Macquarie Point Coastal Inundation Assessment

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The Amendment Record below records the history and issue status of this document.

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1 Introduction

1.1 Purpose of the Report

This report presents the findings of a coastal hazard risk assessment to support the development of responses to the Tasmania Planning Commission (TPC) Guidelines: *Macquarie Point Multipurpose Stadium Project of State Significance* (the Guideline). This report specifically responds to the Guideline section: 8.7.1 coastal inundation.

The purpose of the report is to develop a better understanding of the exposure of the Macquarie Point Multipurpose Stadium Project Site (Project Site) to coastal inundation under current and future climate scenarios. If hazards are identified, the potential effects on the Project Site, potential effects on public health and measures to manage the risks (including emergency management) are to be reported.

The assessment focuses on the following coastal hazards that contribute to coastal inundation:

- Tsunami
- Sea level rise
- Extreme coastal water levels in the River Derwent and inundation
- Wave overtopping

Extreme coastal water level statistics are presented, based on previous studies and new site-specific analysis.

The coastal hazard assessment area includes the Macquarie Wharf; however, it is noted that this area is managed by the Tasmanian Ports Corporation Pty Ltd (TasPorts) and subject to a separate Port Master Plan and redevelopment process (TasPorts, 2021). For completeness, key assessment results relevant to the Macquarie Wharf are also presented and discussed in this report.

1.2 Site Description

The Project Site is located at the foreshore of Macquarie Point in Sullivans Cove, bound to the south by Evans Street, west by Davey Street, Hobart Cenotaph to the north, and Port of Hobart to the east and north-east adjacent to the River Derwent forming part of the TasPorts Macquarie Wharf. The Project Site slopes gently to north-northwest towards Evans Street and the Hobart Cenotaph. It is understood that the proposed development is to be delivered across three broad stages delivering mixed-use precinct with the Project Site being approximately 9.3 hectares.

The Project Site is currently intermixed with carparking, sheds and cleared surfaces with existing structures including the Goods Shed, The Red Square and The Royal Engineers Building. The Project Site will gain vehicular access via Evans Street and two smaller, unnamed roads connected by both the Tasman Highway and Davey Street, which provides access to the existing facilities. TasPorts Macquarie Wharf is accessed via Hunter Street, which is connected to Evans Street, allowing access to the east of the Project Site.

The Hobart Rivulet traverses the northern boundary of the site from west to east before draining into the River Derwent southward of the Doman Boat Ramp.

The Project Site (Mac Point Site), boundaries and locality context are provided Figure 1.1.

Figure 1.1 Project Site and immediate locality context (Source: Macquarie Point Development Corporation, 2023)

1.3 Proposed Development

The proposed development involves the remediation, redevelopment and transition of the Project Site into a mixed-use precinct. It is understood that the precinct will include:

- 23,000 seated roofed stadium acting as a multipurpose sporting, arts, events and entertainment facility;
- An Aboriginal culturally informed zone;
- Mixed zoned comprising restaurants, cafes, hotels, medical facilities and commercial office space;
- Antarctic facilities including commercial spaces and connections; and
- Residential area, new public promenade and food and beverage offerings at Regatta Point.

The Project Site will be accessed via active frontages encouraging pedestrian activity, Evans Street and proposed connecting road to the north via the Residential Development and Public Foreshore Zone.

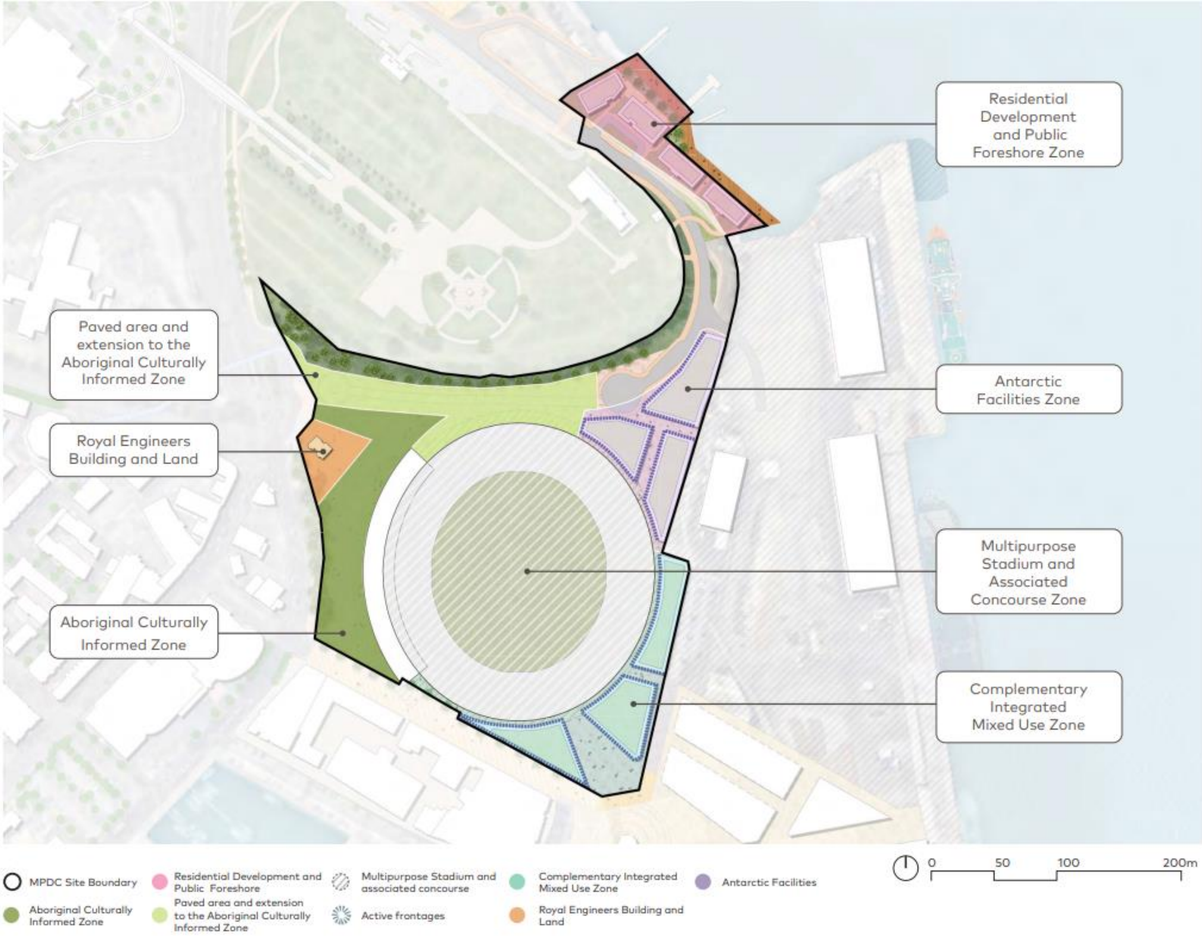


Figure 1.2 Project Site boundary and zones (Source: Macquarie Point Development Corporation, 2023)

2 Background Information

2.1 Previous Studies

Several previous studies and online portals provide background information to support the assessments presented in this report and are listed below.

- Building on tsunami modelling undertaken by Geoscience Australia, the Tasmanian Government completed a more detailed assessment of coastal inundation associated with a 'maximum credible earthquake/tsunami/high tide' scenario in southeast Tasmania: https://d2kpbjo3hey01t.cloudfront.net/uploads/2020/02/MRT-Technical-report-on-tsunami-inundation-in-SE-Tasmania.pdf

Previous studies that have assessed extreme water level statistics based on an analysis of historical data recorded by the Hobart tide gauge include:

- Hunter (2007) *Historical and Projected Sea-Level Extremes for Hobart and Burnie*, Tasmania, prepared by the Antarctic Climate and Ecosystems Cooperative Research Centre and commissioned by the Tasmanian Government https://nre.tas.gov.au/Documents/CCCRMP-Hunter_Report.pdf

- McInnes, et al. (2011) *Climate Futures for Tasmania, Extreme Tide and Sea-Level Events,* prepared by the Antarctic Climate and Ecosystems Cooperative Research Centre https://climatefutures.org.au/technical-reports/extreme-tide-sea-level-events-technical-report/

- UWA (2018) Extreme Sea Levels in Australia, maintained by UWA and Bushfire and Natural Hazards CRC https://sealevelx.ems.uwa.edu.au/index.php

- CSIRO Canute 3 (2023) *Sea level calculator – July 2023 release*, maintained by CSIRO Climate Systems National Environmental Science Program https://shiny.csiro.au/Canute3_0/

The references above adopt the Intergovernmental Panel on Climate Change (IPCC) sea level rise projections when assessing future climate scenarios. The recent sea level rise projections and datasets associated with the IPCC Sixth Assessment Report (AR6) have been adopted for this assessment:

- IPCC (2022) *Sixth Assessment Report (AR6)*, Intergovernmental Panel on Climate Change https://www.ipcc.ch/assessment-report/ar6/

Key references that informed the wave modelling and overtopping assessment include:

- AS/NZS 1170.2.2002 *Structural design actions, Part 2: Wind actions* https://www.standards.org.au/standards-catalogue/standard-details

- EurOtop (2018) Manual on wave overtopping of sea defences and related structures http://www.overtopping-manual.com/

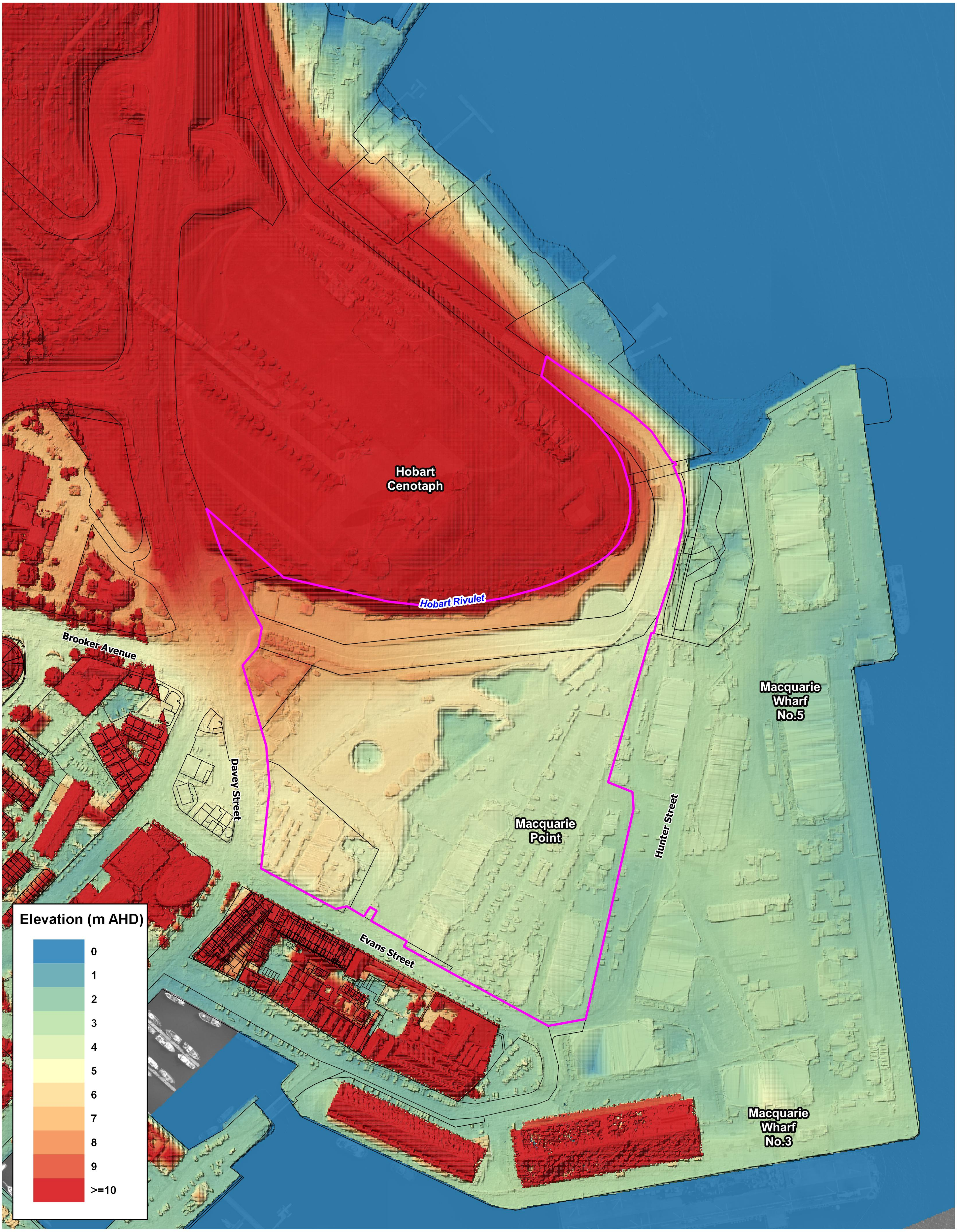
2.2 Key Datasets

The following datasets have been used in this assessment:

- Drone LiDAR topography survey supplied by Macquarie Point Development Corporation (MPDC)
- Fixed-wing aircraft LiDAR topography survey captured in 2010 and accessed via Elvis: https://elevation.fsdf.org.au/
- Hobart Ports Corporation River Derwent Soundings, 16-28 August 2003 & 15-26 October 2004, Job No. H000804.10, Drg No. H000804.10.07, Sheet 32 of 39, supplied by TasPorts
- Hobart tide gauge hourly observations 1987 to 2021, supplied by the Bureau of Meteorology
- Hobart tide gauge hourly observations 1960 to 2019[1], accessed via the GELSA website https://gesla787883612.wordpress.com/

The topography datasets were combined to create a digital elevation model (DEM) of the Project Site and surrounding area. This is shown in Figure 2.1.

