. In total, three models were built. These included a storm tide model, tsunami model and a rainfall only model. The modelled events as part of these models are shown in Table 7-1 below.

Table 7-1 Modelling Scenarios

Event	Rainfall			Storm tide & Rainfall			Tsunami		
	2030	2070	2110	2030	2070	2110	2030	2070	2110
Climate Change Allowance									
20yr	✓	✓	✓	✓	✓	✓	✓	✓	✓
100yr	✓	✓	✓	✓	✓	✓	✓	✓	✓
500yr	✓	✓	✓	✓	✓	✓	✓	✓	✓

Early iterations of model runs indicated that, due to the depth of water in some off shore locations being greater than 8 metres, a number of model stability issues were apparent. To overcome these instabilities, the sea bed level offshore was raised to -2mAHD and TUFLOW quadtree mesh implemented. The use of a quadtree mesh allows users to implement a flexible grid cell size within the model area. All offshore locations were modelled with a 16m cell size, reducing to 2m for onshore locations, transitioning down in cell size through the wave break zone. This is demonstrated below in Figure 7-1.



Figure 7-1 TUFLOW quadtree mesh transition.

7.2 Stormwater network

The stormwater network within the hydraulic model was based on the supplied data. The data provided contained information in regards to pipe characteristics such as length, materials, size and direction. Once this data had been collated it was then subject to a number of checks. These checks included;

- Pipe network continuity;
- Pipe underground/cover;
- Pipe flow direction;
- Pipe invert levels;
- Pipe outlet at or near ground level;
- Inlets located in low points.

There are some aspects of the data such as invert values which have been partially surveyed and/or pipe lengths/size which were missing data. These have been populated by extrapolating from existing data.

7.3 Boundary conditions

Due to the number and type of inflows a variety of boundary conditions were implemented within the hydraulic model. The boundary conditions for each of the models are described below in Table 7-2.

It was assumed that peak rainfall and peak storm tide were coincident, as the critical storm durations were relatively short compared to the time scale of the storm tide and there was no significant time lag in the catchment stormwater response.

Table 7-2 TUFLOW model boundary conditions.

TUFLOW Model	Boundary Condition Description
Rainfall	<p>The rainfall only model uses a set tailwater level set to the MHWS of the greater Geraldton area. As such, this was set to 0.27mAHD for the 2030 climate events. As per the proposal the tailwater level was set to 0.67mAHD and 1.17mAHD for the 2070 and 2110 climate change events respectively. This ensures that appropriate sea level rise predictions are accounted for.</p> <p>Similar to the storm tide modeling rainfall was applied via a rain on grid RF layer. The boundary conditions for the rainfall model are shown in Figure 7-2.</p>
Storm Tide & Rainfall	<p>To model storm tide inflows, two boundary conditions were required. Firstly, flows entering the study area as a result of waves overtopping the coastal foredunes were modelled as SA All inflows. This means that the overtopping flows recorded in the coastal modelling were evenly distributed to all cells covered by individual polygons within this SA layer.</p> <p>In order to model the tailwater conditions as a result of storm tide levels, a combination of HX BC lines, HT BC points and 1D nodes were used. Rainfall within this model was applied via rain on grid RF layers. This applies the rainfall hyetograph evenly across the study area. The boundary conditions for the storm tide model are shown in Figure 5-4 and Figure 7-2. AEP events were combined (1% with 1%) and peaks matched, assuming worst case.</p>
Tsunami	<p>Within the tsunami model, boundary conditions, similar to the storm tide model are a combination of HX BC lines, HT BC points and 1D nodes. Rainfall was not applied to this model. The boundary conditions for the tsunami model are shown in Figure 7-3.</p>

7.4 Initial conditions

At the start of the selected modelling period, the initial water level was set to the corresponding tailwater level for the respective scenario and ARI. This ensured stable modelling conditions at the commencement of each model run.



Figure 7-2 Rainfall boundary conditions.



Figure 7-3 Tsunami boundary conditions.

8 Hydraulic model results

8.1 Background

Mapping has been prepared as required by the project brief. Mapping of each event provides a clear indication of flooding behaviour within the catchment. The maps showing flood surface (WSL) and depth are provided in Appendix B. These outputs have also been provided in ARC/INFO ASCII grid format (.ASC) to the City of Greater Geraldton. General results regarding each of the three hydraulic modelling scenarios are further detailed below.

8.2 Rainfall only

Within the rainfall modelling, the 14 design events selected as part of the hydrology (refer to Section 6) were run through the hydraulic model for each ARI and climate change event. In total, 42 events were modelled. For each ARI and climate change combination, a maximum flooding envelope was created.

