

HYDRODYNAMIC MODELLING OF A WAVE-DOMINATED TIDAL INLET SYSTEM: PORT PHILLIP BAY, AUSTRALIA

by

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Abstract

Tidal inlets are located between open seas and coastal lagoons. They experience complex hydrodynamic patterns owing to the actions of ocean waves and tidal currents - the two main drivers of various coastal processes including sediment transport, shoreline evolution, erosion, inundation and extreme sea levels. Better decision-making, sustainable management and adaptation requires numerical models to predict coastal hydrodynamics to gain a better understanding of how these mechanisms behave and change over time. Especially nowadays, human-induced pressures have been increasingly causing significant disruption to the formerly pristine state of these important coastal systems. However, our existing knowledge and the reliability of existing models are far from adequate. This study aims to provide a better understanding of coastal hydrodynamics, including the impacts of human-induced influences on these fields. The contributions of this research include the implementation of advanced measured-based physical packages (ST6) in a spectral wave model - WWMIII - which provides an important update in terms of predicting water waves focused on coastal regions. This implementation was tested in several water environments before being integrated into a coupled hydrodynamic model system (SCHISM-WWMIII) to study climatology of hydrodynamics at Port Phillip Bay, Australia based on long-term hindcast data. These data were derived from 27-year simulations applied to two bathymetry options: before and after channel deepening at Port Phillip Bay. To this end, these decadal data sets provide a unique opportunity to quantify hydrodynamics and possible impacts due to dredging on these fields in more detail.

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Huy Quang Tran (Melbourne, May 2020)

Declaration

I, Huy Quang Tran, hereby declare that this thesis is entirely my own work and that, to the best of my knowledge, it contains no material previously published or written by another author nor material that to a substantial extent has been submitted to qualify any other degree of the university, except where due reference is made in the text. I acknowledge that I have obtained, where necessary, permission from copyright owners to use any third-party copyright material re-used in this work.

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Chapter 1

Introduction

1.1 Coastal zones

Human activities have a close connection with coastal environments (*Small and Nicholls*, 2003). Thousands of years ago, these regions were selected for occupation based on recreational and economic interests, and the abundance of food resources from these water systems (*Syvitski et al.*, 2005). Nowadays, coastal zones are home to about 28% of the world's population. This portion includes the approximately 11% of people living in low-lying areas - within 10m of the mean sea level (MSL). The portion of the population living in these low-lying regions is predicted to increase twofold by the middle of this century (*Portner et al.*, 2019). Without improved understanding of coastal processes and behaviours, this increasing trend may force a decline in coastal health, posing significant challenges to decision-makers and coastal managers.

Citizens living near coasts experience great exposure and vulnerability to hazards and risks induced by ocean and cryosphere changes, including sea level rise (SLR) and extreme weather conditions (*Portner et al.*, 2019). In the coastal zones, tidal patterns are predicted to change because of SLR, which may increase local coastal hazards (*Neumann et al.*, 2015; *Nicholls et al.*, 2007; *Portner et al.*, 2019). At the same time, extreme ocean waves are expected to increase because of changes in wind patterns (e.g. *Young and Ribal*, 2019; *Young et al.*, 2011). This increasing trend can contribute to coastal extremes causing coastal erosion and flooding. It is important to note that increasing the global MSL will also affect coastal extremes because of the increasing frequency of extreme events. By the end of this century, most coastal areas in the world are expected to suffer from a 100-year event at least once a year (*Portner et al.*, 2019).

Human-induced pressure is another driver disrupting the pristine state of coastal zones causing changes in coastal processes. For example, many ports around the world have been expanded to serve large vessels (*Bradford and Siebinga*, 2009; *Van Maren et al.*, 2015), resulting in channel deepening (*Bradford and Siebinga*, 2009; *Liria et al.*, 2009; *Lopes and Dias*, 2015; *Zarzuelo et al.*, 2015) and land reclamation (*Shao et al.*, 2018). Human-induced disruptions have caused significant changes in hydrodynamics creating a loss of tidal prisms and properties (*Liria et al.*, 2009; *Lopes and Dias*, 2015), which can reduce ebb flows and modify

the current profile, and increase the residual currents, which enhance erosion and deposition processes (e.g. *Zarzuelo et al.*, 2015).

Tidal inlets are part of estuary systems that are connected with barrier-lagoons (*Kreeke and Brouwer*, 2017) and protected by barrier islands from the open seas. Therefore, these water areas are ideal to serve various human interests, including fishing, recreational activities, ship navigation and renewable energy (*Davidson-Arnott*, 2010; *Syvitski et al.*, 2005). From the hydrodynamic perspective, tidal inlets are dynamic systems that shape the coastal zone by controlling the exchange rate of sediment and flow between the ocean and the lagoons (*Davidson-Arnott*, 2010; *Kreeke and Brouwer*, 2017). Although knowledge of tidal inlets has generally increased, underlying processes in these systems are not well understood (e.g. *Bertin et al.*, 2019). The major challenge arises from the constant changes in physical processes, including the actions of waves and currents (e.g. *Dodet et al.*, 2013), changing coastal morphology. These changes can also be driven by both natural processes (e.g. *Duong et al.*, 2016) and human actions, which, in combination, can accelerate the rate of change. Improved understanding of coastal processes will help to ensure current and future generations can live, work and enjoy their lives in sustainable coasts.

1.2 Research aim

The overall aim of this research study is to provide a better understanding of hydrodynamics in tidal inlet - coastal lagoon systems including changes due to human-induced impacts on these coastal environments with the aid of numerical modelling and field observations. Specific objectives are as follows:

1. To update wind input and wave dissipation in an unstructured grid spectral wave model (WWMIII) to reliably represent water waves. This update has important implications as surface waves are linked to different coastal processes including coastal morphology and sediment transport. Further, such update also has methodological value, as WWMIII, coupled with the SCHISM model, is broadly employed for coastal wave modelling.
2. To develop an unstructured grid coupled wave-current model with high resolution integrated with the updated wave physics. This high-resolution coupled modelling system is then used to replicate long-term hindcasts.
3. To study the climatology of tidal inlet hydrodynamics based on the analysis of long-term hindcasts including extreme data analysis.
4. To quantify possible impacts of engineering modifications due to channel deepening on hydrodynamics in tidal inlet environments.

1.3 Thesis structure

This dissertation consists of eight chapters. The next chapter provides background on tidal inlet - lagoon features, highlighting the major forcing mechanisms that dominate coastal processes. Chapter 3 describes the location of the study at Port Phillip Bay, Australia, detailing distinguishing features and processes that are relevant to the hydrodynamics in this coastal environment. This chapter also

describes the field data available and reviews previous studies in this field of this research. Chapter 4 presents the implementation of newly-observation-based physics with results tested for several water environments: deep water, shallow water and the global scale. Chapter 5 describes steps to build up a coupled hydrodynamic modelling system to study the climatology of hydrodynamics in Port Phillip Bay, Australia. Chapter 6 presents results from data analysis of the hydrodynamic climatology in Port Phillip Bay, focusing on sea levels, waves and currents. Chapter 7 investigates the impacts of channel deepening on wave climate, sea levels and coastal erosion. The conclusions and future directions are provided in Chapter 8.

Chapter 2

Background

Tidal inlets are dynamic and complex in nature as these systems are connected with coastal lagoons, which are the main routes for navigation and replete with marine animals and plants. In line with this, humans have chosen these areas to live in, and have cherished these areas for thousand of years. Now, the pristine state of these socio-economic and environmental systems has been disrupted by changes in natural processes as well as human-induced pressures. These include SLRs, extreme events, human activities and engineering interventions. Under such new threats, it is indeed challenging to ensure coastal systems remain stable and healthy. To adapt to and mitigate such changes, further understanding of coastal environments is needed for better decision-making, planning, sustainable design and management. For this, we need further research to investigate the behaviours of the driving forces affecting coastal processes. This chapter provides fundamental concepts of tidal inlets - coastal lagoons and highlights the important underlying processes. It also reviews the state of the art of process-based models, widely used tool to predict such systems.

2.1 Tidal inlet systems

Tidal inlets¹ are important regions in estuary systems. These coastal environments were originally formed during the Holocene period (*De Swart and Zimmerman, 2009; Kreeke and Brouwer, 2017*) in line with SLR resulting in complex sediment processes. Consequently, many regions (e.g. valleys) were filled with sediment from highlands, creating many sand barriers that can still be observed in coastal zones around the world. Tidal inlet systems typically consist of short and narrow channels that serve as the transitional zones between the open seas and the inner water zones, which are often referred to as ‘coastal lagoons’. Interest in tidal inlets dates back thousands of years, since these regions were used as gateways for navigational purposes including commercial shipping and recreational activities (*Kreeke and Brouwer, 2017*). Nowadays, tidal inlets contribute 10-15% of global coasts (*De Swart and Zimmerman, 2009*), and are home to a large portion of the world’s population.

¹*The Dictionary of Earth Science published by Oxford University Press in 1999 defines a tidal inlet as ‘a narrow channel that connects the open sea with a lagoon. Tidal inlets often occur in barrier island systems and are typified by small-scale deltas at each end of the inlets, resulting from the high-velocity, tidal-reversal currents that flow through the channels’*

2.1.1 Geometry

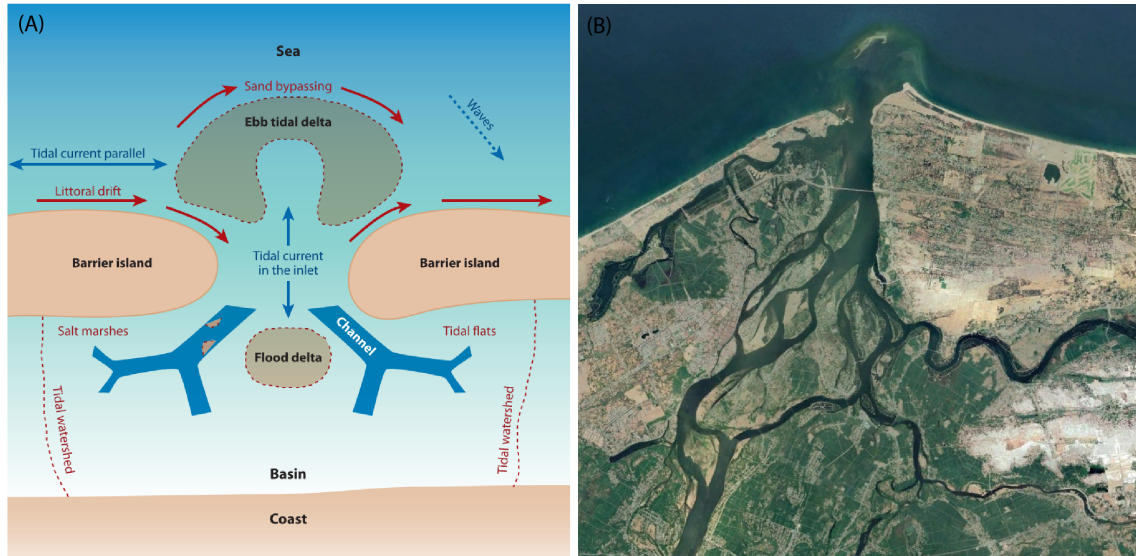


Figure 2.1: Tidal inlets: (A): conceptual case; (B): real example (Cua Dai inlets, central coast of Vietnam).

Tidal inlets can be recognised by the appearance of barrier islands, which protect back-barrier lagoons from the open sea. Figure 2.1 shows a conceptual configuration compared with a real example of a tidal inlet system - Cua Dai - which is located in the central region of Vietnam. Typically, tidal inlet geometry is divided into three regions: the inlet, the flood delta located in the inward side towards the lagoons and the ebb delta on the seaward side. Morphological evolution in each region is influenced by the prevailing hydrodynamic regime, closely linked to the actions of waves and currents. Specifically, *the inlet* consists of main channel systems, which can be curved, straight or orientated perpendicular to the barrier islands. In many cases, there is often a *terminal lobe* with a steep slope at the end section of the channel system. *The flood delta* is located in the back of the barrier islands. The shape of this region is strongly affected by tidal currents; however, it is often hard to find a common shape for the flood tidal delta. *The ebb delta* is situated on the ocean side in front of the barrier islands. The shape of the ebb tidal delta is controlled by the relative dominance of actions of tides versus waves (e.g. *Hubbard et al.*, 1979) - the ebb delta is large when tidal actions are dominate, and small in wave dominated environments. In the latter case, this is partly due to long-shore currents, which can transport the sediment forced in a down-drift side. The names of flood and ebb deltas stems from their relationship with tidal cycles; particularly, the ebb tidal delta is influenced by seaward movements of sands during ebb tidal cycles, while the flood delta is affected by landward sediment movements during flood tides.

2.1.2 Classifications

The traditional classification of tidal inlet systems, which stems from the seminal work of *Hayes* (1979), which is based on the characteristics of hydrodynamic forcing fields including surface gravity waves, tides and the relative dominance of actions across these two mechanisms.

Based on the wave field alone, tidal inlets can be classified into three categories. A tidal inlet domain is said to be low-wave energy when the mean significant wave height (SWH) is below 0.6m. Above this threshold but lower than 1.5m, it is called a medium- wave energy tidal inlet system. A tidal inlet has high wave energy when the mean SWH is above 1.5m. Based on tidal regimes alone, a tidal inlet system can fall into one of five classes (see Figure 2.2), ranging from micro-tidal inlets with a tidal range below 1m to high macro-tidal inlets where the tidal range is larger than 5m. In reality, a tidal inlet domain is shaped by the actions of both waves and tides, and therefore, tidal inlet systems can be classified by interpreting the curve lines in Figure 2.2. For example, a mixed-energy tidal inlet system with waves dominant has a tidal range of below 2m with a mean wave height less than 1m. This is the case for Port Phillip Bay, Australia.

Tidal-dominated inlets

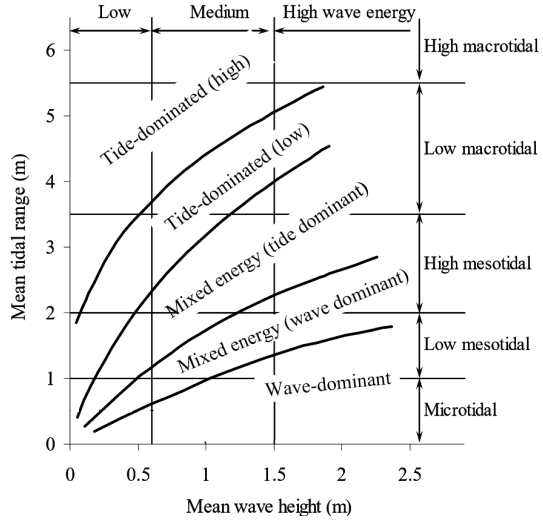


Figure 2.2: Classification of tidal inlets according to hydrodynamic regimes adapted from Lam et al. (2008) which was then improved by Vu (2013).



Figure 2.3: An example of typical shapes of tidal inlets - a tidal-dominated inlet in Victoria River, Northern Territory, Australia (left) and a wave-dominated inlet in Swansea, New South Wales, Australia (right). In the former, the mouth is larger because of the action of tides.

The tidal inlet region is often kept open by tidal actions. The level of opening depends on the tidal prism, which is measured by the flow volume entering the inlet during flood tidal cycles and leaving during the ebb tides. In many cases across the globe, the tidal range in tidal inlets does not exceed 4m.

Wave-dominated inlets

The inlet tends to be narrowed by wave actions because of wave-driven cross-shore and long-shore transport, tidal prisms and wave-induced long-shore sand transport.

2.2 Dominant Forcing

Underlying processes in tidal inlets are complex because of the various agencies involved. In many tidal inlet systems, processes are driven by both wind-waves and currents (Dodet *et al.*, 2013; Olabarrieta *et al.*, 2011), often a major role also for tides. These mechanisms are the main drivers of coastal sediment transport and morphological changes (Roelvink and Reniers, 2011; Stive and Wang, 2003). In extreme conditions, storm surges can be a dominant driver (De Swart and Zimmerman, 2009), caused by atmospheric pressure, and wave setup (e.g. Idier *et al.*, 2019), resulting in a significant change in water levels (see Figure 2.4). This section reviews these dominant fields in coastal regions.

2.2.1 Astronomical tides

Tidal generation

Tides are generated by gravitational forces exerted by the moon, the sun and earth systems that generate the upward and downward movements of the water surface on regular basis (Boon, 2004; Brown, 1989; Dronkers, 2005; Haigh, 2017). The term ‘tides’ here is referred as ‘astronomical tides’ meaning that tidal forces and fluctuations are generated by the gravitational systems that differ from those produced by meteorological mechanisms such as strong wind or barometric pressure. Although the complexity of land features and bathymetry may affect the prediction of astronomical tides, harmonic analysis is still the most-widely used method to predict tidal patterns (e.g. Massel, 2013). A major advance in our knowledge about tidal generation was made by Isaac Newton (1642-1727) with the invention of gravitational theory. This theory stated that every object in the universe attracts every other object. The force between two objects can be expressed in term of the mass product of these objects and the square of the distance measured from one central point to another. This forcing term F is expressed as follows:

$$F = G \frac{M_1 M_2}{r^2} \quad (2.1)$$

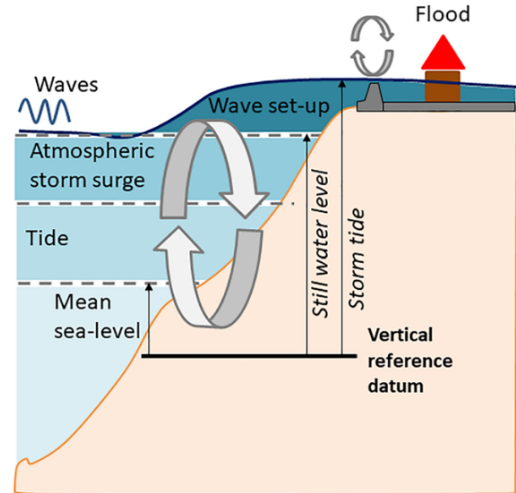


Figure 2.4: Various mechanisms affecting extreme sea levels. Atmospheric forcing fields (wind, pressure) cause storm surges, which increase the water level. The effect is exacerbated by the wave setup (adapted from Idier *et al.*, 2019).

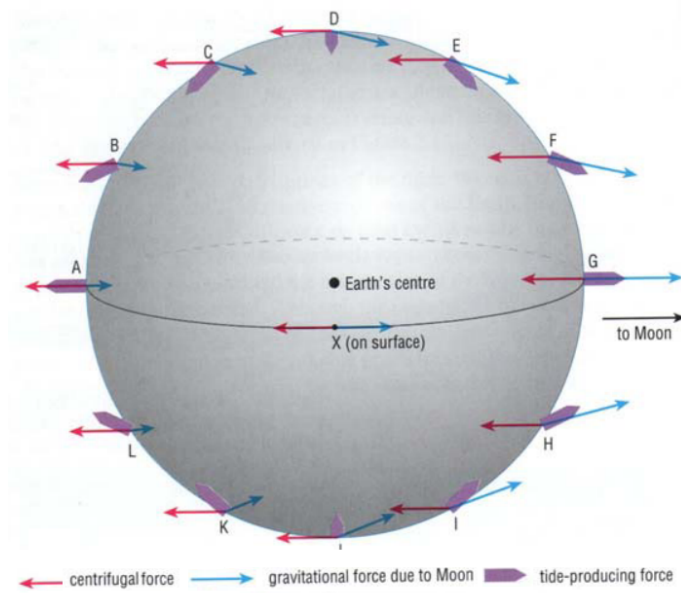


Figure 2.5: An illustration of tidal generating forces on the earth's surface, adapted from Brown (1989). The tidal-producing term is the resultant of centrifugal forces and gravitational forces arising from the moon acting on the earth.

where G is the gravitational constant (equal to $6.674 \times 10^{-11} \text{m}^3/\text{kg} / \text{s}^2$), r is the distance between the two objects and m_1, m_2 are the masses of the two objects. Newton's theory assumes that both the moon and the earth comprise a unique system, with the central mass located within the earth. The reason for this is that the earth is about 81 times larger than the moon. For convenience, each system (e.g. the earth, sun) is considered a single point. Therefore, the total mass at each point is located at its centre.

As a system, both the earth and moon rotate in the shared central mass with a cycle of 27.3 days. A further assumption considers the earth as an ideal system, without continental land masses. This means that the earth's surface is entirely covered by water. Therefore, all points on the earth must be regulated on the same path with an angular speed of $2\pi/27.3$ days and the same centrifugal force (Haigh, 2017). The centrifugal forces are the same everywhere. However, the forcing term due to gravity caused by the moon on earth varies because not every point on the earth has the same distance to the moon - the closer a point to the moon, the greater gravitational forces exerted. The resultant of gravitational and centrifugal forces is known as the tidal generating force (see Figure 2.5). Similar to the moon, the sun also generates forces on the earth. Although the mass of the sun is many times larger than that of the earth, it is located 360 times further away than the moon. Consequently, the forcing term caused by the sun is only 0.46 times approximately that of the moon.

The moon, the sun and the earth do not rotate on the equatorial plane; the earth rotates around the sun, for example, on a plane called *ecliptic* plane. This plane can be inclined up to 23.45° compared with the equatorial plane of the earth. Additionally, the orbital plane of the moon differs by approximately 5° relative to the ecliptic plane. As the moon rotates, its position relative to the earth's latitude can vary up to 28.5° at the high latitudes. Further, it is important to note that the orbital planes of the earth and the moon are not circular, but rather, elliptical. Therefore, the strength of tidal-producing forces varies depending on the distance from the earth to the sun and the moon.

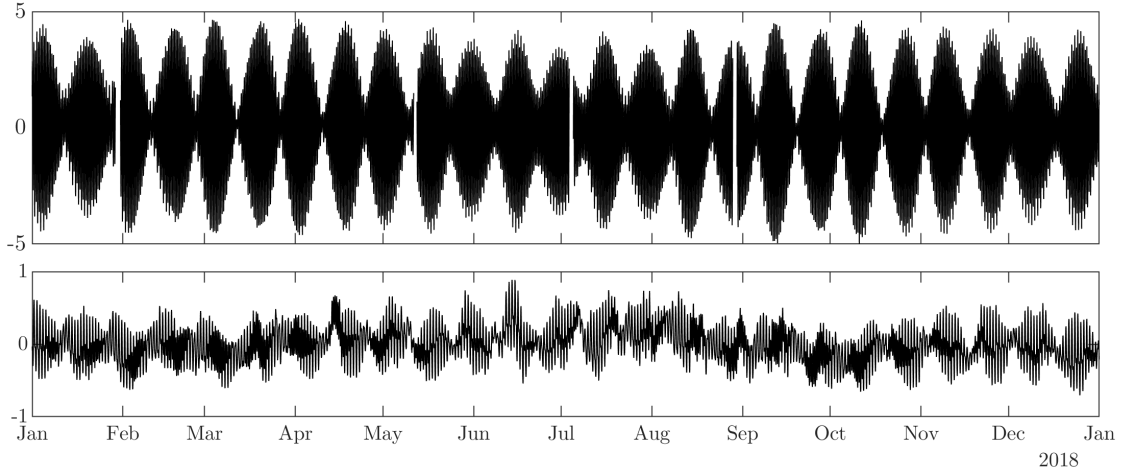


Figure 2.6: Water elevation at Broome, Western Australia (top) and Portland (bottom), South Australia based hourly sea level provided by the Australia Bureau of Meteorology (BOM) for 2018. Whereas the tidal range at Broome is very large ($>9m$), the tidal range at Portland is small.

Tidal regimes

The position of the sun and the moon can influence several characteristics of observed tides. For simplicity, it is assumed that the declination of the sun and the moon are the same (equal to zero). The first situation to consider here is when the moon, the earth and the sun are in line with each other. This is known as *spring tides*, which occur during the new and the full moon (case A in Figure 2.7). *Neap tides* (case B in Figure 2.7) occur when the moon is in a halfway position between the full and the new moon.

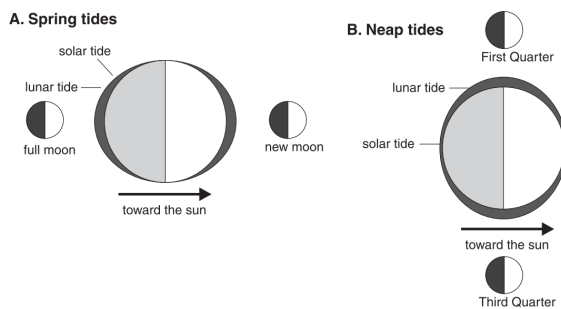


Figure 2.7: Impact of the sun and the moon on tidal patterns adapted from Davidson-Arnott (2010).

Tidal patterns in coastal zones

The rise and fall of water levels along coasts can be observed on a regular basis. The rise is referred to as the *flood tide* while the fall corresponds to the *ebb tide*. Between a flood and an ebb tide, there is a period during which water is free from stress; this is called the *slack tide*. *High tide* occurs during the flood cycle when the water level is approaching its maximum level. Conversely, *low tide* occurs during the ebb tide when the water falls to its lowest level. The difference between the high tide and the low tide is referred as the *tidal range*.

The tidal range can vary significantly from a few centimetres to over 15m depending on the location. The largest tidal range was observed in the Bay of Fundy, Canada with a fluctuation of over 16m. In Australia, the largest tidal range often occurs in Western Australia (up to 10m), but this range is small in some locations. Figure 2.6 provides an example of tidal variations at two different sites at Broome and Portland located in Western and South Australia respectively for the full year of 2018. The tidal fluctuation in Broome can be approximately 10m, while this is only 1m at Portland. In terms of the mean, tides can be classified into three different types. The first is the *macrotidal* tide, which has a range larger than 4m. The term *mesotidal* is applied for a tidal range between 2 and 4m, and if the tidal range is less than 2m, it is called *micro-tidal*. Another way of classifying astronomical tides relates to the daily occurrence of high and low tides. A *semi-diurnal* tide has two high and low tides daily, whereas a *diurnal* tide has only one single high and low tide. *Mixed* tide has either one low tide and two high tides or two low tides and one high tide per lunar day. To determine the tidal pattern at a location with respect to semi-diurnal, diurnal or mixed tides, the following formula can be used:

$$T = \frac{\eta_{K_1} + \eta_{O_1}}{\eta_{M_2} + \eta_{S_2}} \quad (2.2)$$

where η_{K_1} and η_{O_1} on the numerator are the diurnal luni-solar and the principal tidal amplitude respectively. On the denominator, η_{M_2} is the amplitude of the semi-diurnal principal lunar constituent and η_{S_2} is the amplitude of the semi-diurnal principal solar constituent.

Tidal asymmetry

Tides in the ocean are considered frictionless, extremes waves that occur without losing energy (*Dronkers, 2005*). This is not the case in restricted seas as ocean tides propagate onto limited water depths, and also their amplitude can no longer be considered small with respect to the depths. Consequently, tides are distorted and become asymmetric (*Dronkers, 2005*). Tidal distortion can be explained by the nonlinear growth of tidal components (e.g. *Friedrichs and Aubrey, 1988*) as the ocean tides consist of a large number of harmonics with different frequencies.

The energy transfer from one frequency to another can result in symmetric tidal velocities. Such a distortion can induce transport processes. Therefore, the role of

Table 2.1: Classification of tides according to Haigh (2017).

T value	Type
0-0.25	Semi-diurnal
0.25-1.5	Mixed, semi-diurnal
1.5-3	Mixed, diurnal
> 3	Diurnal

tidal asymmetry in coastal regions is critical for sediment transport applications. In tidal inlets, such asymmetric pattern can affect the equilibrium morphology; for instance, the uptake of sediment during a flood and ebb tides is not equal. The asymmetrical pattern between these tidal cycles creates an imbalance in sediment transport which thus may not reach an equilibrium morphology affecting coastal shorelines (*Dronkers, 2005; Friedrichs and Aubrey, 1988*).

Tidal currents

Tidal currents have a unidirectional form (*Davidson-Arnott, 2010; Roelvink and Reniers, 2011*). In coastal system with a large tidal range, tidal currents can erode and transport sediment. In micro-tidal ranges, tidal current velocity is only important in the inlets and entrances of coastal lagoons. Similar to tides, the asymmetrical pattern also occurs in tidal currents. Owing to difficulty in observations, the natural distortion of tidal currents has not been inadequately described (*Friedrichs and Aubrey, 1988*); therefore, process-based models are often employed to predict the nonlinear distortion of tidal currents. Figure 2.8 shows

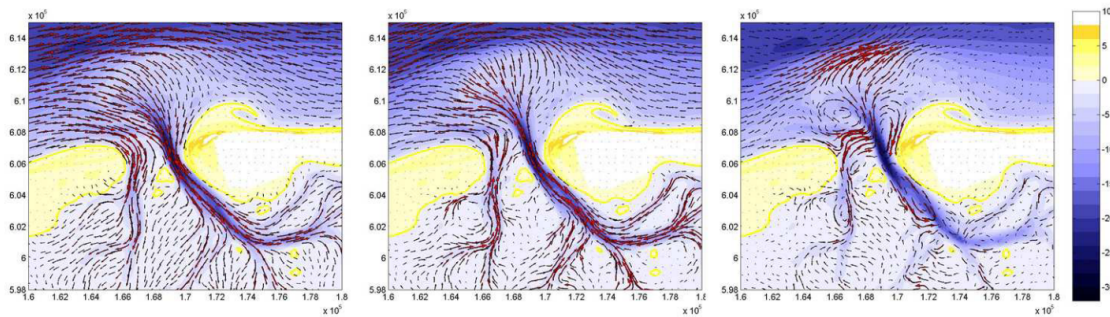


Figure 2.8: Tidal current pattern in Ameland Seagat inlets (*Roelvink and Reniers, 2011*). The first and second panels respectively show the maximum flow pattern during flood and ebb tides. The last panel depicts 3-hour residual currents. The water depth in this inlet system is up to 30m, as indicated in the color bar.

tidal current patterns in a real tidal inlet system - Ameland Seagat in Dutch Wadden Sea (*Roelvink and Reniers, 2011*). As can be seen, the current direction is reversed between the flood and ebb tides. The tidal current is concentrated in the narrow entrance in both tidal cycles, but with different velocities.

2.2.2 Wind-waves

Atmosphere and ocean form a coupled system that results in momentum, heat, and mass exchanges from the atmospheric layers to the ocean through the contact surface - a rough interface where dynamic and unsteady waves are formed and induced in the ocean (e.g. *Massel, 1996, 2013*). Surface gravity waves, generated by wind, are an obvious phenomenon in the ocean. In this thesis, the term of ‘wind-waves’ refers to waves generated by wind blowing over sea surfaces. Wind-generated waves can be *swells* or *wind-seas*. Swells are generated by storms at large distances from deep oceans leaving out of the origin of generation that reach the coast after many hours and days (*Holthuijsen, 2007; Young, 1999*). In contrast, wind-seas are generated by local wind.

Wind-waves, even in their less complex form (in the deep oceans), present a daunting research challenge (e.g. *Babanin, 2011; Babanin et al., 2017*), from the experimental, analytical, modelling and statistical perspectives. Related challenges have been well documented in many studies such as *Young (1999)* and *Holthuijsen (2007)*. The major difficulty is that ocean waves are generated by nonlinear, turbulent wind fields, exhibit dissipated energy due to breaking (*Babanin, 2011*), and can be modulated by ocean currents (*Peregrine, 1976*), which can increase nonlinearity. There is no exact solution to solve wave equations in nonlinear form; therefore, approximations are applied to solve these nonlinear equations.