[1] GELSA note possible datum issues with this data, and it was therefore largely discarded from the analysis



Legend

Project Site Boundary

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Project Site Topography

Drawing: 2-1

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BMT endeavours to ensure that the information provided in this map is correct at the time of publication. BMT does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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3 Tsunami Assessment

3.1 Introduction

A tsunami inundation assessment for southeast Tasmania coordinated by Mineral Resources Tasmania is reported by Kain et al. (2018). Inundation associated with a ‘maximum credible earthquake/tsunami/high tide’ scenario was simulated and used to assess the potential hazard area and impact to maritime industries. The hazard scenario was based on a Mw 8.7 earthquake in the Puysegur subduction zone located off New Zealand’s southwest coast, generating a representative 1 in 13,000-year tsunami event coinciding with ‘Highest Astronomical Tide’ (refer Section 4.2).

Key outputs from Kain et al. (2018) relevant to the Project Site and surrounding areas are summarised in this section.

3.2 Tsunami inundation hazard area

The tsunami inundation hazard area for the Hobart CDB presented by Kain et al. (2018) is shown in Figure 3.1 and includes Macquarie Point and the Project Site. This map shows the Project Site is outside of the tsunami hazard area, with the inundation extent limited to low-lying areas at Sullivans Cove and the seaward perimeter of Macquarie Wharf.

Kain et al. (2018) note the potential need for further assessment of tsunami inundation for the Hobart CBD, particularly in areas where inundation could be influenced by the Hobart rivulet. Considering the topography shown in Figure 2.1, and the results of flood studies that consider the rivulet (e.g. Entura, 2014; Cardno, 2019), the Project Site is expected to remain outside of the tsunami hazard area.

Some further consideration of the tsunami assessment with respect to Macquarie Wharf is provided in Section 7.

Figure 3.1 Modelled tsunami inundation in Hobart CBD and Sandy Bay (Kain et al. 2018)

4 Coastal Water Level Assessment

4.1 Introduction

A site-based assessment of present and future climate coastal water levels was recently completed by BMT (2023) and the key findings are presented below. The assessment is based on tide gauge data analysis, sea level rise projections and previous studies that have derived extreme water level statistics.

4.2 Coastal Water Levels

Present-day tide

Tides occur as a response to astronomic gravitational forces, largely due to the effects of the sun and the moon on the earth. At Hobart, the tides vary in a semi-diurnal pattern (two high, and two low tides per 24-hours) and a moderate spring/neap variation (larger tides during full and new moons). Other longer-term variations also occur that mean certain high tides are larger than others.

The theoretical highest or lowest tide that can be caused by astronomic forces alone under average meteorological conditions is known as the ‘Highest Astronomical Tide’ (HAT) or ‘Lowest Astronomical Tide’ (LAT).

The intertidal areas are defined by land with elevations between HAT and LAT. These areas do not typically contain development and are composed of beaches, estuary banks, marshes, and mangrove habitat, which are largely tolerant of regular inundation. The exception is coastal-dependent development such as ports, harbours, marinas and recreational boating facilities (e.g. boat ramps).

For this assessment, present-day HAT at Hobart is estimated to be 0.87 mAHD. This is based on analysis of the hourly tide gauge observations at Hobart and compares well with other recent planning developments (e.g., a level of 0.86 mAHD was adopted for the New Bridgewater Bridge Coastal Inundation Assessment, with this number supplied by TasPorts, see Burbury Consulting, 2021).

Sea level Rise

Sea level rise (SLR) will cause an increase to Mean Sea Level (MSL) and move the intertidal zone to a higher elevation, exposing low-lying coastal areas to tidal processes.

CSIRO has been previously engaged by the Tasmanian Government to provide updated sea level rise planning allowances for Tasmania (CSIRO, 2016). The planning allowances for Hobart from this study are 0.23 m and 0.85 m for 2050 and 2100, respectively. This is based on projections from the Intergovernmental Panel on Climate Change’s (IPCC) Assessment Report 5 (IPCC, 2013). This study adopted the high emissions scenario (RCP8.5) as the basis for these projections.

In 2021/22, the IPCC released Assessment Report 6 (AR6) which included updated sea level rise projections. AR6 presents sea level rise projections relative to future climate scenarios, known as SSP (shared socioeconomical pathways) scenarios, with SSP5-8.5 the most similar to the previous RCP8.5 scenario. The SSP5-8.5 projections are reported for two confidence levels, medium and low, with medium being similar to the RCP8.5 projections.

For this assessment, the AR6 projections based on SSP5-8.5 the medium confidence level has been used to estimate future extreme sea-levels at Hobart. These projections, along with the other sources, are shown in Table 4.1. The estimated present and future climate HAT tide level is provided in Table 4.2.

Table 4.1 Sea level rise projections at Hobart

Year	Hobart Sea Level Rise Projection (CSIRO, 2016) m	AR6 SSP5-8.5 Medium Confidence m	AR6 SSP5-8.5 Low Confidence m
Present (~2020)	-	0.05	0.06
2050	0.23	0.23	0.24
2070	-	0.40	0.43
2100	0.85	0.78	0.89
2120	-	1.01	1.26

Table 4.2 Future HAT levels (mAHD) based on the adopted SLR scenario

Year	Sea level rise m (SSP5-8.5 Medium Confidence)	Future HAT level mAHD
Present	0.05	0.87
2050	0.23	1.05
2070	0.40	1.22
2100	0.78	1.6
2120	1.01	1.83

Extreme sea levels

Extreme sea levels are caused by the combination of a high tide with another disturbance to the ocean that results in short-term elevated sea levels. A common driver of extreme sea levels are low pressure weather systems that can elevate sea levels through a combination of low atmospheric pressure and wind setup (that can cause water to 'pile up' against the coast). The difference between the predicted astronomic tide and the observed water level is referred to as the 'tidal anomaly' or 'storm surge' during severe weather conditions. The resultant combination of storm surge and the underlying astronomical tide is often referred to as the 'storm tide'.

Open coast storm tide events are often accompanied by large ocean waves that can drive temporary increases to water levels due to breaking waves, known as wave setup. Macquarie Point is well protected from ocean swells, with only locally generated wind waves able to reach the shoreline during severe weather events, which do not generate significant wave setup. The contribution of wave setup is therefore not included in the extreme water levels presented herein, with the levels representing a quasi-static water level (tide plus surge) that may be reached in a storm.

An analysis of the Hobart tide gauge observations to estimate extreme water levels has been completed, using the following methodology:

1. Erroneous or low-quality data from the Hobart tide gauge was removed prior to further analysis. The resulting dataset used for fitting was based on the more recent 1987 to 2023 observations, with the older data (1960 to 1987) excluded due to vertical datum uncertainty.

2. The water levels from the Hobart tide gauge were detrended from any long-term fluctuations (e.g., sea level rise).

3. Harmonic analysis was undertaken on the detrended dataset to extract tidal constituents, which were then used to reconstruct a tide-only signal covering the entire dataset. This was subsequently used to calculate residual sea levels (or tidal anomaly).

4. An extreme value analysis of the residuals was undertaken using a standard peak over threshold approach, with a generalised pareto distribution used to model the residual extremes.

5. A 10,000-year climate of extreme events was constructed by combining a random 24-hour high tide level (Hobart experiences two daily high tides) with a residual water level from the fitted extreme distribution.

6. Extreme water level estimates were then extracted from this synthetic dataset for the 5% and 1% annual exceedance probabilities (AEP), equivalent to 20- and 100-year average recurrence intervals.

The results from this analysis are shown below in Table 4.3, combined with the adopted sea level rise projections (i.e. IPCC AR6 SSP5-8.5 Medium Confidence). A set of coastal inundation hazard maps, based on these levels and a DEM of the Project Site and surrounding areas developed from LiDAR topography datasets (see Figure 2.1) are shown in Annex A.

Table 4.3 Coastal Inundation Hazard Levels (m rel. AHD)

AEP (%)	Future Horizon Year			
	2020	2070	2100	2120
5	1.21	1.56	1.93	2.17
1	1.31	1.66	2.04	2.27

The 2100 1% AEP level in Table 4.3 is generally consistent with the City of Hobart ‘1% 2100 storm surge level’ of 1.94 m above AHD83 (or approximately above mean sea level in 1972). The City add a 1.0 m freeboard allowance to account for wind and ocean swell generated waves and conclude that land below 3.0 mAHD83 may be at risk of coastal inundation. The additional 1.0 m freeboard allowance is to account for uncertainty and the potential influence of waves. A site-based assessment of waves and overtopping potential at the Regatta Grounds shoreline (the ‘Residential Development and Public Foreshore Zone’ shown in Figure 1.2) is provided in Section 5 and Section 6. Considerations for waves and overtopping at Macquarie Wharf is discussed in Section 7, noting this area is outside of the Project Site and subject to separate planning and development assessments.

The levels produced in this assessment are also generally consistent with other estimates for Hobart summarised in Table 4.4 and based on the sources discussed in Section 2.1. It is noted that each study has used differing datasets or methods, and may not be directly comparable (e.g., Hunter 2007 presents extremes in the context of exceedance likelihood in a future climate).

Table 4.4 Comparison between 1% AEP extreme sea-level estimates (m rel. MSL)

BMT	Hunter (2007)	Canute 3 CSIRO (2023)	McInnes (2012)	UWA (2018)
1.26*	~1.4**	1.44	1.24	1.29

*This value does not include SLR in 2020 that is shown in Table 4.3, hence the lower level.

**Hunter (2007) presents the analysis results in terms of exceedance % of specific levels with a distance of 0.1m during a future climate year, and hence the level shown in this table has been estimated.

Asset design life

The probability for an asset to experience a design event of a certain magnitude in its lifetime is related to the design life of the asset. For example, an asset with a design life of 100-years will have a 22%, 39% and 63% chance to experience a 1% AEP event in the first 25-, 50- or 100-years of its life.

Table 4.5 presents exceedance levels which represent the % probability for an asset with either a 50- or 100-year design life to encounter a certain storm tide elevation level during its lifetime. This has been extended to incorporate sea level rise based on the AR6 SSP5-8.5 Medium Confidence projections.

This table shows that an asset built in the present day (~2020) will need to adopt a floor level of about 1.5 mAHD to reduce the risk of inundation during its lifetime to within 3 to 5% for a 50- or 100-year design life respectively. By the year 2100 and due to sea level rise, a floor level around 2.2 mAHD is needed to retain a similar risk profile. Ground elevations across the Project Site are generally above 3.0 mAHD, expect for the Regatta Grounds foreshore area that slopes towards to the shoreline.

Given that increases to mean sea level are gradual and will likely continue beyond 2120, developments that can adapt and/or are resilient to inundation will allow present-day values to be maximised while giving stakeholders certainty that future risks can be managed.

Table 4.5 Coastal inundation hazard exceedance levels

Level	50-year Asset				100-year Asset			
mAHD	2020	2070	2100	2120	2020	2070	2100	2120
0.9	100	100	100	100	100	100	100	100
1.0	100	100	100	100	100	100	100	100
1.1	100	100	100	100	100	100	100	100
1.2	95	100	100	100	100	100	100	100
1.3	46	100	100	100	70	100	100	100
1.4	13	100	100	100	24	100	100	100
1.5	3	100	100	100	5	100	100	100
1.6	0	75	100	100	0	94	100	100
1.7	0	24	100	100	0	42	100	100
1.8	0	5	100	100	0	10	100	100
1.9	0	0	98	100	0	1	100	100
2.0	0	0	60	100	0	0	84	100
2.1	0	0	17	100	0	0	32	100
2.2	0	0	3	79	0	0	7	96
2.3	0	0	0	25	0	0	1	44
2.4	0	0	0	6	0	0	0	11
2.5	0	0	0	0	0	0	0	1

5 Wave Assessment

5.1 Introduction

The coastal inundation levels presented in Section 3 represent quasi-static extreme water levels represented by the combination of ‘tide plus surge’. The inundation associated with these levels (and maps presented in Annex A) may be considered “green water overtopping” whereby the extreme water level is maintained, and the inundation is continuous or persists in the order of minutes or more. Green water inundation is the primary concern when setting planning levels for development due to safety, and because there are no cost-effective engineering options available to resist this type of hazard, except for ground treatment (such as raising the ground level or bunding).

In contrast, “white water overtopping” refers to non-continuous overtopping due to waves breaking at the coastal defence structure and running up and over the crest. White water overtopping can occur at lower (non-extreme) water levels when there is an energetic wave state, and hence the design of edge protection and/or ground levels should be carefully considered to ensure that overtopping rates remain tolerable and any associated risks to people and/or development are manageable.

This section presents the wave modelling to understand the local extreme and prevailing wave climate. These outputs are then used to inform the overtopping analysis presented in Section 6.

5.2 Extreme wave assessment

Hobart is located on the River Derwent estuary and the Project Site is generally protected from ocean swells. Nonetheless, storm events (usually associated with low pressure weather systems) can generate waves within the estuary and its bays if strong winds are present. These types of waves are referred to as locally generated wind-waves and are typically characterised by lower wave heights and shorter periods than ocean swells on an open coast; however, may still be significant when considering coastal hazard risks and engineering design.