Flooding behaviour within the catchment is characterised by local flooding within individual allotments without any clearly defined flow paths. Shown below in Figure 8-1 is a representative design event for the 100year ARI (2070) event. As shown by the Figure, local flooding mechanisms dominate the catchment.

In all ARI events, the majority of the stormwater network is at 100% capacity at the peak of the modelling period and as a result, is unable to convey all flows. It is estimated that the likely capacity of the stormwater network is around the 5year ARI event.



Figure 8-1 Peak rainfall extent 100year ARI (2070) event. High resolution maps provided in Appendix B.

8.3 Storm tide & Rainfall

As part of the storm tide modelling, the design events selected as part of the hydrology were further refined to best represent catchment conditions. Using the rainfall modelling, for each ARI and climate change scenario, 3 design events were used. This was done to reduce model run times while still providing a realistic representation of flooding behaviour within the catchment. For the storm tide modelling, a total of 27 events were run through the hydraulic model. Using these model runs, for each ARI climate change combination, a maximum flooding envelope was created.

Based on results from the hydraulic modelling, in all ARI events, the tidal levels as a result from the storm tide breach the foreshore into urban areas and further exacerbate flooding caused by the rainfall. The 20year ARI (2110), 100year ARI (2110) and 500year ARI (2110) event flood extents are shown below in Figure 8-2, Figure 8-3 and Figure 8-4 respectively. Values within ocean areas are not shown.

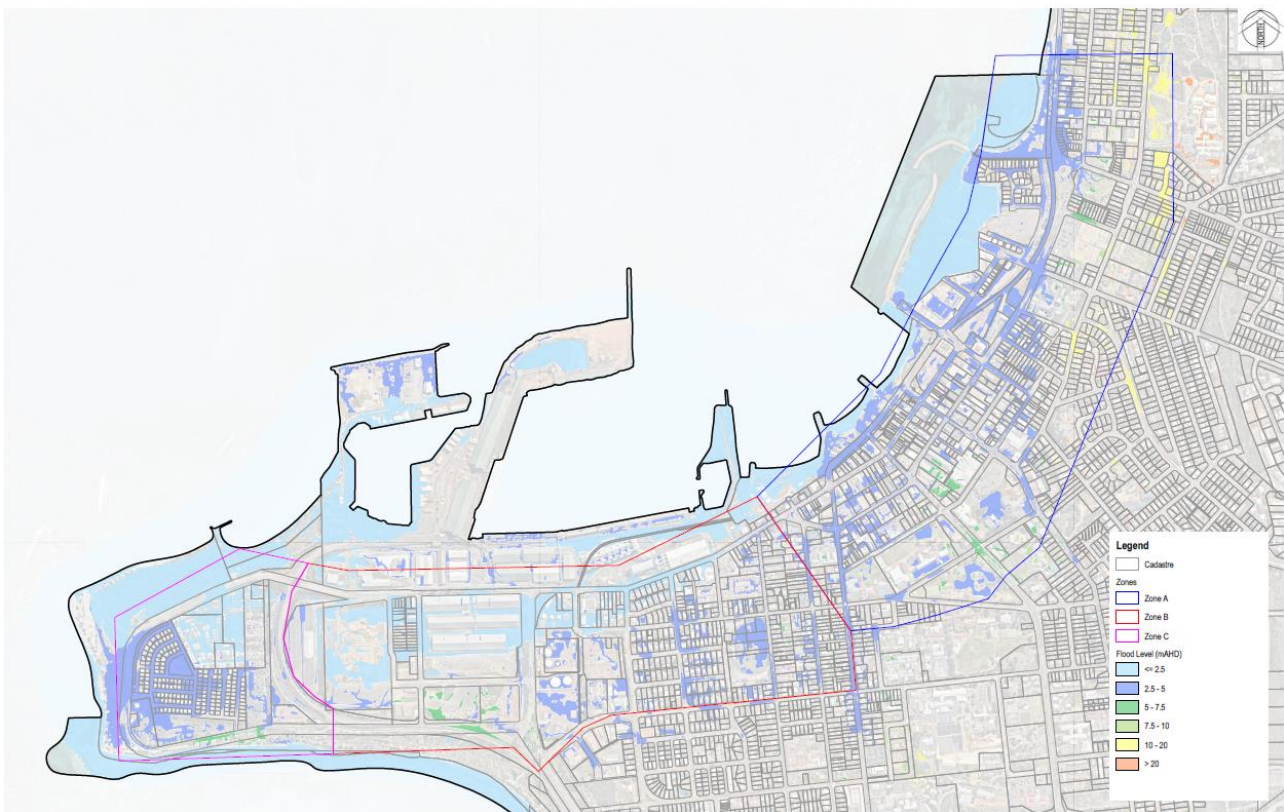


Figure 8-2 Peak storm tide & rainfall extent 20year ARI (2110) event. High resolution maps provided in Appendix B.