Linear wave theory

The most common method to describe surface gravity waves is the small-amplitude approximation or linear wave theory applied for deep water introduced by *Airy (1845)*. The mathematical descriptions are provided in Appendix 1. Essentially, this theory is applied for water with idealised physics and considering gravitational forces as the only external forcing. Mathematically, linear wave theory requires two equations based on the conservation of mass and momentum, to be solved. These equations can be expressed via the potential velocity function (*Holthuijsen, 2007; Young, 1999*), which then provides Laplace and Bernoulli equations. The Laplace equation gives an analytical solution of a *harmonic wave*:

$$\zeta(x, t) = A \sin \omega t + kx \quad (2.3)$$

and *particle velocities*:

$$u_x = \frac{\partial \Phi}{\partial x}, u_y = \frac{\partial \Phi}{\partial y} \text{ with } \Phi = \left[\frac{\omega A \cosh[k(h + d)]}{k \sin(kd)} \right] \cosh(\omega t - kx) \quad (2.4)$$

and the *dispersion relationship*:

$$\omega^2 = gk \tanh(kd) \quad (2.5)$$

where η is surface elevation; ω is radian frequency, k is wave number, A is wave amplitude, d is water depth, and u_x, u_z is particle velocity in the horizontal x and vertical z - space.

From the dispersion relationship (Eq. 2.3), phase speed (or velocity) can be derived as:

$$c = \frac{L}{T} = \frac{\omega}{k} = \frac{g}{\omega} \tanh(kd) = \sqrt{\frac{g}{k} \tanh(kd)} \quad (2.6)$$

This expression demonstrates that the phase velocity depends on wave number and frequency. In other words, the speed of wave propagation is influenced by wave frequency and length. This expression also shows that longer waves propagate faster than short waves. Therefore, longer waves in deep waters propagate in front of shorter waves (e.g. *Holthuijsen, 2007*). In finite water depths, however, $\tanh(kd)$ approaches kd ; therefore, the speed of wave propagation becomes:

$$c_{finite} = \sqrt{gd} \quad (2.7)$$

This means that propagation speed is not affected by wavelength or wave frequency in shallow waters. Because waves propagate as groups in the real ocean, wave group velocity is often used instead of phase velocity. Wave group velocity can be derived by considering two harmonic waves with minor differences in frequency propagating in the same direction. If these two waves have the same amplitude, the total surface elevation becomes:

$$\zeta = \zeta_1 + \zeta_2 = A \sin(\omega_1 t - k_1 x) + A \sin(\omega_2 t - k_2 x) \quad (2.8)$$

By applying trigonometric rules and rearranging, group velocity can be derived as follows:

$$c_g = \frac{\Delta\omega}{\Delta k} = \frac{\partial\omega}{\partial k} = \frac{c}{2} \left[1 + \frac{2kd}{\sinh(2kd)} \right] \quad (2.9)$$

The major feature of this equation is that wave group velocity approaches phase velocity in shallow waters, implying that each individual wave component propagates at the same speed as wave groups.

Waves in coastal zones

In restricted seas, the wave field becomes a different physical object (*Babanin et al.*, 2017). The wind input is changed because of the sea-land interface while dispersion decreases because of limited water depth and nonlinearity increases in line with the coexistence of other physical mechanisms (e.g. currents). For the wind forcing, in contrast to deep seas, the wind field in the coastal region is affected by land and many other forms of obstacles. More specifically, wind cannot blow over a large distance or *long-fetch*. For example, a coastal domain is often bounded in several directions. Such land-sea interfaces can restrict the evolution of both waves and the wind field. The wind on open seas is often stronger than that in bounded seas (e.g. 5%; see *Cavaleri et al.*, 2018). This means that any error in estimates of the wind field can directly contribute to wave condition. Water depth is a major contrasting feature in restricted seas compared with open oceans. Limited water depth affects wave evolution and energy, which can result in *refraction*, *shoaling*. Another difference relative to the deep oceans is that waves in coastal seas typically propagate in the presence of a current, which can be generated by *tides* resulting in *wave-current interactions*. Effects in shallow waters have been well described thoroughly in resources such as *Holthuijsen* (2007); *Massel* (2013); *Young* (1999) using the linear wave theory.

Wave-induced currents

Waves transport both energy and momentum. Towards the coast, waves break and release momentum which is then transferred to the current field (e.g. long-shore current; *Roelvink and Reniers*, 2011). Wave momentum is also termed ‘radiation stress’ (*Longuet-Higgins and Stewart*, 1964), and is used to describe wave-current interaction. A comprehensive description of this topic can be found in *Peregrine* (1976), with research dating date back to *Unna* (1947), and after this, *Longuet-Higgins and Stewart* (1960) who found that a wave train ride on a steady current. *Unna* (1947) proposed that when short water waves propagate onto longer waves such as tidal currents U or ocean swells, the wavelength is shortened at the crests and lengthened at the troughs. Terming the shrinking length and the mean value of the short waves at the crests of the long waves as L' and L respectively,

then:

$$\frac{L'}{L} = 1 - A_L K_L \coth K_L h \quad (2.10)$$

where A_L and K_L are the amplitude and wavenumber of the long waves respectively and h is the mean water depth. In contrast to length, if A' denotes the short-wave amplitude riding on long waves and A_s is the mean of A' , then, as (*Unna*, 1947) found:

$$\frac{A'}{A_s} = 1 + A_L K_L \coth K_L h \quad (2.11)$$

Further evaluation was conducted by *Longuet-Higgins and Stewart* (1960) who verified *Unna* (1947) equations in the form:

$$\frac{A'}{A_s} = 1 + A_L K_L \left(\frac{3}{4} \coth K_L h + \frac{1}{4} \coth K_L h \right) \quad (2.12)$$

In the equations above, $K_L h \rightarrow \infty$ when deep water is considered, implying that Eq. 2.11 and Eq. 2.12 are similar; that is:

$$\frac{A'}{A_s} = 1 + A_L K_L \quad (2.13)$$

However, *Longuet-Higgins and Stewart* (1960) found that when a wave train rides on a steady current U , the energy transfer across any normal plan to the motion can be expressed as:

$$\epsilon c_g + \epsilon U + S_x U + \frac{1}{2} \rho h U'^3 + S_x U \quad (2.14)$$

where ϵ is the mean energy density, the first and second terms account for the body transport by group velocity C_g (by the waves and by current U respectively), and the third term represents transport by kinetic energy of the modified stream U' . The most important term here is $S_x U$, which represents the work done by the current against the radiation stress of the waves. S_x can be written as:

$$S_x = \epsilon \left(\frac{2c_g}{c} - \frac{1}{2} \right) \quad (2.15)$$

Similarly, when short waves ride on long waves (rather than on a current), the radiation stress term above is accounted for by the work done by the long waves against the short waves. Therefore, it is necessary to modify the amplitude of the short-wave components. This was confirmed in a series of publications by *Longuet-Higgins and Stewart* (1964); *Longuet-Higgins* (1953); *Longuet-Higgins and Stewart* (1960, 1961).

Radiation stress is important for coastal applications to describe the interaction between short waves (e.g. wind generated waves) and long waves (e.g. swells, tidal currents). For example, in presence of a current with U velocity, wave frequency is subject to a well-known effect termed the Doppler shift. In this case, phase velocity is shifted by the current. The dispersion relationship in Eq. 2.5 now becomes:

$$\omega = \sigma + k.U \quad (2.16)$$

where σ is the dispersion relationship, $\sigma^2 = gk \tanh kh$ (e.g. *Young, 1999*), k is wave number, h is water depth and g is gravitational force (i.e. 9.8 m/s^2).

In coastal systems, waves and currents act simultaneously; therefore, various effects come into play. When a current opposes the direction of wave propagation, waves experience a transformation that reduces the wavelength and steepens the wave height (*Longuet-Higgins and Stewart, 1961; Peregrine, 1976*) resulting in nonlinear processes (e.g. breaking). Experimental (e.g. *Toffoli et al., 2015*) and field observational (e.g. *Bertin et al., 2019*) studies have shown that waves may block when propagating against an opposing current. According to *Toffoli et al. (2015)*, the blocking condition depends on the current speed and the wave group velocity (e.g. blocking limit $U/C_g \rightarrow -0.5$). Wave-current interaction also induces the bottom (*Grant and Madsen, 1979*) affecting the velocity profile in line with an increase in roughness. In another study, *Craig and Banner (1994)* found waves can enhance surface mixing processes, influencing turbulence kinetic energy.

2.2.3 Storm surges

Storm surge is the abnormal level generated during a storm above the normal water surface. It is estimated as the residual after subtracting the predicted astronomical tidal elevation from the measured level. Storm surge events typically occur when atmospheric pressure is low, and the wind blows up towards the shoreline. Therefore, bathymetric and shoreline features may also contribute to storm surge levels (e.g. *Idier et al., 2019*). Regarding the water elevations produced by storms, another relevant term is ‘storm tide’, the total water level generated by storm surges and astronomical tides. Therefore, to summarise, storm tide is the absolute level as measured while storm surge is the change in the water level compared with the normal predicted level (e.g. astronomical tide). Conceptual mechanisms affecting the elevation of storm tides, and surges are depicted in Figure 2.9. Storm surge is

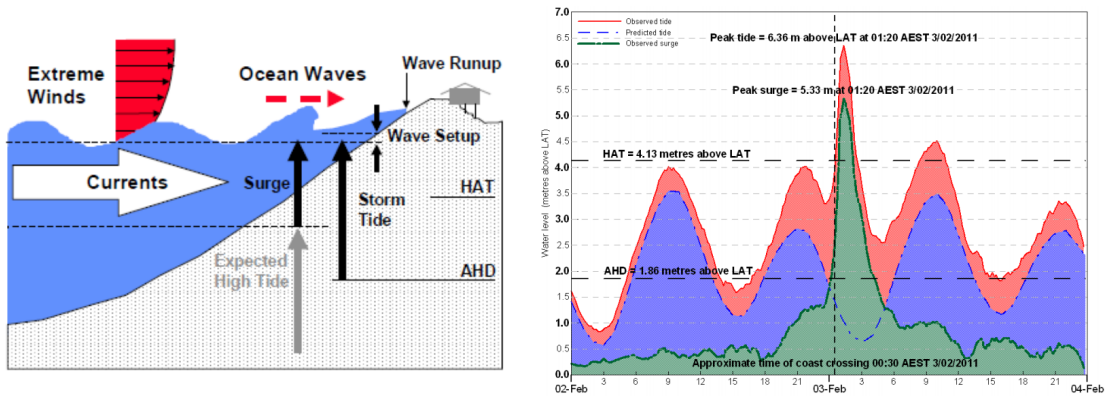


Figure 2.9: Impact of storm surges on water elevations. Left panel presents conceptual components of storm tide from (*Harper et al., 1998*); right panel is a example of water levels during Cyclone Yasi at Cardwell, Queensland in 2012 (adapted from *DSITIA, 2012*).

one of mechanisms causing coastal flooding associated with extreme sea levels. The degree of storm surge elevation differs substantially depending on the coastal region. Many studies have shown that there is a strong interaction between tidal regimes

and surge patterns. Important features resulting from this interaction can include a phase shift in the tidal signals and modulation of surge levels (e.g. *Idier et al.*, 2019). All of these effects have implications for the proper selection of numerical techniques, forcing terms, spatial resolution and physics to predict waves and hydrodynamic conditions in the restricted water environment.

2.2.4 Seiches

Seiches can be observed in some areas such as basins, bays and harbours; these are considered as long waves. Seiches are generated by meteorological factors such as wind gusts, pressure jumps and seismic oscillations due to earthquakes. These waves can be captured in by tidal gauges.

2.3 Modelling approaches

Tidal inlet-lagoon systems are constantly influenced by the actions of waves and currents that drive morphological changes. Figure 2.10 presents typical feedback mechanisms (*Roelvink and Reniers*, 2011) in coastal systems. Wave-current actions play an important role in driving sediment transport and modifying the bed form. For these reasons, modelling morphology requires inputs from hydrodynamic models which can be integrated in a coupling system (e.g.

Lesser et al., 2004; *Roland et al.*, 2012; *Warner et al.*, 2008). Numerical models use mathematical equations derived from physics laws such as the conservation of mass, momentum and energy to describe fluid motion. These equations cannot be solved using an analytical approach unless simplifications are made. For a practical perspective, shortcuts are applied to solve these equations effectively using discretisation with approximations. This section describes general modelling approaches to predict waves, currents and morphological changes.

2.3.1 Wave modelling

Wave models can be generally classified into two types: phase-resolving and phase-averaged wave models. From a practical point of view, phase-resolving models are computationally expensive; such models are not widely used in ocean wave modelling. Thus, this dissertation focuses on phase-averaged wave models. Spectral wave models employ the wave action balance equation (*Komen et al.*, 1994, 1984) to describe the evolution of wind-generated waves. The physical aspects will be described in more detail in Chapter 4. The general form is given by:

$$\frac{\partial N}{\partial t} + \nabla_{x,y}(N \cdot \dot{X}) + \frac{\partial}{\partial \sigma}(\dot{\varphi} N) + \frac{\partial}{\partial \varphi}(\dot{\sigma} N) = S_{sum} \quad (2.17)$$

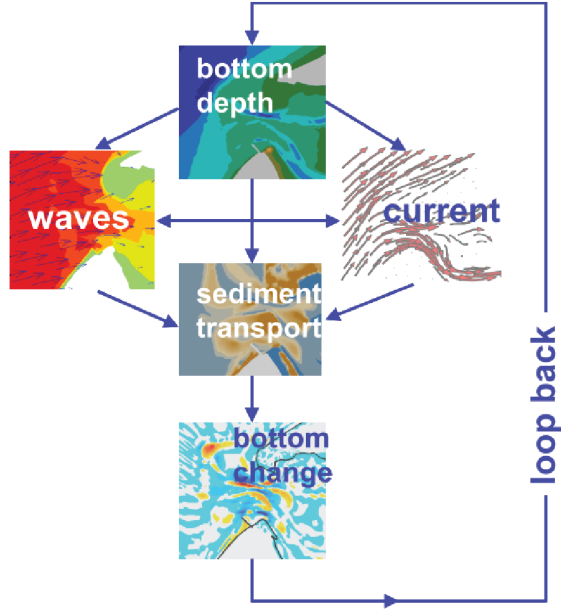


Figure 2.10: Feedback mechanisms in coastal system (*Roelvink and Reniers*, 2011).

where first term on the left represents the wave action evolving at time $N = E/\sigma$ with relative radian frequency σ , the second term is the advection in the geographical space $\dot{X} = \vec{C}_g + \vec{U}$ with wave group velocity $\vec{C}_g$ and current velocity $\vec{U}$. The last two terms are advection terms in spectral and φ direction space. S_{sum} is the sum of all source (positive) and sink (negative) terms, including wind input, wave breaking and dissipation due to bottom friction. Comprehensive reviews of this area have been provided by *Babanin et al.* (2017); *Cavaleri et al.* (2018, 2007). Figure 2.11 visualises three classes of factors affecting the accuracy of spectral wave models, as proposed by *Ardhuin* (2012) and *Roland and Ardhuin* (2014). The importance level descends from left to right. For forcing fields, the wind quality has been improved

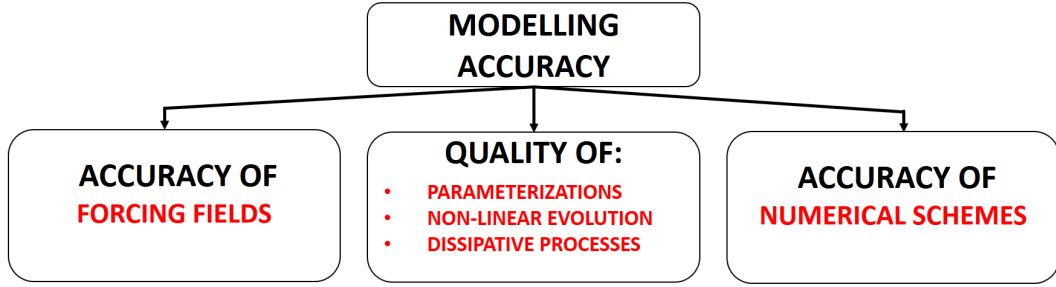


Figure 2.11: Three groups of factors affecting the accuracy of spectral wave models.

with a significant reduction in the error of wind speed (e.g. *Janssen*, 2008). In terms of physics, it is generally accepted that there has been a lack of development of new ideas in the field of physics (*Cavaleri et al.*, 2019). Therefore, attempts should be made to improve parameterisations. Behind the physics, attention has also been focused on numerical schemes applied to discretise wave propagation across both time and space. These include implicit finite difference, implicit and explicit residue distribution (*Roland*, 2008), and explicit finite volumes (*Qi et al.*, 2009). Despite relaxing the downscale issues, such frameworks contain various problems. For example, while time derivatives of regular grids can provide first-order accuracy, unstructured grid wave modelling requires second-order for geographical space and third order accuracy for spectral space (*Cavaleri et al.*, 2018; *Roland*, 2008). Further, care must be taken in terms of time steps, which are often based on two schemes: explicit and implicit. The former is derived from the simple Euler method limited by the CFL criterion with a restriction in the computational time step. However, the increasing high-performance computation has made the explicit time-stepping scheme feasible at high resolutions. The latter is not constrained by CFL, and is now available in unstructured grid models (*Roland*, 2008) with grid-resolution of multiple-kilometres. Another aspect of spectral wave modelling is the grid algorithm required to solve the governing equations. A regular grid is often used to solve deep water equations such as WW3 (*Tolman*, 1992). However, the downside of this grid approach is the difficulty in dealing with spatial gradients as it features a fixed spatial resolution. Unstructured grid modelling provides a more efficient means of dealing with complex environments. This grid approach was first introduced in TOMAWAC (*Benoit et al.*, 1997) and then implemented in WWM (*Roland*, 2008; *Roland et al.*, 2012). Unstructured grid wave models are considered a quasi- standard approach,

and fully coupled with circulation models for operational and forecasting purposes.

2.3.2 Ocean modelling

The Navier-Stokes equations (NSEs) have a central role in describing flow forms. These equations can be written as follows (Warner *et al.*, 2008; Zhang and Baptista, 2008; Zhang *et al.*, 2016):

Momentum equation:

$$\frac{du}{dt} = \frac{\partial}{\partial z} \left(\nu \frac{\partial u}{\partial z} \right) - g \nabla \eta + \Gamma \quad (2.18)$$

Continuity equation (2 or 3-dimensional):

$$\nabla \cdot U + \frac{\partial w}{\partial z} = 0 \quad (2.19)$$

$$\frac{\partial \eta}{\partial t} + \nabla \cdot \int_{-h}^{\eta} u dz = 0 \quad (2.20)$$

Transport equations:

$$\frac{\partial T}{\partial t} + \nabla \cdot (uT) = \frac{\partial}{\partial z} \left(\mathbb{k} \frac{\partial T}{\partial z} \right) + f \quad (2.21)$$

where $\nabla = (\frac{\partial}{\partial x}, \frac{\partial}{\partial y})$, $\frac{d}{dt}$ is material derivative, x, y, z are the coordinates, t is time, η is free surface elevation, h is the water depth, Γ represents other forcing terms (e.g. horizontal viscosity, Coriolis, and atmospheric pressure), g is gravitational force, ν and $\mathbb{k}$ are the vertical eddy viscosity and diffusivity respectively and T is trace concentration (e.g. sediment). These equations are often solved with the aid of the generic length-scale models proposed by Umlauf and Burchard (2003).

2.3.3 Morphodynamic modelling

Importance of quantifying the bed level: Modelling coastal morphological behaviour is a very challenging task; primarily, it requires understanding of hydrodynamics and sediment transport. One of the main concerns in coastal morphodynamics is quantifying changes in the bed level, which often lead to coastal erosion or accretion. The governing equation to predict the bed level is written as follows (Roelvink and Reniers, 2011):

$$(1 - \mu) \frac{\partial z_o}{\partial t} + \frac{\partial S_{ox}}{\partial x} + \frac{\partial S_{oy}}{\partial y} = L_d + L_e \quad (2.22)$$

On the left, the first term represents the rate of change in the bed level z_o at time t with porosity μ , the second and third terms are the change in bed-load transport in the x and y directions respectively. The two terms on the right account for the behaviour of suspended sediment - the first term L_d is the rate of deposition and the second term L_e is the rate of erosion. If there is no distinction between transport due to the suspended load and the bed-load, the computational Eq. 2.22 can be

simplified as:

$$(1 - \mu) \frac{\partial z_o}{\partial t} + \underbrace{\frac{\partial S_x}{\partial x} + \frac{\partial S_y}{\partial y}}_{\text{transport gradient}} = 0 \quad (2.23)$$

In Eq. 2.23, it can be easily seen that erosion occurs when increasing transport produces a positive transport gradient (the second and third terms), leading to erosion. Conversely, accretion occurs in the case of negative transport. In the real-world context with respect to time scale, the positive gradient also means that the removed volume of sediment is larger than the storage in a certain region. This highlights the importance of the gradient of transport because this can help to determine whether erosion will occur or not.

Importance of wave actions: Waves are the main drivers of morphological changes in many open coasts, especially in sandy beaches. Wave actions change not only the shape of the cross-shore lines but also the gradients in the long-shore direction. While changing the cross-shore profile is more significant during extremes (e.g. storms), leading to erosion, long-shore currents can deport sediment along shorelines which can result in either sediment erosion or accretion. When alongshore bars exist on the coast, the profile of the cross-shorelines is affected by the incoming waves with bars moving in both onshore and offshore directions during calm conditions. This quasi-moving trend creates an equilibrium condition, influenced by both wave and current actions. Because of the offshore propagating trend during extremes, the equilibrium condition of the beach profile can be lost as a result of storms.

Importance of tidal regimes: Tides play a crucial role in affecting the morphological features of tidal inlet and estuary regions. For example, the tidal direction of propagation is a key component in determining the shape and direction of the ebb delta. When tides travel perpendicularly to the coastline, a symmetrical development in the tidal inlets is expected; however, asymmetrical features are often observed because tides do not propagate in an ideal direction.

Over-wash and swash motions: During storm extremes, swash motions are driven by low-frequency wave motions (e.g. infra-gravity waves), which can be generated by the grouped short waves.

Types of morphological models: There are typically three types of models to deal with coastal morphodynamics. The simplest form is the coastal profile model, which is intended to study cross-shore processes only. This type of model is often used to evaluate storm effects without man-made structures and investigate the long-term evolution of sandbars and nourishment features on the coastal profile. The second type is coastline models where the shape of cross-shore profiles is presumably retained. Within this assumption, long-shore transport is considered fully adapted to the actions of incident waves. Coastline models can be used for large-scale applications because the spinning-up distance of long-shore currents is typically large. The most complicated is the coastal area models which have further been developed into depth-averaged models (2DH, two dimensional horizontal) and three-dimensional models, which can predict vertical flow motions

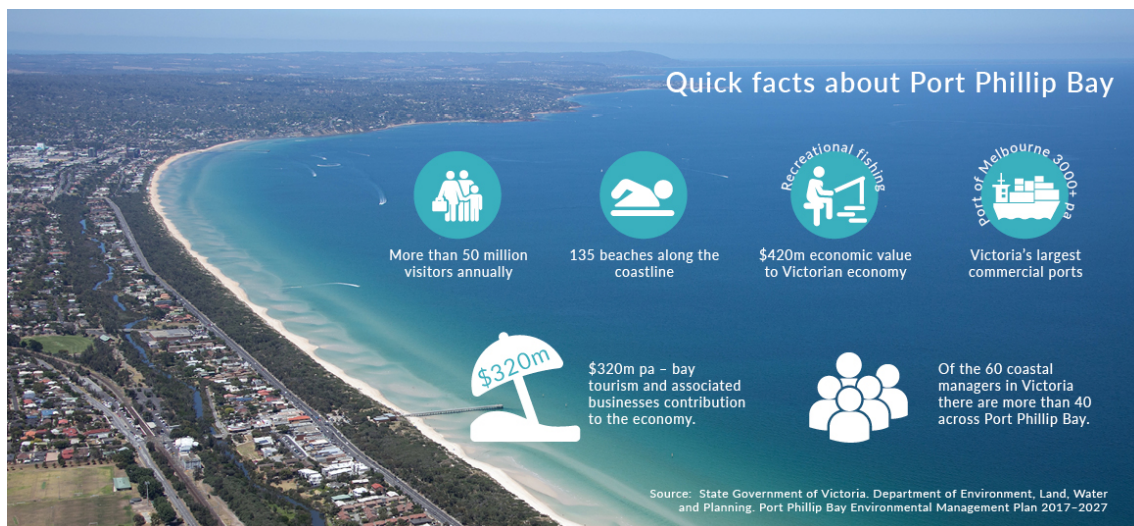
as well as sediment transport. Coastal area models are often applied for applications with different scales. A typical application is to model tidal inlet morphodynamics which cannot be handled by coastal profile or coastline models because tidal inlets often have complex bathymetry and contain channels that may be dissected into different shoals, meaning it is impossible to separate long-shore and cross-shore profiles.

2.4 Conclusions

Tidal inlets are commonly found in many coastal regions around the world. Because these coastal systems are complex in term of physical processes, better decision-making, sustainable management and adaptation needs numerical models to gain better understanding of these coastal processes.

Chapter 3

Location of study



Port Phillip Bay (PPB) is an important route for ship navigation to the Port of Melbourne which is one of the largest ports in Australia. The bay receives about 50 million visitors each year. It is challenging to ensure this unique water environment remains healthy, especially in light of the projected doubling of the population over the next few decades (The Department of Environment Land and Planning, 2017) and other changes in climate patterns. Therefore, a better understanding of driving forces is vital for decision-making, planning and sustainable coastal management.

3.1 Background

PPB is the largest bay in Victoria, Australia. It was formed about 18,000 years ago during the glacial maxima time because of SLR that increased the MSL up to 120m below the present time. At that time, the bay was a dry valley connected with Tasmania (Short, 1996). The bay covers approximately 1,934 km² and has 333 km of shoreline. There are about 130 beaches, accounting for 68% of the total coastlines. The bay is surrounded by densely populated areas, including the city of Melbourne, the capital of Victoria. It is connected with a large catchment network (see Figure 3.1) serving more than 4.5 million people - some 18% of Australia's population. In terms of environmental and social perspectives, there is a strong and important

link between this coastal environment and quality of life for many Victorians (*The Department of Environment Land and Planning, 2017*).

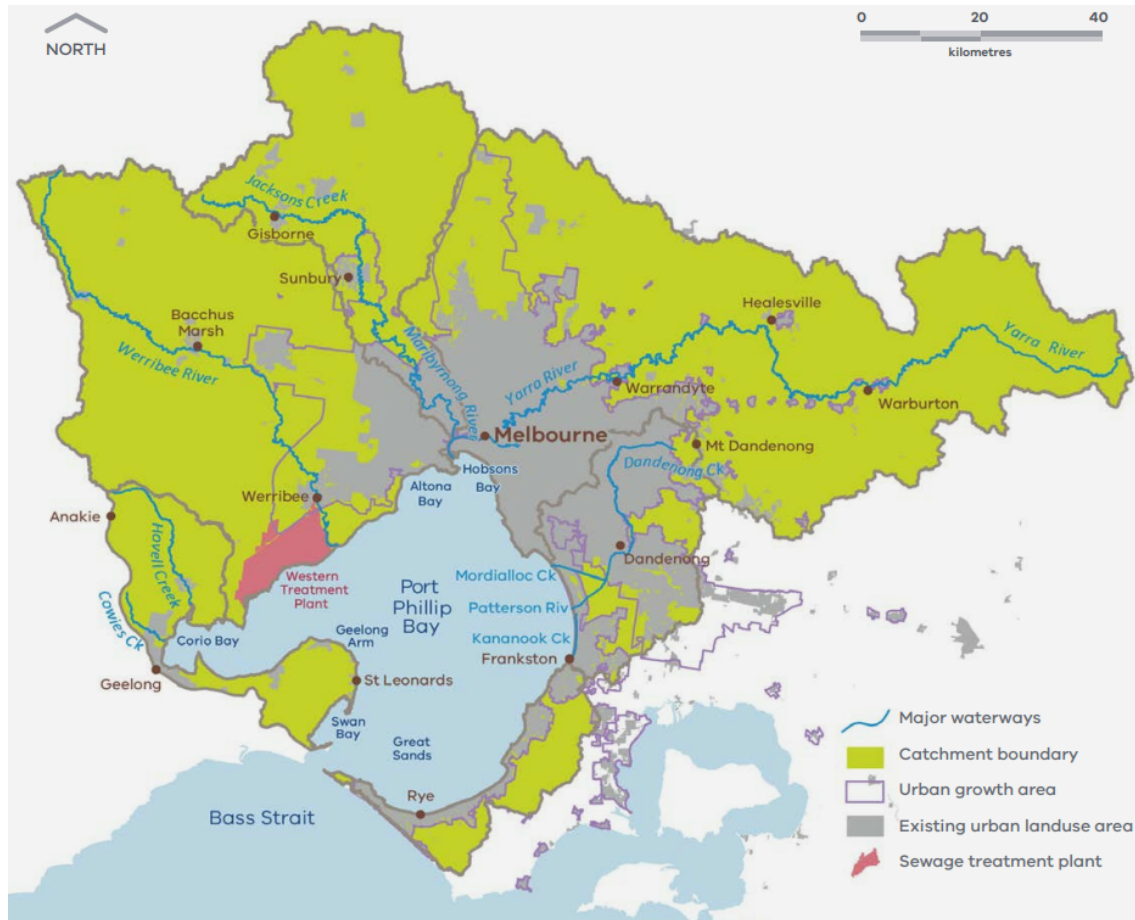


Figure 3.1: PPB is connected with a large catchment network which serves more than 4.5 million people (*The Department of Environment Land and Planning, 2017*).

In terms of the national economy, the bay is home to Port of Melbourne - one of the largest ports in Australia for containers and general cargo. Each year, this port receives about 3,500 commercial vessels, worth US\$ 27 million. Counting containers alone, this port system contributes to about one-third of national trades (*Bradford and Siebinga, 2009*). This volume is expected to increase by 2035 in line with the port expansion. Additionally, this coastal environment is a rich source of fish and marine habitats.

In terms of physical processes, the bay is a complex system with irregular shorelines and topography. The most complicated portion is located in the southern region of PPB. In this area, there is a tidal inlet system connected with a narrow entrance (a mere 3km wide) where the bathymetry is very steep. Specifically, there is a deep and narrow canyon > 90m deep and 150m wide, curved by relatively shallow regions where the water depth falls below 20m. Towards the inner bay, there is a main navigational channel system that dissects the shallow inlet region into two parts, featuring a sand island and many sand bars. Outside the inlet system, there is also a shallow C-curved region called the offshore bar. The entrance of PPB (Port Phillip Head [PPH] or simply the Heads) also exhibits

energetic waves that propagate from the Southern Ocean and interact with strong inverse tidal currents all year around.

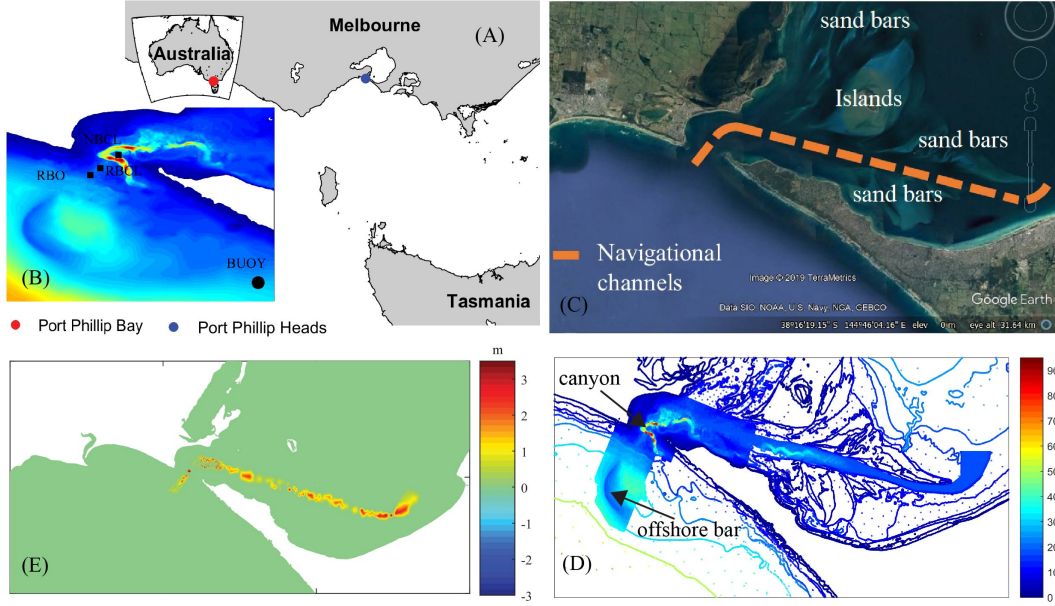


Figure 3.2: Location of PPB marked as a red dot on the map of Australian (A); the blue dot is the entrance of the bay. Locations of field data (waves, currents) at PPH used for the model calibration and validation are shown in (B): Three locations (NBCL, RBCL, ORBCL) as a straight line. An illustration of high-resolution bathymetry (e.g. 0.5m along shipping channels) is shown as contours in (D). Difference in depths (increasing is positive) after and before dredging bathymetry (measured in 2010 and 2007 respectively) is shown in (E), considering only changes within the shipping channels with fixed shorelines.