Wave modelling has been used to assess extreme locally generated wind-waves within the estuary. A summary of the assessment results is presented below, and the technical detail of the modelling is provided in Annex B. Example model outputs showing the locally generated wind-wave conditions during an approximately north-easterly and south-easterly 100-year wind are shown in Figure 5.1.

Table 5.1 presents omnidirectional wave extremes at three locations (refer to Figure 5.2) around the Macquarie Point area of interest. These omnidirectional values represent the highest predicted significant wave height (H_s)[2] due to a wind blowing across the harbour from the north to south (clockwise) directional sectors, and the associated wave peak period (T_p). A full set of wave extreme results for each wind direction and return period are shown in Annex B.

[2] The significant wave height H_s is defined as the average height of the top 1/3 highest waves.

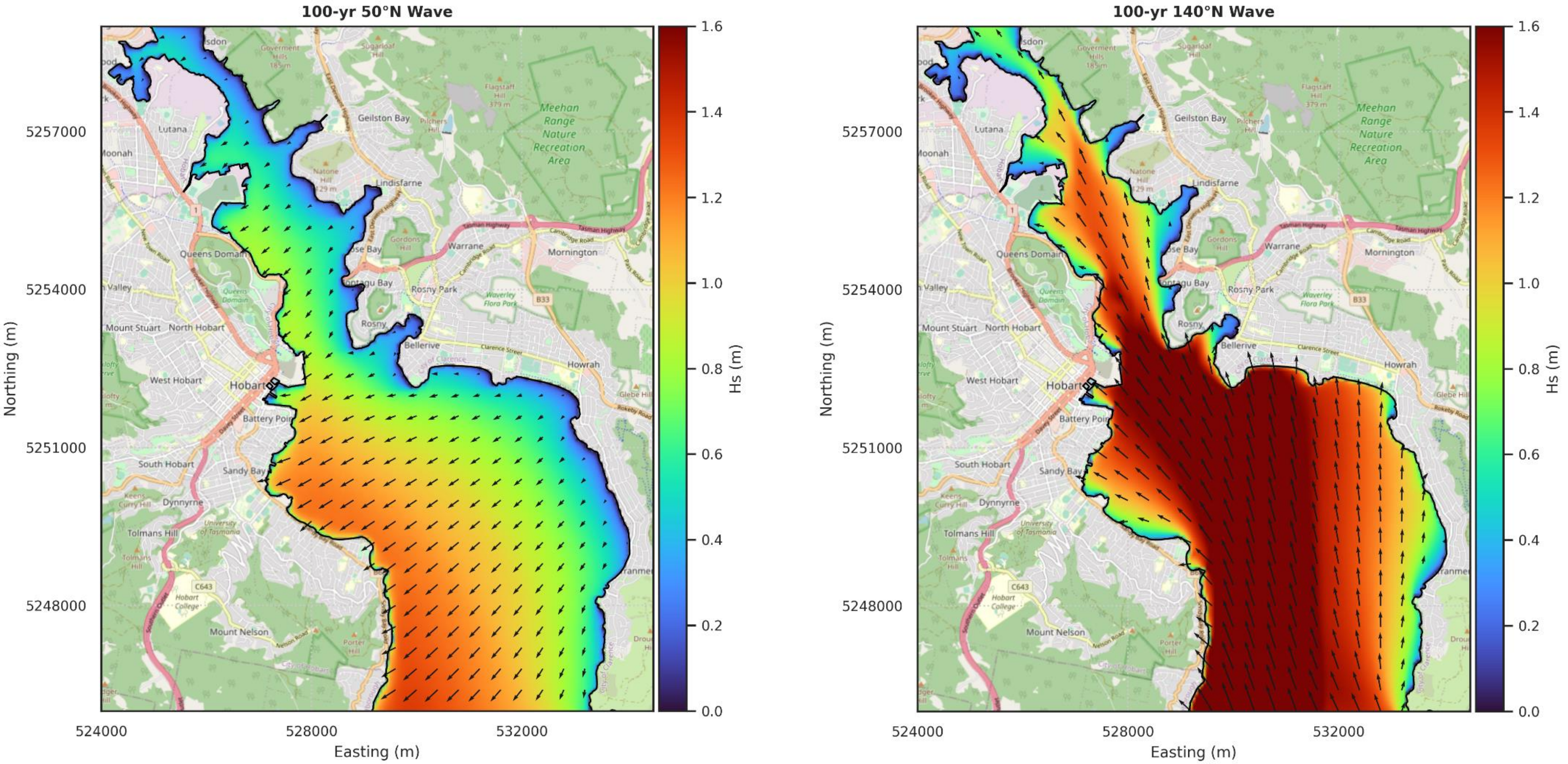

Figure 5.1 SWAN model outputs showing 100-year (1% AEP) significant wave height for a 50°N (left) and 140°N (right) extreme wind

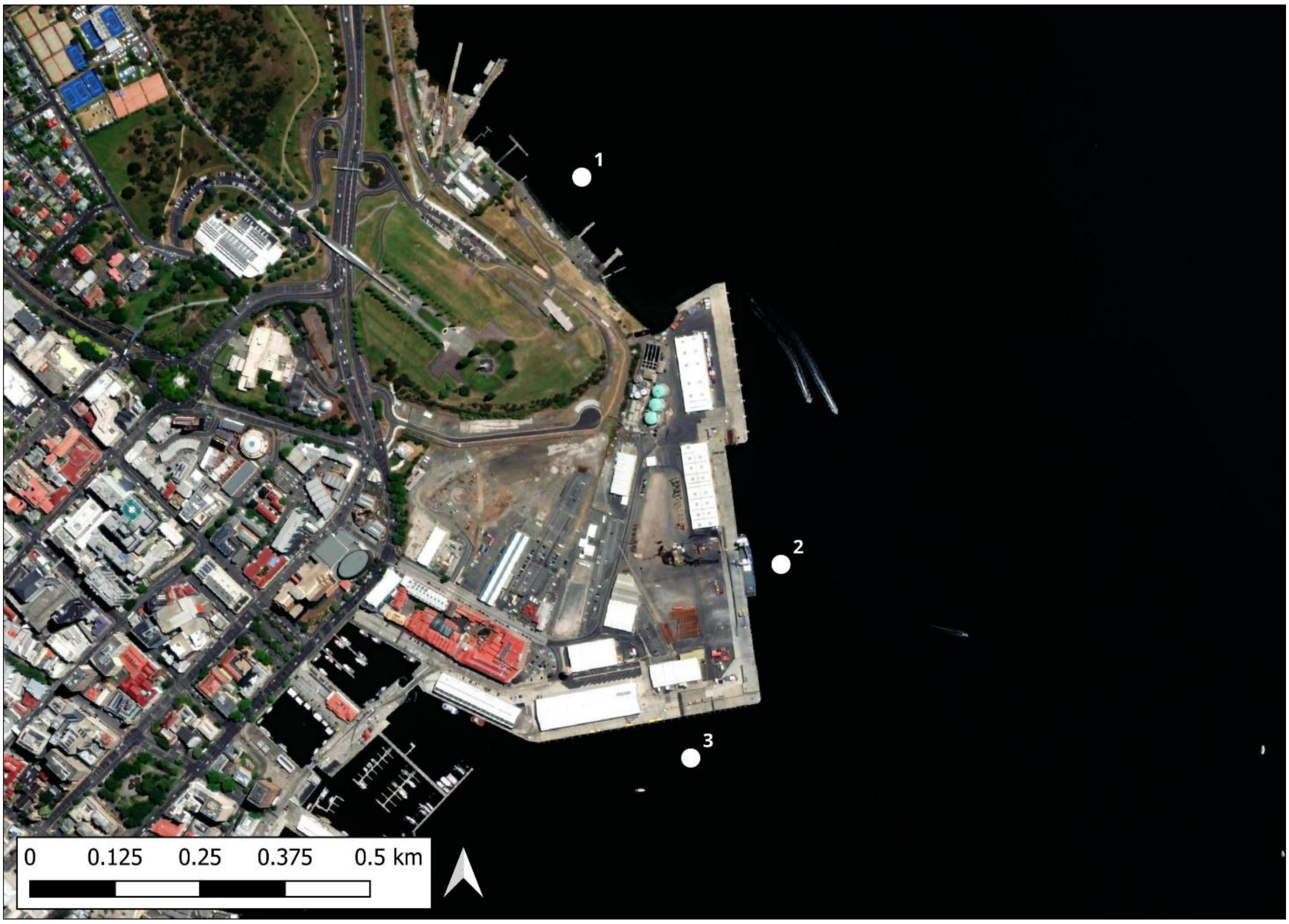

Figure 5.2 Wave model output locations

Table 5.1 Omnidirectional wave extremes

ARI (AEP)	Location 1 (Regatta Grounds)		Location 2 (Wharf East)		Location 3 (Wharf South)	
	H_s	T_p	H_s	T_p	H_s	T_p
20 (5%)	0.77	3.07	1.36	4.49	1.36	4.52
100 (1%)	0.87	3.21	1.54	5.06	1.56	5.04

5.3 Long-term wave hindcast assessment

Wave modelling has been used to develop a continuous timeseries of nearshore wave conditions (height, period, and direction) based on historical winds and water levels at Macquarie Point. The wave model described in Annex B was used for this purpose. The wave model boundary conditions were defined using historical data from:

- The Hobart Airport weather station (ID: 094250) operated by the Bureau of Meteorology[3]
- Reliable water level observations (hourly) from 1987 to 2021 recorded by Hobart tide gauge and supplied by the Bureau of Meteorology

[3] The Hobart (Ellerslie Road) weather station (ID: 094029) wind data was also considered. The higher wind speeds recorded by the Hobart Airport weather station (ID: 094250) were adopted for the wave modelling as a conservative approach (i.e. the higher wind speeds will lead to slightly larger wave heights).

The resulting nearshore significant wave height timeseries offshore from the Regatta Grounds (location 1 in Figure 5.2) is shown in Figure 5.3 and a scatter plot showing the significant wave height and water level pairs is shown in Figure 5.4. These outputs are used in inform the overtopping analysis presented in Section 6.

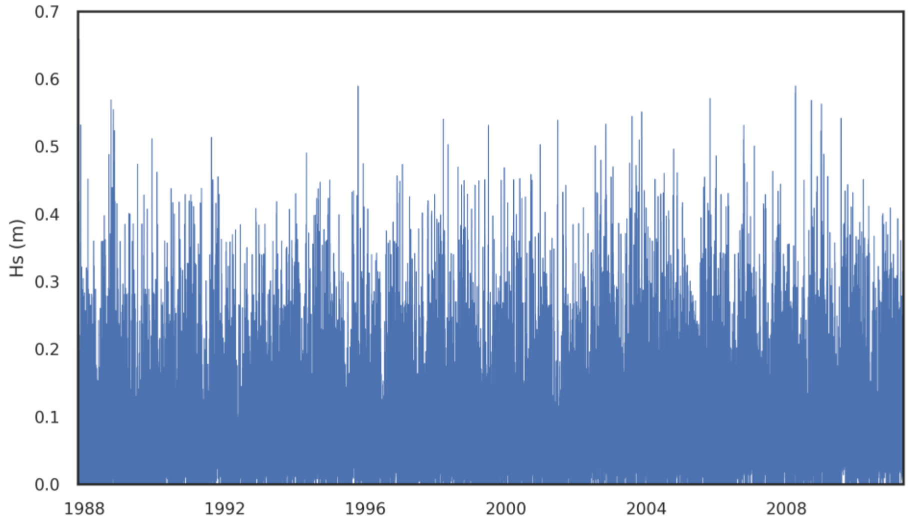

Figure 5.3 Significant wave height timeseries at Regatta Grounds

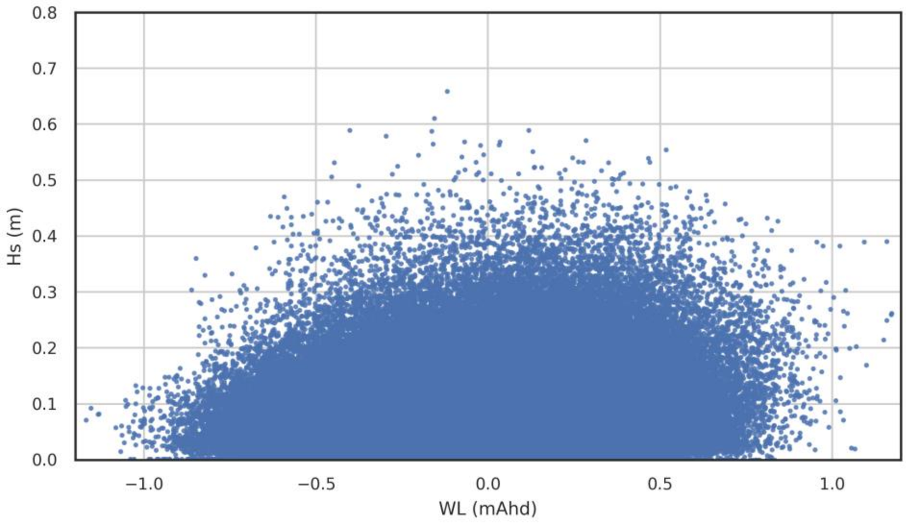

Figure 5.4 Scatter plot of significant wave height and water level pairs at Regatta Grounds

6 Regatta Grounds Detailed Wave Overtopping Assessment

6.1 Introduction

Engineering measures to reduce overtopping at sea defence structures include edge or foreshore protection (such as sloping rock revetments, vertical walls, promenades) and often including edge details (such as wave return walls and crest walls). Direct assessment and design to reduce overtopping to tolerable levels will usually commence at the design development (options assessment) and concept design stages using numerical modelling and empirical methods. In some cases, the overtopping potential of a proposed sea defence design is further assessed using scaled physical models as part of the detailed design stage.