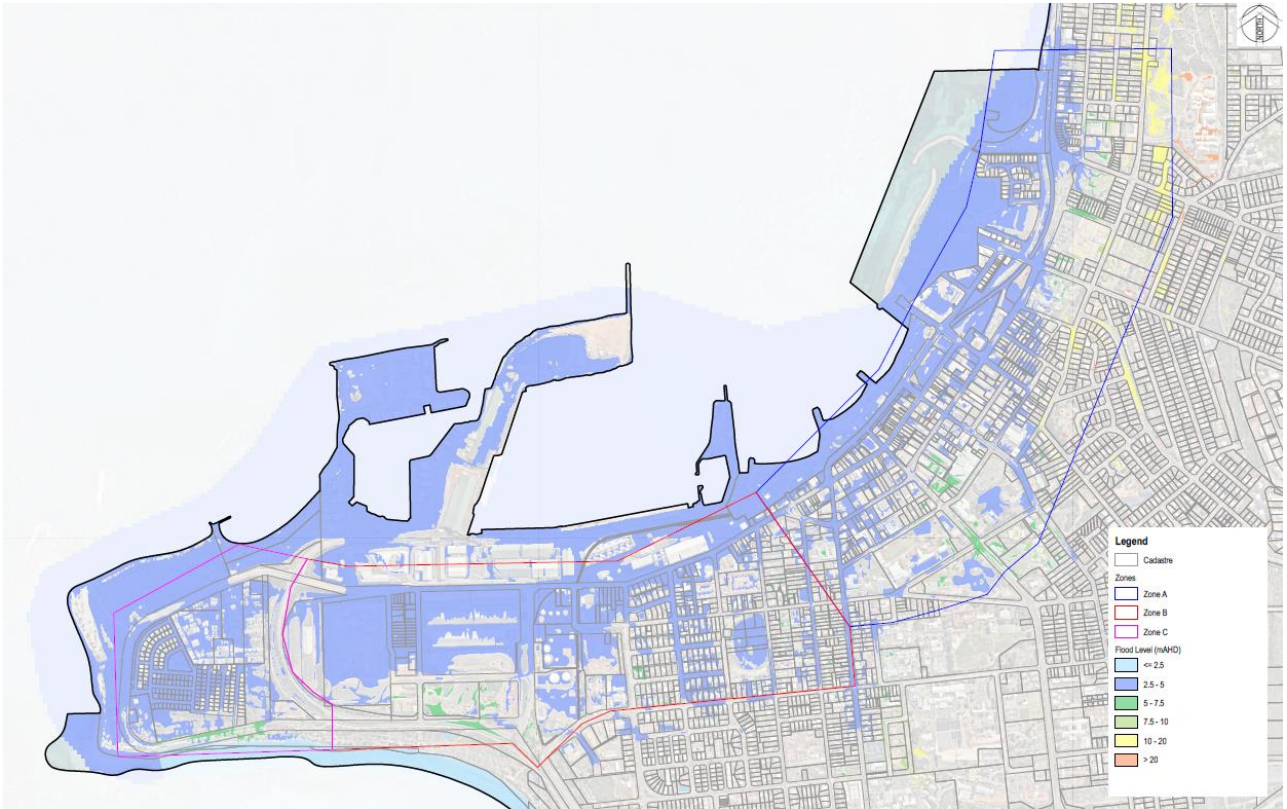


Figure 8-3 Peak storm tide & rainfall extent 100year ARI (2110) event. High resolution maps provided in Appendix B.