Recently, the natural features of tidal inlets in PPB have been disrupted by engineering modifications. In particular, a capital dredging project was carried out in PPB during 2008 - 2009 as part of a port upgrade to serve larger vessels up to 7,000 TEU with 14m draught by 2035 at all tidal conditions. The major works were carried out within the south channels. In this region, the channel system was environmentally deepened by up to 3m, equivalent to approximately 14.5 million m^3 of sand and 0.55 million m^3 of mud (*Minister of Planning*, 2007).

3.2 Field observations

Several types of field measurements taken within PPB were used in this study, including waves, currents, water levels and winds. These data were collected via several types of sensors. Further, the numerical models also required the bathymetry. Details of these field data are described in the following sections.

3.2.1 Acknowledgment

Data used in this study came from several sources. The wave and current data at PPH, offshore wave buoys and two locations in the south channels of PPB were obtained from Lawson and Treloar and Cardno Ltd, Melbourne, Port of Melbourne, and the Victorian Regional Channel Authority (VRCA). The water levels at various



Figure 3.3: Several types of instruments used to measure waves and currents at PPH (AWAC, ADCP), SC (Waverider), and offshore (Triaxis).

locations within PPB, and the wind data at the South Channel station were provided by the Bureau of Meteorology (BOM) and the National Tidal Centre (NTC, BOM). The high-resolution bathymetry was provided by Port of Melbourne, and Cardno Ltd. Bathymetry outside PPB (i.e. Bass Strait) was provided by NTC. Dr. David Provis (Cardno Ltd, Melbourne) is also acknowledged as the original author of some figures and texts in this chapter.

3.2.2 Observations of waves and currents at Port Phillip Heads

Wave and current measurements were managed by Port of Melbourne Corporation (PMC) and VRCA in various locations within the entrance of PPB. Only bottom-mounted instruments are suitable to deploy in this area to ensure safe navigation. Two types of sensors - acoustic wave and current recorder (AWAC) and acoustic Doppler current profile (ADCP) - were deployed to measure wave and velocity profiles at the entrance of PPB (see Figure 3.3).

The AWAC produced by Nortek AS (Norway). The instrument can simultaneously measure wave and current profiles using two separate acoustic techniques. The first is an inverted echo sounder, which measures the free surface by measuring the returning period of a narrow-beam acoustic pulse from the water surface. The other technique employs the acoustic Doppler technique to determine the water velocity along the water column at various levels. An AWAC is often made from a plastic cylinder with an approximate height of 25cm. On the top of the instrument, there are three acoustic transducers arranged in 120° - spacing circle angled at 20° vertically. These transducers generate acoustic pulses by analysing the reflected

energy from water particles and bubbles to determine the shifted frequency due to Doppler effect. The current profile can be recorded at various layers in the water column. The wave spectrum can be determined by combining the current records via the Acoustic Surface Track (AST) technique. AWACs are often mounted on the seabed with a frame made of stainless steel. The ADCP is produced by Teledyne RD Instruments. This instrument applies the same technique as the AWAC to measure the current and wave profiles; however, it differs in that it is equipped with four acoustic beams arranged perpendicularly to each other from a horizontal view and inclined 20° from the vertical axis. For the ADCP, surface elevation is determined by calculating the average of the returned surface from the beams. ADCPs can be deployed at the seabed via a similar method to AWACs.

Locations of AWAC and ADCP were not fixed in time; they were sometimes swapped. As PPH is known to create hazardous conditions for ship navigation, these sensors were deployed to characterise severe sea state conditions. For example, three bottom-mounted sensors were often deployed in a straight line along the Great Ship Channel to measure the variability of wave conditions after passing complex seabed regions at the deep canyon from the offshore. Because of channel deepening and maintenance works, field measurement campaigns at PPH were often discontinuous, and each sensor was often deployed for only 2-3 months. Wave and current parametric data were derived from 20 minute-frequency sampling. Wave and current data at PPH provide useful information used for model calibration and validation, and to investigate the variability of hydrodynamic conditions across both time and space.

3.2.3 Observations of offshore waves

The first Waverider buoy (see Figure 3.3) was deployed offshore of PPB in 1993. However, early wave measurements were discontinuous until 2002. After that, waves have been measured via Triaxis buoys, manufactured by Axys Environmental System, Canada. A Triaxis buoy is made from hemispherical hull of stainless steel. At the top, it has a hemispherical dome made of clear polycarbonate, which allows sunlight to enter the solar panels inside the buoy to recharge the battery. Buoys have three accelerometers, a flux-gate compass and three sensors (angular rate gyroscopic), allowing the measurement of the buoy's motion and the establishment of the directional wave spectrum. The buoys were deployed moored with rubber cords, to provide flexibility to move following wave motion. The offshore wave buoy data obtained in this study cover 15 consecutive years with minor gaps. The outputs are parametric and have a 1.5D spectral format with a time interval of 30 minutes. All the raw data were processed with the aid of Manufacturers' software and passed Cardno's quality control processes. The frequency ranged from 0.06 to 0.48Hz with an interval of 0.005Hz. Locations of field observational data are shown in Figure 3.2.

3.2.4 Observations of waves along shipping channels

Limited wave and current data are available for PPB. The only source of data in the inner region is provided by Lawson and Treloar (Cardno, Melbourne). In 2003, following a request from the PMC, field measurements were undertaken to measure waves affecting vessel movements in the South Channel of PPB. Two Waverider ®

buoys (see Figure 3.3) were deployed in the vicinity of the South Channel. One was a non-directional buoy moored about 150m south of Beacon No.15 in the South Channel and another was a directional buoy moored approximately 8m off Rosebud - about 2.3 km south west of the first buoy.

3.2.5 Observations of sea levels around Port Phillip Bay

Sea levels within the bay have been observed by tidal gauges at several locations since the 1960s. Measurements were started at Lonsdale in 1962. This station provides the longest track record with 54 full years of data with the only gap longer than one month (between August and November) in 2012. Another location that can provide up to 51 years of records is Williamstown, where the field measurement campaign started in 1966. Further detail is provided in Table 3.1. The water levels within PPB were obtained from the BOM (Tidal Unit) and contain hourly data from seven stations: Lonsdale, Williamstown, Hovell Pile, Queenscliff, Geelong, West Channel Pile, Stony Point. Details of water levels used in this study are given in Table 3.1. The elevations from these stations were based on the lowest astronomical tides (LATs); for consistency with the modelled results, these data were converted into MSL. To do this, the mean values calculated from each of the entire timeseries and minus the LATs at each station were subtracted from the measured water levels:



Figure 3.4: Locations of tidal gauges to measure sea levels used in this study.

$$MSL_{EL} = EL_i - \text{mean}(\text{sum}(E_i)/n)$$

Table 3.1: Length of observational records of sea levels used in this study.

Station/(Location)	First full year	Number of full year	Before dredging	After dredging
Lonsdale (1)	1962	54	46	7
Geelong (2)	1965	42	32	8
Hovell Pile (3)	1991	26	16	8
Queenscliff (4)	1991	23	13	8
West Channel Pile (5)	1991	26	16	8
Williamstown (6)	1966	51	41	8
Stony Point (7)	1963	35	25	7

It is noted, however, that water levels in PPB have been measured at an interval of 5 minutes, which may contain daily oscillations due to seiches (e.g. *Woodworth et al.*, 2019), resulting in ‘piling up’ or sudden changes in waters. However, this

study uses hourly data derived from 5-minute records. Therefore, the actual highest water elevations may be slightly higher than those hourly data.

3.2.6 Wind observations

There are limited wind data available for PPB (*Cardno Lawson Treloar*, 2007a,b). Within this region, the BOM operates two wind stations to measure wind in PPB located at Point Wilson and the South Channel. In this study, the observed wind data are not suitable as model inputs because the measurements have many gaps and the temporal scale is short (only about 15 years of data are available). For this reason, the present study only uses the measured wind for comparison purposes (e.g. checking the model wind inputs).

3.2.7 Mapping bathymetry

Bathymetric measurements were conducted by Port of Melbourne during summer 2007/2008 and 2009/2010. The main purpose of these mapping tasks was to measure the sea floor before and after the channel deepening project. Thus, high-resolution mapping was conducted in the tidal inlet regions, including shipping channels and the entrance of PPB. Several mapping methods were used, including laser airborne depth sounding (LADS) and multi-beam echo sounder (MBES) systems. The output files were in rectangular grids with a resolution of 6.0m in the region around PPB and 0.5m along shipping channels. These data are provided by industry (*Cardno Ltd*, Melbourne) described in *Cardno Lawson Treloar* (2007a,b).

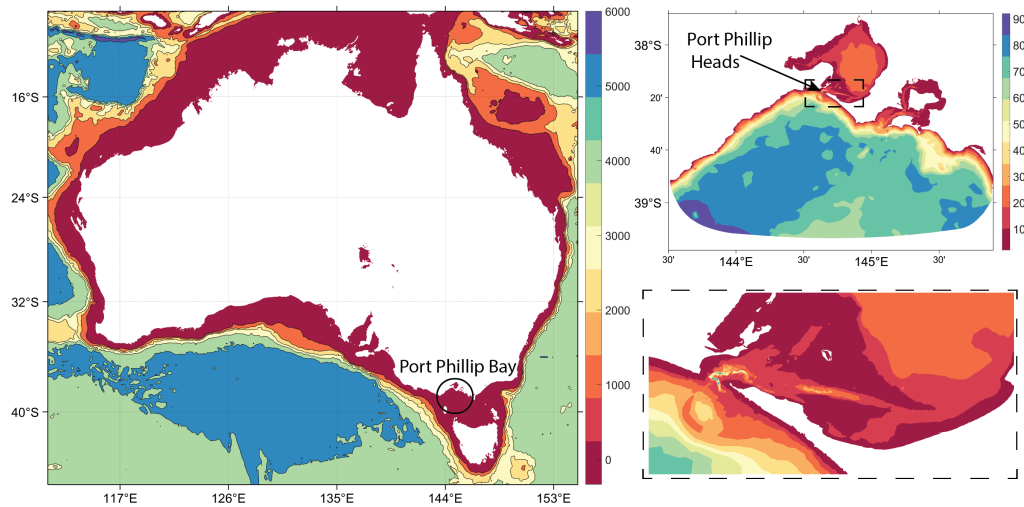


Figure 3.5: Seafloor features of Australian waters (left), PPB (right, top) and the southern region including PPH (right, bottom). Generally, PPB is a shallow lagoon. The deepest point inside the bay is around 20m. The complex section is located at the entrance of the bay with the presence of a deep, narrow canyon up to 90m in depth.

To create sufficient spatial domain for modelling purposes, this study uses two other bathymetric sources to cover the Victorian coasts and offshore waters. For Victorian coastlines, 500m bathymetry is provided by the Cooperative Research Centre for Spatial Information (CRCSI, <http://www.crcsi.com.au>); this was conducted by DELWP, Victorian Government, Port of Melbourne, the Royal

Australian Navy (RAN), Deakin University and Geoscience Australia. These data are freely available from (http://thredds.aodn.org.au/thredds/catalog/Deakin_University/bathymetry). For offshore waters, the data were from the BOM (National Tidal Unit via personal communication). Figure 3.5 illustrates the bathymetric features of Australian waters, zoomed in on the region of PPB and PPH. All depths are in metres and reduced to MSLs.

3.3 Characteristics of forcing mechanisms at PPB

This section reviews some of the main drivers of physical processes in PPB based on field data analysis and previous studies.

3.3.1 Wind

The wind pattern in PPB is influenced by two trade winds – the westerly wind belts of the subtropical winds and the southern hemisphere winds. During summer, the subtropical range (see Figure 3.6) shifts southward along with the thermal equator. As a result, pressure systems also move further to the south. With the hot weather, the mainland becomes a low pressure region. Therefore, the wind pattern moves towards the land during the summer. The opposite trend holds for the wind in winter (*Cardno Lawson Treloar, 2007b*). Figure 3.7 plots wind rose data measured at the South Channel station from 1998 to 2013 for the two seasons. During summer, about 50% of the wind blows from the Southern Ocean towards PPB. During winter, about 40% of the wind comes from the mainland (e.g. the north).

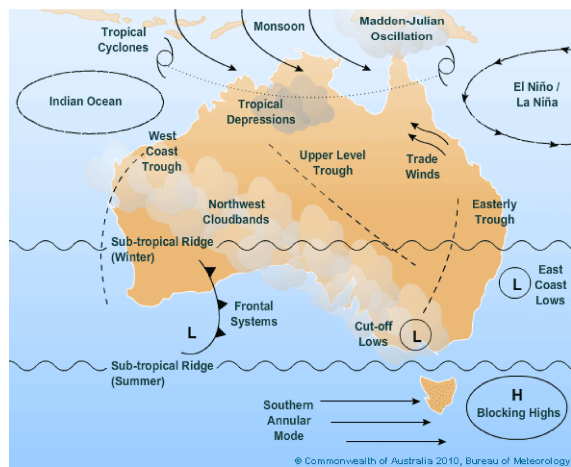


Figure 3.6: Major drivers of climate patterns in Australia (source: BOM). The subtropical range shifts to the south during summer months, and moves to the north in winter affecting wind pattern in PPB.

3.3.2 Tides

Tides in PPB are driven by the ocean tides from Bass Strait adjacent to the entrance to PPH. Tides in Bass Strait are the result of the tides at either end of the strait - the Indian Ocean west of King Island and the Tasman Sea east of Flinders Island. The tides enter the strait from the east and west, and then move into the centre of the strait with complex interactions due to the islands at either end (King and Flinders Islands) and the openings at the northern and southern end of each island. Subsequent to these interactions, the two tidal waves effectively meet in the central south west of Wilsons Promontory. In this area, there are larger tidal elevation changes but smaller tidal currents than in the remainder of Bass Strait. As the tide rises and falls in Bass Strait, water flows in and out of PPB through the entrance. In line with the relatively narrow heads and the constricted channels through the Great Sands, the tide levels in PPB are not the

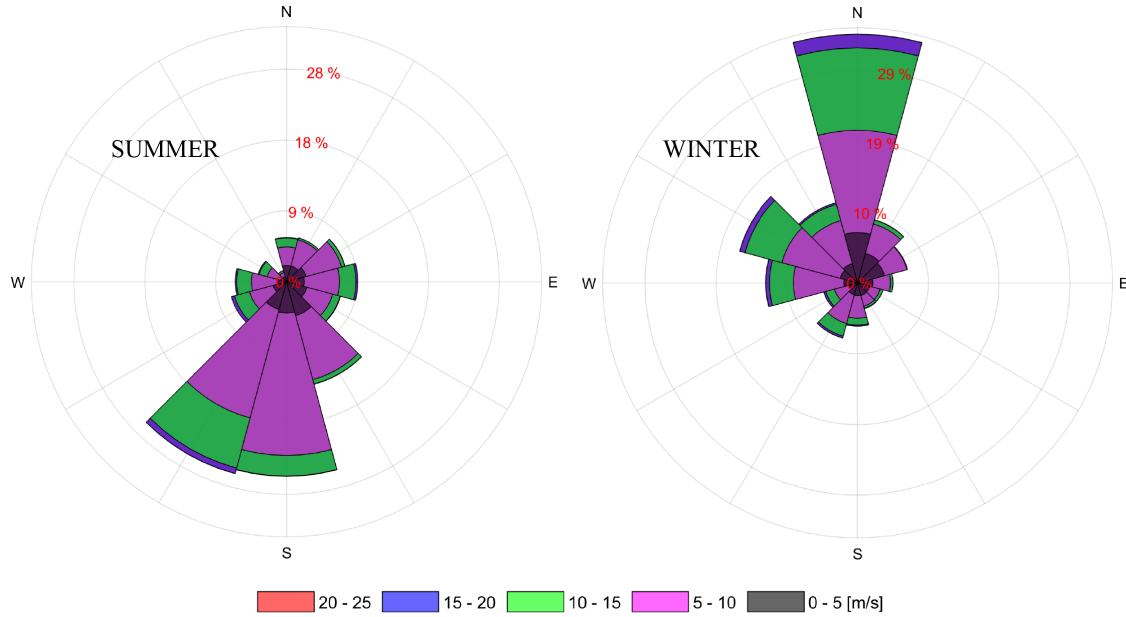


Figure 3.7: Measured wind data from 1998 to 2013 (mean speed, and direction) at South Channel station during summer and winter months.

same as those in Bass Strait. As the tide rises in Bass Strait, water flows in through PPH and through the channels to the body of the bay. However, when it is high tide at the entrance, water is still flowing into the bay and this continues as the level in Bass Strait falls and that in the bay continues rising until they are the same. Then, as Bass Strait tide levels fall, water begins to flow out of the bay since it is now higher than the strait. A similar sequence occurs at low tide, with the level in the strait falling lower than the bay and beginning to rise again before water starts to flow into the bay. Figure 3.8 shows the large reduction in tidal variation from Bass Strait, represented by Rip Bank, through the PPH to Queenscliff. There is then a further reduction along the South Channel to Beacon 6 (SCH6CON) and then a slight increase to Hovell Pile. This change in the amplitude of the tide thus represents a major semi-diurnal constituent.

For completeness, Figure 3.9 plots changes in tidal amplitude and time lag with respect to distance for major tidal constituent M2 only. There is a marked drop in amplitude from Rip Bank to Queenscliff with Point Lonsdale in between; the minimum is found at Beacon 6, before a small rise to Hovell Pile. It also shows a very small increase in amplitude from Hovell Pile to Williamstown. This figure also shows the change in timing of the tide, indicated by the red line. The values shown are the time lags at each point for the occurrence of high water following that at Rip Bank in the entrance. The values are taken from measured data and show a fairly steady change in the timing, with the tides at Hovell Pile occurring over three hours after those at Rip Bank. Again, of note is the value for Williamstown, which is almost identical to that at Hovell Pile. This suggests that the tides in PPB do not change very much north of Great Sands.

Referring again to Figure 3.8, there is another noteworthy feature that helps to further understand the tides of PPB. There are a number of points where the tidal elevation time series passes through the same point on the graph; for example, around hour 20 and hour 26, the water level at all locations is identical. This

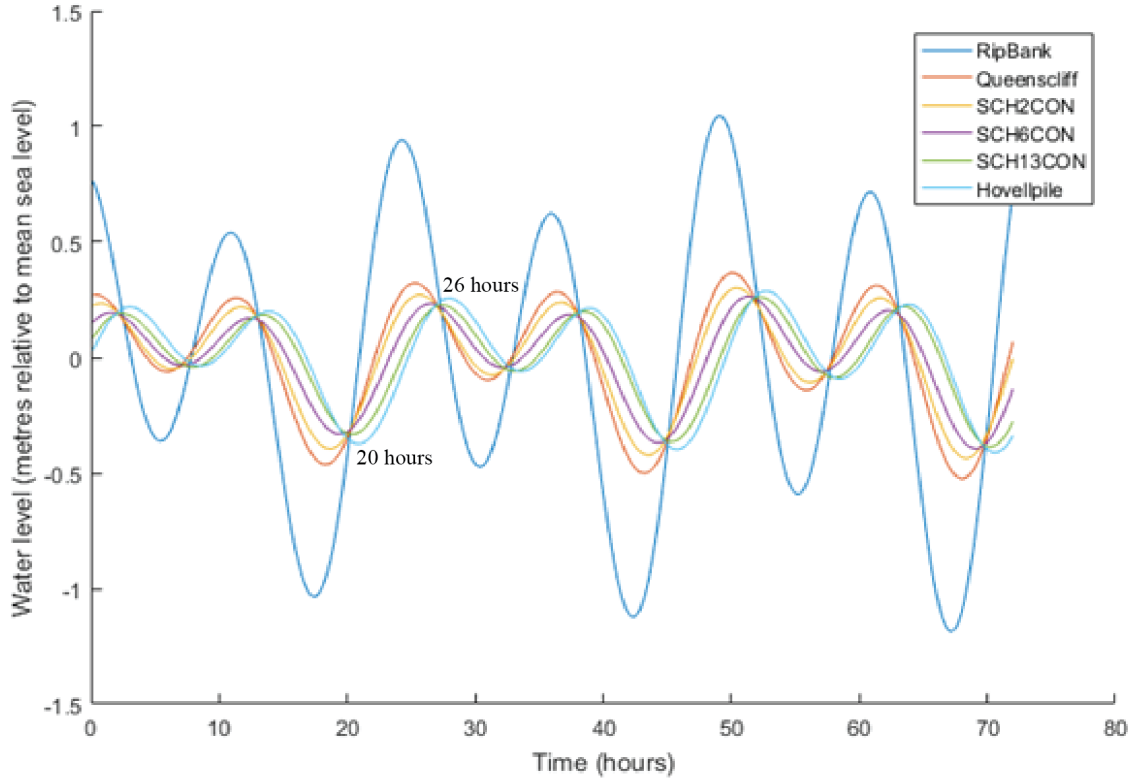


Figure 3.8: Characteristics of tidal elevations at selected locations along South Channel showing the change in amplitude and phase (time) lag. There is a significant change in tidal elevation from Bass Strait (as represented by Rip Bank) through PPB to Queenscliff. A further reduction can be observed along the tidal inlet (SCH6CON, the South Channel) to Hovell Pile (Hovellpile).

occurs twice in each tidal cycle, with one of crossing points above the mean level (zero) and the other below the mean level. These points indicate that the water surface is the same as the Bass Strait outside PPH, through the entrance, across the Great Sands and indeed through the body of the bay. When the water surface is flat, there is no slope or gradient to drive tidal currents -giving rise to slack water, or zero tidal streams.

3.3.3 Tidal currents

Tidal currents are the dominant driver of water movements in the southern portion of PPB, including the entrance areas (*Cardno Lawson Treloar, 2007b*). For example, Figure 3.10 shows measured current magnitude and direction at NBCL, RBCL, RBO (see Figure 3.2 for these locations) in PPH from 2006 to 2012 recorded by bottom mount sensors (e.g. AWACs). The strongest current magnitude ~ 3.4 m/s was recorded during a strong ebb tide at RBCL. It is noted that the actual strongest current speed can be larger than this value because field measurements were not continuous, and thus may not contain all extreme currents. For current direction, it can be reversed due to tidal effects. The direction at RBCL is more confined than other two locations because the flow movement within this point is restricted by the shipping channel. Tidal currents are only strong in the vicinity of PPH and the southern region of the bay. Understanding the tidal flows in these regions is essential for various applications. For example, Figure 3.11 shows a strong correlation between

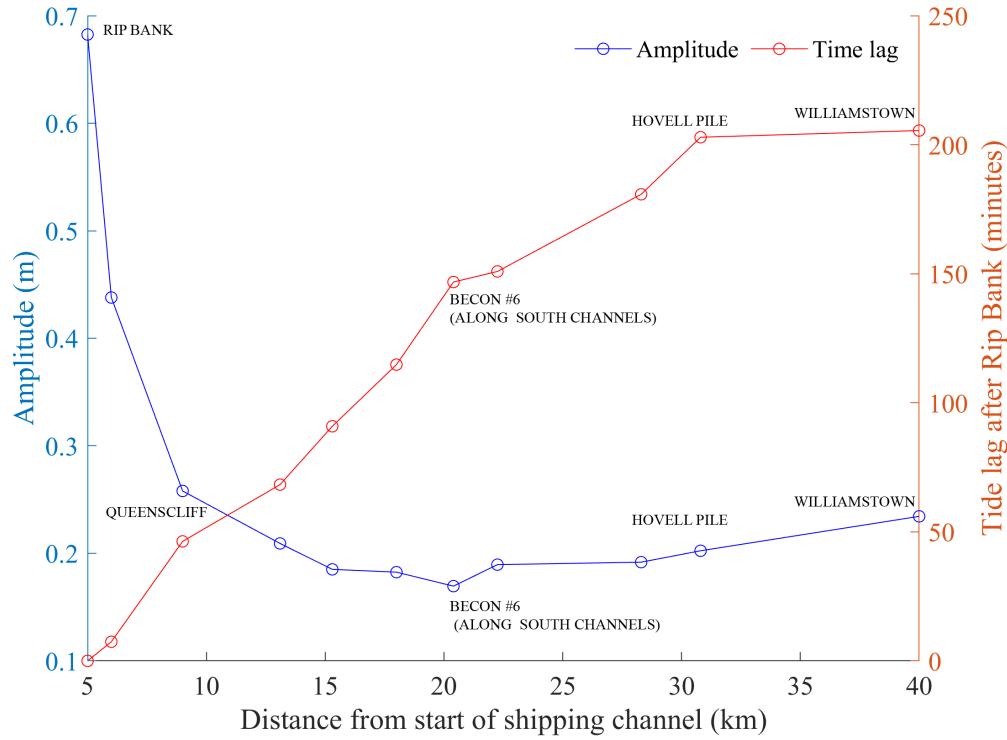


Figure 3.9: Amplitude, phase and time lag of the tide along South Channel from Rip Bank to Hovell Pile with values for Williamstown shown on the right at a distance of 40 km.

the movement of plumes observed by satellite with the pattern of modelled tidal currents during a strong ebb tide. Therefore, an accurate account for tidal currents is important for modelling of turbidity and sediment transport. Another application requiring attention on tidal currents at PPH is for ship navigation as a strong current can be hazardous for vessel movements and it can increase wave height and modify the wave direction of propagation (e.g *Peregrine*, 1976; *Wolf and Prandle*, 1999).

3.3.4 Waves

Waves in PPB are fetch-limited, which means the wave field is generated by local wind (*Association of Bayside Municipalities*, 2017; *Cardno Lawson Treloar*, 2007a,b). Therefore, the evolution of the wave field largely depends on the direction and distance over which the wind can blow. The ocean waves do not propagate too far into the inner region of PPB. The reason is that the entrance of PPB acts as low-pass filter that does not allow short-wave motions to enter into the lagoon. Further, ocean waves can quickly lose energy when they propagate onto the shallow Great Sand.

At PPH, wave patterns are very complex. Large swells from the Southern Ocean (Bass Strait) propagate onto PPH, then interact with strong tidal currents (*Lesser et al.*, 2007; *Provis*, 2015; *Provis and Aijaz*, 2005). As a result, waves increase in height and break. Figure 3.12 plots all measured SWH and mean wave direction during 2006-2012 for both ebb and flood tides at NBCL, RBCL, RBO. Distances from the outer point (RBO) to RBCL and NBCL are about 1,000m and 1,500m respectively. SWH at RBCL is usually higher than the two other locations as

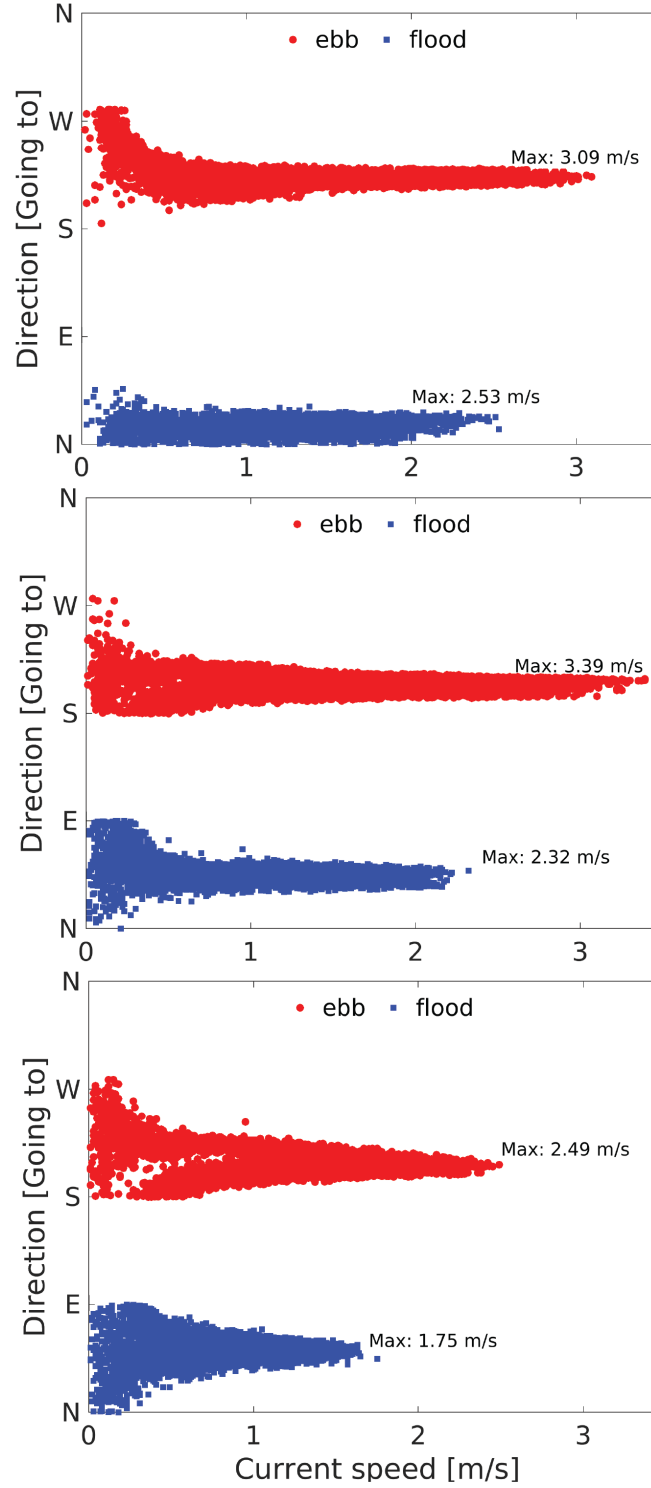


Figure 3.10: Measured current velocity and direction at PPH from 2006 to 2012. Data were recorded at NBCL (top panel), RBCL (middle panel) and RBO (bottom panel) by bottom mount sensors (e.g. AWAC) - see Figure 3.2 for these locations. The actual current magnitude can be stronger than these values as measurements were not continuous, and thus may not contain all extreme currents. Within PPH, currents are stronger during ebb tides than flood tides.

waves interact with strong currents at this point prior to passing the deep canyon. After this point, waves reduce their height and lose energy at NBCL. Therefore, SWH at NBCL is often smaller than that at RBCL.