In this section the overtopping potential at the Regatta Grounds foreshore area (refer location 1 in Figure 5.2) is considered using wave model outputs and empirical methods described in the EurOtop (2018) manual. The objectives of this assessment are to:

- Estimate the overtopping potential under design wave conditions and present and future climate scenarios,
- Estimate the frequency of inundation by overtopping for the ground floor of buildings at 2.0 mAHD under current and future climate scenarios,
- Estimate the year (based on sea level rise projections) when the unmitigated frequency of inundation and overtopping risk is too high, and
- Provide initial advice on protection measures to maintain safety for pedestrians, based on tolerable limits defined by the EurOtop (2018) manual.

The EurOtop methods are not suited to wharf structures and the Macquarie Wharf area is subject to a separate planning and development process (TasPorts 2021), therefore overtopping was not assessed at locations 2 and 3 shown in Figure 5.2. Considerations for the Macquarie Wharf area is briefly discussed in Section 7.

6.2 Overtopping formula

For each generic structure/foreshore type the overtopping analysis considers:

- Wave height
- Wave period and length
- Wave direction
- Structure and foreshore height, slope, angle
- Water level (including sea level rise)

The general mean overtopping discharge on a slope is given by (EurOtop 2018):

$$\frac{q}{\sqrt{g.H_{m0}^3}} = \frac{0.026}{\sqrt{\tan\alpha}}\gamma_b.\xi_{m-1,0}.exp\left[-\left(2.5\frac{R_c}{\xi_{m-1,0}.H_{m0}.\gamma_b.\gamma_f.\gamma_\beta.\gamma_v}\right)^{1.3}\right]$$

Equation 6.1

Where q is the mean overtopping discharge (l/s/m), g is acceleration due to gravity (m/s^2), H_{m0} is the significant wave height estimated from spectral analysis[4] (m), ξ is the breaker parameter based on wave steepness (-) and R_c is the crest freeboard of the structure (m).

EurOtop (2018) defines several "influence factors" (or reduction coefficients) to represent generic structure types and/or additional overtopping mitigation measures on a slope (generic structure types and mitigation is discussed further below). An influence factor is included in the denominator of the exponential part of Equation 6.1: γ_b is the influence factor for a berm (or promenade), γ_f is the influence factor for roughness elements, γ_β is the influence factor for oblique wave attack and γ_v is the influence factor for a wall at the end (top) of a slope.

6.3 Overtopping limits

Wave overtopping limits suggested by EurOtop (2018) have been revised downwards from previous versions of the manual, reflecting the importance of individual wave volumes to seawall performance requirements and safety to property and people. The overtopping limits relevant to this study (in terms of the small nearshore wave heights) are provided in Table 6.1 and have been drawn from Chapter 3 of EurOtop (2018).

Table 6.1 General limits for overtopping for property or people behind the defence

Hazard type	Limiting overtopping rate	Reference
Building structure elements, H_{m0} = 1-3 m	< 1 l/s/m mean overtopping < 1,000 l/m max volume	Table 3.2 EurOtop II
People at seawall crest, H_{m0} = 1 m	< 10-20 l/s/m mean overtopping < 600 l/m max volume	Table 3.3 EurOtop II

6.4 Extreme wave overtopping assessment

Extreme wave overtopping rates at the Regatta Grounds foreshore based on Equation 6.1 are provided in Table 6.2. This assessment assumes the 100-year ARI (1% AEP) omni-directional design wave conditions offshore from the Regatta Grounds (location 1 in Figure 5.2, Table 5.1) coinciding with a "normal" high tide at Hobart (~0.4 mAHD). These results correspond to overtopping at the shoreline structure crest which is around 1.1-1.2 mAHD based on the topography data shown in Figure 2.1.

Table 6.2 suggests that the tolerable overtopping rates for safety of people and vehicles at the seawall crest are not exceeded under the present (2020) climate 100-year ARI (1% AEP) scenario but may be exceeded before 2070 due to the reduced freeboard caused by sea level rise. This preliminary assessment suggests a freeboard (the vertical distance from the still water level to seawall crest) around 0.5 m is needed to maintain tolerable limits of overtopping at the Regatta Grounds shoreline.

Overtopping in the context of proposed buildings and shoreline/foreshore variability at the Regatta Grounds is considered further in Section 6.5.

[4] H_{m0} and H_s have been used interchangeably in this assessment as H_s derived from numerically modelled waves is typically equivalent to H_{m0} (e.g. CEM 2006)

Table 6.2 Regatta Grounds 100-year ARI (1% AEP) overtopping assessment

Year	Sea-level rise, SLR (m)	Still-water level, SWL (m)	Freeboard, Rc (m)	Mean overtopping rate, Q (l/s per m)	Maximum Overtopping Volume, Vmax (l/m)
2020	0.05	0.45	0.55	7.50	461
2070	0.40	0.8	0.20	52.0	911
2100	0.78	1.18	-0.18	123	1370
2120	1.00	1.41	-0.41	439	3890

Notes: EurOtop (2018) limits for overtopping for people and vehicles: mean overtopping rate Q = 10-20 l/s per m, maximum overtopping volume 600 l/s per m. Green shade indicates overtopping is below tolerate limit, pink shade indicates overtopping exceeds tolerable limit.

6.5 Wave hindcast overtopping assessment

Outputs from the wave hindcast modelling described in Section 5 have been used to analyse overtopping magnitude and frequency with a focus on proposed development at the Regatta Grounds. The general approach to this assessment follows:

1. Development of generic structure types and nearshore/foreshore characteristics at the Regatta Grounds based on the bathymetry/topography data and preliminary architectural plans.

2. Wave overtopping simulation based on the long-term wave hindcast modelling (refer Section 5.3) and generic structure types, considering both current and future climate (sea level rise) scenarios.

Overtopping structures

The following generic structure types and foreshore characteristics have been considered in the context of estimate overtopping potential at the Regatta Grounds and the proposed building footprints shown in Figure 6.1:

- Shallow sloping (1V:5H) structure/foreshore with a 5 m (minimum) wide promenade before reaching a ground elevation of 2 m AHD (indicative of site 1 in Figure 6.1),

- Average sloping (1V:4H) structure/foreshore with a 5 m (minimum) wide promenade before reaching a ground elevation of 2 m AHD (indicative of site 2 in Figure 6.1),

- Steep sloping (1V:2H) structure with a narrow promenade (less than 5 m) before reaching a ground elevation of 2 m AHD (indicative of site 3 in Figure 6.1), and

- Vertical wall with a 2 m AHD crest.

Figure 6.1 Proposed building footprints (red outline) at the Regatta Grounds (note: ground floor at or above 2 m AHD)

Tolerance limits

The results presented below focus on the mean overtopping discharge limit for building structure elements. An 'overtopping event' is defined as having a mean overtopping discharge >1 l/s/m (i.e. limiting overtopping rate for buildings, refer Table 6.1) and the 'intolerable limit' is defined as being when the average number of overtopping events per year is greater than one (1) event.

Individual wave volume limits should also be considered as the shoreline protection and foreshore designs progress, noting these limits are not expected to be a constraint given the relatively small waves at the proposed development site.

Wave hindcast overtopping simulation results

The present and future climate overtopping potential at the development site was simulated by resampling of the long-term nearshore wave parameters, coincident water level and sea level rise projection (refer Section 4.1).

The overtopping analysis for the generic structures described above is summarised in Figure 6.2 and Table 6.3. These results estimate the statistical likelihood of an 'overtopping event' occurring in any year between 2020 and 2120.

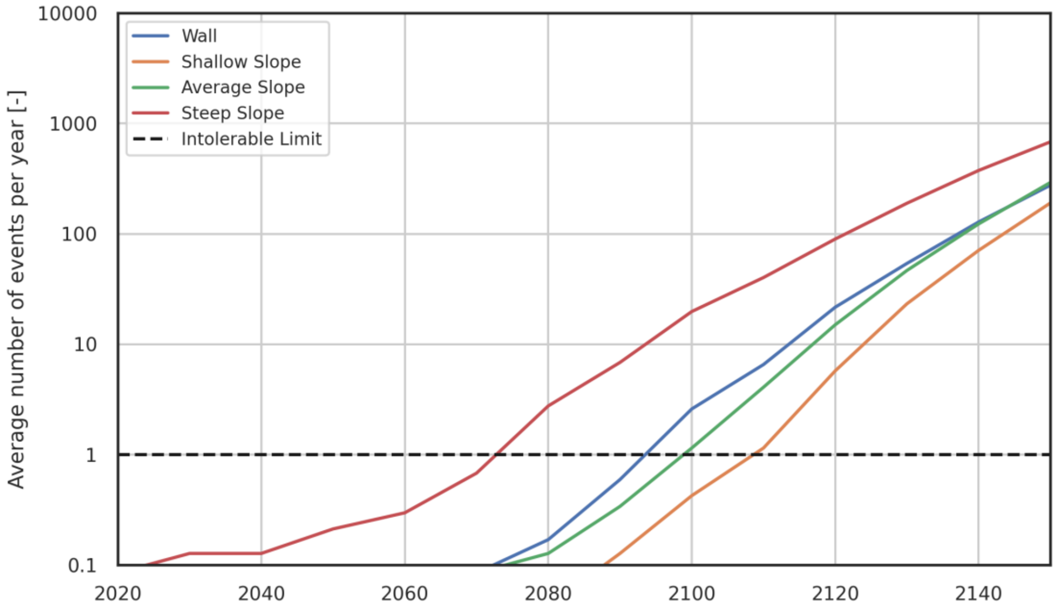

Figure 6.2 Average number of overtopping events[5] per year for generic structures

Table 6.3 Average number of overtopping events per year for generic structures

Structure Type	2020	2050	2070	2100	2120
Steep Slope	0.1	0.2	0.7	19.8	89.7
Wall	0.0	0.0	0.1	2.6	21.6
Average Slope	0.0	0.0	0.1	1.1	15.0
Shallow Slope	0.0	0.0	0.0	0.4	5.7

The overtopping analysis results in Figure 6.2 and Table 6.3 indicate:

- The average number of overtopping events increases with time due to sea level rise.
- Overtopping increases with structure and foreshore steepness (noting the vertical structure produces less overtopping than the steep structure and narrow promenade scenario).
- A steep slope (1V:2H) may experience intolerable overtopping under the present day (2020) scenario and has a relatively high likelihood of intolerable overtopping by 2070.
- The overtopping potential for the average slope (1V:4H) and vertical wall is similar, with both structure types expected to experience intolerable overtopping by 2100.
- The shallow slope structure type may not experience intolerable overtopping after 2100.

[5] An 'overtopping event' has a mean overtopping discharge >1 l/s/m

Wave hindcast overtopping simulation results: structures with mitigation

Considering the relatively small waves and low overtopping volumes expected at the development site, simple mitigation measures can further reduce the likelihood of an intolerable overtopping event occurring. The overtopping analysis results in Figure 6.3 and Table 6.4 consider the 'average slope' generic structure type with the following mitigation:

- Average slope with a 0.3 m vertical crest wall
- Average slope with a 5 m (minimum) wide promenade
- Average slope with a 5 m (minimum) promenade and 0.3 m vertical crest wall

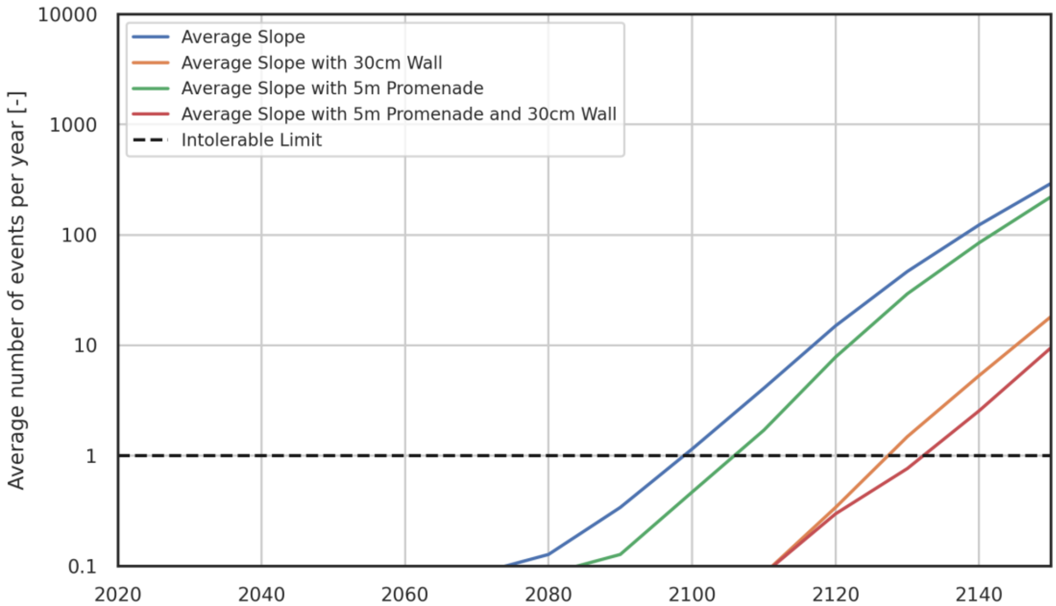

Figure 6.3 Average number of overtopping events per year for generic structures with mitigation

Table 6.4 Average number of overtopping events per year for generic structures with mitigation

Structure Type	2020	2050	2070	2100	2120
Average Slope	0.0	0.0	0.1	1.1	15.0
Average Slope + 0.3m Wall	0.0	0.0	0.0	0.0	0.3
Average Slope + 5m Promenade	0.0	0.0	0.0	0.5	7.8
Average Slope + 5m Promenade + 0.3m Wall	0.0	0.0	0.0	0.0	0.3

These results indicate:

- Each proposed mitigation measure reduces the likelihood of intolerable overtopping occurring with benefits extended to at least 2100.
- The 0.3 m vertical wall provides the most benefit in comparison to the 5 m wide promenade.
- The combination of a 5 m wide promenade with 0.3 m vertical wall provides benefits to 2130.