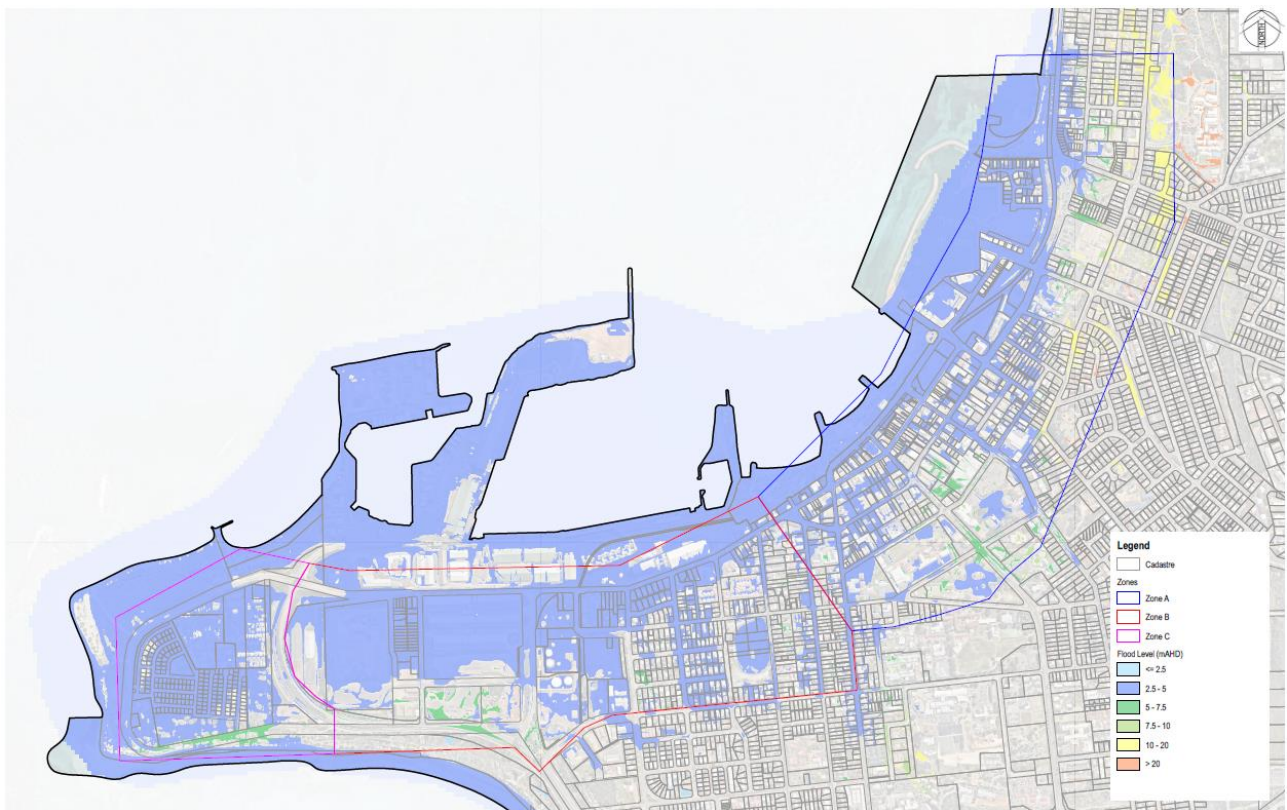


Figure 8-4 Peak storm tide & rainfall extent 500year ARI (2110) event. High resolution maps provided in Appendix B.

8.4 Tsunami

Based on the results from the hydraulic model it was observed that, generally, peak tsunami levels were contained within the foreshore. Shown below in Figure 8-5 is the peak 500year ARI (2110 climate) flooding extent. Minor breakout flows are observed in some areas as shown. This is attributed to either backflow from the stormwater network causing minor flooding, or this may be due to some areas being lower than the initial water level of the respective model as shown in Figure 8-6.



Figure 8-5 Peak Tsunami Extent 500year ARI (2110) event. High resolution maps provided in Appendix B.



Figure 8-6 Terrestrial elevation below 1.6mAHD.

8.5 Flood level in the Port

Flood levels in the Port area were extracted at a number of locations (Figure 8-7) and summarised in Table 8-1, Table 8-2 and Table 8-3.