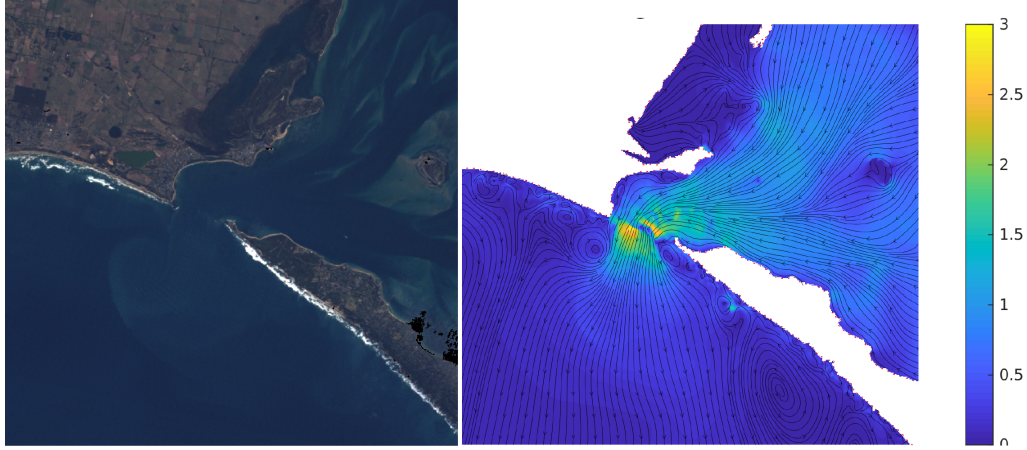


Figure 3.11: The importance of tidal currents for plume movements at PPB. The turbidity is at about 23:20 on 25 August 1994 (left panel) captured by Landsat L5 Satellite (<https://www.usgs.gov/land-resources/nli/landsat/landsat-5>). The movement of these plumes is driven by a strong ebb tide (right panel) forming a rip current up to 3m/s.

Another complex pattern in PPB is the occurrence of freak waves - defined as a wave that is more two times larger than SWH <https://www.nationalgeographic.org/article/rogue-waves/>. Figure 3.13 a, b and c compare SWH (H_s) from AWAC sensors at three locations aligned in a straight line (see Figure 3.2 for locations), maximum wave height ($H_{\max}$) and the ratio ($H_{\max}/H_s$) measured by AWAC sensors at RBO, RBCL, NBCL. It can easily be seen that there are at least three occurrences of rough waves where $H_{\max}/H_s > 2$. In nature, freak waves are not necessarily the largest waves; they are unusual waves, which can be generated by strong opposing currents (e.g. see *Toffoli et al.*, 2015) and other meteorological factors. Numerical models to replicate rough waves are very computationally expensive (e.g. see *Babanin et al.*, 2017), and are not a subject of this study.

3.3.5 Storm surges

Key drivers of storm surges in Victoria

Storm surges occur year around in Victoria (*McInnes et al.*, 2005), suggesting that wind pattern is a major driver of storm surges. *McInnes et al.* (2005) investigated the influence of meteorological patterns on storm surges in eastern Victoria and found that 70% of storm surge events were caused by cold fronts. The other relevant meteorological patterns were Tasman lows and east coast lows (see Figure 3.14), which accounted for 23% and 6% respectively. As storm surges generally are highly complex, knowledge about these remains incomplete. In Victoria, the interaction between tides, surges and waves is not yet fully understood (e.g. due to lack of field data to validate numerical models).

Phenomena of storm surges in PPB

Variations in water level in PPB are caused not only by astronomical tides but also by other mechanisms, including wind and atmospheric pressure generated by climate patterns (e.g. *McInnes et al.*, 2009). Further, wind can also generate waves

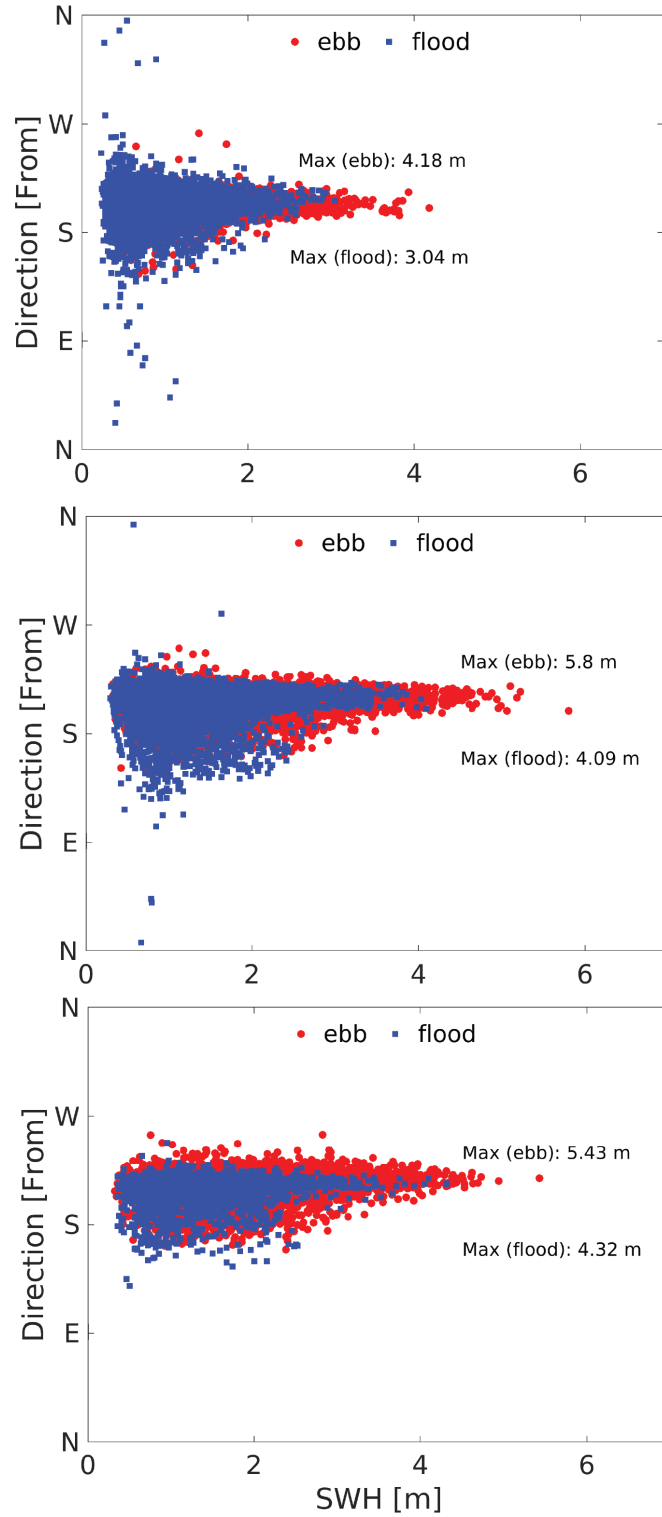


Figure 3.12: Significant wave height and mean wave direction at PPH between 2006 and 2012. Data were recorded by bottom mount sensors (e.g. AWAC) at NBCL (top panel), RBCL (middle panel) and RBO (bottom panel) - see Figure 3.2 for these locations. The actual wave height can be larger than these values as measurements were not continuous, and thus may not contain all extreme waves. Within PPH, waves often steepen during an ebb tide and flatten during a flood tidal cycle as they interact with strong tidal currents.

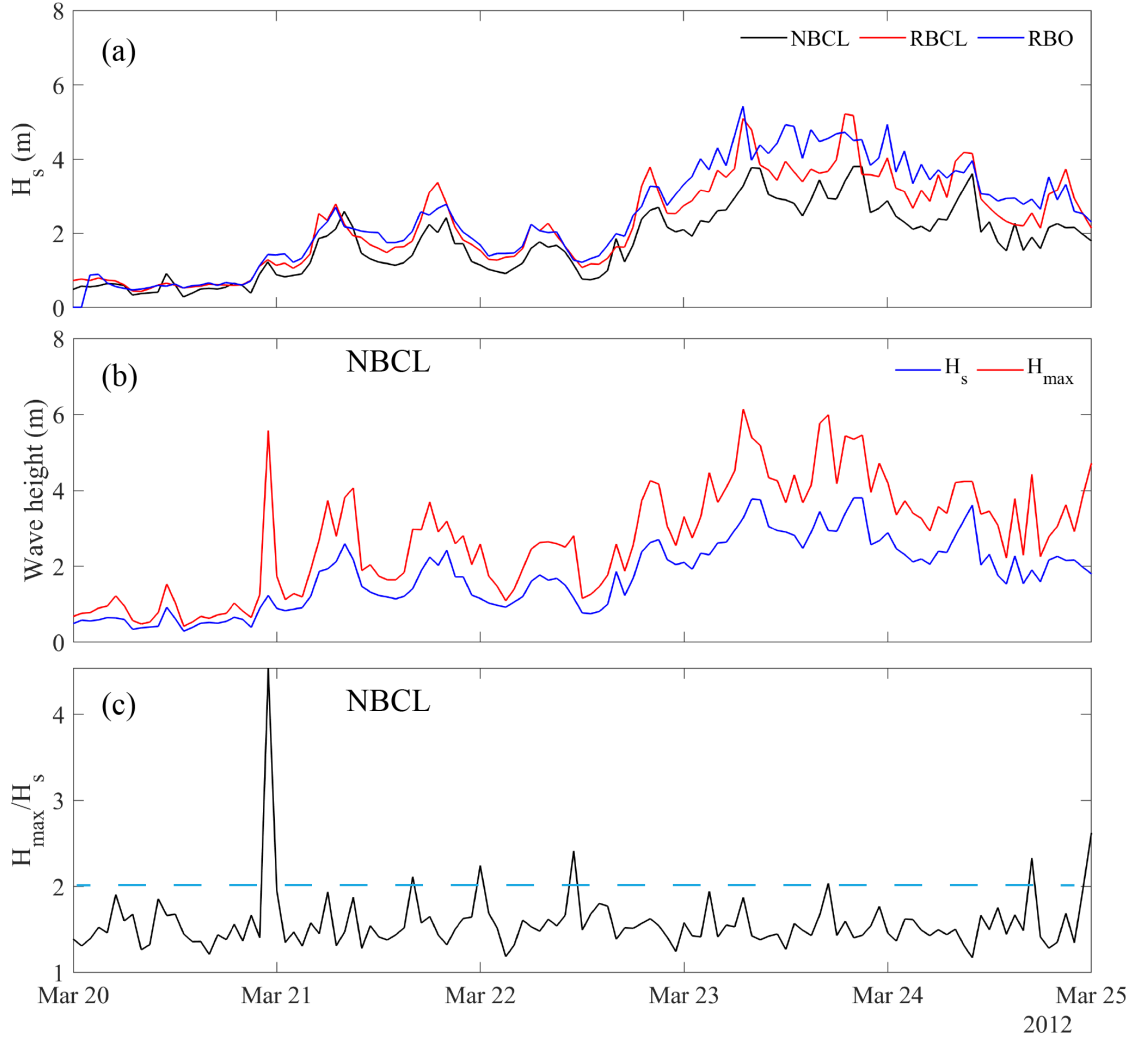


Figure 3.13: Wave pattern at PPH in March 2012 measured by AWAC sensors: (a) - comparison of SWH at three locations at NBCL, RBCL and RBO (see Figure 3.2); (b) a comparison of SWH H_s and H_{max} at NBCL ; (c) - ratio H_{max}/H_s at NBCL.

inside the bay resulting in wave setup. Therefore, understanding storm surges in PPB requires knowledge of both wind waves and climate systems..

Wind blowing over the surface of the water leads to a change in elevation, with, for instance, water ‘piling up’ against the coast which the wind is blowing. Atmospheric pressure causes changes in sea level, with high pressure lowering the sea level and low pressure causing an increase in sea level - this is referred to as the inverse barometric effect. These changes in level can result in very long-period waves propagating across the ocean and, in particular, along the edge of continents, where they interact with the continental shelf to form ‘shelf waves’. There are various types and propagation forms for such waves and all involve a variation in surface elevation at the coast. The impacts of shelf waves are felt in PPB, even though the waves themselves may be created by a weather system a great distance away. Since all these factors combine to make up surface variations (other than the astronomical tidal components), they are considered storm surges in this study, since their combined impact is greatest during storms. Storm surges

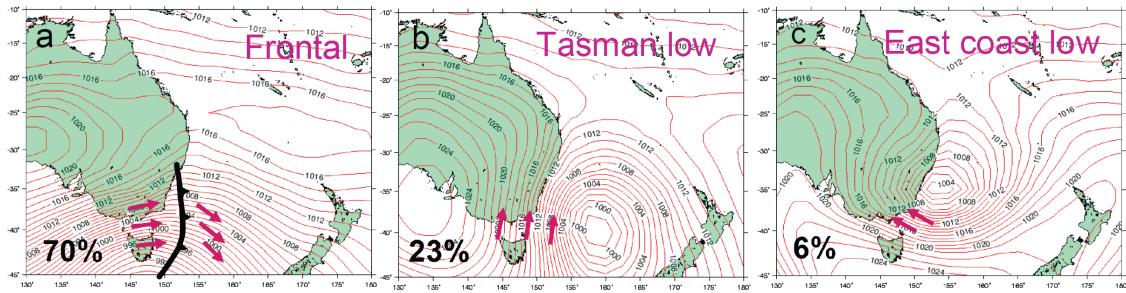


Figure 3.14: Three weather patterns responsible for storm surges in Victorian regions according to McInnes et al. (2005).

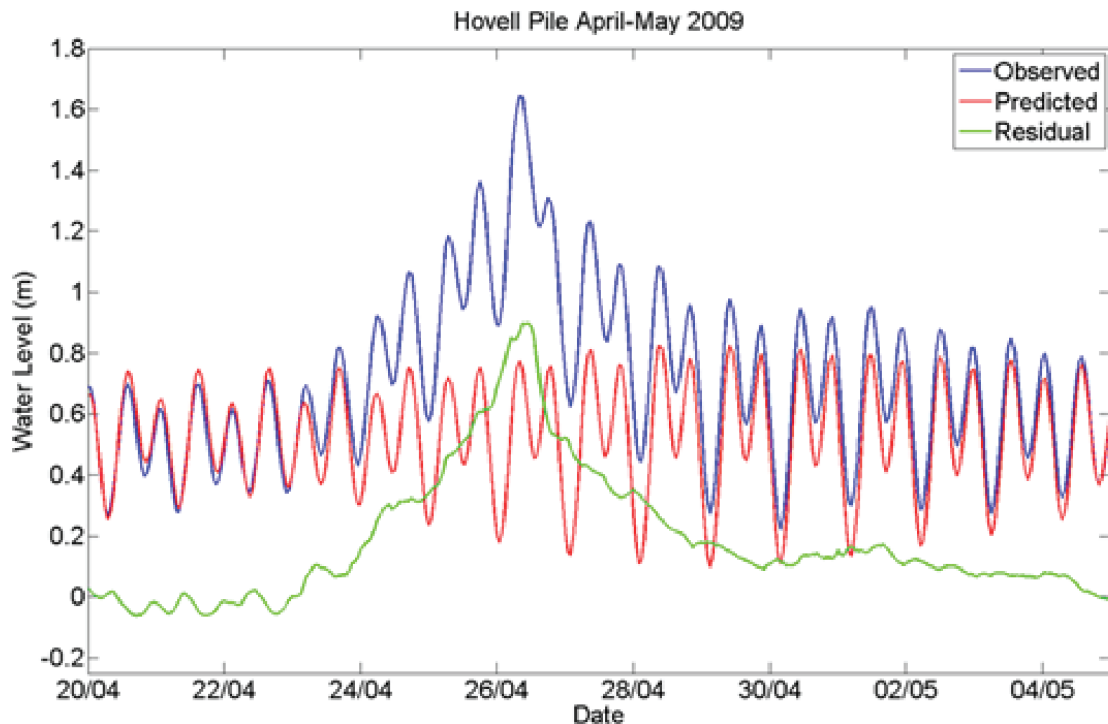


Figure 3.15: A comparison of observed sea levels at Hovell Pile (in meter, CD) with astronomical tides during a storm surge event (May 2009) with the tidal residual about 0.9m.

happen during and after a storm; they describe a situation where the meteorological effects of wind, atmospheric pressure and wave setup increase the water elevation over a long period (e.g. several days). Subtracting the predicted surface elevation generated by the astronomical tide from measured sea levels provides the tidal residual, giving an estimate of the magnitude of meteorological effects in observed sea level variations. Positive tidal residuals are commonly called storm surges. A combination of large positive residuals and high astronomical tides creates extreme sea levels (Cardno Lawson Treloar, 2007a,b).

Figure 3.15 gives an example of a storm surge event that occurred during April/May 2009 at Hovell Pile, causing a residual of up to 0.9m. This event occurred in autumn suggesting that extreme events in PPB can happen year around. To give further insight into the phenomenon of storm surges in PPB, Figure 3.16 compares the maximum measured sea levels (in metres, MSL) recorded

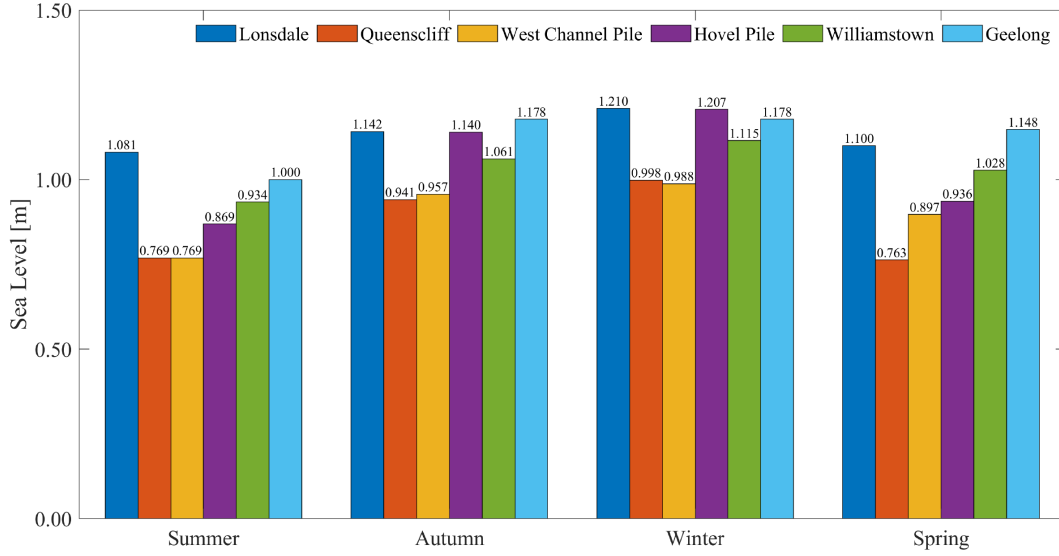


Figure 3.16: Maximum water levels at different stations (AHD) applied datum correction which is based on Association of Bayside Municipalities (2015) -ABM. Calculations are based on measured sea elevations from tidal gauges provided by the Australian Bureau of Meteorology (BOM). These levels may differ slightly from other studies (e.g. Association of Bayside Municipalities, 2015) as data come from different sources and lengths of data. The major features in this figure include (1) there is only minor difference between the extreme sea levels at stations from Hovell Pile to Geelong; (2) the extreme sea level pattern in summer is similar to spring, while the autumn is similar to winter; and (3) the maximum extreme sea levels at several stations only occur during a single event (see Table 3.2).

at several stations around PPB for all seasons. Lonsdale station often experiences a greater extreme than other locations inside PPB, because of a larger tidal range induced by ocean tides. Inside PPB, the gap between this station and other stations varies seasonally as extreme sea levels inside PPB can be affected by waves, and wind climate. Table 3.2 provides the timing of the most extreme sea

Table 3.2: Statistics of timing of maximum observed water levels at tidal gauges within Port Phillip Bay for different seasons.

Station	Summer	Autumn	Winter	Spring
Lonsdale	05-Jan-2014 18:00	26-Apr-2009 03:00	12-Jul-1964 05:00	05-Nov-1994 16:00
Geelong	09-Feb-1996 21:00	27-May-1994 07:00	29-Jun-1980 06:00	06-Nov-1994 20:00
Hovell Pile	02-Feb-2005 21:00	26-Apr-2009 07:00	12-Jul-2016 11:00	06-Nov-1994 20:00
Queenscliff	05-Jan-2014 18:00	26-Apr-2009 04:00	05-Jul-2011 06:00	07-Sep-2012 08:00
West Channel Pile	02-Feb-2005 21:00	26-Apr-2009 07:00	05-Jul-2011 08:00	06-Nov-1994 20:00
Williamstown	09-Feb-1996 20:00	26-Apr-2009 07:00	24-Jun-2014 04:00	06-Nov-1994 20:00

level at each station matching the values in Figure 3.16. Several interesting features can be observed in this table. First, the most extreme case at all stations can happen as a single event. In spring, for instance, the highest water level occurred in a single storm in November 1994. Especially, during this event, the maximum sea levels at all inner stations occurred at the same time - 20:00 on 6 November 1994. Second, there is often a time lag between Lonsdale and the inner

stations. This phenomenon can be clearly seen in autumn and spring. For example, in autumn, the maximum water level at Lonsdale occurs 1 hour ahead of Queenscliff. Another interesting pattern from Table 3.2 is that the majority of all extreme events occurred in the first and last week of the month.

Wave setup at PPH

Wave setup is expected within PPB entrance. In this region, wave decay from

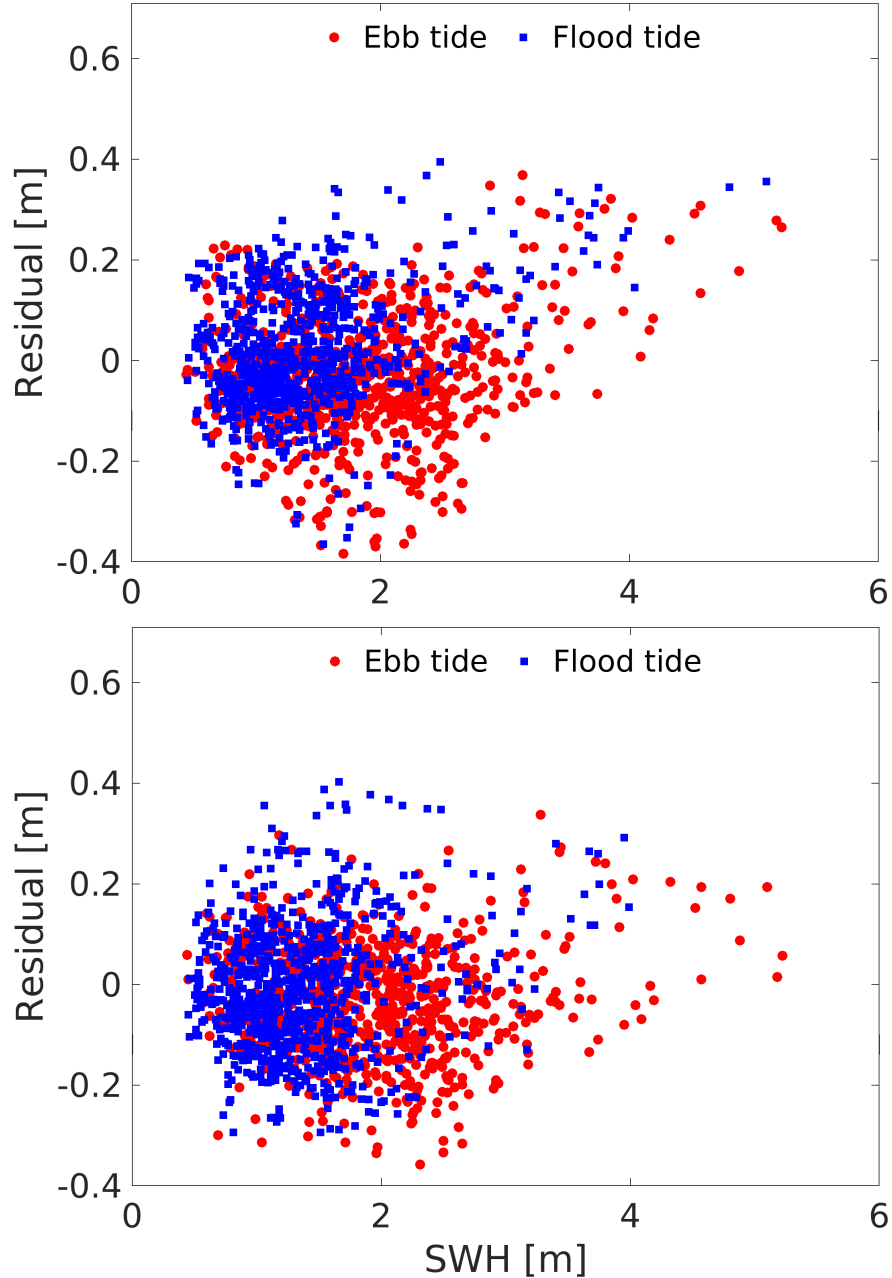


Figure 3.17: A relationship between tidal residuals within PPH. Top panels shows tidal residuals at Point Lonsdale extracted from a tidal gauge and SWHs at RBCL measured by AWAC from February to April 2012 compared with modelled results based on this study.

bottom friction and wave breaking can result in less setup with large waves. This

phenomenon has been investigated by several authors (e.g. *Dunn et al.*, 2000; *Hanslow et al.*, 1996). It can be identified by comparing tidal anomaly from measured water levels with wave height (*Dunn et al.*, 2000). To provide an insight into the wave setup in PPH, Figure 3.17 shows a relationship between tidal residuals based on both field measurements (residuals extracted from a tidal gauge at Point Lonsdale and SWHs at RBCL from AWAC) and modelled results at the same locations estimated by this study. Clearly, both measured and modelled data show a reduction in tidal residuals when waves are large. As shown, negative tidal anomaly often occurs during an ebb tide which produces a strong current, resulting in more wave breaking due to wave-current interaction. Therefore, it is essential to include wave-current effects in this region to estimate extreme sea levels. It is expected that including wave setup within PPH would reduce extreme sea levels in this area.

Previous studies related to storm surges in PPB

Several studies have focused on storm surges and extremes in Victoria and have covered PPB (*Hubbert and McInnes*, 1999; *McInnes et al.*, 2009; *McInnes and Hubbert*, 1996; *McInnes et al.*, 2011, 2005). However, storm surge phenomena in PPB have not yet fully assessed nor understood. *McInnes and Hubbert* (1996) and *Hubbert and*

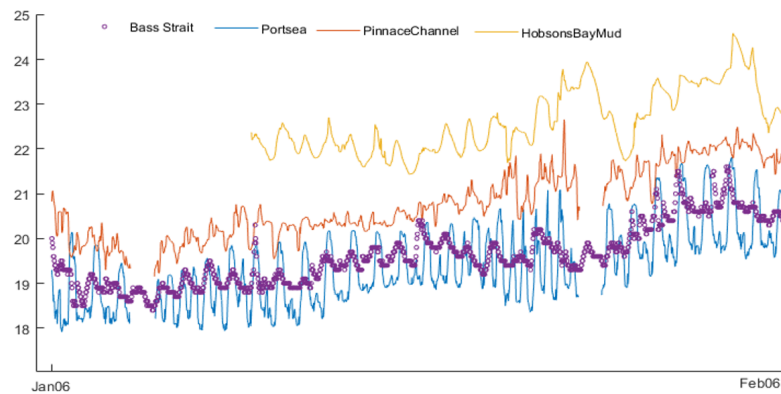


Figure 3.18: Measured temperature in January 2006 at several locations inside and outside PPB.

McInnes (1999) examined climate change impacts on Victorian coastlines using numerical modelling. These authors only simulated two storm events in Port Phillip Bay which occurred in 1994. The major drawbacks of these studies included the use of a coarse grid (a 500-m grid resolution) and that the influence of wind waves on storm surges was not considered. Neglecting the interaction between sea levels and waves can alter the estimate of return levels (e.g. *Arns et al.*, 2015). In this study, it is demonstrated in a later chapter that the contribution of wave setup in extremes creates a nonlinear pattern in constraint to the linear form found in these previous studies (e.g. see Figure 13 in *Hubbert and McInnes*, 1999). These limitations were not resolved until *McInnes et al.* (2009), who modelled sea level extremes with the presence of wave setup using the GCOM2D model (*Hubbert and McInnes*, 1999) coupled with SWAN (*Booij et al.*, 1999). Still, *McInnes et al.* (2009) exhibits several limitations. First, it used a coarse atmospheric forcing in terms of both temporal and spatial scales (e.g. $1.875^0 \times 1.785^0$ wind $2.5^0 \times 2.5^0$ pressure grids with 6-hour intervals) that did not allow the capture of the wind and wave patterns inside PPB. The reason is that the longest fetch of PPB is approximately 60km, equivalent to only 0.5 degree. Another limitation in *McInnes et al.* (2009) is that 6-hour wind forcing can underestimate the peak of storm

events in a small bay. Further, it is unclear if *McInnes et al.* (2009) prescribed an open boundary to allow swells and ocean waves propagating into their domain.

3.3.6 Wind-driven circulation

According to *Cardno Lawson Treloar* (2007b), wind-driven circulation is the dominant factor for water movement in the northern portion of PPB. Therefore, it is an important mechanism for flushing and sediment transport processes in this region. However, wind-generated currents are generally weak including in extreme events. During calm conditions, the current speed is least than 0.1 m/s; it reaches about 0.2 m/s (in shallow areas) during extreme events.

3.3.7 Baroclinic forcing

Baroclinic forcing is related to sea surface temperature and salinity. Inside PPB, temperature responds within a shorter time than flushing, which involves a complex interaction between tidal volume (or prism) and tidal phase. *Scientific and Industrial Research Organisation* (1996) shows that the water temperature in PPB is controlled by effects other than water exchange with the ocean; however, water exchange is the main factor affecting temperature. Therefore, it is unlikely that the predicted change in flushing time will have a significant impact on water temperature. In terms of salinity, *Scientific and Industrial Research Organisation* (1996) and *Cardno Lawson Treloar* (2007b) show that salinity in PPB is caused by the balance between the following processes: (1) inputs of freshwater from rain, rivers and drains, which tend to lower salinity; (2) evaporation of freshwater from the water surface, which tends to increase salinity; and (3) exchange of water with Bass Strait which is normally more saline than the bay, and thus tends to increase salinity.

3.4 Conclusions

PPB is an import coastal water environment. Previous studies have focused on hydrodynamics in this region; however, gaps remain, and these form the research interest for this study:

- Extreme sea levels: The influence of wave setup has not yet been investigated in sufficient detail (e.g. *McInnes et al.*, 2009). This is important issue as non-linear interaction between wave-tide-surges must be taken into account dealing with coastal region (*Arns et al.*, 2020).
- Wave climate: As long-term field measurements were missing, numerical modelling was applied to characterise the wave climate in this region (e.g. *Association of Bayside Municipalities*, 2017). However, there has been no study based on long-term hindcast simulations, which could provide sufficient data to determine the climatology of hydrodynamics in PPB.
- The impact of channel deepening on hydrodynamics has been investigated, but the potential impact of this engineering modification has not yet been fully understood because of a lack of data.
- The determination of which swells can propagation onto the body of PPB was lacking. This information can help determine whether ocean waves can affect the wave climate in any region in PPB.

Chapter 4

Observation-based physics in a spectral wave model -WWMIII

4.1 Introduction

Oceanic waves are considered major forces acting on marine activities that require wind-wave predictions. Numerical wave models have been operating since the 1950s (e.g. *Gelci*, 1957; *Pierson et al.*, 1955) to predict sea conditions. The outputs from these models are an essential source of information that can assist in determining sea states and improving our understanding of marine environments; this is important for many purposes, including offshore engineering, port navigation, coastal hazards and research activities. Numerical wave models are broadly grouped into two types: phase-resolving and phase-averaged (spectral) wave models (e.g. see *Babanin et al.*, 2017, for a recent review). For practical applications, the former is computationally expensive and not used for ocean scales. Therefore, spectral wave models are often employed to simulate the wave field in different water environments ranging from deep oceans to coastal waters. These models predict the evolution of the wave field under wind actions using the radiative transfer equation (RTE). Based on the conservation of wave actions rather than the energy, the main concept underlying RTE is that the random wave field is considered as a linear superposition of many harmonics (e.g. varying frequencies) propagating in different directions:

$$\frac{D}{Dt} \left(\frac{E}{\sigma} \right) = \frac{S}{\omega} \quad (4.1)$$

On the left hand side, D/Dt is the Lagrangian derivative, which estimates the rate of change in the wave action density (E/ω). The variance density E typically represents a five-dimensional variables: frequency (ω), direction (θ), spatial scale (x, y) and time (t). On the right-hand side, S accounts for various physical processes including the wind input (positive), wave dissipation (negative) and wave-wave interactions, among others. Many studies have shown that improvements in the spectral wave model can be obtained by detailing the physical processes on the right-hand side of Eq. 4.1.