Planning and design for the Regatta Grounds foreshore area should account for the need to adapt shoreline and foreshore protection structures to mitigate an increased frequency over overtopping due to sea level rise. The engineering options can be incorporated with broader foreshore planning to achieve cost effective outcomes, minimise impacts to visual amenity, maximise social and recreational values, while limiting wave overtopping to tolerable rates.

6.6 General design principles to reduce overtopping

As the planning and design for development at the Regatta Grounds foreshore progresses, the following general design principals can be followed to reduce the risk of impact to buildings:

- Where possible, adopt shallow sloping structures and foreshore areas.
- Increase the distance between the habitable ground levels and the estuary as much as possible (using promenades and/or pedestrian access areas). This distance will reduce overtopping energy and discharge that reaches the buildings.
- Utilise (relatively) inexpensive crest details to minimise overtopping (such as wave walls)
- Combinations of these design options can be used to good effect.

7 Macquarie Wharf Area Considerations

7.1 Introduction

The TasPorts Macquarie Wharf area provides a buffer between the Project Site and the coastal inundation hazard. The wharf is subject to a separate Port Master Plan and redevelopment process to ensure it continues to provide benefits to Hobart and the Tasmanian economy. Significant capital investment and upgrades are to be delivered from 2018 over a 15-year period (TasPorts, 2021).

Aspects of the assessment for the Project Site that are relevant to the Macquarie Wharf area are briefly discussed below.

7.2 Tsunami effects and consequences

Lewarn (2017) provided maritime advice on the potential effects and consequences of tsunami impacts at Macquarie Wharf. The potential consequences of a 'maximum credible event' include major damage to infrastructure and vessels and serious/critical injury to life. An event of this magnitude would cause widespread impact to the principal Hobart port areas and may indirectly impact the Project Site while the surrounding areas were reconstructed.

7.3 Wharf elevation and freeboard

Based on the topographic LiDAR dataset supplied by MPDC and shown in Figure 2.1, the elevation of the wharf deck surface around the seaward perimeter varies between approximately 1.6 to 2.9 m AHD. The highest elevations are along the southern face (berth 1, berth 2 and berth 3) and lowest elevations along the eastern face (berth 4, berth 5 and berth 6).

Table 7.1 provides a simple summary of the coastal inundation hazard levels, wharf deck surface elevation and freeboard (distance between the water level and deck surface). The southern face of the wharf is above the design water level for all scenarios, with approximately 0.6 m freeboard for the 1% AEP water level in the year 2120. The freeboard along the eastern face is relatively smaller, with the 1% AEP water level exceeding the deck by the year 2070.

Table 7.1 Wharf deck freeboard for coastal inundation water level scenarios

Year Horizon	Coastal Water Level (mAHD)	Southern Face (deck elevation ~2.9 mAHD) Freeboard (m)	Eastern Face (deck elevation ~1.6 mAHD) Freeboard (m)
5% AEP 2020	1.21	1.69	0.39
5% AEP 2070	1.56	1.34	0.04
5% AEP 2100	1.93	0.97	-0.33
5% AEP 2120	2.17	0.73	-0.57
1% AEP 2020	1.31	1.59	0.29
1% AEP 2070	1.66	1.24	-0.06
1% AEP 2100	2.04	0.86	-0.44
1% AEP 2120	2.27	0.63	-0.67

*negative freeboard indicates the water level has exceeded the deck surface elevation

The variation in deck elevation and consequence with respect to inundation is illustrated in the coastal inundation maps provided in Annex A. Inundation is initiated along the eastern face of the wharf for the 2070, 2100 and 2120 future climate scenarios. As discussed in Section 3, no allowance for waves is included in Table 7.1 and Annex A and it is noted that waves may influence the inundation extent and potential for damage at the wharf.

7.4 Overtopping and design waves

Further detailed assessments to support the engineering design process are needed to understand and mitigate the additional risk associated with wave breaking at the wharf. The assessments presented in Section 5 and 6 focus on the potential contribution of waves on the coastal inundation hazard, with a focus on the 100-year ARI (1% AEP) wave conditions and future climate sea level scenarios. The empirical relationships used to estimate overtopping are applicable to sloping shoreline gradients with (or without) shoreline protection structures, such as sloping rock revetments, vertical walls and/or promenades (characteristic of the Regatta Grounds shoreline area). These relationships are not suited to deepwater wharf areas.

The coastal water level and wave assessments presented in this report imply that wave loads on the structure are likely to increase under future climate sea level scenarios. Further assessments and interpretation to meet the requirements of *AS 4997-2005 Guidelines for the design of maritime structures* (and other relevant standards and guidelines) are needed to support the planning and design of upgrades to Macquarie Wharf, including breaking wave uplift and overtopping on the wharf deck.

The coastal inundation assessment results for the Project Site assume that the wharf is maintained and upgraded to accommodate future climate coastal hazards, and therefore continues to provide an adequate buffer to the Project Site.

8 Conclusion

The key outcomes from the *Macquarie Point Multipurpose Stadium Project of State Significance* coastal inundation assessment are as follows:

- Based on assessments presented by Kain et al. (2018) there is no risk of tsunami inundation at the Project Site based on a 'maximum credible earthquake/tsunami/high tide' scenario.
- The City of Hobart consider land below 3.0 mAHD83 as being at risk of coastal inundation, based on the 2100 1%AEP coastal inundation level being 1.94 m AHD83 and addition of a 1.0 m freeboard allowance. BMT's independent assessment of the coastal inundation level (not including the freeboard allowance) is generally consistent with the City's definition.
- Ground elevations across the stadium area within the Project Site are above 3.0 mAHD and there is no risk of coastal inundation up to the 2120 1% AEP. No mitigation measures related specifically to coastal inundation are considered necessary in the design and construction of the stadium.
- There are no effects on public health or necessary measures to manage risks (including emergency management) associated with coastal inundation up to the 2120 1% AEP coastal inundation event.
- The Regatta Grounds foreshore area slopes below 3.0 mAHD towards the shoreline. Considering the Regatta Grounds foreshore area:
  - Under present climate (~2020) conditions, there is a 5% chance that an asset with a 100-year design life and floor level at 1.5 mAHD will be impacted by the 1% AEP coastal inundation event. By the year 2100 and due to sea level rise, a floor level around 2.2 mAHD is needed to retain a similar risk profile. The City's requirement for freeboard and minimum habitable floor levels at 3.0 mAHD83 suggest the 1%AEP coastal inundation risk is low (<5% chance for an asset with a 100-year design life) beyond the 2100 planning horizon.
  - Land below 3.0 mAHD83 may still be suitable for non-residential development to support commercial and retail activities, subject to appropriate designs to ensure safety and resilience in the lower lying areas.
  - The overtopping assessment suggests areas behind steep sloping structures (1V:2H) and a narrow (less than 5 m wide) promenade may experience intolerable overtopping under the present day (2020) scenario and have a relatively high likelihood of intolerable overtopping by 2070. These types of structure should be avoided where possible.
  - Overtopping potential and consequence decrease with structure/foreshore steepness and width of promenade. The average (1V:4H) and shallow (1V:5H) structure/foreshore with wide (minimum 5 m) promenade may experience intolerable overtopping by 2100.
  - Given the relatively small waves and overtopping volumes assessed, simple mitigation measures can reduce the likelihood of an intolerable overtopping event impacting buildings.
  - Engineering measures to reduce coastal inundation and/or wave overtopping at sea defences may include ground treatment (i.e., raising the ground level or bunding), edge protection (e.g., sloping rock revetments, vertical walls) and edge details (e.g., wave return walls and crest walls). These options should be considered as part of the broader foreshore planning to achieve cost effective outcomes, minimise impacts to visual amenity, maximise social and recreational values, and achieve the required protection for buildings.
- The coastal water level and wave assessments imply that wave loads on the Macquarie Wharf structure are likely to increase under future climate sea level scenarios. The coastal inundation

assessment results for the Project Site assumes the wharf is maintained and upgraded to accommodate future climate conditions.

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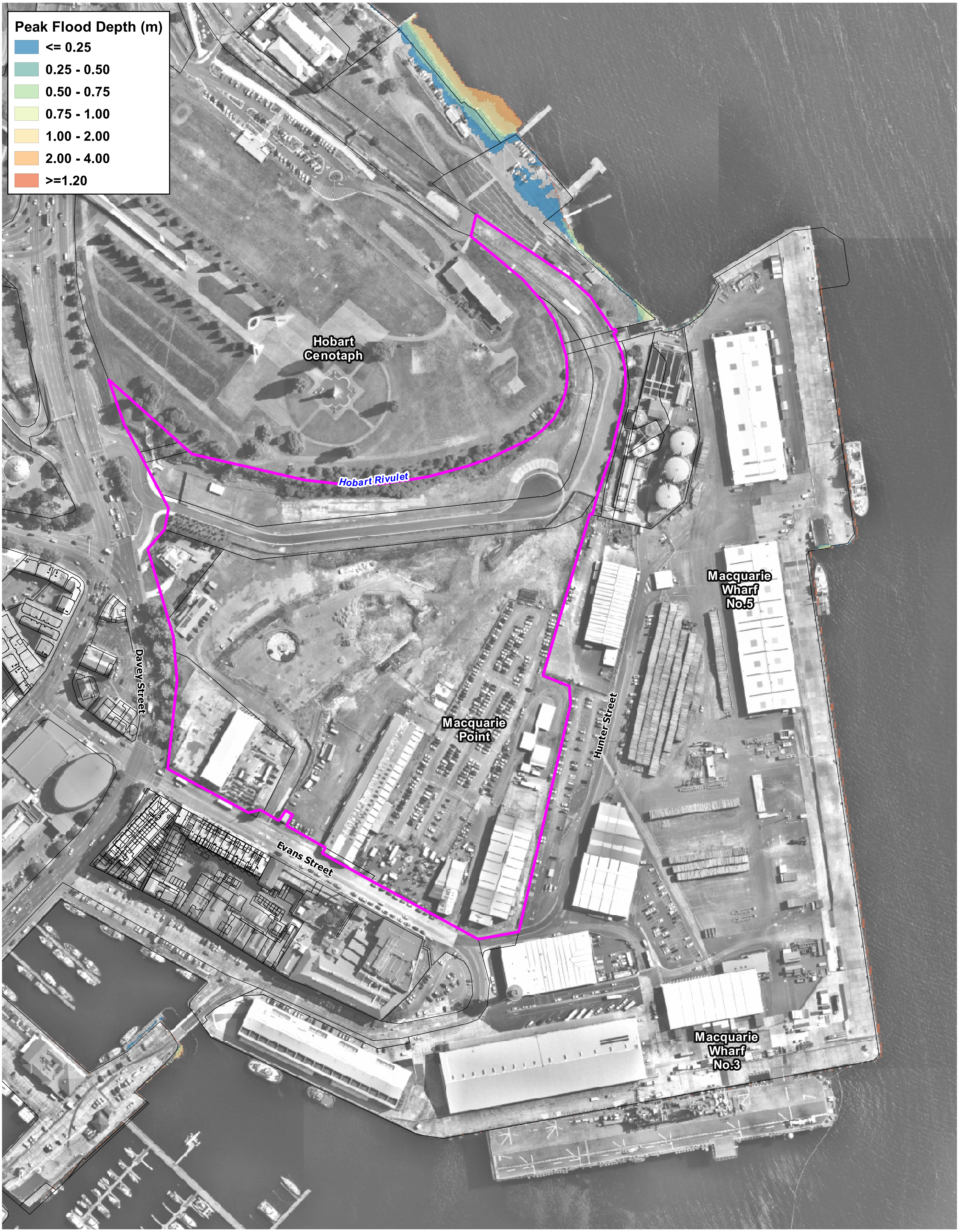
Annex A Coastal Inundation Maps



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Cadastre

Title:

Coastal Inundation Hazard: 5% AEP in 2020

Drawing: **A-1**

Rev: **A**

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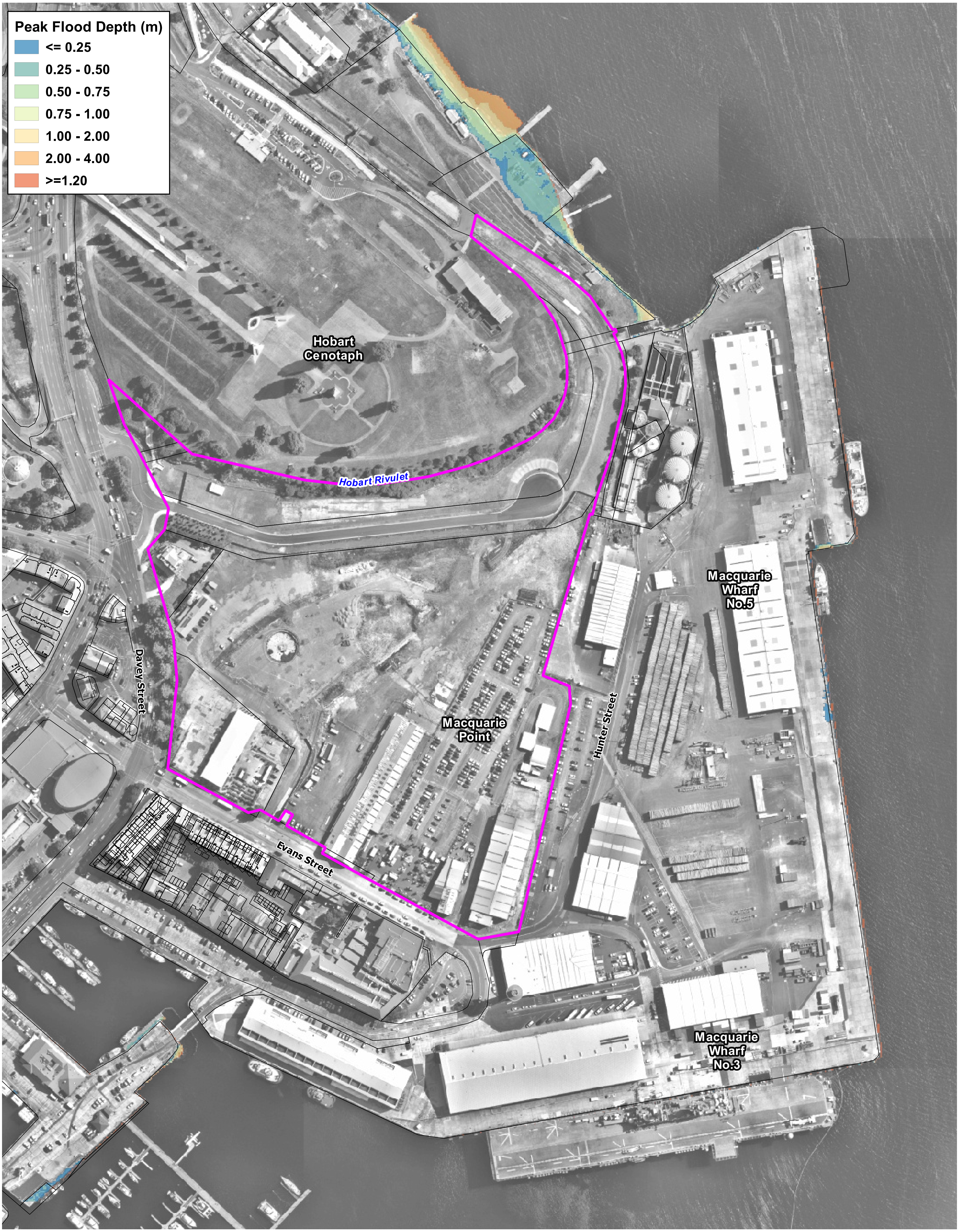

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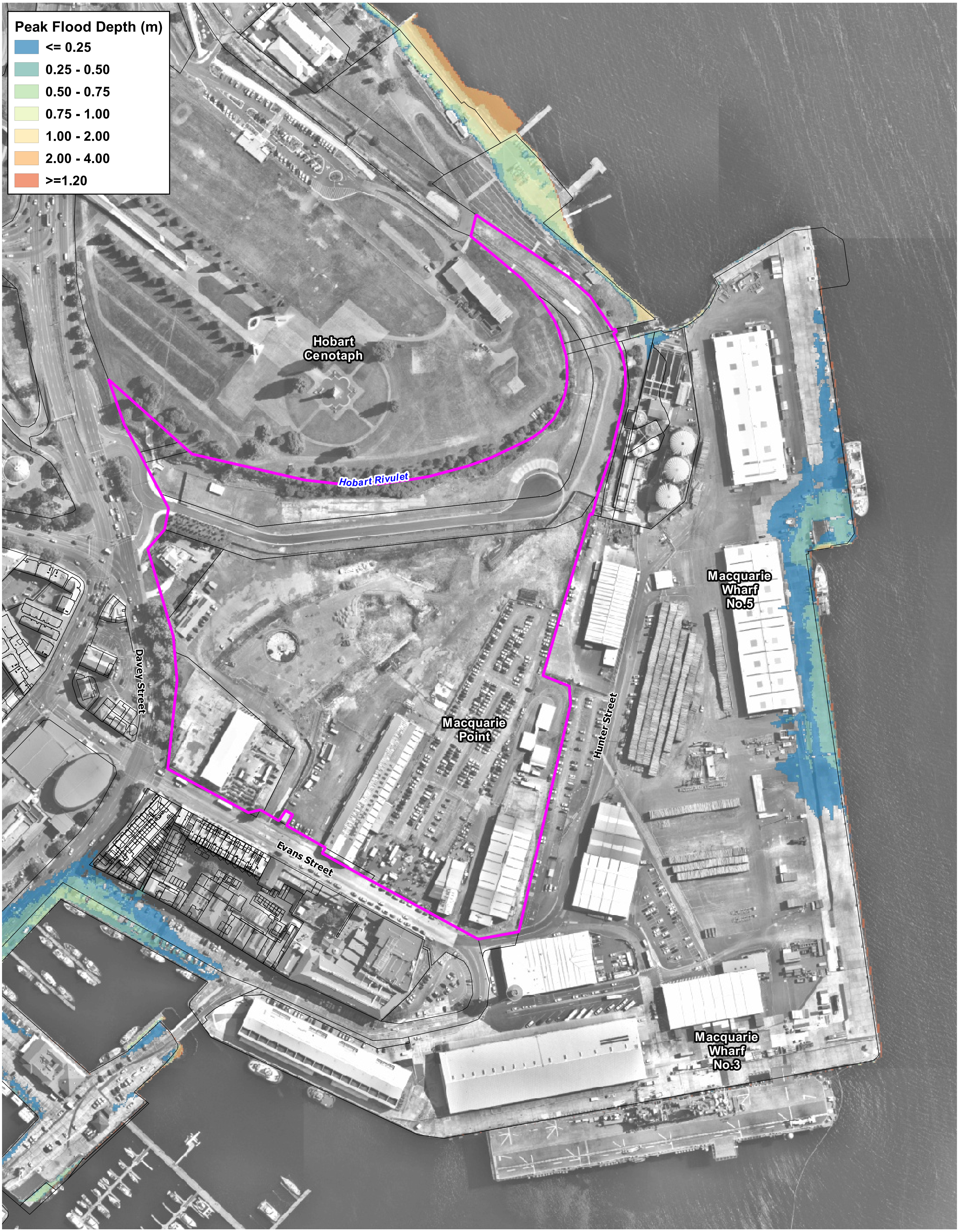

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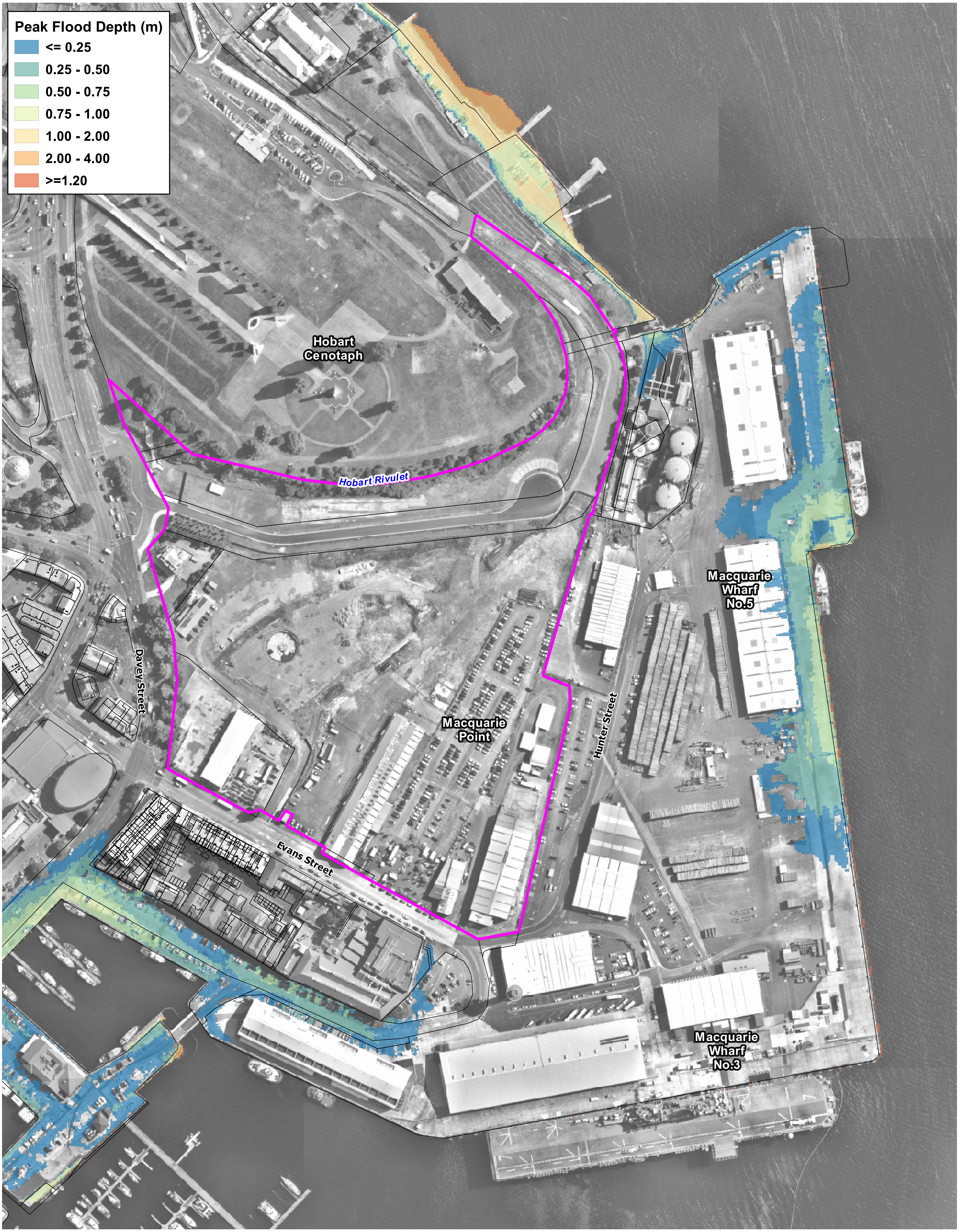

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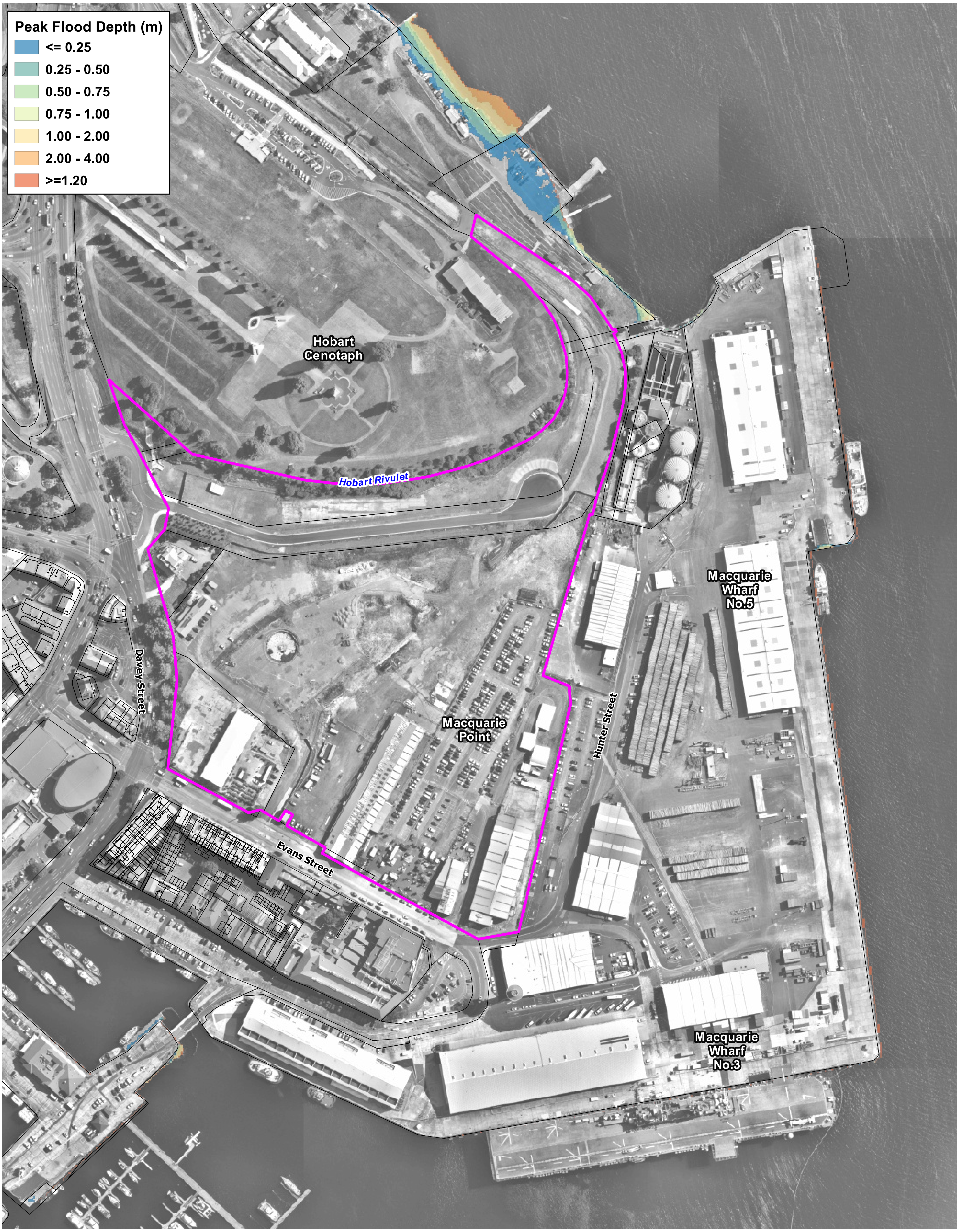