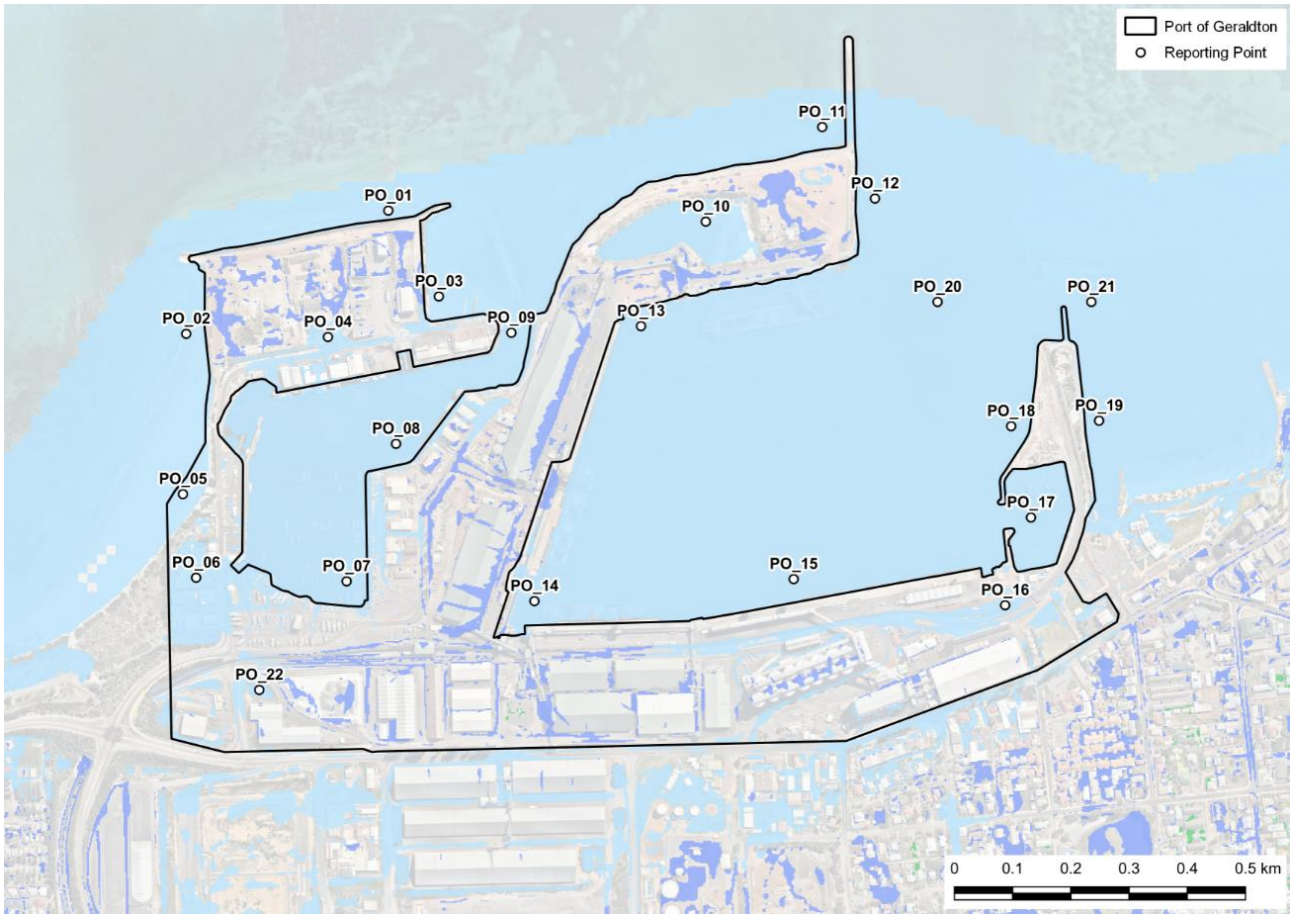


Figure 8-7 Point location where flood levels were extracted.

Table 8-1 Rain flood level in the Port area.

Year	2030			2070			2110		
	5% AEP	1% AEP	0.2% AEP	5% AEP	1% AEP	0.2% AEP	5% AEP	1% AEP	0.2% AEP
PO_01	0.66	0.66	0.66	1.45	1.45	1.45	2.29	2.29	2.29
PO_02	-	-	-	1.09	1.09	1.09	1.74	1.74	1.74
PO_03	0.56	0.56	0.56	1.25	1.25	1.25	2.01	2.01	2.01
PO_04	2.44	2.45	2.48	2.45	2.46	2.49	2.46	2.47	2.51
PO_05	-	-	-	-	-	-	1.39	1.39	1.39
PO_06	-	1.85	1.86	-	1.86	1.87	1.85	1.86	1.87
PO_07	0.39	0.39	0.39	0.85	0.85	0.85	1.43	1.43	1.43
PO_08	0.36	0.36	0.36	0.86	0.86	0.86	1.47	1.47	1.47
PO_09	0.44	0.44	0.44	0.99	0.99	0.99	1.65	1.65	1.64
PO_10	0.48	0.54	0.57	0.83	0.89	0.92	1.36	1.43	1.46
PO_11	0.52	0.52	0.52	1.18	1.18	1.18	1.91	1.91	1.91
PO_12	0.34	0.34	0.34	0.78	0.79	0.78	1.40	1.40	1.40
PO_13	0.37	0.37	0.37	0.82	0.82	0.82	1.39	1.39	1.39
PO_14	0.40	0.40	0.40	0.89	0.89	0.89	1.53	1.53	1.53
PO_15	0.35	0.35	0.35	0.83	0.83	0.83	1.43	1.43	1.43
PO_16	2.01	2.04	2.05	2.02	2.07	2.08	2.04	2.12	2.14
PO_17	0.32	0.32	0.32	0.75	0.75	0.75	1.30	1.30	1.30
PO_18	0.38	0.38	0.38	0.89	0.89	0.89	1.49	1.49	1.49
PO_19	0.38	0.38	0.38	0.94	0.93	0.93	1.60	1.60	1.60
PO_20	0.34	0.34	0.34	0.81	0.81	0.81	1.41	1.41	1.41
PO_21	0.33	0.33	0.33	0.80	0.80	0.80	1.38	1.38	1.38
PO_22	1.63	2.01	2.08	1.82	2.08	2.13	2.06	2.18	2.21