In this study, the main focus is on the wind source terms and wave dissipation. To this end, it investigates the performance of spectral wave modelling within the

unstructured grid wave model WWMIII through the implementation of a newly advanced physical package - ST6 - which consists of wind-input and dissipation functions, derived from field observations. Before this work, ST6 has been implemented in the regular grid SWAN by *Rogers et al.* (2012) and *Rapizo et al.* (2017), and in WW3 models by *Zieger et al.* (2015) and *Liu et al.* (2019). Following this, several test cases have been conducted for different water wave environments, including deep-water waves without swells (Lake Michigan), deep water waves with swells (global domain) and shallow water waves without swells (PPB).

4.2 Limitations of existing parameterisations for wind source terms and wave dissipation

Spectral wave models use radiative transfer (Eq. 4.1) to represent the evolution of wind driven surface gravity waves. For the sake of clarity, this equation can be rewritten as follows:

$$\underbrace{\frac{\partial A}{\partial t}}_{\text{time space}} + \underbrace{\nabla_s[(c_g + U)A]}_{\text{geographical space}} + \underbrace{\nabla_k[c_k A]}_{\text{spectral space}} = \underbrace{\frac{S}{\omega}}_{\text{source/sinks}} \quad (4.2)$$

On the left-hand side, A is the action density which is a function of intrinsic radian frequency ω and wave number k , and is written as:

$$A = \frac{E(\omega, k)}{\omega} \quad \text{with} \quad \omega^2 = gk \tanh kH \quad (4.3)$$

c_g is wave group velocity, U is current speed, ∇ operators represent the geographical divergence (horizontal) and spectral space (with subscript k), g is the gravitational acceleration, and H is water depth. On the right-hand side, S represents various possible physical processes:

$$S = S_{at} + S_d + S_{no} + S_{tr} + S_{bo} + S_{deb} + S_{oth} \quad (4.4)$$

where S_{at} is wind input, S_d is dissipation, S_{no} is nonlinear, S_{tr} is triad, S_{bo} is wave-bottom, S_{deb} is deep-induced breaking and S_{oth} is other terms such as wave-ice interaction. These terms are also spectral functions. As this is the main subject of the present study, the following sections review the main drawbacks of existing wind source and wave dissipation functions.

4.2.1 Wind-input source terms (S_{at})

Waves are generated by wind travelling over the sea surface. Absorbing energy from the wind, waves develop across both time and space. The wind input is always the most important mechanism in wave generation models.

The physics underlying wind input date back to the 1920s (*Jeffreys*, 1924, 1925) with the novelty of airflow separation to support wave growth. *Jeffreys* considered wind blowing over the water surface (η) and proposed an expression to represent the normal wind tension including two terms: air pressure (p) and wind-induced pressure variations (Δp). The second term can be vary across different portions of

the surface, leading to the following expression (*Massel, 1996; Young, 1999*):

$$\Delta p = c_s \rho_{air} (U - C)^2 \frac{\partial \eta}{\partial x} \quad (4.5)$$

where c_s represents the sheltering effect (accounting for the resistance of the wave field to the atmosphere), U is wind speed and C is wave speed. The pressure fluctuations (Δp) can produce an energy flux on the water surface written as:

$$\frac{\partial E}{\partial t} = \alpha c_s (U - C)^2 (ak)^2 C \quad \text{with} \quad \alpha = \frac{\rho_{air}}{2\rho_w g} \quad (4.6)$$

The sheltering effect requires a phase shift (e.g. 180 degrees) of the pressure differences compared with the waves. For deep ocean, wave growth should meet the following constraint:

$$c_s \rho_{air} (U - C)^2 C > 4\epsilon g \quad (4.7)$$

where ϵ is the viscosity coefficient. This constraint requires the determination of the empirical sheltering coefficient $c_s \sim 0.3$ and the minimum wind speed $U_{min} \sim 1.1 \text{ m/s}$. Although *Jeffreys* (1924, 1925) found that the wave growth is exponential, in this work, tangential stresses were neglected. Further improvements of *Jeffreys* (1924, 1925)'s work were suggested by *Sverdrup and Munk* (1947), who considered tangential stresses in wave growth and introduced the energy balance equation. However, a seminal critical review by *UrSELL* (1956), significant in the history of oceanography research, showed that none of above studies adequately described wind-wave generation.

The cornerstones of the modern wind-wave generation models were laid by *Phillips* (1957) and *Miles* (1957). *Phillips* (1957) proposed a theory to examine possible mechanisms for wind-wave evolution, with the main mechanism considered pressure differences due to turbulent eddies. The expression for the wind source terms (S_{at}) is (*Young, 1999*):

$$S_{at} = \frac{2\pi^2 \omega}{\rho_w^2 C^3 C_g} \Gamma(k, \omega) \quad (4.8)$$

where C_g is wave group velocity, C is phase velocity and $\Gamma(k, \omega)$ is the atmospheric wave number frequency spectrum (three-dimensional). Eq. 4.8 produces linear growth, which is important in the initial stages of wave evolution. *Miles* (1957) formulated an idealised model referred to as the quasi-laminar wave generation model, and applied this to inviscid flows with suitable boundary conditions without considering turbulent stresses. As a result, the study claimed that a possible mechanism for wind-wave generation is the resonance of waves and wave-induced pressure differences. Miles's theory predicted exponential growth of the wave spectrum (*Massel, 1996; Young, 1999*):

$$S_{at} = \gamma A \quad \text{with} \quad \gamma = \frac{\rho_{air}}{\rho_w} \beta \left(\frac{u_*}{C} \cos \theta \right)^2 \quad (4.9)$$

where β comprises two dimensionless parameters $\beta = \beta(u_* \cos \theta, \Omega)$, u_* is friction velocity, $\Omega = gz_0/u_*^2$ is dimensionless roughness and z_0 is the roughness length. It

is generally accepted that Miles's theory provided a supplement to *Phillips* (1957) to describe initial growth and wave evolution. However, it is noted that *Miles* (1957, 1959) applied to high-frequency waves, and neglected effects of viscosity and air turbulence on the critical layer. Further, Eq. 4.9 requires the estimating of wind friction velocity u_* and roughness length z_0 , based on the similarity laws and logarithmic wind profile using Charnock's formula:

$$C_z = \left(\frac{u_*}{U_z} \right)^2 \quad \text{with} \quad U_z = \frac{u_*}{\kappa} \ln \left(\frac{z}{z_0} \right) \quad \text{and} \quad z_0 = a \frac{u_*^2}{g} \quad (4.10)$$

where C_z is the drag coefficient and κ is the Kaman constant. In practice, the wind profile is estimated at 10m height: $z = 10m$, $U_z = U_{10}$, $C_z = C_{10}$. Research has found that the wind drag coefficient is a key parameter to be estimated. It largely depends on the wave age: (1) a young sea corresponds to the sea state where waves are generated by wind, when $U_{10}/C > 1$ or $C/u_* < 10$; (2) an old sea refers to the saturated sea, when $U_{10}/C < 1$ or $C/u_* > 10$. For example, an additional stress arises air flows over the water waves compared with a flat surface (e.g. *Massel*, 1996). This added term, introduced by *Janssen* (1989), is called wave-induced stress. *Janssen* (1989) found that wave-induced stress accounted for up to 95% of the total stress in young sea conditions, and thus the wave growth of *Miles* (1957, 1959) needed some improvement. As a result, an adapted version of Miles's expression was proposed by *Janssen* (1991) for wave growth:

$$\gamma = \frac{\rho_{air}}{\rho_w} \frac{\beta_{max}}{\kappa^2} \exp(z) z^4 \left[\frac{u_*}{C} \max(\cos(\theta - \phi), 0) \right]^2 \quad (4.11)$$

where θ , ϕ are the wave and wind directions respectively and β_{max} is a turning parameter. In comparison to Eq. 4.9, Eq. 4.11 considers the wave age:

$$z = \ln \left[\frac{gz_0}{C^2} \right] + \frac{\kappa}{\cos(\theta - \phi) \left(\frac{u_*}{C} + z_\alpha \right)} \quad (4.12)$$

where z is the wave age coefficient controlled by turning parameter z_α . However, Miles's theory was based on an idealised model that did not consider turbulent eddies. *Janssen* (2004) added swell damping effects to the second parts of Eq. (4.11). The final version of the wave growth rate:

$$\gamma = \frac{\rho_{air}}{\rho_w} \left[\frac{\beta_{max}}{\kappa^2} \exp(z) z^4 \left[\frac{u_*}{C} \max(\cos(\theta - \phi), 0) \right]^2 + 2\kappa \left(\frac{u_*}{C} \right)^2 \left(\cos(\theta - \phi) - \frac{C}{U_w} \right) \right], z \leq 0 \quad (4.13)$$

where the second term represents a correction for swell damping effects, which is of an order of thousands of kilometres and U_w is wind speed.

4.2.2 Wave dissipation (S_d)

Wind energy transfer to water waves at both temporal and spatial scales can amplify the wave amplitude - waves become unstable then break. The breaking process results in energy dissipation or loss (*Hasselmann*, 1974; *Holthuijsen*, 2007; *Komen et al.*, 1984; *Massel*, 1996; *Young*, 1999).

The pioneering work to describe wave dissipation was conducted by *Hasselmann* (1974) who formulated an analytical model that considers the loss of wave energy as random pressure pulses. This study was then adapted by *Komen et al.* (1984). The general form of the S_{di} formulation is as follows (for comprehensive reviews, see *Ardhuin, 2012; Ardhuin et al., 2010; Cavaleri et al., 2007*):

$$S_d(k, \theta) = \alpha_d \sqrt{g} k_a^{4.5} H_s^4 \left[(1-b) \frac{k}{k_a} + b \left(\frac{k}{k_a} \right)^2 \right] F(k, \theta) \quad (4.14)$$

$$k_a = \left[\left(\frac{4}{H_s} \right)^2 \int_0^{f_{cut}} \int_0^{2\pi} k^a E(k, \theta) dk d\theta \right]^{1/a}; \quad H_s = 4 \sqrt{\int \int E(k, \theta) dk d\theta} \quad (4.15)$$

where α_d is the dissipation constant, $F(k, \theta)$ is spectral density with wave number k and direction θ , H_s is the significant wave height, k_a is the energy mean-weighted mean wave number calculated from the entire spectrum, a and b are adjustable parameters based on the user's experience, α_d is a non-dimensional constant and f_{cut} is the cut-off frequency. The problem with Komen's dissipation formulation, including the updated version by *Bidlot et al.* (2005), is the use of mean steepness, which is adjusted for the entire spectrum. Such an adjustment attempts to fit the energy loss as a turning knob, which may not be appropriate given to the underlying physics (*Ardhuin et al., 2010; Cavaleri et al., 2007*). Applying mean steepness to the entire spectrum may cause discrepancies in predicting sea states when both wind sea and swells coexist.

Several modifications to wave dissipation have been implemented to overcome the shortcomings of Komen's expression. *Banner and Young* (1994) made significant progress by pointing out the essential changes in wave dissipation made by a nonlinear term. Subsequently, *Alves and Banner* (2003) proposed a new expression for the dissipation term based on local saturation $B(k)$ derived from the azimuth-integrated wavenumber $\chi(k)$ from *Banner et al.* (2002):

$$B(k) = k^4 \chi(k) \quad \text{with} \quad \chi(k) = \int_{-\pi}^{\pi} F(k, \theta) d\theta \quad (4.16)$$

The physical meaning of Eq. 4.16 is that spectral saturation depends on wave steepness. The saturation-based dissipation term with $B(k)$ dependency is as follows:

$$S_d(k, \theta) = \alpha_d \left[\frac{B(k)}{B_{st}} \right]^{s/2} \underbrace{(E_t k_p^2)^M}_{T1} \underbrace{\left(\frac{k}{k_m} \right)^N}_{T2} \sigma F(k, \theta) \quad (4.17)$$

where B_{st} is the saturation threshold, $E_t k_p^2$ is an integrated spectral steepness parameter with peak wave number k_p , E_t is the threshold saturation level defined as $E_t = \int_0^{2\pi} \int_0^\infty F(k, \theta) k dk d\theta$, M and N are user-defined coefficients, which can be determined by calibration, k_m is the mean wave number and s is a smooth function

that can be defined by:

$$s = \frac{s_0}{2} + \frac{s_0}{2} \tanh \left[10 \left(\sqrt{\frac{B(k)}{B_{st}}} - 1 \right) \right] \quad (4.18)$$

Alves and Banner (2003) defined several B_s values ranging from 0.0031 to 0.00425 with $s_o = 4-8$; these values are two times larger than those in *Banner et al.* (2002). *Westhuysen et al.* (2007) remarked that such large coefficients can impose a high level of dependency of dissipation on the spectral density, up to an order of 5. Further, $B(k)$ is still supplemented by several factors including the turbulent effect $T1$ and wave-wave interactions $T2$ resulting in an oversimplification of turbulence-wave influences. To overcome these shortcomings, the influence of $T1$ and $T2$ was removed in *Westhuysen et al.* (2007) by setting $M = 0$ and $N = 0$; Converting Eq. 4.17 into frequency domain $\omega = \sqrt{gk}$, *Westhuysen et al.* (2007) proposed a new version of the wave dissipation term as follows:

$$S_d(\omega, \theta) = \alpha_d \left[\frac{B(k)}{B_{st}} \right]^{s/2} \sqrt{gk} E(\omega, \theta) \quad \text{with} \quad B(k) = c_g k^3 E(\omega) \quad (4.19)$$

In this expression, α_d , B_{st} and s are retained as calibration coefficients. However, for non-breaking waves, when the value of s approaches zero, the dissipation expression of *Westhuysen et al.* (2007) exhibits some uncertainties.

After the work of *Westhuysen et al.* (2007), *Ardhuin et al.* (2010) attempted to produce a new set of wave dissipation terms incorporating the realistic features of wave breaking introduced by *Babanin and Young* (2005). However, *Ardhuin et al.* (2010) drew on *Westhuysen et al.* (2007)'s formulation to derive their dissipation expression, which was then combined with the wind source functions introduced by *Bidlot et al.* (2007); *Janssen* (2004).

In conclusion, previous studies have developed various formulations for the wind source and wave dissipation terms. No study has been fully derived from observational data, because measurements of wave growth and dissipation are extremely challenging. Therefore, it remains questionable whether further improvements can be made given both wind source and wave dissipation terms are not empirically based on observations. The answer to this question is the subject of the present study.

4.3 Observation-based wind input and wave dissipation functions

Comprehensive findings from field measurements of wind-wave growth during the Australian Shallow Water Experience (AUSWEX) at Lake George, Australia (e.g. see *Babanin et al.*, 2001) have brought new knowledge about the physics of wind-wave energy exchanges. This is the only field experiment conducted under a broad range of atmospheric forcing conditions (e.g. from light to strong winds). The outcomes from this study provide a unique opportunity to derive a new set of wind input and wave dissipation functions.

4.3.1 Source term functions

The wind input source function proposed in this study is based on *Donelan et al.* (2006, 2005). Compared with previous studies, the new features of this source term include : (1) full air separation with a reduction of wind input in strong wind forcing, (2) the growth rate as a function of the wave steepness and (3) the influence of wave breaking in the wind input. Wave steepness is linked to the phase shift and the normalised magnitude of the wave-induced pressure. This term only follows the potential flow theory when wave steepness approaches zero. Because wave steepness can be expressed as the saturation-based spectrum derived by *Phillips* (1984), wind input can be parameterised as nonlinear function of the wave spectrum:

$$S_{at} = \frac{\rho_{air}}{\rho_w} \omega \gamma \Gamma(k, \theta) \quad (4.20)$$

where γ is the wave growth rate measured at 10m-height above the mean water surface, which can be determined by:

$$\gamma = \underbrace{s_c}_{T1} \underbrace{\sqrt{B_{st}}}_{T1} \underbrace{W(k, \theta)}_{T3} \quad (4.21)$$

The three terms in Eq. (4.21) are explained as follows. The first term (*T1*) represents the sheltering effect, which includes the reduction of momentum transfer due to airflow separation, determined by:

$$s_c = 2.8 - 1.0 \left[1 + \tanh \left\langle 10 \sqrt{B_{st}(k)} W(k, \theta) - 11 \right\rangle \right] . \quad (4.22)$$

The second term (*T2*) is the saturation-based spectrum, which is based on the local wave number, written as:

$$B_{st}(k) = A(k) \Gamma(k, \theta). \quad (4.23)$$

where $A(k)$ is the calculated width of angular distribution proposed by *Babanin and Soloyev* (1987): $A^{-1}(k) = \int_0^{2\pi} \Gamma_n(k, \theta) d\theta$ with $\Gamma_n(k, \theta) = \Gamma(k, \theta) / \max(\Gamma(k, \theta))$.

The third term (*T3*) is the wind forcing relative to the wave direction, calculated as:

$$W(k, \theta) = \left[\frac{U_\alpha}{C} \cos(\theta - \theta_w) - 1 \right]^2 \quad (4.24)$$

where U_α is the wind speed scaled by friction velocity u_* : $U_\alpha = \alpha u_*$ with $\alpha = 28$ proposed by *Komen et al.* (1984). The directional distribution is assessed by considering the effect of both following and adverse wind inputs, calculated as:

$$\begin{aligned} W(k, \theta) &= \underbrace{W_f(k, \theta)}_{\text{favourable wind}} - \underbrace{n W_a(k, \theta)}_{\text{adverse wind}} \\ W_f(k, \theta) &= \max^2 \left[0, 28 \frac{u_*}{C} \cos(\theta - \theta_w) \right] \\ W_a(k, \theta) &= \min^2 \left[0, 28 \frac{u_*}{C} \cos(\theta - \theta_w) \right] \end{aligned} \quad (4.25)$$

where n is the turning coefficient, u_* is the wind friction velocity, which can be determined from the total stress τ (e.g. the total momentum flux from the wind to

the waves): $u_*^2 = \tau / \rho_{air}$, C is the phase velocity, $\theta - \theta_w$ is the angular difference between the wave and wind direction. *Tsagareli et al.* (2010) calculated the total wind stress, which is similar to those of *Janssen* (1989, 1991). In particular, total stress consists of two terms:

$$\tau = \tau_w + \tau_\nu = \tau_w + \rho_{air} C_V U_{10}^2 = \rho_{air} C_z U_{10}^2 = \rho_{air} u_*^2 \quad (4.26)$$

where τ_ν is viscous stress and C_z, C_v respectively are wind and viscous drag coefficients. For wind drag, *Rogers et al.* (2012) employed the formulation proposed by *Hwang* (2011) which is consistent with saturation and reduced in high winds, and is calculated as:

$$C_z = (8058 + 967U_{10} - 16U_{10}^2) \times 10^{-7} \quad (4.27)$$

However, constraint: $u_* = 2.026$ m/s is applied for wind speeds higher than 50.33 m/s. For viscosity, *Tsagareli et al.* (2010) proposed drag coefficient C_v as a function of wind speed U_{10} as follows:

$$C_v = (1.1 - 0.05U_{10}) \times 10^{-3} \quad (4.28)$$

τ_w is wave-influenced (normal) stress, which is constrained by the total stress at the surface. This term is a vector integral as demonstrated in *Zieger et al.* (2015).

4.3.2 Wave dissipation

The observation-based formulation of wave (white-capping) dissipation was derived from a series of studies done by *Babanin and Young* (2005); *Babanin et al.* (2010, 2001); *Banner et al.* (2000); *Donelan* (2001); *Rogers et al.* (2012); *Young and Babanin* (2006). Two novel features were included in this formulation: (1) the threshold constraint of the wave breaking and spectral saturation (e.g. *Banner et al.*, 2000), (2) a two-phase function for dissipation mechanisms (e.g. *Babanin et al.*, 2010; *Young and Babanin*, 2006). The physical meaning of the first inclusion is that waves do not break until wave steepness goes beyond a threshold. The second inclusion implies wave dissipation arises due to two distinct mechanisms. First, wave breaking is linked to local effects (e.g. instability of waves at that frequency), in contrast to the approach by *Komen et al.* (1984), which considered the entire spectrum. Second, wave dissipation of short waves is induced by long waves. Details of the white-capping dissipation are as follows:

$$S_d(k, \theta) = \left\langle \underbrace{D_1(k, \theta)}_{\text{Inherent breaking}} + \underbrace{D_2(k, \theta)}_{\text{Cumulative effect}} \right\rangle \Gamma(k, \theta) \quad (4.29)$$

where the first term D_1 represents the inherent breaking and the second term D_2 is the cumulative influence of short wave breaking induced by long waves. D_1 only occurs at frequencies equal to or below the peak frequency; otherwise, D_2 activates and becomes dominant because the peak downshifts further. The physical constraint of breaking is determined by the threshold, calculated as:

$$\Gamma_T(k) = \frac{\wp_b}{A(k)k^3} \quad (4.30)$$

where $A(k)$ is angular narrowness (e.g. *Banner et al.*, 2002, $A(k) \approx 1$) with wave number k and $\wp_b = 0.001225$ is an empirical coefficient (*Babanin et al.*, 2007). The exceeded energy above the threshold $\Gamma_T(k)$ can be determined by $\Delta(k) = \Gamma(k) - \Gamma_T(k)$; these values are required to calculate each dissipation mechanism in Eq. 4.29 via the following:

$$D_1(k) = c_1 A(k) \frac{\omega}{2\pi} \left\langle \frac{\Delta(k)}{\Gamma_n(k)} \right\rangle^{m_1} \quad (4.31)$$

$$D_2(k) = c_2 \int_0^k A(k) \frac{C_g}{2\pi} \left\langle \frac{\Delta(k)}{\Gamma_n(k)} \right\rangle^{m_2} dk \quad (4.32)$$

where $\Gamma_n(k)$ is the generic spectrum density, which can be either $\Gamma_T(k)$ (similar to *Ardhuin et al.*, 2010) or $\Gamma(k)$ (as in *Tsagareli et al.*, 2010), c_1, c_2, m_1, m_2 are turning parameters (e.g. see *Rogers et al.*, 2012).

It is noted that Eq. 4.31 and 4.31 are constrained by a threshold $\Gamma_T(k)$. This means that S_d in Eq. 4.29 will not be activated if the dissipation energy is below this critical level. However, observational studies have demonstrated that wave breaking still occurs even if the energy is below $\Gamma_T(k)$ - a case known as swell dissipation. Further, for swells, the direction of propagation may not be relevant to the wind. Therefore, an additional term is required to estimate wave dissipation due to swells. This study uses the formulation implemented in WW3 by *Zieger et al.* (2015) derived from studies done by *Babanin* (2011); *Babanin and Haus* (2009); *Young et al.* (2013). Essentially, swell attenuation is relevant to wave-turbulence production, and expressed as follows:

$$S_{LF}(k, \theta) = -\frac{2}{3} c_3 \omega [B_{st}(k)]^{0.5} \Gamma(k, \theta) \quad (4.33)$$

where c_3 is the scaling coefficient. In *Zieger et al.* (2015), the authors proposed $c_3 = B_{sw} H_s k_p$. Therefore, B_{sw} is the only turning parameter in Eq. 4.33.

In summary, the observation-based physical package (ST6) for wind input and wave dissipation requires only six turning parameters: n for the wind input in Eq. 4.25, c_1, c_2, m_1, m_2 for wave dissipation in Eq. 4.31 and 4.32 and B_{sw} for swell attenuation in Eq. 4.33.

4.4 Implementations and calibrations

The implementation of ST6 physics in WWM requires an upgrade in the Fortran source code. Following a suggestion by *Liu et al.* (2019), the α in Eq. 4.24 was changed from 28 to 32 (wind scale factor) to improve the model's skills in the tail level. For this reason, this study applied the same turning parameters as used in *Liu et al.* (2019), (e.g see $ST6^D$ in Table 1 of their study for the calibration). The updated physics in WWMIII were then applied to simulate waves in three different water environments: (1)- deep water waves without swells - Lake Michigan (US); (2) - deep water waves with swells at a large scale with one year of global hindcasts (2011); and (3) - shallow water waves without swells - PPB. All simulations were forced with CFSR wind data (*Saha et al.*, 2010, 2014). For global hindcasts, wind forcing has a spatial resolution of 0.5 degree and a 3-hour time interval to reduce

computational costs. However, Lake Michigan and PPB domains were run with hourly data and 0.25 degree - spatial resolution. It is noted that, in line with computational expensive at global scale, only ST6 was tested and compared with buoy and altimeter data, while both ST4 (Ardhuin *et al.*, 2010) and ST6 were applied for Lake Michigan and PPB.

4.4.1 Lake Michigan

Two months of simulations for September and October 2002 were conducted for Lake Michigan. The modelled results were then compared with NBDC (National Buoy Data Center, <https://www.ndbc.noaa.gov/>) buoy 45007, located in the southern portion of Lake Michigan with water depth greater than 100m. Waves in this lake are predominately by young seas, without interaction with swells. The computational domain along with buoy location is shown in Figure 4.1.

Figure 4.2 show timeseries and scatter plots, with basic statistical parameters obtained from in calibration and validation. For September 2002, both ST6 and ST4 produced the same correlation (CORR) of 0.89 and root mean squared error (RMSE) of 0.27. However, ST6 performed better than ST4 in terms of R^2 and bias (BIAS). For example, the R^2 of ST6 is 0.79, 0.01 higher than ST4 while the bias in ST6 is 5cm lower than its counterpart. For October 2002, ST6 performs better than ST4 in all statistical aspects. In terms of correlation, the difference between the two physics is only 0.01. However, ST6 has a R^2 of 0.81 which is 0.05 higher than ST4. The absolute bias of the newly physics is also 0.03m lower than the semi-empirical ones. It can be seen from scatter plots that in both September and October, ST4 underestimated the SWH, as the scatter plots incline to the x-axis, which represents the field data. This is made even clearer by examining the timeseries in panel e and f in Figure 4.2. In September, ST4 underestimated SWH by up to 1m during the largest events of the month although ST6 did also not produce that peak. The evidence for underestimation of large waves in ST4 can also be observed in October, since this package did not capture the four largest events; in contrast, ST6 was able to correctly produce three of four events. This underestimation of high wave height could result in lower estimates of extreme values.

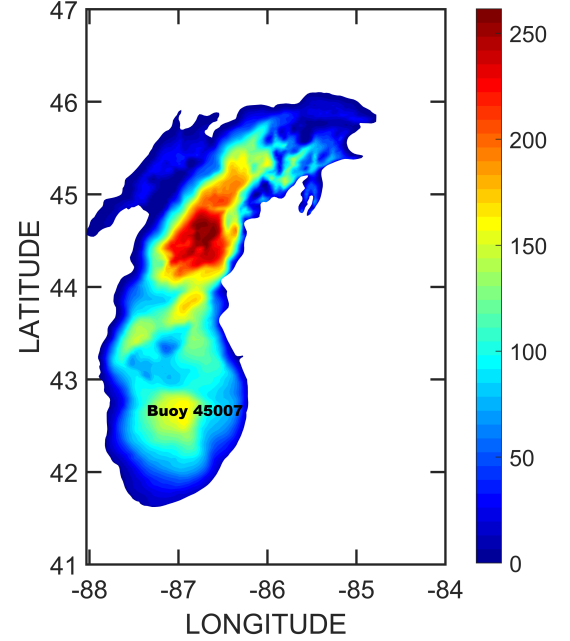


Figure 4.1: The computational domain of Lake Michigan with the water depth larger up to 200m in the north. The buoy (Buoy 45007) used to calibration and validation is located in the south of the lake at the water depth deeper than 100m.

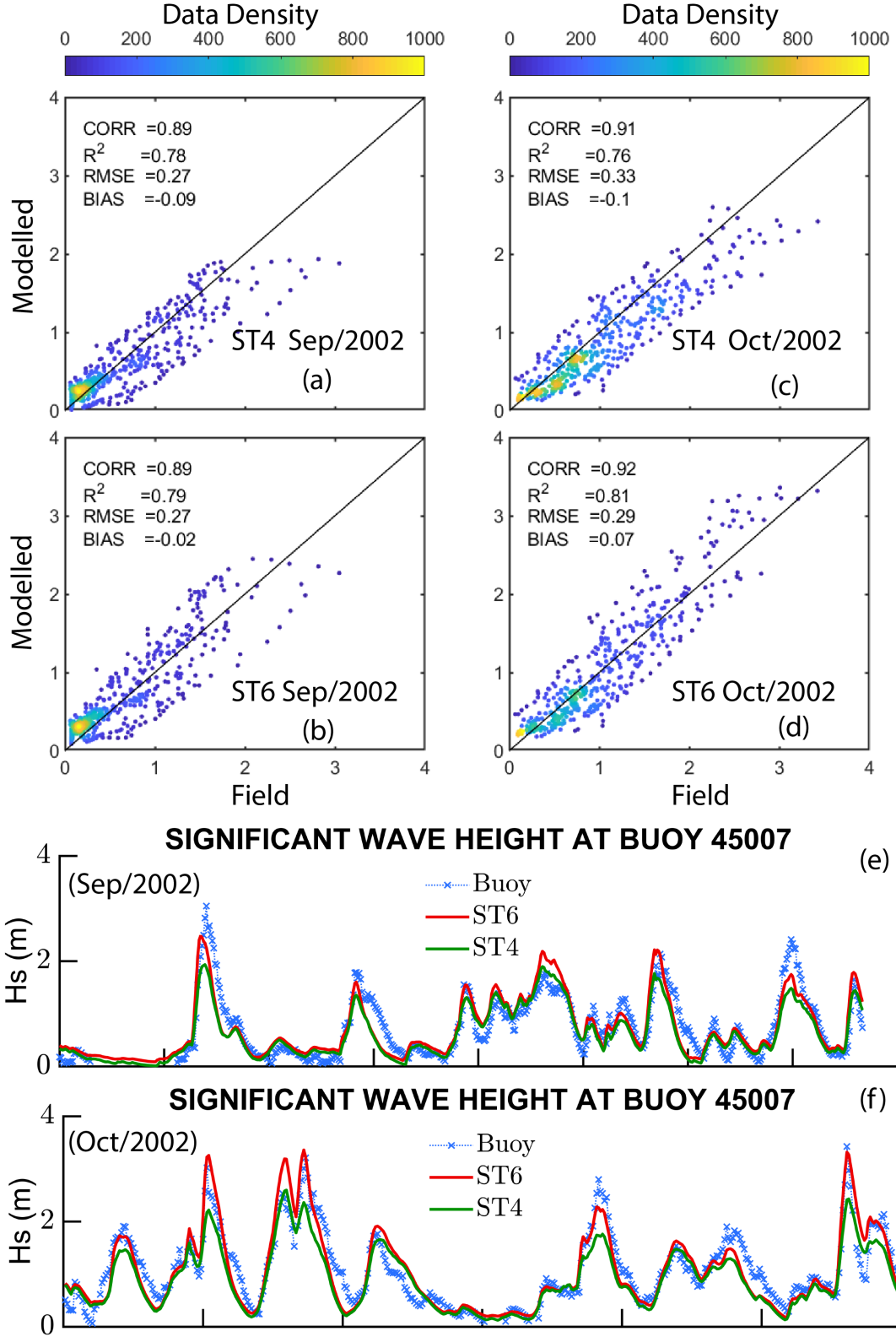


Figure 4.2: A comparison between SWH as simulated by ST4 and ST6 physics at buoy B45007, Lake Michigan for September and October 2002. Timeseries for each month are shown in panels e and f respectively. The scatter plots for ST4 physics in these months are shown panels a and b, while ST6 scatter plots are shown in panels c and d.