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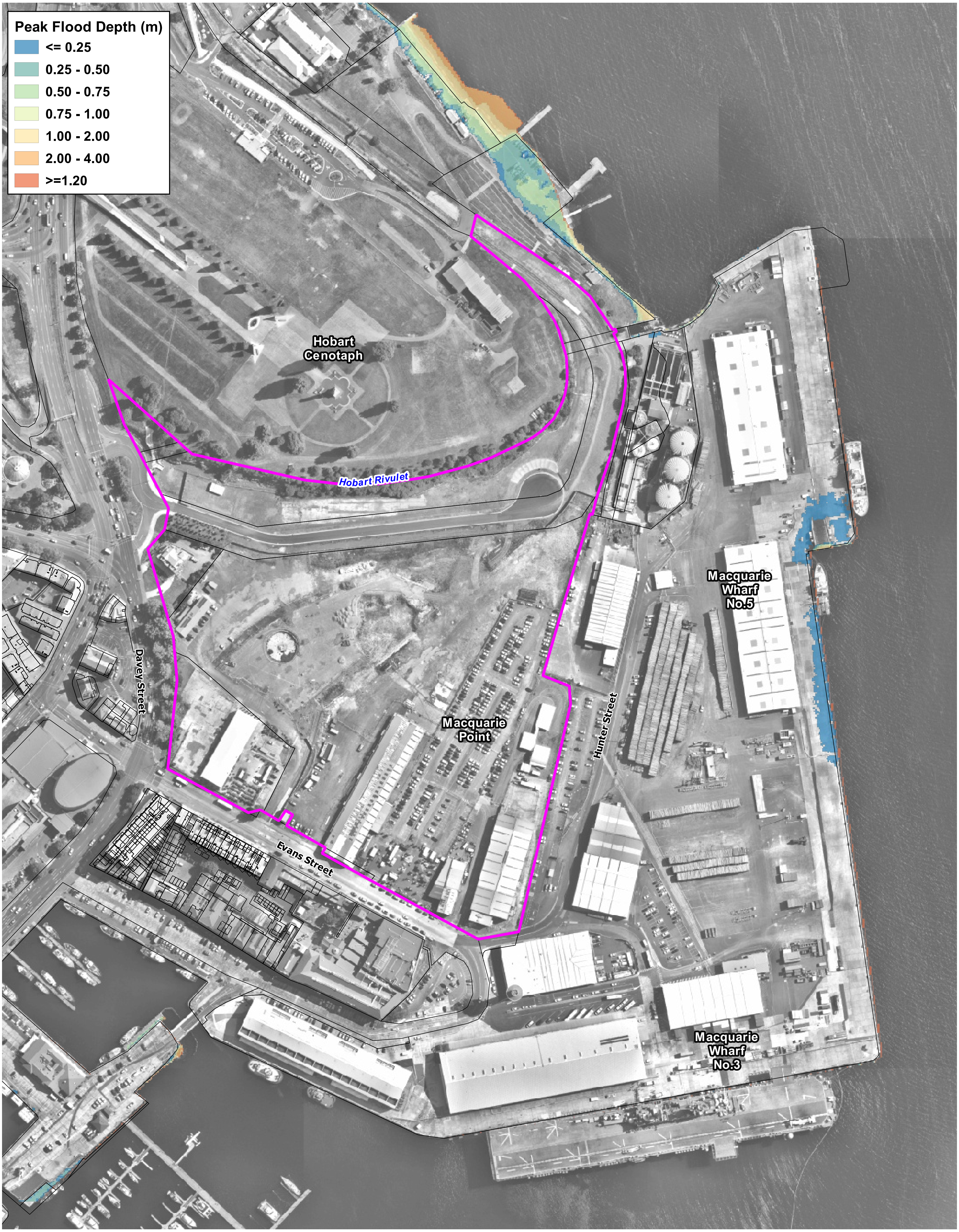

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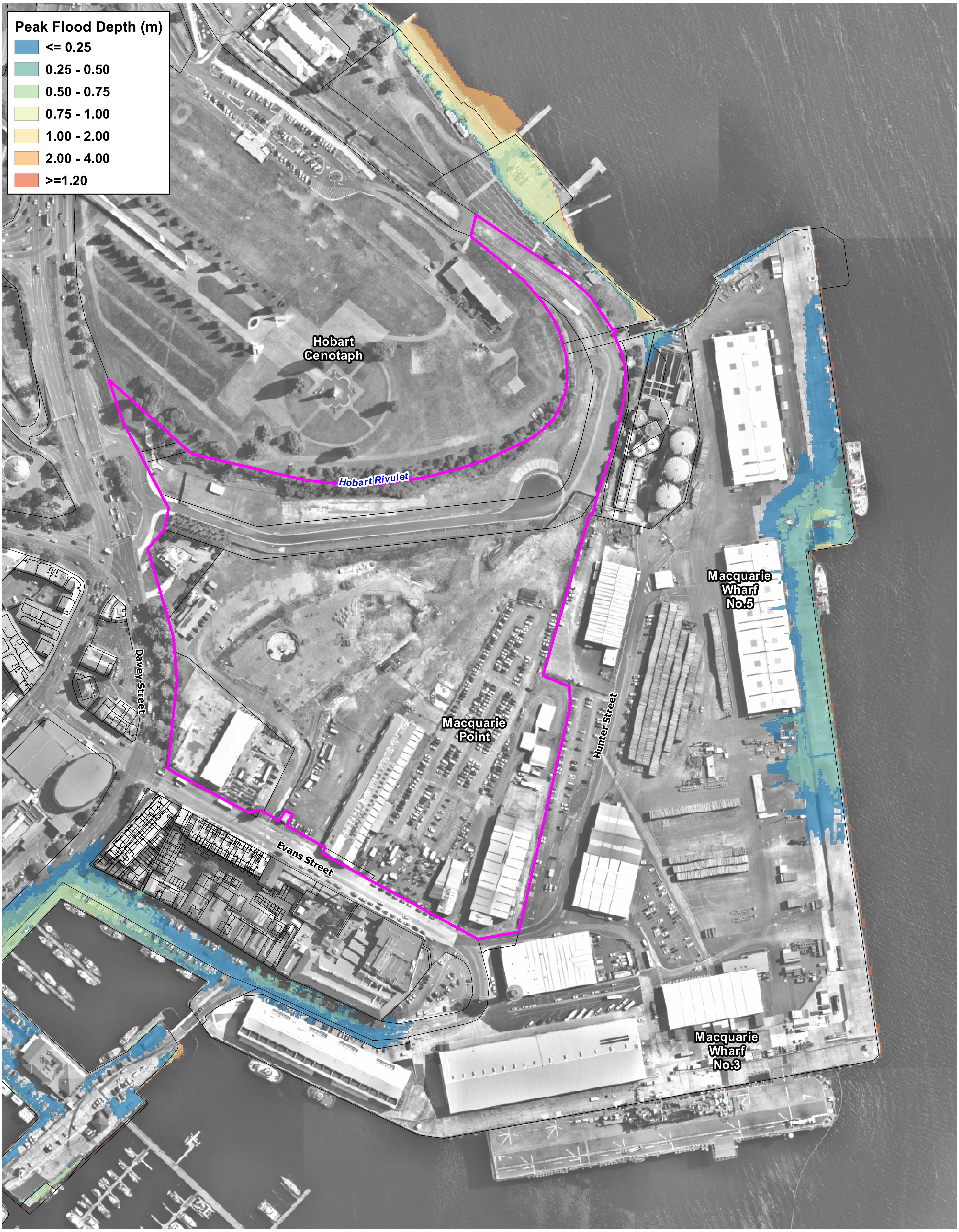

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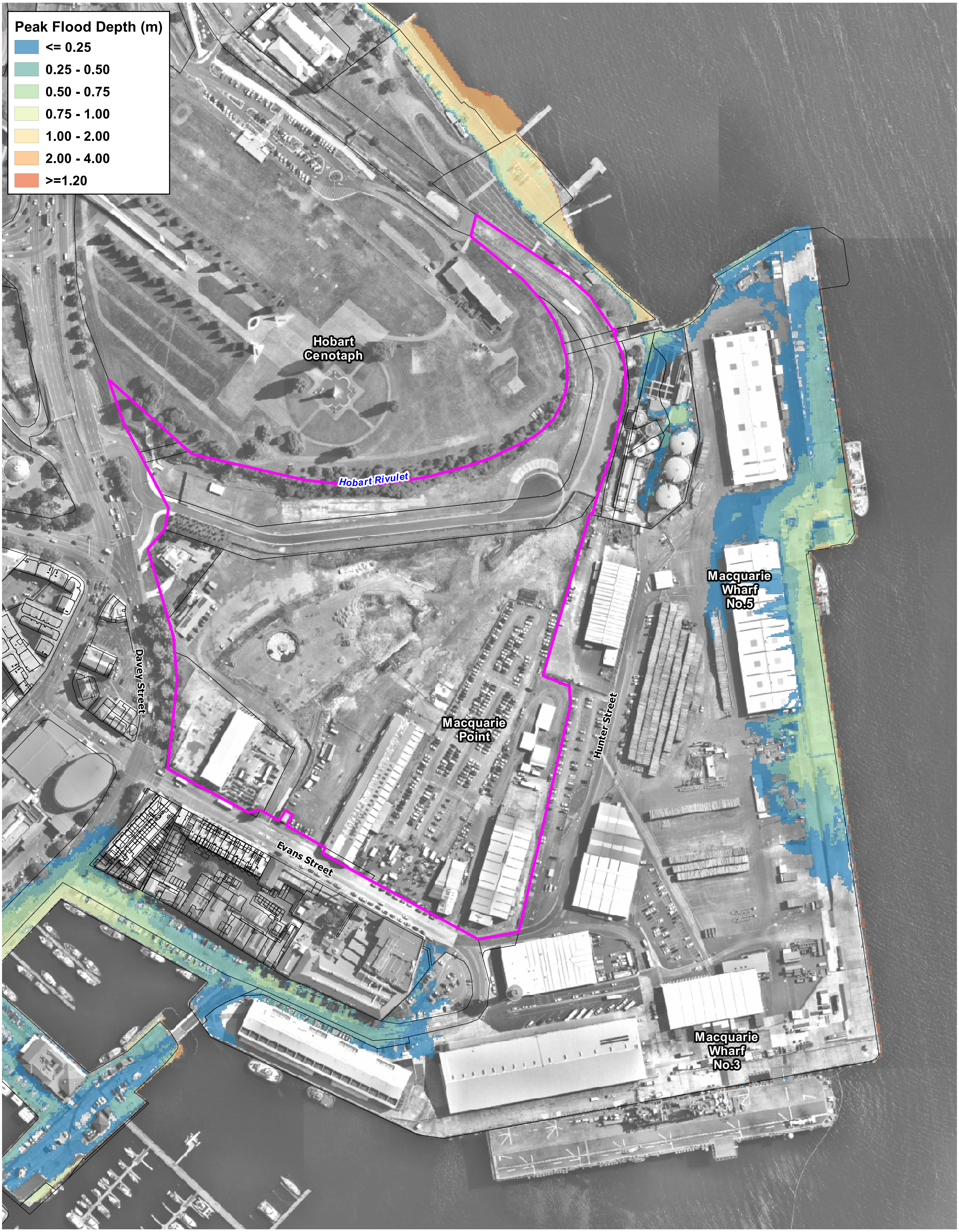

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Annex B Wave Modelling

B.1 Overview

The SWAN modelling package (Delft University of Technology, 2006) has been developed to assess extreme locally generated wind-waves at Macquarie Point. SWAN (Delft University of Technology, 2006) is a third-generation spectral wave model, which can simulate the generation of waves by wind, dissipation by white-capping, depth-induced wave breaking, bottom friction and wave-wave interactions in both deep and shallow water. SWAN simulates wave/swell propagation in two-dimensions, including shoaling and refraction due to spatial variations in bathymetry and currents. This is a global industry standard modelling package that has been applied with reliable results to many investigations worldwide.