Table 8-2 Storm tide & rainfall flood level (mAHD) in the Port area.

Year	2030			2070			2110		
	5% AEP	1% AEP	0.2% AEP	5% AEP	1% AEP	0.2% AEP	5% AEP	1% AEP	0.2% AEP
PO_01	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_02	1.35	2.19	2.59	1.67	2.51	2.91	2.18	3.03	3.43
PO_03	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_04	2.42	2.53	2.74	2.42	2.68	3.01	2.50	3.11	3.48
PO_05	1.36	2.09	2.50	1.67	2.41	2.83	2.18	2.95	3.35
PO_06	1.98	2.30	2.69	1.98	2.62	2.97	2.18	3.06	3.41
PO_07	1.35	2.30	2.70	1.67	2.62	2.99	2.18	3.09	3.46
PO_08	1.35	2.30	2.70	1.67	2.62	2.99	2.18	3.09	3.46
PO_09	1.35	2.30	2.70	1.67	2.62	3.01	2.18	3.12	3.51
PO_10	0.50	1.07	2.82	0.82	1.98	3.09	1.60	3.15	3.53
PO_11	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_12	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_13	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_14	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_15	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_16	2.00	2.31	2.70	2.02	2.62	3.03	2.23	3.13	3.53
PO_17	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_18	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_19	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_20	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_21	1.35	2.30	2.70	1.67	2.62	3.02	2.18	3.13	3.53
PO_22	1.41	2.07	2.65	1.89	2.51	3.00	2.09	3.10	3.46

Table 8-3 Tsunami flood level (mAHD) in the Port area.

Year	2030			2070			2110		
	5% AEP	1% AEP	0.2% AEP	5% AEP	1% AEP	0.2% AEP	5% AEP	1% AEP	0.2% AEP
PO_01	0.58	0.62	0.9	0.91	0.95	1.16	1.43	1.49	1.69
PO_02	-	0.62	0.88	0.91	0.94	1.14	1.45	1.46	1.66
PO_03	0.59	0.63	0.92	0.92	0.96	1.18	1.43	1.5	1.71
PO_04	-	-	-	-	-	-	-	-	-
PO_05	-	-	-	-	-	-	1.43	1.47	1.66
PO_06	-	-	-	-	-	-	-	-	-
PO_07	0.60	0.67	0.99	0.94	1.00	1.24	1.45	1.57	1.78
PO_08	0.60	0.66	0.98	0.94	1.00	1.24	1.45	1.55	1.77
PO_09	0.59	0.63	0.92	0.92	0.97	1.19	1.43	1.51	1.72
PO_10	0.52	0.52	0.52	0.84	0.84	0.84	1.35	1.35	1.35
PO_11	0.59	0.63	0.91	0.94	0.96	1.18	1.48	1.51	1.72
PO_12	0.61	0.68	1.00	0.94	1.00	1.28	1.47	1.58	1.8
PO_13	0.63	0.73	1.1	0.97	1.06	1.38	1.50	1.68	1.88
PO_14	0.64	0.73	1.11	0.97	1.07	1.39	1.50	1.69	1.89
PO_15	0.63	0.73	1.1	0.97	1.06	1.38	1.50	1.68	1.88
PO_16	-	-	-	-	-	-	-	-	-
PO_17	0.63	0.72	1.09	0.96	1.05	1.37	1.50	1.66	1.87
PO_18	0.63	0.71	1.06	0.96	1.04	1.34	1.49	1.64	1.86
PO_19	0.61	0.68	1.00	0.95	1.00	1.28	1.47	1.58	1.82
PO_20	0.62	0.69	1.03	0.95	1.02	1.32	1.48	1.61	1.83
PO_21	0.61	0.68	1.00	0.95	1.00	1.28	1.47	1.58	1.81
PO_22	0.52	0.52	0.52	0.84	0.84	0.84	1.35	1.35	1.35

9 Conclusion

Summary

This Project Modelling Report outlines the technical foundation of the assessment of the flood and inundation hazards in the City of Greater Geraldton coastal zone, including the Port of Geraldton and the suburbs of West End, Geraldton and Beresford.