4.4.2 Global simulations

To further demonstrate the performance of ST6 as producing a realistic sea state suitable for practical applications, global hindcast simulations were conducted for the one-year period of 2011. This is the same period selected by *Ardhuin et al.* (2010) and *Liu et al.* (2019) for validation of ST4 and ST6 physics in the WW3 model (*Tolman, 1992*). However, those studies were conducted for rectangular grids with a 0.5-degree resolution, while this study uses an unstructured grid. ST6 validation was undertaken by comparisons of modelled SWH with available situ-measurements including buoy data from the US coasts and calibrated altimeter data provided by *Ribal and Young (2019)* and *Young and Ribal (2019)*.

Buoy data

Buoy data used in this study were drawn from buoys located around the US coasts operated by the National Data Buoy Center (NDBC). The quality of this in-situ dataset has been undertaken by and obtained from the National Oceanographic Data Center. A full list of all buoy locations can be found at <https://www.ndbc.noaa.gov/>. Buoy locations are shown in Figure 4.4.

Altimeter data

Satellite radar altimeters have been operated since 1985 with the first launch of GEOSAT. Currently, up to 13 satellites have been operated. These various missions can provide information on surface waves in the form of SWH. SWH is estimated based on the sea surface variance σ^2 (*Holthuijsen, 2007; Zieger et al., 2009*). From this, SWH is derived as $SWH = 4\sqrt{\sigma^2}$. Figure 4.4 shows an example of SWH between 06:00AM and 09:00AM on 01-October 2011 from five missions available in 2011. A detailed description of those satellites (drawn from *Ribal and Young, 2019*) is described below.

Jason-1: This satellite was launched in December 2001 using the orbit of the former TOPEX mission. This mission was operated by the CNES and NASA. Originally, Jason-1 was planned to operate over a 5-year period; however, it continued until 2013. Measurements were recorded based on a 10-day repeat cycle.

Jason-2: This satellite is one of the ongoing altimeters launched in June 2006 by a consortium of agencies including CNES, NASA, NOAA and EUMETSAT. This mission provides high-accuracy measurements relative to previous counterparts (e.g. Jason-1, TOPEX), with the same repeat cycle of 10 days.

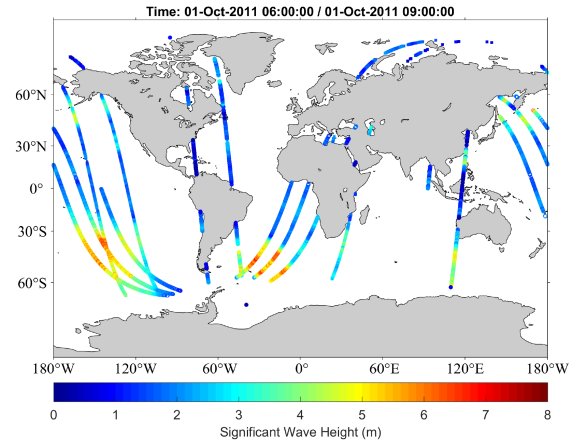


Figure 4.3: An example of SWH produced from satellites between 06:00AM to 09:00AM on 1 October 2011 for the whole world. All data within 50-km of the coastlines are excluded, as data are not reliable in coastal regions (e.g. see section 2.5 in Cavalieri et al., 2018, for a review).

Envisat: Envisat stands for the Environment Satellite, which was launched in March 2002 by the European Space Agency (ESA). This mission provides sea surface information with a 35-day cycle up to October 2010. After this, the orbit of Envisat was updated with a shorter repeat cycle of 30 days. The operation of Envisat ended in May 2012.

Cryosat-2: This satellite was launched in April 2010 by the ESA for the purpose of polar observations; in line with this, this mission covers almost all polar areas. Cryosat-2 was equipped with an interferometer and an altimeter named Doris and Sival, which use Ku-bands to provide oceanic measurements.

HY-2: The *Hai Yang 2A* mission is operated by the National Satellite Ocean Application Service (NSOAS, China). HY-2 is China's first dynamic radar altimeter, and was launched on 15 August 2011 to measure marine dynamics. This mission uses dual-frequency Ku- and C-bands to detect the ocean surface environment. SWH as measured by HY-2 in 2011 was based on a 14-day cycle from an altitude of 971km.

Validation

Simulated SWHs from WWMIII were extracted from simulations with 3-hour time intervals for every grid point (~ 500.000 nodes) in a computational domain covering the entire globe. These data will be compared against the altimeter data. To validate the buoy data with the modelled results, extracted locations were set in model configurations with hourly outputs. For altimeter data, an extra process involved collocating and matching satellite and modelled SWHs into the same grid point. A rectangular grid with 1-degree resolution was created for collocating satellite and modelled data into this grid. Once all data were collocated, the match-up process was undertaken. A detailed description of these processes can be found in *Appendini et al.* (2015).

Figure 4.4 b shows WWMIII validation results using ST6 for 2011 with buoy data in a scatter plot. Good agreement between the model with the buoy data was obtained. For example, both data yield a correlation of 0.9 with a R^2 of 0.81. The RMSE between the two datasets is below 0.5m (around 0.49) with a bias of -0.1m. This small bias can be clearly seen in the q-q plot, presented in panel (c). In this panel, the model data slightly deviate from the red line at the top quantile levels. For WW3, *Liu et al.* (2019) also reported a high correlation of above 0.9 with biases ranging from 0.11m to 0.29m for $ST6^D$. Therefore, it has been evidenced that the ST6 physics in the unstructured grid WWMII can well replicate SWH at buoy locations.

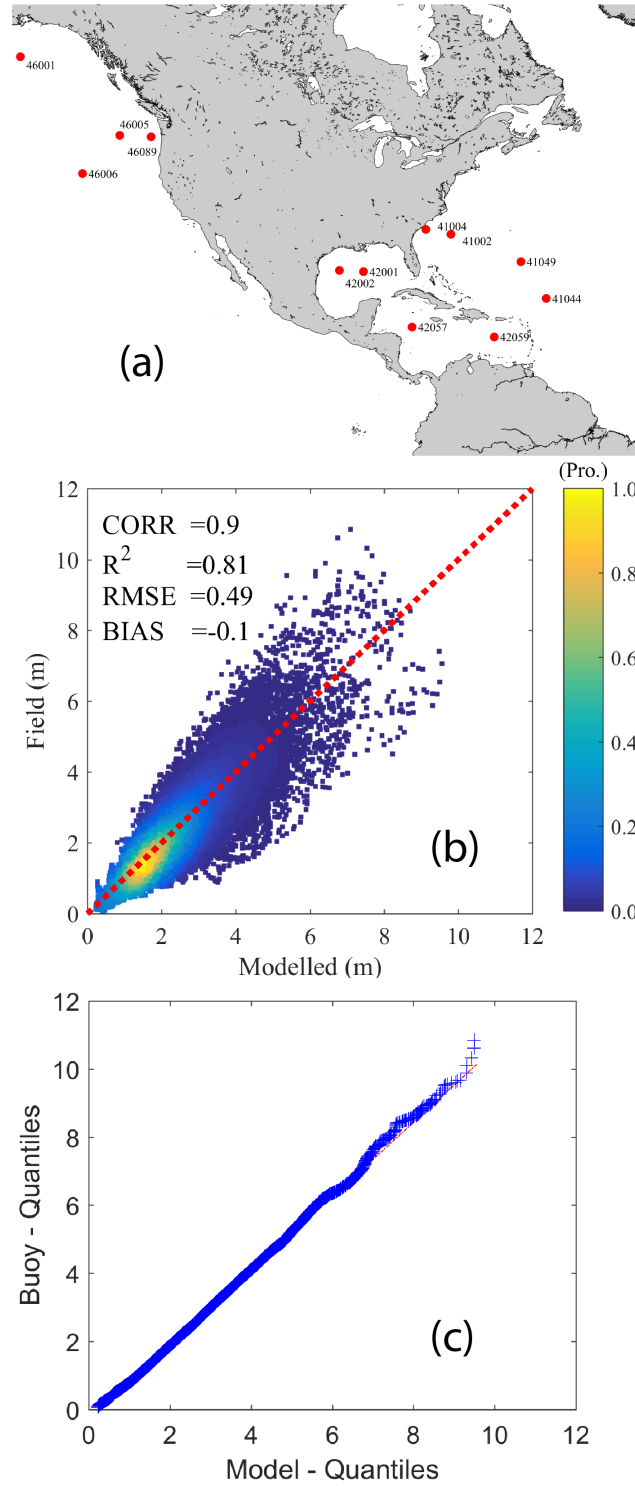


Figure 4.4: Validation of modelled results (WWMIII) with buoy data available in 2011. Selected buoy locations are shown in panel (a). The scatter plot in panel (b) compares the SWH from WWMII configured with ST6 physics against the buoy data. A high agreement of 0.9 between the two data types was achieved, with a low bias of 0.1m. The q-q plot in (c) shows that the WWM model slightly underestimates the buoy data at high-quantile levels. This minor underestimation is reflected in the small bias of -0.1m in panel (b).

For the altimeter data validation, Figure 4.5 maps spatial variations in term of correlation (panel a), bias (panel b) and RMSE (panel c). In terms of correlation, an agreement greater than 0.9 was obtained for a large part of the Pacific Ocean and the North Atlantic. The low correlation is concentrated at the south pole and some areas around the coastlines. WWMIII with ST6 physics generally produces low biases, ranging from -0.4 to 0.4m. Areas of low values are located in the eastern and southern Pacific Oceans, as also indicated by *Liu et al.* (2019). These authors also reported that the major areas of negative biases are located in the Indian, Pacific and central Atlantic Oceans. The underlying reason is the underestimation of wind speed in CFSR version 2. In WWMIII, the largest biases are often located around islands and in the Southern Ocean. In terms of the effect of islands, WWMIII does not include features to resolve blocking effects due to obstacles, as proposed by *Tolman* (2003); however, the blocking effect due to islands and other mismatches are not fully described. For example, *Liu et al.* (2019) reported a more noticeable underestimation in the region near Indonesia. However, this is not the case in the present study. As shown in panel b of Figure 4.5, WWMII generates low bias around the islands of Indonesia. For the south, *Ardhuin et al.* (2011) found that the positive bias from the latitude of 50° further to the south pole could be alleviated by implementing blocking effects for icebergs. Figure 4.5 shows the map of spatial variation in RMSE. Clearly the largest errors are located at the two poles and around some big islands. At the poles, a large RMSE could be due to sea-ice, which is not included in WWMIII. It is noted that a large error in the Southern Ocean has been reported in many studies (e.g. *Ardhuin et al.*, 2010; *Liu et al.*, 2019; *Zieger et al.*, 2015). For example, a RMSE of up to 1m can be produced by WWM around New Zealand islands. This could be due to blocking effect of obstacles (*Tolman*, 2003). However, it is doubtful that this is the only reason, as the model can also produce low RMSE in the vicinity of other big islands and coasts, such as around the African and Asian coastlines. The only other possible reason is that unstructured grid features result in a large error in matching up with altimeter data, as unstructured grids are often refined along coastlines while regular grids with coarse resolutions (e.g. 0.5 degree) as done in WW3 (e.g. *Liu et al.*, 2019) does not contain coastline features. Other than that, WWMIII with ST6 physics can produce a low RMSE for a large section of oceans, including the Indian, Atlantic and Pacific Oceans, with a low bias of around 0.1m only.

Therefore, we conclude that WWMIII can reasonably replicate the SWH at global scale with a high correlation of above 0.9 covering nearly two-thirds of all oceans. The model also has a low bias ranging from -0.4 to 0.4m. Mismatches at several locations are explainable due to the influence of wind forcing, as also reported in other studies. RMSE is generally small, except for at the two poles, which may be affected by sea-ices.

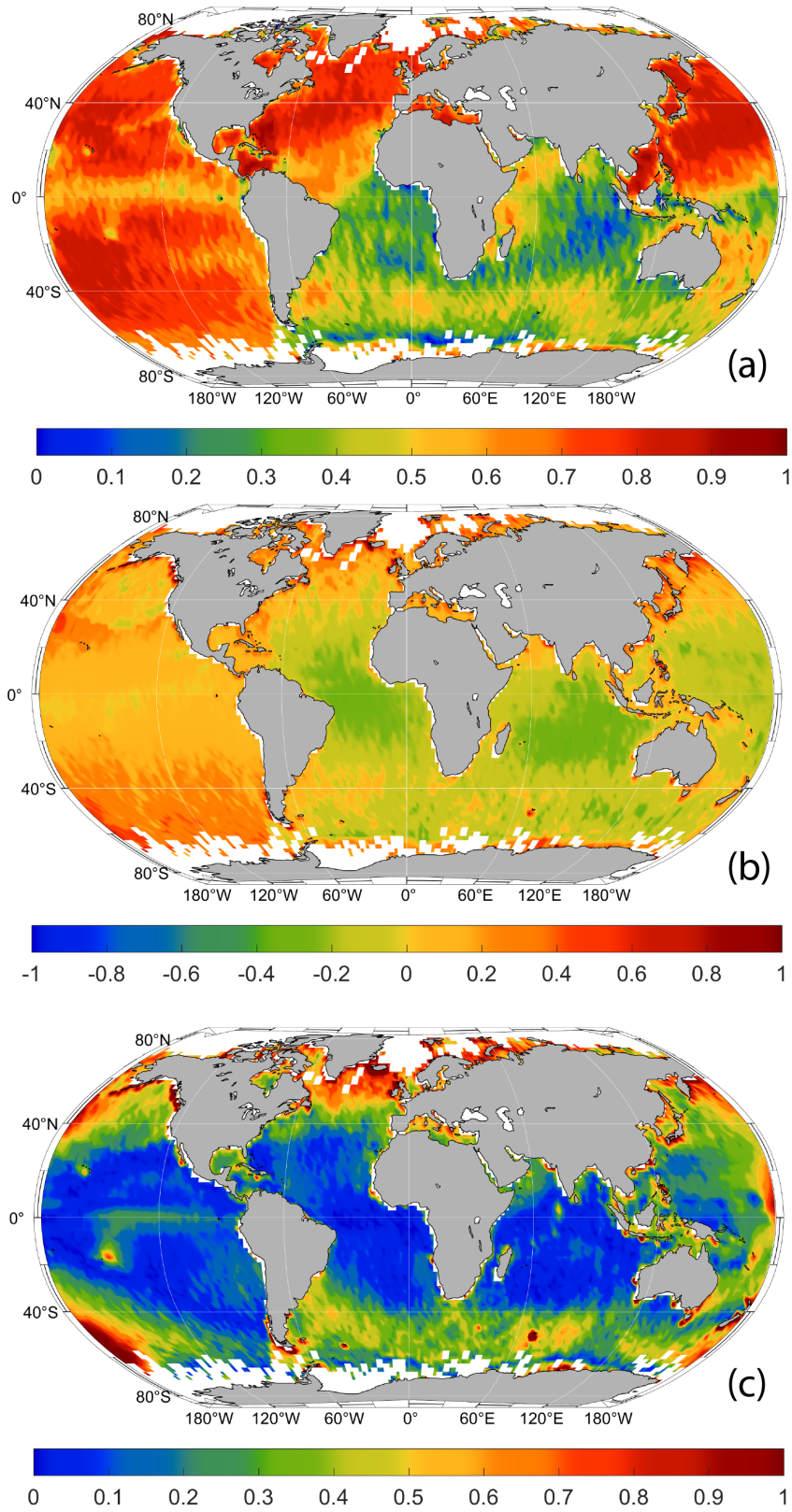


Figure 4.5: Validation of ST6 physics in WWMII with altimeter data for 2011. The top panel (b) shows a spatial correlation (CORR). Panel (b) represents the bias (BIAS), and panel (c) on the bottom shows the root-mean-squared error (RMSE).

4.4.3 Shallow water environments

The last step in the validation of ST6 physics was to test with waves in a shallow water environment; in this case, PPB. PPB is a shallow bay with water depth no greater than 30m. Waves inside the bay are generally caused by wind blowing over the water surface. To validate ST6 physics, WWMII is forced with only wind, CFSR as done for Lake Michigan and global hindcasts. Measured wave data are required to validate the model. Although wave buoys have been deployed in the vicinity of entrance of PPB for 15 years, wave measurements inside PPB are scarce. The most relevant data were undertaken before the channel deepening project provided by industry (Cardno Ltd, Melbourne) in 2003. Two wave buoys were deployed at the same time at Rosebud and Beacon #15 near the shipping channel, to measure the waves due to vessels. Further, there is a wind station close to the location of the buoy deployments, which could provide additional information to interpret the results. The locations of buoys and wind stations are shown in Figure 4.7.

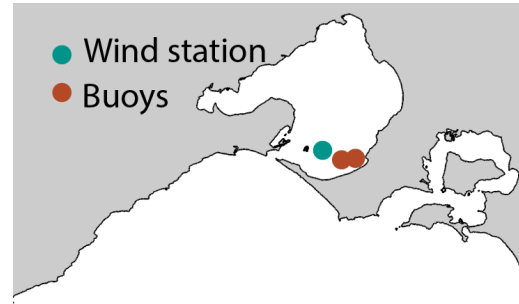


Figure 4.6: Locations of measured data used to validate ST6 physics and wind stations near the shipping channel in Port Phillip Bay.

Figure 4.7 shows validations of ST6 in WWM against field data and ST4. The top panel compares the wind forcing CFSR with field wind data at the South Channel, matching the timing of simulations. In b and c panels, the modelled and measured SWHs are compared, together with wind and wave mean directions for both ST4 and ST6 physics. It can be seen that ST6 performs better than ST4 at high wind speed. Evidence of extreme events can be interpreted the wind speed plot as the model wind forcing also matched with field wind data. For example, at the peak point, when the wind speed reaches about 80 km/h (22 m/s at about 12:00 noon 24 August 2003), measured wave height at both stations (Rosebud and Becon#15) reaches about 1.2m. Waves at buoys are large due to northerly wind, which can blow over a long distance from the north. The forcing wind speed and direction are matched with field wind data. For this extreme event, ST4 (black dot) underestimates the measured SWH about 0.4m. It is noted that the buoy data used here were drawn from near the shipping channel; wave height may be affected by vessel movements. However, during extreme events, it would be expected that no vessels would be operating.

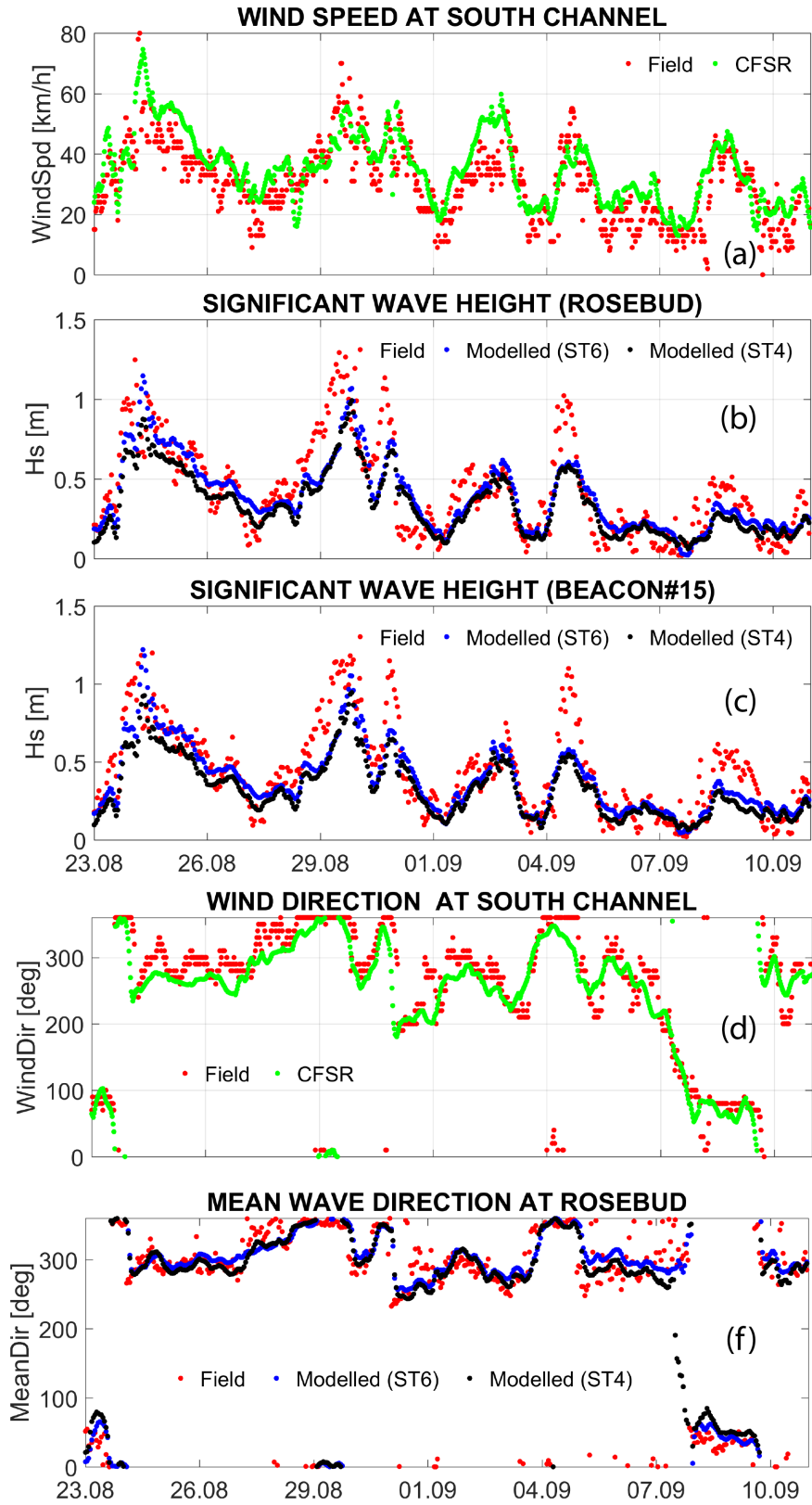


Figure 4.7: Comparison of (a) wind forcing from CFSR with wind data measured at the South Channel station inside PPB, SWH from WWMII with ST6 and field data at Rosebud (b) and Beacon 15 (c), and wind direction between CFSR and field data (d) and wave mean direction from WWM and field data (f).

4.5 Conclusions

This research study implemented the observation-based physical package ST6 in the unstructured grid spectral wave model - WWMIII. This was an important task, aimed to include physics in spectral wave modelling. Physical features in ST6 have significantly reduced the number of turning knobs to only six parameters: n for the wind input (Eq. 4.25), c_1, c_2, m_1, m_2 for wave dissipation (Eq. 4.31 and 4.32) and B_{sw} for swell attenuation (Eq. 4.33). Further, this implementation will also serve as an important resource for practical applications and public use since this updated version has been publicly available after this study.

Following the implementation, the updated WWMIII version was comprehensively tested and validated: several test cases, ranging from deep-water waves without swells (Lake Michigan), through large-scale simulations (global) with the presence of swells, to wind-seas in shallow water (PPB). CFSR wind was used to force the WWM testing, using two different physical packages: ST4 and ST6. Two months were selected for calibration and validation. Modelled SWHs were then compared with available buoy data. Model results revealed that ST6 performs slightly better than ST4 for high wind speeds. At the global scale, simulations were conducted for a one-year hindcast of 2011. Modelled results were then validated through deep-water buoy measurements around US coasts and with satellite altimeter data from the five missions available for this year. A further step was also conducted to test ST6 for shallow water waves at PPB, where waves are generated by local winds only. For this domain, ST6 again outperforms ST4, especially at high wind speeds: ST4 could underestimate SWH inside PPB by as much as 0.4m.

This upgrade of physics in WWMIII was then integrated with the high-resolution unstructured grid ocean model SCHISM, again after thorough calibration and validation procedures. After that, this coupling system was used to provide long-term hindcasts of hydrodynamics at PPB.

Chapter 5

Implementation of a coupled hydrodynamic modelling system in Port Phillip Bay

5.1 Introduction

Numerical models contain ideas about natural processes that are captured in mathematical formulations with approximations (e.g. *Cavaleri et al.*, 2018, 2007; *Roelvink and Reniers*, 2011). Advanced technology and human knowledge have increased the complexity and accuracy of numerical models; however, not all models are designed to serve the same purpose. Therefore, one of the initial steps is to select an appropriate framework suitable for a specific application. Further, numerical modelling cannot be used by itself; it is often used in conjunction with observational data, and in this regard, model calibration and validation are required. These processes involve adjusting numerical parameters to optimise model performance and verifying if the model can reasonably capture reality with selected model configurations. A general guide for these processes is described in *Williams and Esteves* (2017).

This chapter presents procedures to develop a coupled hydrodynamic modelling system for PPB. The next section describes the numerical modelling framework used in this study. After that, the model setup including bathymetry data processing, mesh generation and model forcing are also described, prior to presenting model calibration and validation.

5.2 Numerical modelling framework

The aim of using numerical models in this study is to replicate hydrodynamic conditions in a complex tidal inlet- lagoon system with irregular bathymetry where stochastic actions of waves and tidal currents play a pivotal role in various processes. This system requires the utilisation of a numerical modelling framework that involves coupling a circulation model with a wave model. This coupled system should provide a high resolution to capture the complexity of coastal shorelines and bathymetry, while still achieving computational efficiency. To this end, we selected the unstructured grid modelling system SCHISM (Semi-implicit Cross-scale Hydroscience Integrated System) based on *Zhang et al.* (2016) - a

upgraded version of the original work done by *Zhang and Baptista* (2008). Details of this modelling framework can be found at (<http://ccrm.vims.edu/schismweb/>). The major feature of SCHISM is to solve the advection in the momentum terms using semi-implicit schemes, resulting in numerical stability and reducing constraints in time steps (e.g. see *Guérin et al.*, 2018). The governing equations of SCHISM include:

$$\text{Momentum: } \frac{dU}{dt} - \frac{\partial}{\partial z}(k_1 \frac{\partial U}{\partial z}) + g\nabla\eta - S = 0 \quad (5.1)$$

$$\text{Continuity: } \nabla U + \frac{\partial w}{\partial z} = 0 \quad \text{and} \quad \frac{\partial \eta}{\partial t} - \nabla \int_{-H}^{\eta} U dz = 0 \quad (5.2)$$

$$\text{Transport: } \frac{\partial c}{\partial t} + \nabla(U.c) - \frac{\partial}{\partial z}(k_2 \frac{\partial c}{\partial z}) - K_h = 0 \quad (5.3)$$

where dU/dt is the time derivative of horizontal velocity U with respect to time t , the free surface η represents the horizontal x, y Cartesian coordinate system and the vertical coordinate z direction (up) with corresponding vertical velocity w , H is the water depth, k_1 and k_2 represent the vertical eddy and diffusivity respectively, g is gravitational acceleration and the S term in the momentum equation represents other forces (e.g. tidal, atmospheric forcing) and c in the transport equation represents the concentration of tracers (e.g. temperature, salinity, sediment transport). These equations are closed using the generic length-scale model proposed by *Umlauf and Burchard* (2003). At the first computational step, SCHISM solves the momentum and transport equations using the semi-implicit finite element method. The implicit scheme is applied to solve the elevation gradient, vertical viscosity and boundary layer and the divergence terms. Using this numerical scheme, stability constraints (e.g. large time step) can be obtained. After solving the horizontal velocity and elevation, the vertical velocity is computed using a finite volume method along each prism. SCHISM can directly be coupled with the spectral wave model WWMIII developed by *Roland* (2008) and *Roland et al.* (2012). This is a third-generation phase-averaged, unstructured grid wave model that allows the computation of wave spectra over realistic coastal and ocean regions. From spectral information, key wave parameters such as SWH, wave direction and peak period can be extracted from the model. WWMIII solves the wave action balance equation, which accounts for various physical processes such as wind energy and dissipation due to white-capping and bottom friction. The governing equation reads as follows:

$$\underbrace{\frac{\partial A}{\partial t}}_{\text{time space}} + \underbrace{\nabla_s[(c_g + U)A]}_{\text{geographical space}} + \underbrace{\nabla_k[c_k A]}_{\text{spectral space}} = \underbrace{\frac{S}{\omega}}_{\text{source/sinks}} \quad (5.4)$$

where A represents the action density as a function of intrinsic radian frequency ω and wave number k written as:

$$A = \frac{E(\omega, k)}{\omega} \quad (5.5)$$

In the linear theory, the temporal and spatial scales of waves obey the dispersion relationship:

$$\omega = gk \tanh kH \quad (5.6)$$

where g accounts for gravitational acceleration, H is water depth and c_g is wave group velocity. In the presence of a current, the current speed U can modify wave propagation. ∇ operators represent geographical divergence (horizontal) and spectral space (with subscript k). The S term on the right of Eq. 5.4 represents the energy of various source/sink terms such as wind forcing, white-capping and bottom dissipation. In WWMIII, the geographical space is solved with a residual distribution scheme, which incorporates the advantages of both finite elements and the finite volume methods (e.g. *Roland et al.*, 2012). The spectral space is integrated using the UQ method, a similar method as that in WW3 (*Tolman*, 1992). The source terms are solved in three fractional steps using the TVD Runge-Kutta method, which is a third-order scheme applied for dissipation terms (e.g. bottom friction, breaking), a dynamic scheme for nonlinear interaction (e.g. triads) and a semi-implicit scheme for deep water source terms. The coupling system can run in parallel modes using MPI (message passing interface), with identical sub-domains decomposed from the same unstructured grid with the user-defined time steps. This means that SCHISM-WWM can provide high accuracy as it does not require grid interpolation, while computational efficiency can be achieved by using different time steps and integration schemes. These key features make the coupled SCHISM-WWMII system highly efficient for simulating waves and currents in PPB.

5.3 Model setup

5.3.1 Bathymetry processing

Before generating a computational mesh, the bathymetry must be prepared. In this study, bathymetry data were gathered from several sources at different resolutions ranging from 0.5m to 500m. Within the southern area of PPB including the entrance, bathymetry data were mainly measured by via LADS surveys with a high resolution (up to 6m). These bathymetric measurements were taken during summer 2007/2008 (before channel deepening) and summer 2009/20010 (after channel deepening). Along shipping channels, the bathymetry after dredging was measured by MBES with a 0.5m resolution in a rectangular grid format. These high-resolution data used MGA94 (Zone 55) map projection. For other areas within PPB, bathymetry was taken from the Victorian coastal digital elevation model (VCDEM 2017) with a 500m resolution. For the remaining area (outside the bay) of the computational domain, bathymetry was provided by the BOM using the WGS84 projection.

The above bathymetry data must be combined into a single set that covers the whole domain. As the use of correct bathymetry is crucial in hydrodynamic modelling, it is important to check the chart datum of all bathymetric data. If there is any difference in chart datum, it must be converted into MSL or Australian height datum (AHD). Further, all bathymetry data must apply the same map projection; therefore, coordinate transformation is also required. It was not appropriate to convert the bathymetry in WGS84 into MGA94 projections because the entire domain slightly overlaps two different MGA94 zones; therefore,

accuracy may not be preserved in such a transformation. In other words, converting bathymetry in MGA94 projections within the southern region of PPB into WGS84 projections is much easier because this region is small, allowing for high accuracy in the coordinate transformation.

5.3.2 Building computational mesh

Notes on spatial grid resolution:

Understanding the appropriate grid size is crucial for modelling success because spatial grid resolution is often linked to the resolution of input forcing data including bathymetry (e.g. *Cavaleri et al.*, 2018). A fine computational domain is not fully taken when using coarse forcing fields. Therefore, in regions of complex bathymetry, it is often necessary to refine grid size to correctly capture topographic features. Further, spatial resolution affects the accuracy level of the equations to be solved; for instance, a computational grid is needed to resolve high gradients in the wave field. Therefore, mesh generation is considered a very important process in numerical modelling.

Advantages of using an unstructured grid framework:

Because of complexity in coastal geometry and bathymetry, an unstructured grid framework is often considered the optimal solution. This grid option can provide flexibility in adjusting the mesh size; for example, a high-resolution grid is often focused in the region of interest or where the bathymetry is complex. In regions of less interest, or where the bathymetry features are not significantly variant, mesh size can be made coarser to reduce computational costs.