B.2 Grid Extents and Bathymetry

A set of rectilinear grids have been developed to encompass the south-east Tasmanian coastline. The open-coast of the coastline was included in the model to validate the assumption that open-coast wave events do not penetrate and influence waves at Hobart.

Bathymetry has been interpolated onto these domains based on the following sources (in order of precedence):

- Electronic navigation chart depth soundings from AU444147 and AU443147 (Australian Hydrographic Office)
- SeaMap Tasmania Bathymetric Data - 5m contours (Institute for Marine and Antarctic Studies, 2007)
- Geoscience Australia Bathymetry and Topography (Geoscience Australia, 2009)

The adopted SWAN grid extents for the inner two models (most relevant) are shown in Figure B.1, along with the adopted bathymetry).

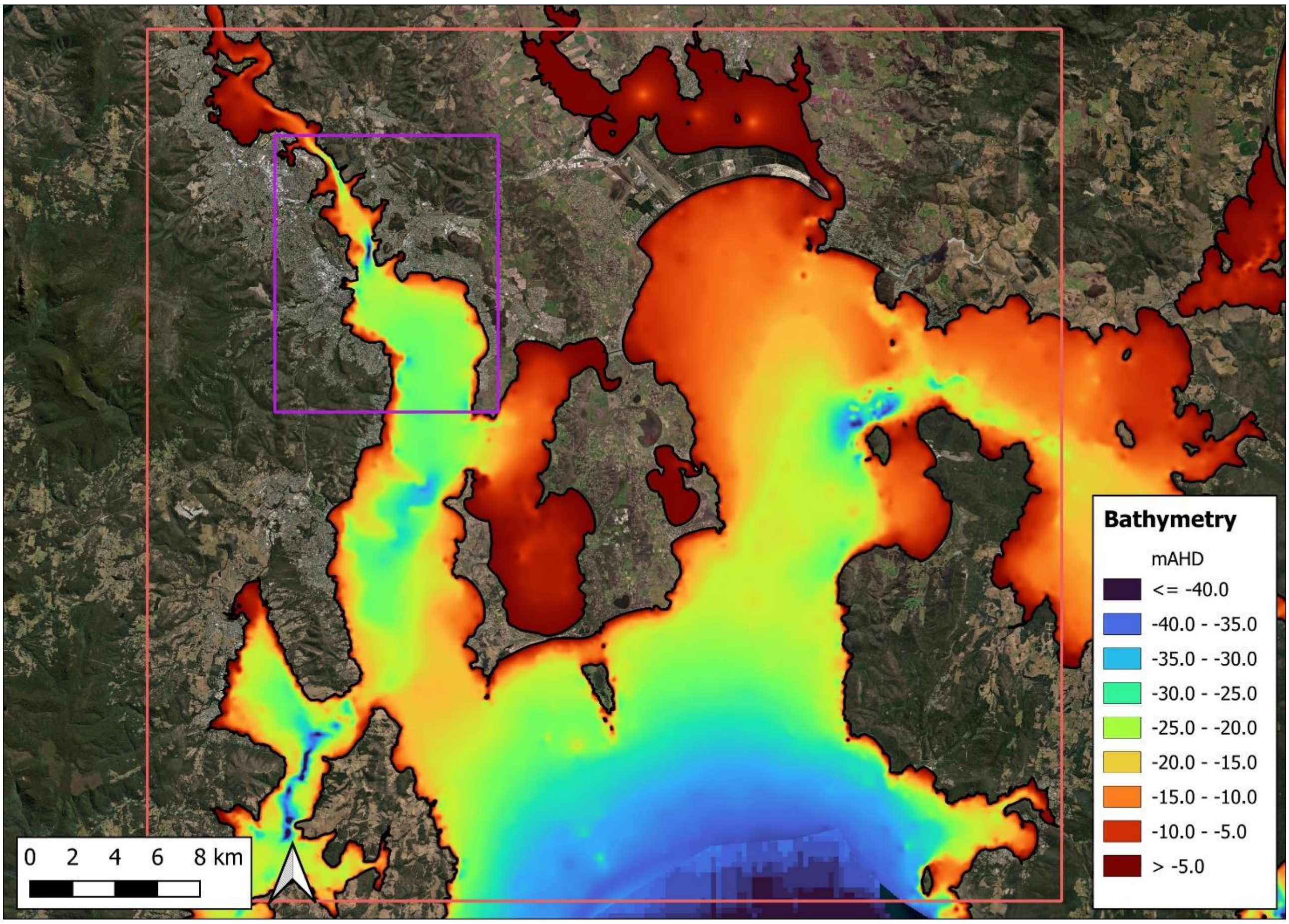


Figure B.1 SWAN model extents (only showing inner grids) and adopted bathymetry

B.3 Extreme Simulations

A set of simulations have been undertaken to assess locally generated wind-waves from extreme winds. Extreme wind speeds from the Australia Standards (AS1170.2; Standards Australia, 2021) were ran for directions clockwise from north to south in 10° increments. The wind speed conditions were converted from 3s gusts to 1-hourly average conditions based on the wind duration averaging factor presented in the Coastal Engineering Manual (U.S. Army Corps of Engineers, 2006). The adopted wind conditions (prior to interpolation into 10° segments) are shown in Table B.1.

Table B.1. Modelled Extreme Wind Speed Conditions (AS1170.2)

ARI	N	NE	E	SE	S
20-year	23.3	22.0	22.0	22.0	22.0
100-year	25.9	24.3	24.3	24.3	24.3

Model outputs were interrogated at 3 locations around the Macquarie Point development area, shown in Table B.2. , Table B.3. and Table B.4. for locations 1, 2 and 3 respectively.

Table B.2. SWAN Model outputs at Location 1

Wind Dir	20-year ARI (5% AEP)			100-year ARI (1% AEP)		
°N	Hs (m)	Tp (s)	Hsd (°N)	Hs (m)	Tp (s)	Hsd (°N)
0	0.76	3.06	11	0.87	3.21	12
10	0.77	3.07	15	0.87	3.20	16
20	0.76	3.01	20	0.87	3.19	21
30	0.74	2.86	27	0.86	3.17	27
40	0.73	2.80	36	0.84	2.89	37
50	0.73	2.68	49	0.83	2.83	49
60	0.74	2.63	64	0.84	2.81	64
70	0.74	2.61	79	0.84	2.74	78
80	0.74	2.63	91	0.84	3.57	89
90	0.75	3.61	99	0.85	3.88	98
100	0.75	3.73	105	0.85	4.07	104
110	0.76	4.00	110	0.85	4.13	109
120	0.75	4.05	113	0.85	4.19	112
130	0.73	4.07	116	0.83	4.22	115
140	0.71	4.10	118	0.81	4.19	117
150	0.67	4.06	121	0.76	4.20	120
160	0.62	4.61	123	0.71	4.19	122
170	0.56	4.82	126	0.64	5.16	125
180	0.49	4.89	130	0.57	5.14	129

Table B.3. SWAN Model outputs at Location 2

Wind Dir	20-year ARI (5% AEP)			100-year ARI (1% AEP)		
°N	Hs (m)	Tp (s)	Hsd (°N)	Hs (m)	Tp (s)	Hsd (°N)
0	0.64	3.16	21	0.73	3.23	21
10	0.66	3.15	24	0.76	3.21	24
20	0.68	3.13	29	0.78	3.18	30
30	0.70	2.81	37	0.81	3.14	38
40	0.72	2.64	49	0.83	2.83	48
50	0.76	2.71	63	0.87	2.85	64
60	0.82	2.73	80	0.93	2.93	79
70	0.89	3.24	95	1.02	3.54	95
80	0.98	3.60	108	1.11	3.68	108
90	1.08	3.75	118	1.23	4.06	118
100	1.17	4.03	125	1.35	4.21	125
110	1.27	4.22	130	1.42	4.33	130
120	1.32	4.30	134	1.49	4.47	134
130	1.35	4.38	137	1.54	4.73	137
140	1.36	4.49	140	1.54	5.06	139
150	1.35	4.66	142	1.52	5.10	141
160	1.31	4.70	144	1.47	5.11	144
170	1.25	4.70	146	1.40	5.09	146
180	1.16	4.65	148	1.32	5.01	148

Table B.4. SWAN Model outputs at Location 3

Wind Dir	20-year ARI (5% AEP)			100-year ARI (1% AEP)		
°N	Hs (m)	Tp (s)	Hsd (°N)	Hs (m)	Tp (s)	Hsd (°N)
0	0.28	2.31	31	0.33	2.48	29
10	0.32	2.37	45	0.37	2.54	44
20	0.38	2.38	57	0.44	2.48	56
30	0.46	2.50	68	0.53	2.65	68
40	0.54	2.64	78	0.62	2.84	78
50	0.64	2.83	87	0.73	3.02	87
60	0.74	3.12	96	0.86	3.27	96
70	0.86	3.40	104	0.98	3.61	104
80	0.98	3.64	112	1.11	3.76	112
90	1.08	3.77	119	1.25	4.08	118
100	1.18	4.03	124	1.36	4.20	124
110	1.28	4.19	129	1.43	4.31	128
120	1.32	4.27	132	1.50	4.52	133
130	1.35	4.38	136	1.55	4.71	136
140	1.36	4.52	139	1.56	5.04	139
150	1.34	4.62	143	1.53	5.09	142
160	1.31	4.64	146	1.48	5.10	145
170	1.25	4.63	148	1.41	5.07	148
180	1.17	4.64	151	1.33	4.77	151

B.4 Hindcast Model

As part of the first revision of this project, the SWAN model described above has been re-used to develop a long-term hindcast of waves and water-levels at the Hobart Regatta Grounds esplanade.

Boundary Conditions

Historical wind observations from the Bureau of Meteorology (BoM) at Hobart Airport (station number 094008) have been used to drive the SWAN model. These measurements were available up until 2011 with wind speed and direction recorded every 3 hours. The data was recorded at a height of 4m above ground level and was consequently converted to a height of 10m as required for SWAN. Hobart Airport is located approximately 13-14km east of Macquarie Point and the harbour and thus provides a relatively nearby source of wind data.

Water-levels were based on the Battery Point tide gauge with hourly data.

Simulation Setup

The hindcast model was ran using a lookup table approach, with a set of stationary simulations first run to determine conditions for distinct scenarios (listed in Table B.5.).

Table B.5. SWAN Lookup Table Scenarios

Parameter	Simulated Values
Wind x-velocity (m/s)	-30, -25, -20, -15, -10, -5, 0, 5, 10
Wind y-velocity (m/s)	-30, -25, -20, -15, -10, -5, 0, 5, 10, 15, 20, 25, 30
Water-level (m rel. AHD)	-1, 0, 1

A linear interpolation scheme was then used to determine the full wave height, period, and direction timeseries by querying the boundary conditions hourly between 1987 to 2011 (based on overlapping datasets). The results from this provided an approximately 24-year historical timeseries of modelled waves and water-levels which were subsequently used for overtopping.

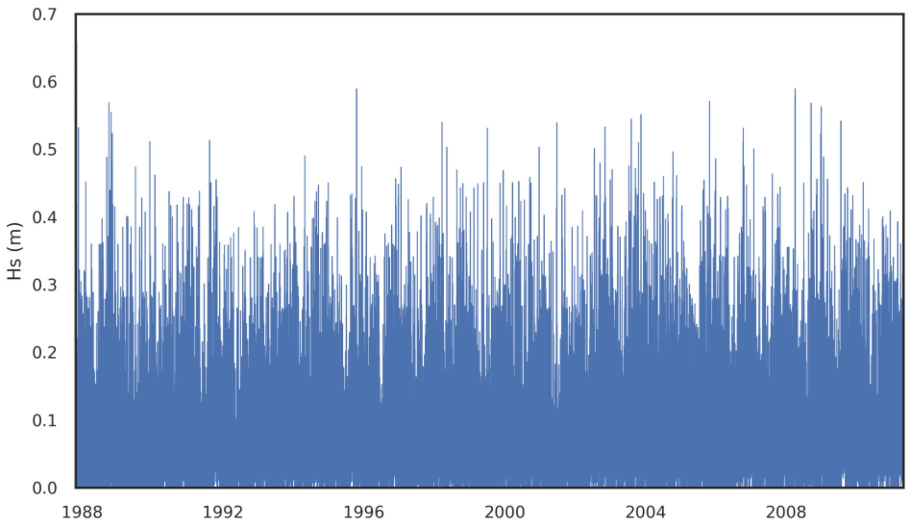
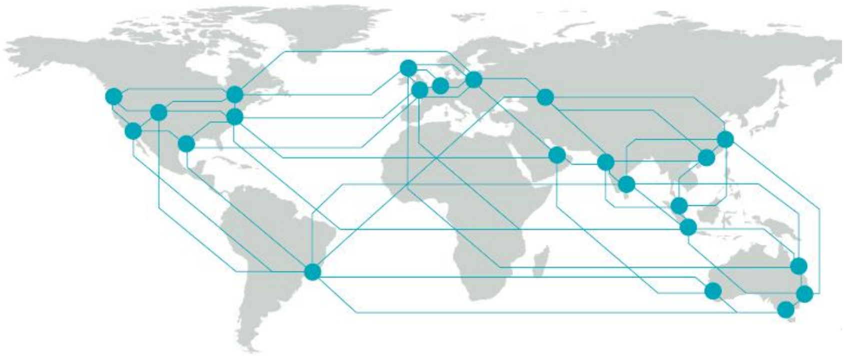


Figure 9.1 Example timeseries of significant wave height (m)



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