The flood and inundation hazards assessment considered a range of processes including rain, storm tide (including wave set-up and overtopping) and tsunamis. Three design events (i.e. 20yr ARI or 5%AEP, 100yr ARI or 1%AEP and 500yr ARI or 0.2%AEP) and three timeframes (i.e. 2030, 2070 and 2110) with adequate allowance for mean sea level rise.

A comprehensive high-resolution hydraulic model was developed that integrates detailed coastal boundary conditions and hydrological conditions modelled specifically for each scenario. The Port and the City provided local survey information used to inform the digital elevation model and develop the drainage infrastructure. The design rainfalls were obtained from the Bureau of Meteorology (BoM) and temporal pattern from the regional Australian Rainfall and Runoff (ARR) Data Hub. The design storm tide levels were based on the City's Coastal Hazard Risk Management and Adaptation Plan (CHRMAP).

Modelling results indicate that:

- > The tsunami cases modelled present minimal risk to the Geraldton area. This is demonstrated in Figure 8-5 and Appendix B with minimal flooding shown. Some areas of inundation on terrestrial locations are a result of the initial tailwater level being higher than the existing ground level. This may indicate that there is backflow through the stormwater network to some of these locations.
- > During the rain events, isolated locations within the study area are inundated by flood waters up to 2m deep. These are generally confined to drainage features and inlet locations. Some locations shown are where basement carpark entrances have been captured in the terrain survey or where terrain filling has introduced localised ponding. Typically, depths on land during the 500yr ARI rain event are less than 0.5m with flow paths typically confined to road reserves and existing drainage reserves/park areas. Furthermore, of those residential properties that are impacted by flooding, depths are generally less than 300mm.
- > The most significant coastal inundation hazards are attributed to storm tide & rainfall events and wave overtopping.
 - Residential areas in Zone C become inundated during the 20yr ARI event, with depths increasing to approximately 2m in the 500yr ARI. Foreshore street in Zones A and B are inundated by flood depths of up to 1.8 m during the 500yr ARI event. During this event, a significant number of residential and commercial properties are inundated.
 - In the Port area, the two reclaimed land areas are expected to experience significant overtopping under all scenarios to 2110, with the fishing boat harbor northern reclaim being submerged under the 100yr ARI event.
 - Wave overtopping rates at the extraction locations during storm tide exceed limits of tolerable discharges provided in overtopping guidelines for pedestrian, and vehicles safety and damage to building and defence structures (Table 5-12).

This Project Modelling Report provides the basis for subsequent mapping and planning considerations. It is accompanied by:

- > Inundation and flood maps (water level and water depth) for each scenario;
- > Inundation and flood animations for each scenario;
- > Data package containing all GIS layers created and model output files;
- > Stormwater Infrastructure Plan for the City, which rationalises the existing drainage network and considers future drainage upgrades; and
- > Emergency Response Plan for the City, which further characterises high flood hazard areas and outlines appropriate response.

Recommendations

Development level and coastal protection

Typically, buildings and structures should, as a minimum, be designed with a constructed floor level that provides adequate inundation immunity for the life of the project. Depending on the governing policy, finished floor level should be above the 1 in 100yr inundation level plus a freeboard allowance of 0.5 m (SPP2.9), alternatively development level should be above the 1 in 500yr coastal inundation level (SPP2.6).

Finished floor level are provided in the accompanying Emergency Response Plan.

A risk-based approach is recommended to be followed that considers the changing inundation hazard events frequency overtime (as sea levels rise) and the cumulative probability of such events to be exceeded during the life of the project.

Where coastal protections are likely to get inundated and overtopped by waves, there is a need to:

- > Consider the adequacy of existing rock works, adjacent pavement and neighbouring property development level going forward; and
- > Ensure adequate mitigation measures that meet standard personal safety and infrastructure integrity requirements are in place to address storm tide inundation and wave overtopping risk.