Define the computational domain:

Computational domain is the area of the specified modelled grid. Before building a computational grid or mesh, it is important to define the area which the computed grid should cover. This also allows the determination of the open boundary, which can influence the inflow mechanisms such as ocean waves and tides propagating onto the modelled domain. Previous hydrodynamic studies in PPB (e.g. *Cardno Lawson Treloar*, 2007b) have defined the open boundary limit with an extension of about 25-35km from the entrance of PPB. The main reason for this is that those studies used the water level at Lorne to force the open boundary in the circulation model and also often used offshore buoy data to represent the ocean waves at the open boundary. Indeed, the Lorne station is approximately 40km from the entrance of PPB. It is nontrivial to correctly locate the open boundary to coincide with the water level at Lorne because this requires an adjustment to the open boundary limit (e.g. moving forwards and backwards between 25 and 35km). Another drawback of using the Lorne water level is that the observations at that station are intermittent, implying it cannot be used to simulate long-term hindcasts. This study uses neither the water levels at Lorne nor buoy data for the open boundary forcing; instead, the boundary is extended up to 200km from the entrance of the bay, for several reasons. First, that extension is located at deep water (approximately 100m deep), and therefore can capture ocean tides and ocean waves. Second, for the purpose of long-term simulations, this extension conveniently allows the use of global ocean wave hindcasts to force the open boundary.

Mesh refinement:

The SCHISM modelling system requires a computational unstructured grid that contains a collection of triangles with unconstrained connectivity with flexibility of grid size. To resolve the complex features at PPH, the grid size should be refined and the mesh quality checked. Grid refinement ensures the computational grid can represent morphological features in nature such as irregular shorelines and complex bathymetry. At the entrance of the bay, there is a narrow and deep canyon, which is 150m wide and up to 90m in depth. It is necessary to have at least two grid cells passing over the cross-section of the canyon, implying the grid resolution must be below 50m. Indeed, this is just an initial guess for the grid size in the region of interest, because the quality of the mesh must be checked afterwards.

Checking the mesh quality:

There are several methods used for a mesh check. The simplest is visual checking based on the user's experience; for example, mesh quality is poor when the angle making a triangle is too small or too large. Another method of checking mesh quality is via software (such as SMS or Aquaveo). However, the most commonly used method is to apply the created mesh to run a short simulation to produce a quick visualisation. This approach often provides the best outcome because a good mesh can provide reasonable patterns (e.g. current magnitude, direction) that cannot be seen when checking the mesh using a visual method or software.

Courant-Friedrichs-Lewy (CFL) condition:

Numerical modelling is constrained by the CFL criterion. In ocean models, the CFL constraint is determined by:

$$\text{CFL} = \frac{(|U| + \sqrt{gH})\Delta T}{\Delta X} \quad (5.7)$$

where u is the current velocity, $\sqrt{gH}$ is wave celerity at water depth H , g is gravitational force and ΔT and ΔX are the time step and the grid size respectively. Explicit models require a CFL value of below 1 to achieve model stability, but the CFL range in SCHISM can differ from this. Figure 5.2 illustrates the time step for a given grid size taken from the SCHISM Manual <http://ccrm.vims.edu/schismweb/>. The left side of the curve represents the change in diffusion error, which may increase quickly when CFL is small. If the CFL is larger than a certain point, the model may produce a large truncation error. It has been shown in Ye *et al.* (2018) that SCHISM can accept large time steps (e.g. 240 second). Figure 5.1 shows a canyon only 150 wide and up to 90m at the entrance of PPB, where tidal currents can

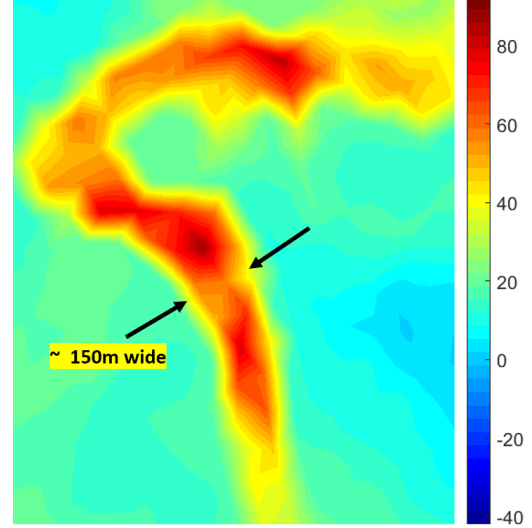


Figure 5.1: The presence of a deep, narrow canyon at the Heads of Port Phillip Bay creates some challenges in mesh generations.

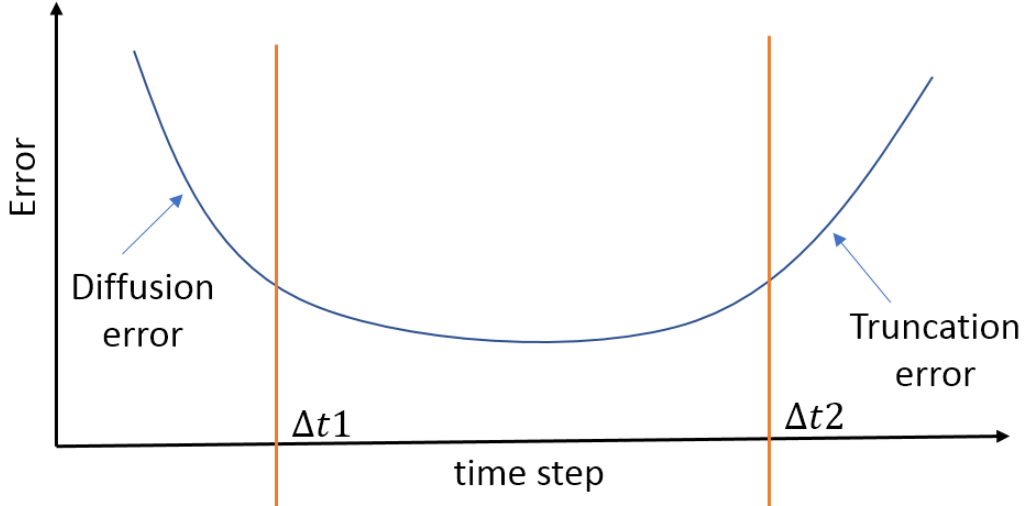


Figure 5.2: Error curve of SCHISM model for time step with a given grid size. The error changes in a nonlinear manner. Diffusion error is dominant when time step $< \Delta t_1$ (left) while truncation error significantly increases when time step $> \Delta t_2$ (right).

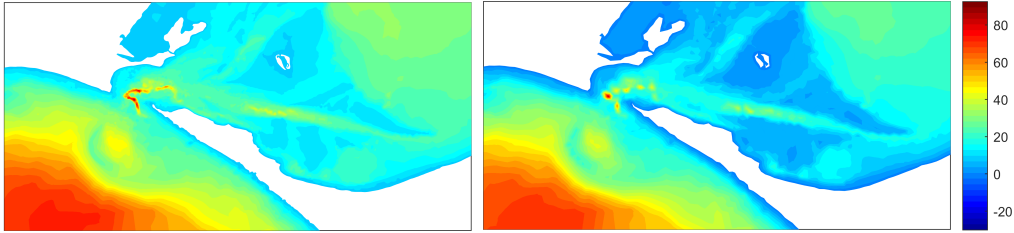


Figure 5.3: A comparison of bathymetry between two grid options for the deep canyon and the Heads. The finer mesh (left) well represents the bathymetry features (e.g. deep canyon) while the coarse mesh (right) does not.

reach 3.5 m/s creating two challenges for mesh generation. First, the grid must reasonably capture the water depth in the deep and narrow canyon, which requires a small grid size; Second, the CFL condition must be considered in the case of strong current speeds (e.g. > 3 m/s) because the CFL formulation in Eq. 5.7 is proportional to wave celerity and current velocity. The first challenge is illustrated in Figure 5.3, which shows the representation of the bathymetry captured in two mesh options. On the left, the grid resolution for the canyon is 150m, while on the right the grid size is 400m. The larger grid size does not properly capture the bathymetry of the canyon. The second challenge is illustrated in Figure 5.4, where the CFL numbers can quickly be modified in cases of strong currents. Therefore, it is important to generate a mesh to meet the CFL numbers in strong currents.

Bathymetry interpolation:

The computational mesh requires a bathymetry be attached to it. Bathymetry interpolation is the final step of a mesh generation. Once mesh quality is visually checked, the bathymetry is then interpolated into the mesh. Care should be paid regarding the choice of interpolating method. Normally, the linear interpolation is used; however, it is useful to compare different methods by visualising the interpolated bathymetry (from the mesh) to compare this with the original one (before interpolation). The interpolated bathymetry with a reasonable method

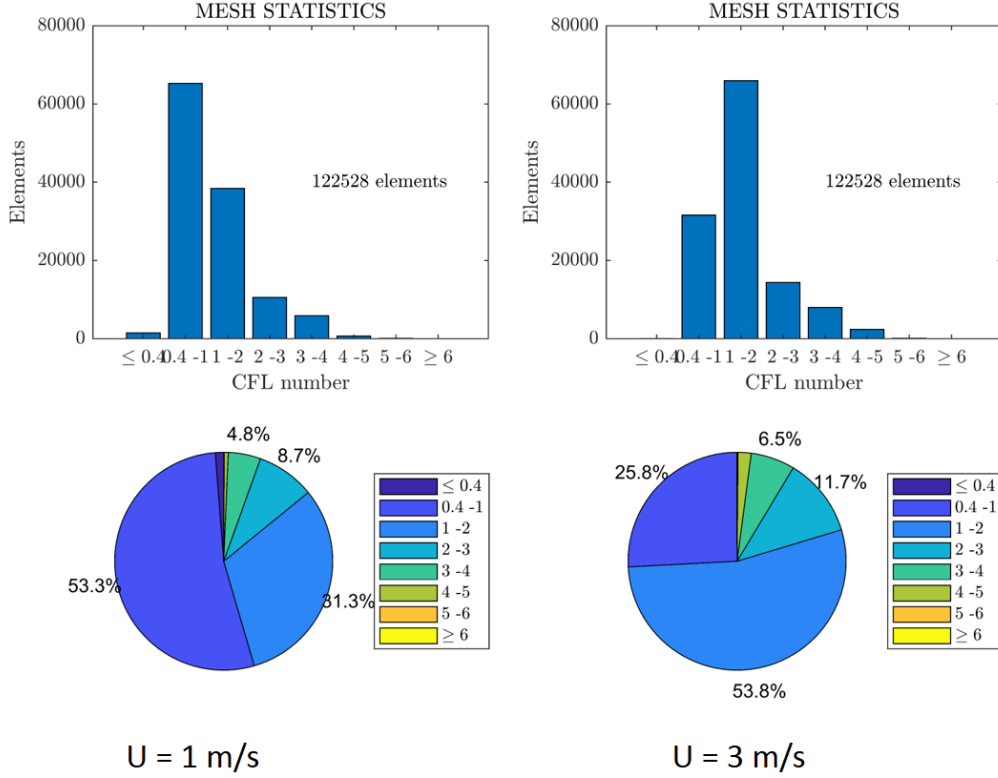


Figure 5.4: A comparison of CFL numbers computed from the same mesh for current speeds of 1 m/s and 3 m/s. With strong currents, the CFL number in many elements of the mesh increases.

should provide similar features to the original one.

Selected mesh:

The selected mesh comprises 54,432 nodes with 103,543 elements, with resolution ranging from about 20m to 3km offshore. The finest mesh was devised for the region of interest at the entrance of PPB where the bathymetry is fairly complex.

5.3.3 Model Forcing

Coastal models are typically applied for a particular region in contact with external regions via the open boundary. Therefore, the open boundary characteristics on the open boundary must be specified. The following sections describe the types of forcing required to run SCHISM and WWMIII.

Elevations/tidal forcing

In coastal areas, tides and winds can generate water fluctuations and currents; these can be very strong in some regions such as PPH. For coastal hydrodynamic modelling, the computational domain is often limited at the open sea, which is referred to as the open boundary. Along such boundary, the water level can also fluctuate across both time and space. Such variations can be due to astronomical forcing (e.g. ocean tides). In hydrodynamic modelling, water elevations at the open boundary must be specified. In practice, at the very least, water elevations (or tidal velocities) are used to force the open boundary of the circulation model to capture inflow from outside of the computational domain. Additionally, non-tidal signals may also be required. These non-tidal signals can be generated by

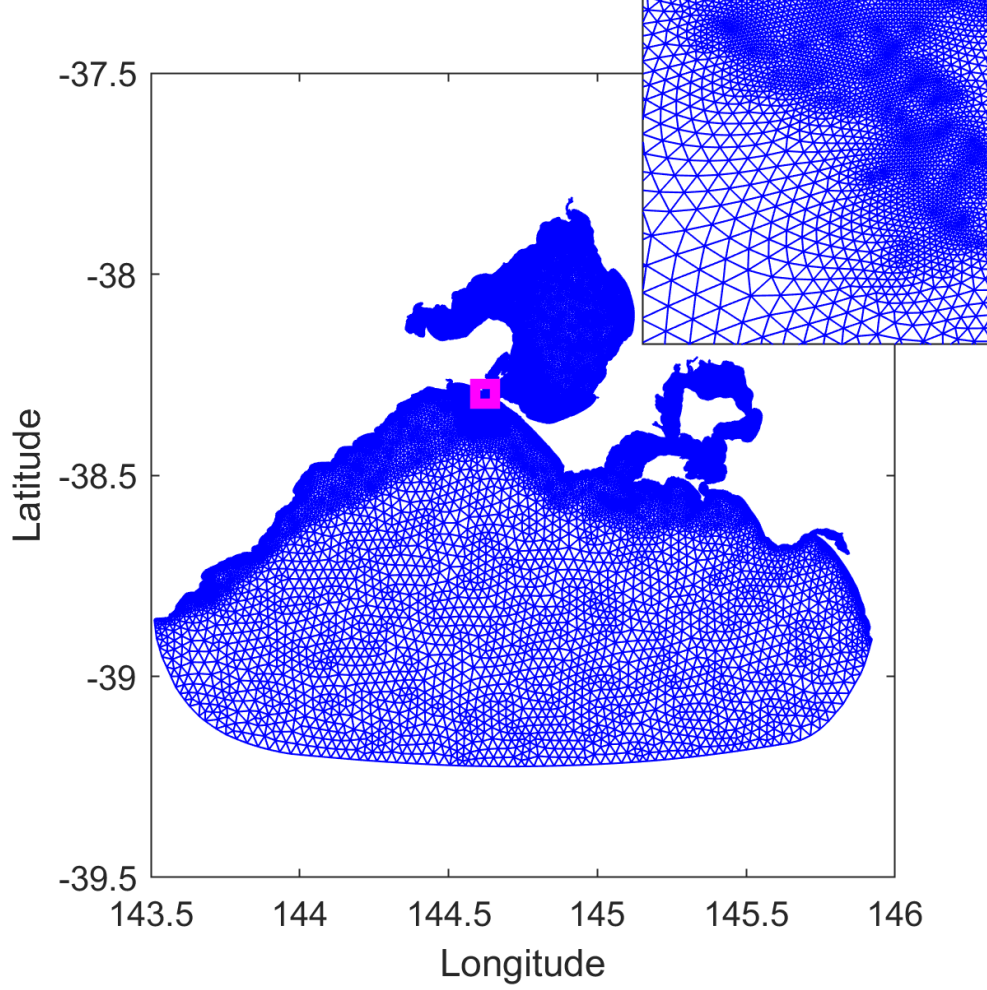


Figure 5.5: Visualisation of the final mesh used in this study.

meteorological forces (e.g. surges) generated from outside the computational domain.

To specify water variations at the open boundary, this study uses the latest development of the global finite element solution tide model (FES 2014) introduced by AVISO (*Carrere et al.*, 2015), which includes improvements in the grid resolutions, especially in coastal regions and the continental shelf with new global bathymetry. This database was assimilated with long-term altimeter data from Envisat, Topex/Poseidon, Jason-1, Jason-2, TPN-J1N, ERS-1, ERS-2 and field measurements at various tidal gauges. The distributed FES 2014 database provides a high resolution of $1/16^\circ$, which is 50% finer than the FES 2012 model.

The circulation model SCHISM was configured in depth-averaged (2DH) barotropic modes using eight tidal constituents (Q1, O1, P1, K1, N2, M2, S2, K2) based on FES 2014 database. These data were produced by Noveltis, Legos and CLS and distributed by AVISO+, with support from Cnes (<https://www.aviso.altimetry.fr/>). The non-tidal signals are driven by HYCOM models (<https://www.hycom.org/>).

Atmospheric forcing

SCHISM allows the inclusion of several atmospheric parameters: wind, air pressure, temperature, specific humidity and fluxes of short and long waves. These forcing terms are in netcdf formats. Within PPB, except for the wind field, none of these parameters are available. Though available, measured wind data within the study domain were not continuously recorded. Therefore, for the purpose of long-term hindcasts, data from global atmospheric models were instead used to force numerical models; these can be freely downloaded from ECMWF or the National Centres for Environment Prediction (NCEP) (Saha *et al.*, 2010, 2014). This study uses hourly atmospheric forcing data 0.25°-resolution from the CFSR database provided by NCEP (Saha *et al.*, 2010, 2014). These data were produced from the coupled atmospheric-ocean-land-surface-sea ice modelling system with the aid of assimilation techniques. Figure 5.6 shows q-q plots of field wind data at the South Channel station in PPB with CFSR and ERA5 wind (Hoffmann *et al.*, 2019) produced by the most advanced wind models. CFSR wind agreed better with the field data than ERA5.

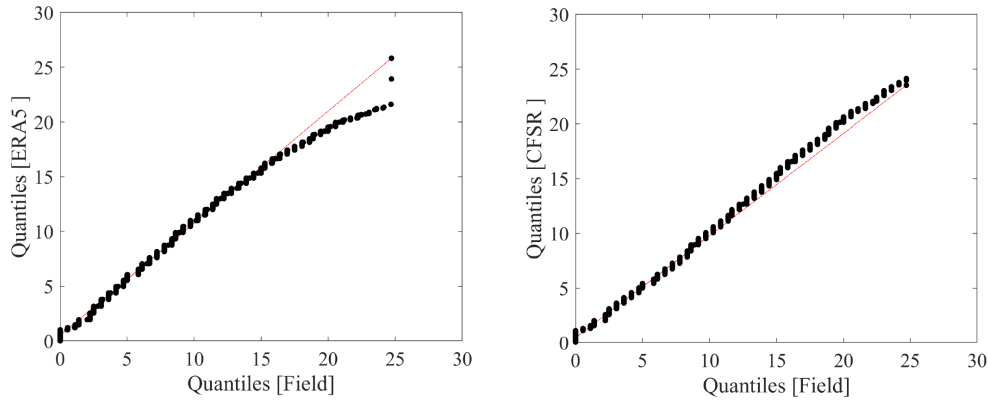


Figure 5.6: A comparison between CFSR/Field and ERA5/Field Q-Q plots at South Channel station. The CFSR wind source provides a reasonable agreement with the field data, while ERA5 underestimates high wind speeds at this location.

Wave forcing for the boundary

In the wave model, ocean wave conditions are often forced at the open boundary unless the computational domain is very large or located in a basin. In semi-enclosed sea regions such as PPB, wave information at the open boundary is required to allow for waves entering into the computational domain towards the entrance of PPB. This study uses the WW3 hindcasts data provided by the French Research Institute for Exploitation of the Sea IFREMER (Rascle and Ardhuin, 2013) at `ftp://ftp.ifremer.fr/ifremer/ww3/HINDCAST`. This data source can provide up to 30 years of data with 0.5-degree resolution and 3-hourly intervals in parametric formats, which can directly be read by WWMIII.

5.4 Model Calibration and validation

5.4.1 Procedures

The purpose of model calibration is to adjust model configurations to optimise model performance. Model accuracy can be determined by comparing model parameters with available field data. In ocean modelling, the main calibration parameter is the bottom friction or drag coefficient. Because the presence of the field data to determine the bed form are scarce, for simplicity, a constant value is often applied for the whole domain(e.g. *Williams and Esteves, 2017*). However, adjustments of the drag coefficient are essential to optimise model performance.

In this study, two bottom friction formulations were tested for SCHISM. Following a few short simulations to examine the stability of these formulas, the quadratic drag formulation was chosen as other formulations (e.g. Manning) produced noise. Once the drag formulation was selected, the coefficient was adjusted to obtain reasonable agreement between the model solutions and the available field data. During the calibration, different time steps were tested. To this end, a time step of 120 seconds was chosen for SCHISM, and 360 seconds for WWMIII as in the final stage. The final calibration of SCHISM was selected with a quadratic formulation with the same drag coefficient of 0.0028 for every grid point in the domain. Once the coefficient was chosen in the calibration step, the calibrated configuration was used to validate SCHISM by running the model over a different time period (e.g. another month).

5.4.2 Skill assessments

Model performance is evaluated using skill metrics. Data from each station were extracted in timeseries. A summary of statistics is provided in Table 5.1.

Table 5.1: A statistical summary for model calibration and validation.

LOC	VAR	June and July-2007				Feb and Mar-2012			
		COR	R^2	RMS	BIA	COR	R^2	RMS	BIA
ORB	EL	0.97	0.9	0.17	0.03	0.98	0.94	0.13	0.01
	VEL	0.94	0.87	0.19	0.01	0.92	0.85	0.21	(0.01)
	SWH	0.91	0.8	0.33	0.01	0.87	0.72	0.38	(0.04)
RBCL	EL	0.97	0.9	0.17	0.03	0.98	0.94	0.13	0.01
	VEL	0.96	0.91	0.21	(0.08)	0.97	0.87	0.26	(0.11)
	SWH	0.92	0.82	0.32	0.03	0.89	0.77	0.34	(0.04)
NBCL	EL	-	-	-	-	0.98	0.94	0.1	0.01
	VEL	-	-	-	-	0.96	0.92	0.19	(0.05)
	SWH	-	-	-	-	0.87	0.59	0.34	0.06
LOC	VAR	2007 (full year)				2012 (full year)			
		COR	R^2	RMS	BIA	COR	R^2	RMS	BIA
OFFSHORE	SWH	0.91	0.82	0.28	(0.08)	0.94	0.89	0.26	(0.07)

EL = water elevation; VEL = current velocity; SWH = significant wave height

LOC = location; VAR = variable; RMSE = root mean squared error;

COR = linear correlation; R^2 = root-squared; BIA = bias

'-' = data were not available; () = negative value

Bias

$$BIAS = \sqrt{\frac{\sum(X_f - \bar{X}_f)^2}{N}} - \sqrt{\frac{\sum(X_m - \bar{X}_m)^2}{N}} \quad (5.8)$$

Correlation

$$R = \frac{\sum (X_f - \bar{X}_f)(X_m - \bar{X}_m)}{\sqrt{\sum (X_f - \bar{X}_f)^2 \sum (X_m - \bar{X}_m)^2}} \quad (5.9)$$

Coefficient of Determination

$$R^2 = 1 - \frac{\sum (X_f - X_m)^2}{\sum (X_f - \bar{X}_f)^2} \quad (5.10)$$

Root mean squared error

$$RMSE = \sqrt{\frac{\sum (X_f - X_m)^2}{N}} \quad (5.11)$$

where X_m is modelled data and X_f presents field measurements.

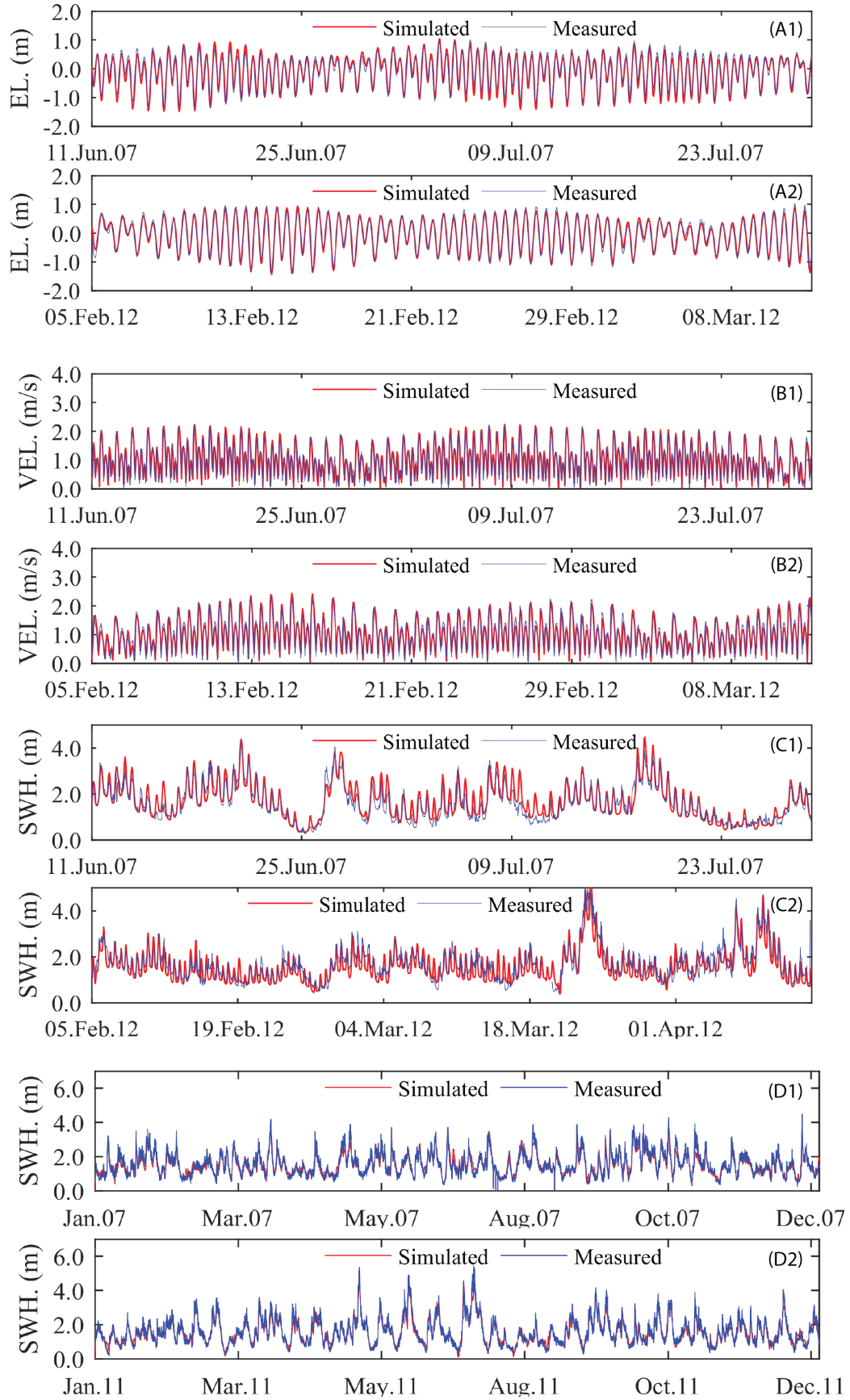


Figure 5.7: Comparisons between measured and field data obtained during calibration and validation. For SCHISM at ORB, plots for water level and velocity are shown in (A1, A2) and (B1, B2). For WWMIII, the SWH at offshore buoy location is shown in (D1, D2). The SWH for the coupling model at ORB is shown in (C1, C2).

Figure 5.7 shows time series plots that compare the measured and field data at OBRCL obtained during the calibration and validation of SCHISM for water level (A1, A2) and current speed (B1, B2). Similarly, the wave model was simulated alone without imposing a current. This step is to ensure ocean waves are well replicated by WWMIII. Therefore, field data at offshore buoy locations were used for calibrating and validating WWMIII, as the wave profile at this location is not affected by currents. Comparisons of modelled SWH and buoy data are presented in (D1, D2) of Figure 5.7. Once these steps were completed, SCHISM was fully coupled with WWMIII model to represent the wave profile at the entrance where waves interact with strong tidal currents. Comparisons of the modelled SWH and field data at ORBCL are shown in (C1, C2) of Figure 5.7. As indicated by the statistics, both SCHISM and WWMIII well replicate the available field data; therefore, this coupling system can confidently be used for long term simulations.

It is noted that the calibration presented above is for the depth-averaged (2D) SCHISM model. A further step with testing the 3D version of SCHISM was also conducted to explore model performance in more detail. For the 3D model, the first step was to build a vertical grid. Z- and S-coordinates are commonly used grid transformations in ocean/circulation models; however, these systems have their own issues. For instance, Z-coordinates often have artificial staircases, which can affect the bathymetry and create an ‘unreal’ drag at the bottom if applied in shallow regions. To overcome this problem, a new coordinate system called LSC^2 (localized sigma coordinates with shaved cell) has recently been introduced by Zhang *et al.* (2015), which allows the setting of vertical coordinates locally. Particularly, each node in the grid (or mesh) has its own vertical coordinates using the concept of degenerate prisms. Therefore, this new vertical grid transformation was selected for PPB. To build the vertical grid, a 32-layer reference master grid was chosen. The thickness of the bottom layer, stretching constant and functions for the bottom and surface were also specified. Figure 5.8 shows a cross-transect from the vertical grid. It starts from the inner region of PPB crossing the entrance and goes further up to the offshore region. Three points along the transect are also plotted (marked a, b, c), which show that the designed vertical grid can well capture the bottom without staircases - an advantage of using LSC^2 in SCHISM. The final vertical grid has a maximum 35 layers at the deepest points, a minimum 5 layers in the shallow regions, and 13 layers on average. To force the SCHISM 3D model, the same tidal and wind data for the 2D model were used with non-tidal signals (e.g. elevation, U-V velocities...) drawn from HYCOM. The calibration and validation results are shown in Figures 5.9 and 5.10. Overall, the 3D SCHISM performs well as regards surface currents and currents in the vertical profile with a high correlation and R^2 achieved.

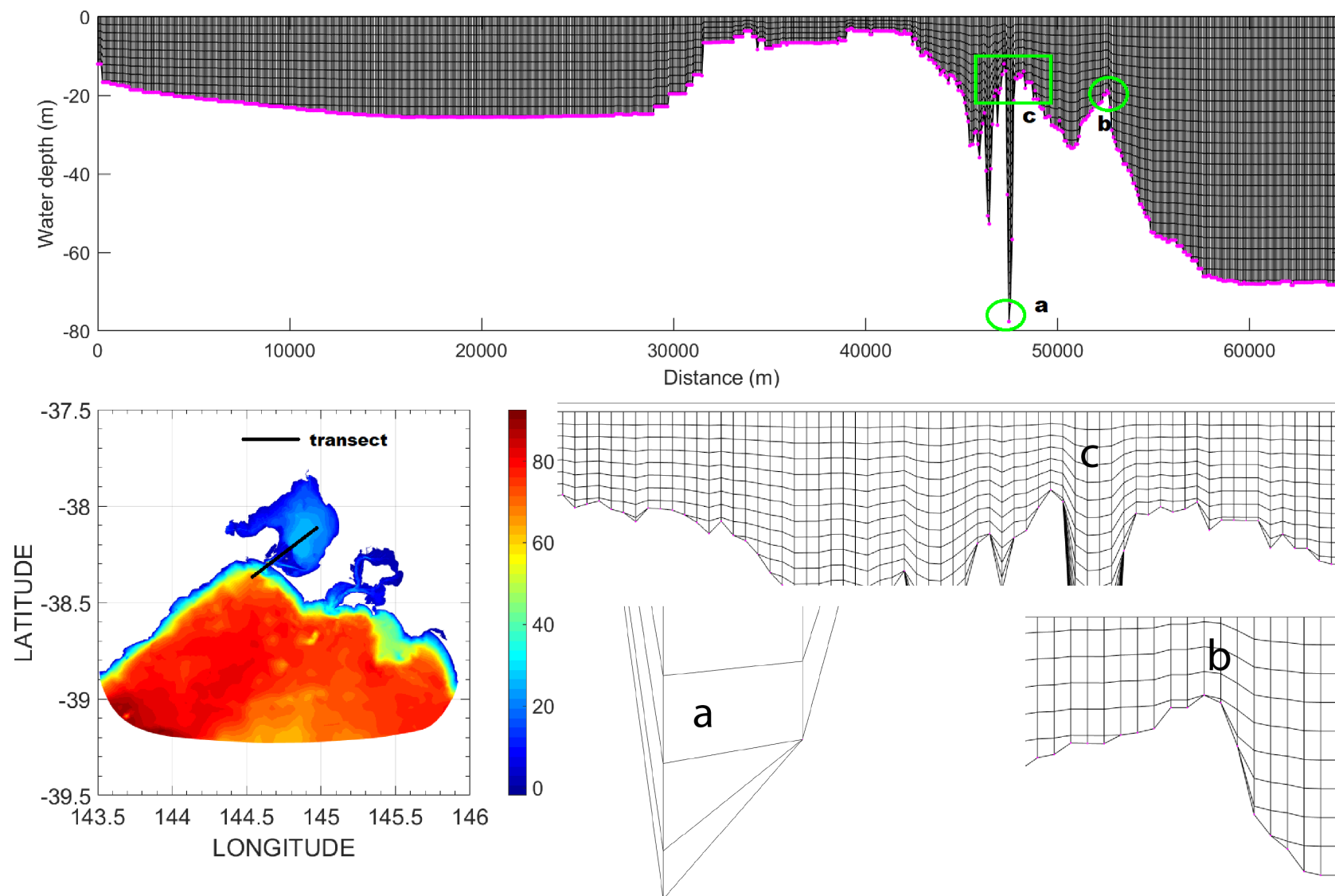


Figure 5.8: An example of a vertical grid at the transect crossing the entrance of PPB where the bathymetry is very steep. In the offshore bar region (shallow bar in the offshore), there are no ‘staircases’. The vertical grid follows the water depth well.