Design water level review

Considering the recent extreme water level review findings and the significance of the storm tide level in the coastal inundation extent, it would be beneficial to ascertain the extreme water level in the study area independently from previous studies. A more detailed assessment of the design water levels that would consider synthetic cyclone climatology and synoptic storm tide modelling is recommended to establish these extreme water levels. This could be part of the next CHRMAP periodic review - a process whereby the most recent and reliable information (including the design water level) can be examined and the CHRMAP updated as required.

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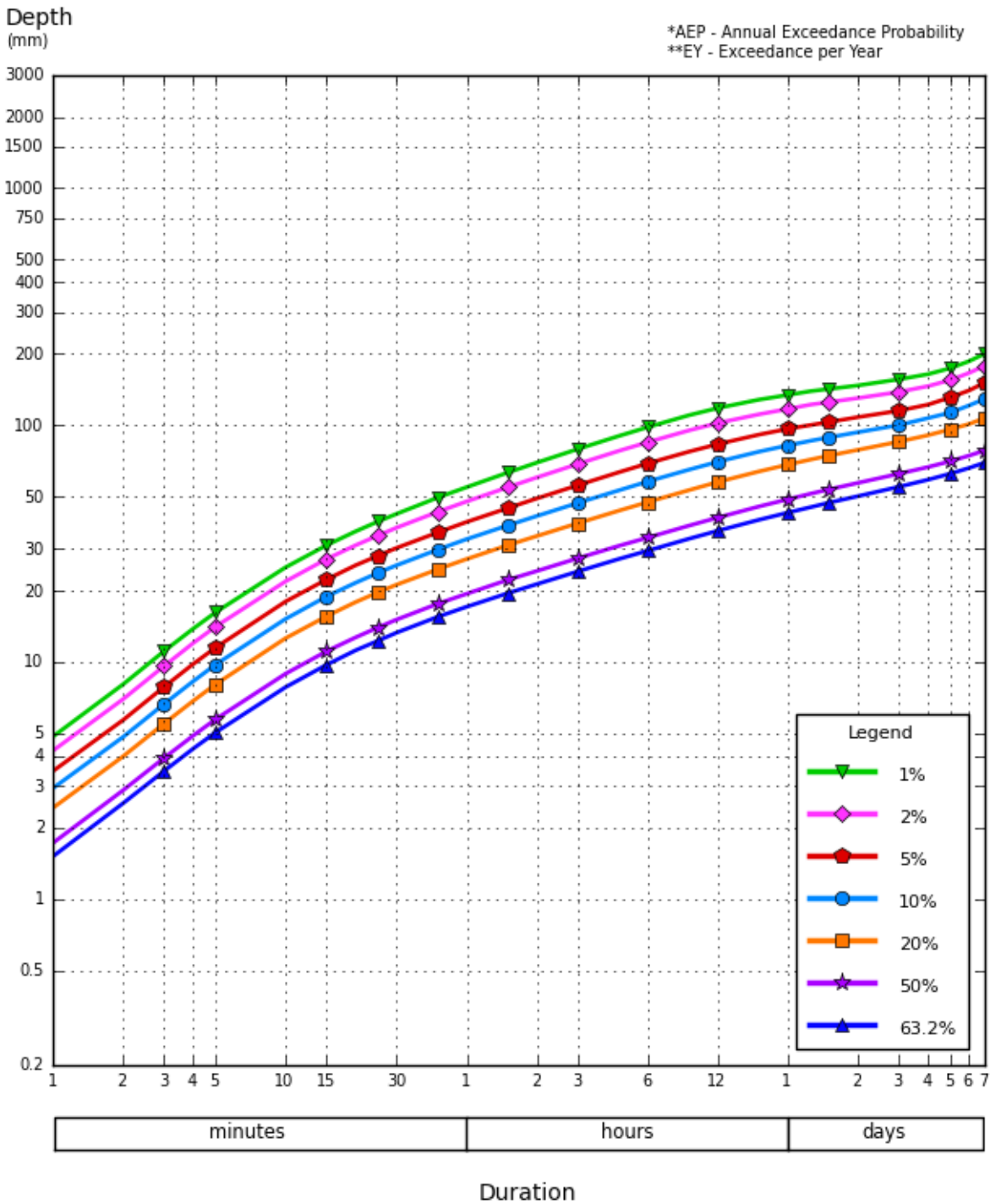
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APPENDIX

A

IFD DATA

Frequent and Infrequent IFDs



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APPENDIX

B

FLOOD MAPS

Flood maps, including flood water surface level (WSL) and flood depth, are provided in a separate (large) pdf document.