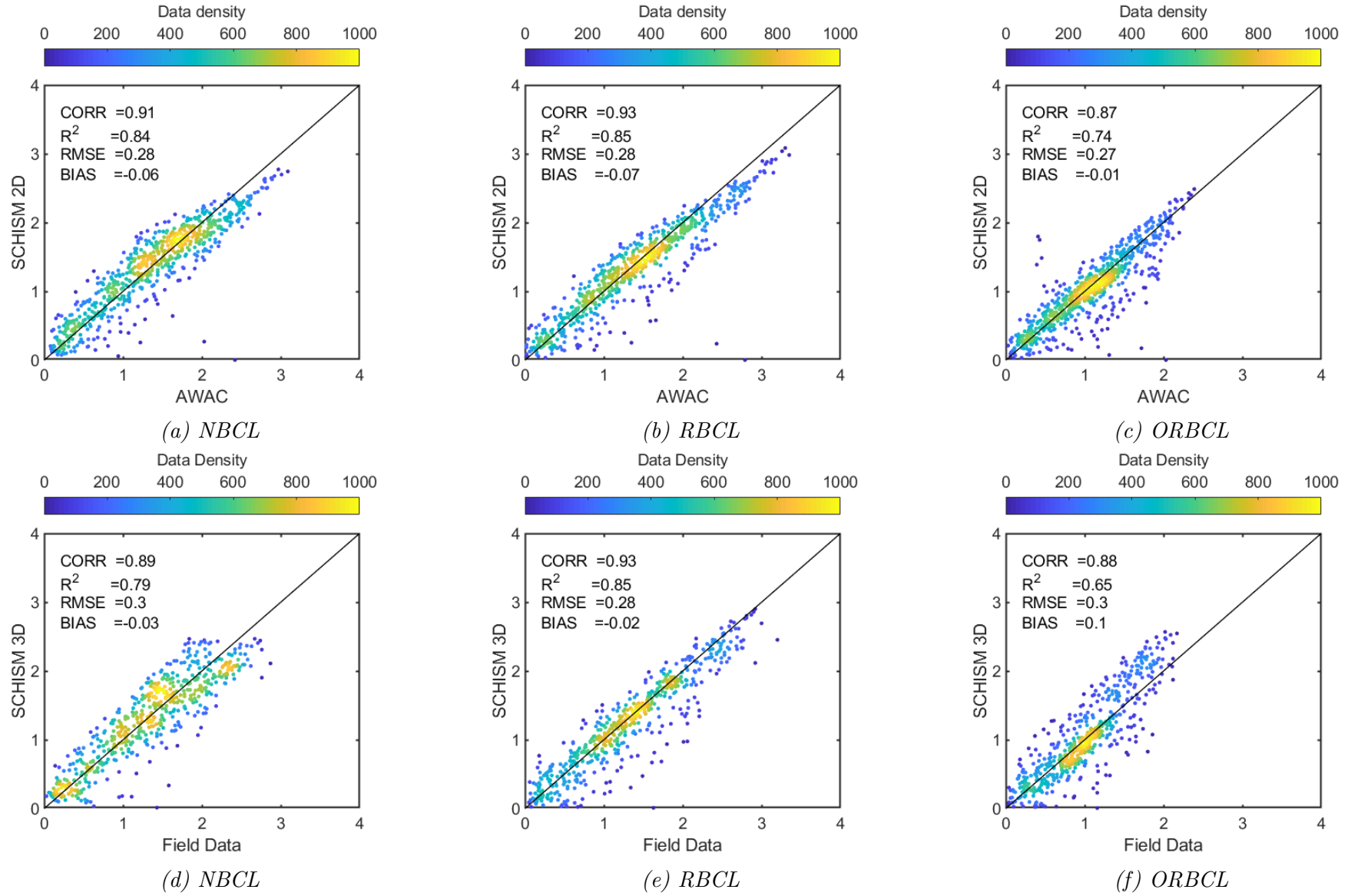


Figure 5.9: Comparisons between field current (AWAC) and 2D SCHISM and 3D SCHISM (surface current).

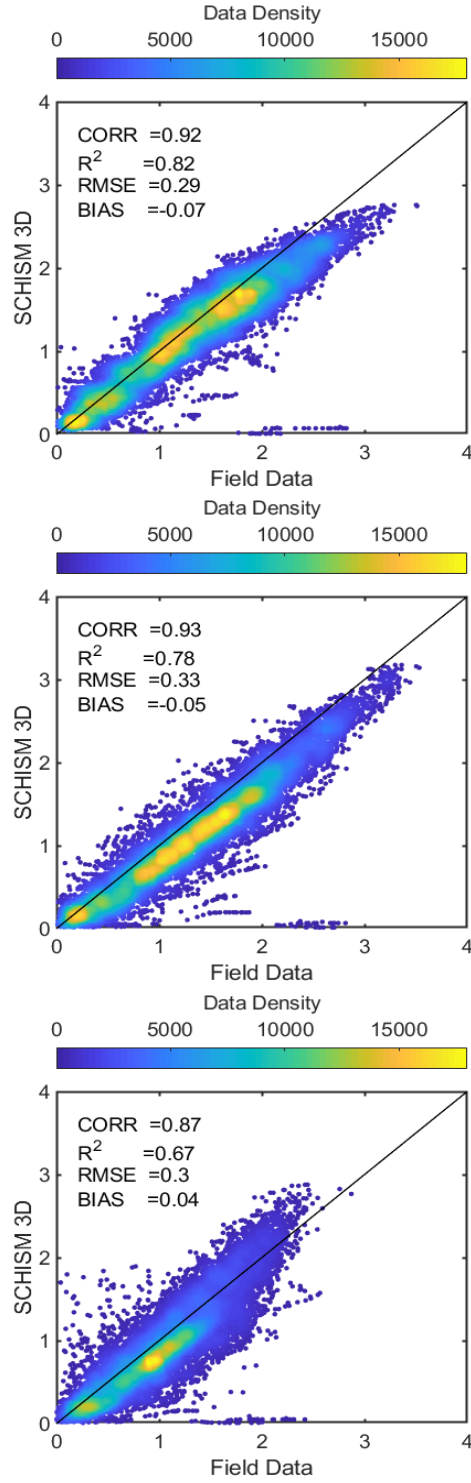


Figure 5.10: Comparisons between field currents (AWAC) and 3D SCHISM for all layers in the water columns (18) at three locations: NBCL (top panel), RBCL (middle panel) and ORBCL (bottom panel). Statistics shown in these plots are an average over all layers.

5.5 Conclusion

This chapter presents procedures to implement a coupled hydrodynamic modelling system based on an unstructured grid framework: the circulation model SCHISM coupled with the updated version of spectral wave model WWMIII (see Chapter 4). Each of these modules was carefully tested and validated against field data available in PPB.

In this study, the computational grid is extended up to a water depth of 100m to ensure ocean waves and tides can be well captured in the modelled domain. The models were forced with atmospheric forcing data from CFSR, which shows a better agreement with field wind data compared with ERA5 at the South Channel station. This study also used the finest resolution of atmospheric forcing fields available from CFSR: 0.2-0.25 degree (spatial resolution) and hourly data. These high resolutions are a few times finer than previous studies (e.g. *McInnes et al.*, 2009). During model setup, it was found that the quality of the computational mesh is crucial for the model results. The complexity of bathymetry and hydrodynamic patterns at the entrance of PPB created challenges in mesh generation; in particular, the presence of a narrow and deep canyon at the entrance of PPB requires mesh refinements to correctly represent the bathymetry in this region. Further, a strong current at the entrance required additional tests to ensure that the mesh can survive in strong currents (e.g. within the CFL limits). It was also found that quadratic drag bottom friction provides better results than other options (e.g. Manning). Results from model calibration and validation show good agreement between the model and field data. It is thus concluded that the configured numerical models from this study can be confidently used to replicate hydrodynamic conditions in PPB. Due to computational expensive, only 2D model configurations were used for long-term hindcasts.

It is concluded that, to the best of our knowledge, to date, this is the most suitable and comprehensive numerical modelling system applied to simulate PPB hydrodynamics. Particularly, the model update with ST6 physics would provide reliable results for estimates of extreme waves. Further, a coupled modelling system would better represent wave-current interaction, wave-setup at PPB compared to previous studies done without coupling waves and currents.

Chapter 6

Port Phillip Bay hydrodynamic climate

6.1 Introduction

Information on the hydrodynamics of PPB is important in many regards, including coastal engineering design, management and port operations. To date, there has been no study based on long-term hindcasts that can provide sufficient data to assess the climatology of this region. Therefore, this study is dedicated to broadening knowledge of hydrodynamic climatology in PPB. Relative to previous studies, the new contributions this present study makes are at least twofold. First, observation-based physics were incorporated into an unstructured grid numerical modelling system, to provide reliable physics to replicate the surface waves in PPB. This important implementation provides insights into the contribution of the wave field to extreme sea levels – a topic that had not been explored in detail. Second, the use of a validated coupled model with a high-resolution unstructured grid with advanced wave physics allowed us to simultaneously investigate temporal and spatial changes in both waves and sea levels based on long-term hindcast simulations. In particular, for extreme sea levels, the availability of a long track record could be added to the data analysis, allowing us to compare and contrast estimated return levels based on hindcasts and historical data, and rendering the results presented in this study more meaningful and reliable.

After presenting the methodology, the next sections discuss the results for three important aspects of hydrodynamics in PPB - sea levels, waves and currents - in terms of seasonal variations and extremes. Several case scenarios are provided to examine the contribution of waves to extremes and circulation. The return values of sea levels (storm tides) and SWHs are also estimated. These return levels are also compared with results from the present study.

6.2 Methodology

For sea levels, the focus is on seasonal variations and extremes, with a detailed analysis of the variability across the four seasons (see Table 6.1 for the meteorological seasons in Australia) based on various thresholds and taking into account impacts of wave setup on extreme sea levels, which has not been investigated before. As the occurrence of storms in PPB does not depend on

seasonality, the contribution of wave setup in this study is examined for several test cases taken from the most extreme events covering all seasons. Both return periods of sea levels (storm tides) and surges are investigated. For the wave field, this study examines the seasonal variability of the three main wave parameters: SWH, mean wave direction and peak period. The return periods are estimated for SWH only. For currents, the focus is on wind-driven circulation and extreme currents.

Estimates of return levels were taken for storm tides and SWHs. From 27 years of hindcast data, this study estimates different return levels up to 100-year return periods based on the POT approach (e.g. *Coles*, 2001). This is a widely used method to estimate the return period for waves (e.g. *Meucci et al.*, 2019; *Takbashi et al.*, 2019; *Vinoth and Young*, 2011; *Young and Ribal*, 2019; *Young et al.*, 2011) and extreme sea levels (e.g. *McInnes et al.*, 2009). Essentially, this method estimates the return level as follows:

Season	Months
Summer	From Dec to Feb
Autumn	From Mar to May
Winter	From Jun to Aug
Spring	From Sep to Nov

Table 6.1: Seasons in Australia

$$H(x) = 1 - \left\{ 1 + \gamma \left(\frac{x - T}{\beta} \right)^{-1/\gamma} \right\} \quad (6.1)$$

Eq. (6.1) requires the determination of three parameters: location T or threshold, the scale (β) and shape (γ) parameters. There is no clear guidance or automatic approach to select the thresholds for non-stationary estimations; it is often selected based on some high percentile level, such as 90th or 95th percentile. In this study, return levels were estimated for several thresholds, which allowed us to explore how threshold adjustments can affect the estimated results. The two unknown parameters in the generalised pareto distribution (GPD) in Eq. 6.1 can be estimated using one of the common fitting approaches, such as probability weighted moment (PWM), least squared (LS), method of moments (MOM) or maximum likelihood (ML). An intensive review of those methods can be found in *Bermudez and Kotz* (2010a,b). In the present study, the built-in functions *gpfite*, *gpcdf* in MATLAB® are used to estimate the two-unknown parameters based on the ML method, and compute the cumulative distribution. The purpose of using *gpfite* is to fit the data to the GPD function while *gpcdf* takes the unknown parameters estimated from *gpfite* to provide the probability based on the defined return period.

6.3 Results

This section presents the results for three key aspects of hydrodynamics - sea levels, surface waves and currents - based on 27 years of hindcast simulations. Where relevant, field data and test cases are used to assist in interpreting results.

6.3.1 Sea levels

This study combines field sea level observations available from tide gauges and 27-year hindcast data to provide insights into sea levels in PPB. To this end, both seasonal variability and extremes are investigated in tandem with the contribution of waves to sea levels.

Seasonal variations

Figure 6.1 presents spatial variations in sea levels across different seasons for means (MSL) and 90th and 99th from 27 years of hindcast simulations 1990-2016. For the means, a clear picture emerges - the MSL inside PPB is always higher than that outside, regardless of seasonality. For instance, in summer, the MSL inside PPB is higher than offshore ranging from 3cm in the southern portion of PPB to 5cm in the north. This reveals the respective influence of temperature on the MSL in PPB and the Bass Strait. One example for the temperature pattern was previously shown in Chapter 3 (Figure 3.18), where the temperature inside PPH is much higher than those offshore. The dependency of sea level fluctuations on temperature has been reported by many authors. Although sea level variations are caused by various physical processes in the earth's systems, *Chen et al.* (2000, 2005); *Marcos and Woodworth* (2017); *Menendez and Woodworth* (2010) all suggested that the two main drivers of affecting seasonal variability in sea levels are salinity and temperature. Because the ocean water is warmer in summer months than in winter, the thermal expansion also differs from one season to another. As a result, the MSL in the summer is often higher than in the winter (e.g. *Cazenave et al.*, 2000). An interesting pattern in MSL is that in spring, this gets higher near the eastern coasts of PPB than in the west while in summer, the highest values occur in the north. This suggests that the wind climate may also affect MSL in PPB as wind blows from Bass Strait towards the north in summers, resulting in wind-wave setup (e.g. *Guérin et al.*, 2018). This can also occur on the eastern coastlines in spring due to the westerly wind.

For high percentiles of sea levels, it is clear that the water level inside PPB is much lower than that outside, in line with the narrow entrance where the ocean tides are distorted and filtered out (e.g. see *DiLorenzo*, 1988). The seasonal variability of the 90th, 99th percentile water levels is unclear because these high percentiles do not reflect the most extreme events in PPB. For example, the 90th percentile represents the threshold at which 90% of data falls below this level. On average, the 90th percentile water level can occur daily in PPB, as this threshold reflects levels of over 11,800 events (10% x 27 years x 365 days x 12 events) in 27 years of hindcasts. For 90th percentile, *Association of Bayside Municipalities* (2015) provided calculations based on observed yearly data at several stations around PPB. Table 6.2 compares the variability in the 90th percentile water levels based on *Association of Bayside Municipalities* (2015) with the present study. Please note that in terms of the lowest recorded 90th percentile sea levels, *Association of Bayside Municipalities* (2015) included two unusually low values 0.176m in Williamstown in 1967 and 0.26m in Geelong in 1971 - which are not included in the period of hindcast data used in this study. However, in both studies, the highest recorded 90th percentile sea levels were recorded in 2013 at least three stations, except for the year 2016, which was not included in *Association of Bayside Municipalities* (2015). A major pattern in the 90th percentile sea levels in this table is that Geelong is higher than Williamstown in term of values, while Queenscliff is lower than Hovell Pile for both the lowest and highest values. The higher value of the 90th percentile sea level at Geelong relative to Williamstown may be explained by the fact that this threshold level is linked to high-tide waters, which would occur daily in a flood tide. During this tidal cycle, PPB acts as a

Table 6.2: Variability of 90th percentile sea levels at several stations in PPB according to Association of Bayside Municipalities (2015) and based on the present research.

Station	Association of Bayside Municipalities (2015)				Present study			
	Min		Max		Min		Max	
	90 th	Year	90 th	Year	90 th	Year	90 th	Year
Queenscliff	0.222	1997	0.406	2013	0.300	1997	0.390	2016
Hovell Pile	0.265	1995	0.510	2013	0.300	1995	0.430	2013/16
Williamstown	0.176	1967	0.425	2013	0.360	1997	0.424	2013/16
Geelong	0.260	1971	0.460	1989	0.35	1997	0.449	2013/16

storage water system because the tidal stream propagates from the ocean into PPB towards Williamstown and Geelong. However, Geelong is narrow and relatively far from the entrance; thus, the water level could be raised for longer at the reversing time between the flood and ebb tides. The explanation for the lower value of the 90th percentile water level at Queenscliff than Hovell Pile is not clear. Association of Bayside Municipalities (2015) speculated that this may be due to the difference in tidal ranges in these two locations. At the 99th percentile, this level reflects 'one-week' largest event on average. Similar to 90th percentile, the water elevation inside PPB is reduced ocean tides have passed the narrow entrance. The most notable feature is that, after passing the entrance, the water level of the nearly entire PPB is the same from Hovell Pile to Williamstown. This reflects the pattern of tidal range inside PPB, which is almost identical over a large portion of the Bay.

Return periods

To gain further insights into extreme water levels in PPB, different levels of up to 100-year return periods were estimated. These extremes are associated with storm tide levels which include tidal elevations and surges. Before mapping 100-year storm tides, this study estimated different return levels with various thresholds at several stations within PPB. The purpose of this was to compare and contrast results from this study with other available results. Four locations: Lonsdale (PPH), Hovell Pile, Williamstown and Geelong were selected for comparison as these have long sea level records. Previous studies have also used measured sea levels at these stations to estimate return values including *McInnes et al.* (2009) and *Association of Bayside Municipalities* (2015). Because the hindcast data run from 1990 to 2016, this study also re-estimated the measured data available during these 27 years for consistency.

Table 6.3 summarises estimated return levels based on two studies (McInnes09C, McInnes09D in table) by *McInnes et al.* (2009) and *Association of Bayside Municipalities* (2015) (ABM2015 in table) in comparison with the present study based on the current climate scenarios. In *McInnes et al.* (2009), to estimate storm tide levels with the aid of numerical models, surges and storm tides were evaluated separately. Further, *McInnes et al.* (2009) used the numerical models, which neglected the wave contribution to extreme sea levels. In that way, the wind drag coefficient was often adjusted to match surge levels (e.g. *Wu et al.*, 2018). Further, evidence from *Arns et al.* (2020) indicated that extreme sea levels could

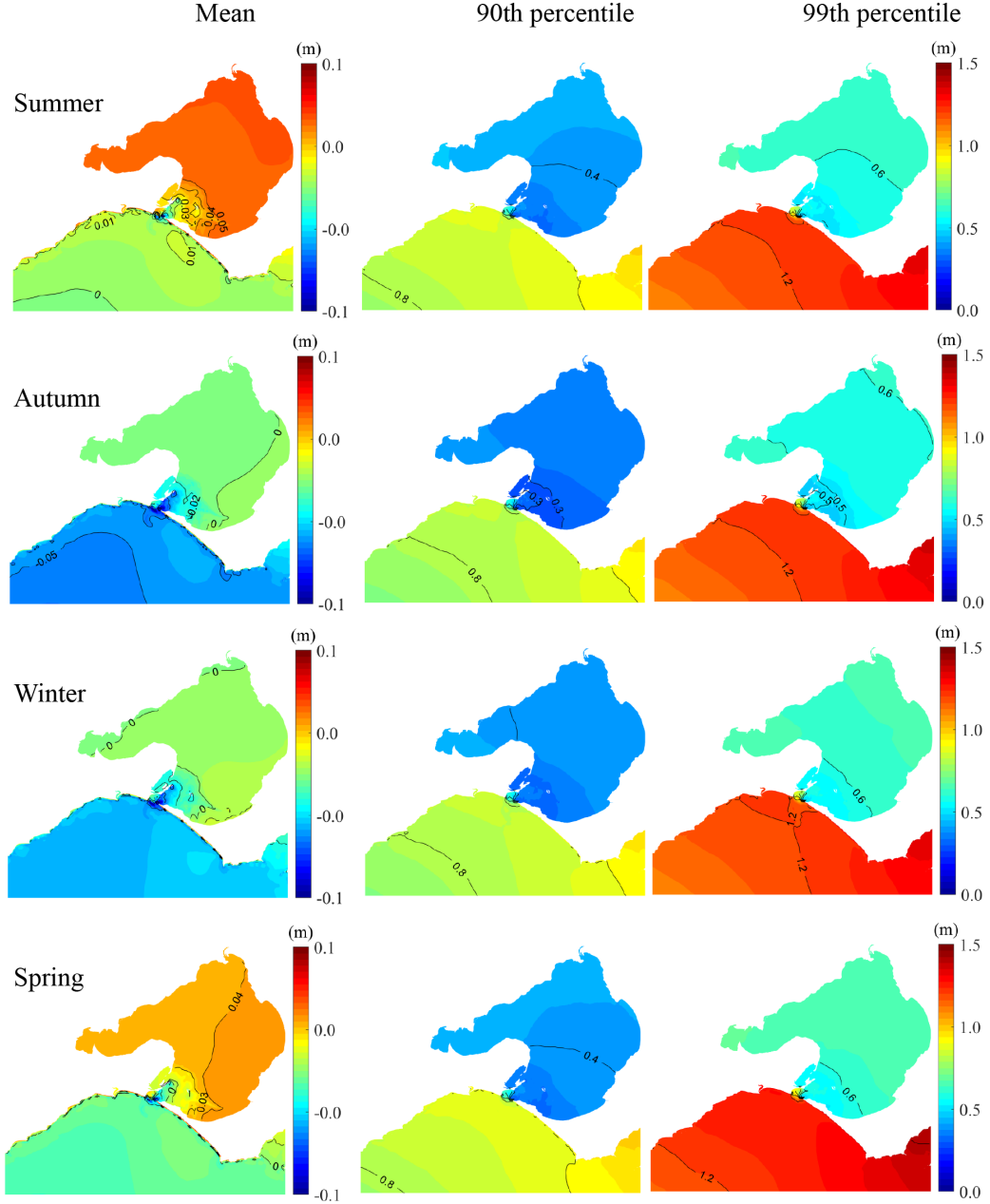


Figure 6.1: Seasonal variations of sea levels with respect to means, 90th and 99th percentiles. The MSL in PPB shows a strong influence from seasonality. The mean inside PPB is always higher than that outside in Bass Strait. This suggests that the difference in temperature plays an important role. The evidence of temperature's influence on MSL can also be seen across seasons. For example, means are higher in summer than in winter. For high percentiles, there is no clear dependence of water levels on seasonality.

be overlooked by 30% if nonlinear effects are not considered. For these reasons, a proper physical approach should be considered by using wave-circulation coupling models, as in the present study. Association of Bayside Municipalities (2015) estimated return levels based on the average recurrence interval (ARI), which is determined by using the annual exceedance probability (AEP); for instance, the 100-year return value is estimated from 1% of the AEP. Therefore, using different methods could yield different results.

Figure 6.2 provides maps of 100-year return values of storm tides estimated in the present study and those proposed by *McInnes et al.* (2009). The same pattern emerges from these maps - the largest storm tide level occurs on the eastern side of PPB because in extreme conditions, wind often blows from the west or the Southern Ocean towards PPB, causing higher surge levels in the east. However, *McInnes et al.* (2009) appeared to record higher values than the present study. The most significant difference in the storm tide levels between the two studies occurs from the entrance of PPB to the offshore regions; for instance, as indicated in the white colour in *McInnes et al.* (2009), the storm tide level in the offshore region of PPB is between 1.7 - 1.8m, which is 0.3 - 0.4m higher than estimated levels in the present study. As noted above, *McInnes et al.* (2009) derived storm tides from modelled results without coupling with a wave model; therefore, their study did not consider wave-tide-surge interactions.

Table 6.3: Comparison between storm tide levels proposed by McInnes et al. (2009)/McInnes09C, McInnes et al. (2009)/McInnes09D, Association of Bayside Municipalities (2015)/ABM2015 and the present study. The measured values are calculated based on sea level data from 1990 to 2016 using peak over threshold (POT) approach for consistency with the modelled data derived from 27 years of hindcasts. Four different thresholds are applied to estimate the return levels in this study. Using different methods to estimate the return levels with various lengths of recorded sea levels may yield different results.

Return	McInnes09C	McInnes09D	ABM2015	This study [based on data from 1990-2016 and applied various thresholds]							
Year				90 th		95 th		99 th		99.5 th	
Lonsdale				Measured	Modelled	Measured	Modelled	Measured	Modelled	Measured	Modelled
100	1.36 ± 0.05	1.41 ± 0.27	-	1.24 ± 0.04	1.33 ± 0.02	1.29 ± 0.06	1.32 ± 0.02	1.29 ± 0.08	1.32 ± 0.05	1.28 ± 0.09	1.33 ± 0.07
50	1.29 ± 0.04	1.35 ± 0.26	-	1.22 ± 0.03	1.32 ± 0.01	1.27 ± 0.05	1.30 ± 0.02	1.27 ± 0.08	1.30 ± 0.05	1.27 ± 0.09	1.31 ± 0.07
20	1.19 ± 0.03	1.27 ± 0.26	-	1.20 ± 0.03	1.30 ± 0.01	1.24 ± 0.05	1.28 ± 0.02	1.24 ± 0.07	1.27 ± 0.04	1.24 ± 0.07	1.28 ± 0.06
10	1.09 ± 0.03	1.16 ± 0.26	-	1.20 ± 0.03	1.20 ± 0.01	1.24 ± 0.05	1.28 ± 0.02	1.24 ± 0.07	1.27 ± 0.04	1.24 ± 0.07	1.28 ± 0.06
Hovell Pile											
100	-	1.09 ± 0.11	1.08 ± 0.09	1.07 ± 0.04	0.85 ± 0.02	1.08 ± 0.05	0.87 ± 0.03	1.13 ± 0.09	0.95 ± 0.08	1.13 ± 0.12	1.02 ± 0.14
50	-	1.06 ± 0.10	1.03 ± 0.08	1.05 ± 0.04	0.83 ± 0.03	1.06 ± 0.06	0.85 ± 0.03	1.11 ± 0.09	0.91 ± 0.08	1.11 ± 0.11	0.96 ± 0.12
20	-	0.99 ± 0.09	0.97 ± 0.07	1.03 ± 0.04	0.81 ± 0.02	1.03 ± 0.05	0.82 ± 0.03	1.06 ± 0.08	0.87 ± 0.06	1.06 ± 0.09	0.90 ± 0.09
10	-	0.93 ± 0.09	0.92 ± 0.05	1.03 ± 0.04	0.81 ± 0.02	1.03 ± 0.05	0.82 ± 0.03	1.06 ± 0.08	0.87 ± 0.06	1.06 ± 0.09	0.09 ± 0.09
Williamstown											
100	1.09 ± 0.03	1.12 ± 0.10	1.12 ± 0.06	1.08 ± 0.05	0.90 ± 0.02	1.11 ± 0.06	0.93 ± 0.04	1.09 ± 0.08	1.02 ± 0.10	1.11 ± 0.10	1.09 ± 0.15
50	1.05 ± 0.02	1.09 ± 0.09	1.08 ± 0.05	1.06 ± 0.04	0.88 ± 0.02	1.08 ± 0.05	0.90 ± 0.04	1.07 ± 0.08	0.98 ± 0.09	1.08 ± 0.10	1.03 ± 0.13
20	1.00 ± 0.02	1.03 ± 0.09	1.03 ± 0.04	1.03 ± 0.04	0.86 ± 0.02	1.05 ± 0.05	0.87 ± 0.03	1.04 ± 0.07	0.93 ± 0.07	1.05 ± 0.08	0.97 ± 0.11
10	0.95 ± 0.02	0.96 ± 0.09	0.99 ± 0.04	1.03 ± 0.04	0.86 ± 0.02	1.05 ± 0.05	0.87 ± 0.03	1.05 ± 0.07	0.93 ± 0.07	1.05 ± 0.08	0.97 ± 0.11
Geelong											
100	1.04 ± 0.03	1.06 ± 0.10	1.08 ± 0.05	1.02 ± 0.04	0.90 ± 0.02	1.04 ± 0.05	0.90 ± 0.03	1.08 ± 0.10	0.98 ± 0.08	1.08 ± 0.11	1.03 ± 0.12
50	1.00 ± 0.03	1.03 ± 0.10	1.06 ± 0.04	1.00 ± 0.03	0.89 ± 0.02	1.02 ± 0.05	0.89 ± 0.03	1.06 ± 0.09	0.94 ± 0.07	1.06 ± 0.10	0.98 ± 0.10
20	0.95 ± 0.02	0.98 ± 0.09	1.03 ± 0.04	0.98 ± 0.03	0.86 ± 0.02	0.99 ± 0.04	0.86 ± 0.02	1.02 ± 0.08	0.90 ± 0.06	1.02 ± 0.09	0.93 ± 0.08
10	0.89 ± 0.02	0.91 ± 0.09	1.03 ± 0.04	0.98 ± 0.03	0.86 ± 0.02	0.99 ± 0.04	0.86 ± 0.02	1.02 ± 0.08	0.90 ± 0.06	1.02 ± 0.09	0.93 ± 0.08

Wave-setup on extreme sea levels

During storms, abnormal increases in sea level are primarily determined by wind stress and pressure gradients (e.g. *Jelesnianski*, 1992; *McInnes and Hubbert*, 1996). However, the contribution of wave radiation stress (*Hansen*, 1978; *Longuet-Higgins and Stewart*, 1964) on the setup is non-negligible (e.g. *Guérin et al.*, 2018; *Wu et al.*, 2018), as this may increase the mean surface elevation owing to wave breaking along coastlines, affecting the undertow (e.g. *Faria et al.*, 2000) and coastal erosion (e.g. *Thornton et al.*, 1996). This section presents experiments to investigate the wave contribution to extreme sea levels in PPB using numerical models. The most severe storm for each of the four seasons was selected. These events have been well documented in the literature (e.g. *Cardno Lawson Treloar*, 2007b; *Hubbert and McInnes*, 1999; *McInnes and Hubbert*, 1996) with an abnormal sea level up to 1m recorded at tidal gauges. Simulations were undertaken to examine the increase of water levels due to waves. For each event, numerical models were applied with two forcing options. The first option is to run the flow model (SCHISM) without waves, but all wind and tides included. The second option is to couple the flow with wave models using the same wind and tide forcing as in the first option. For each case, the contribution of wave actions to sea levels is then estimated by subtracting the water elevations in simulations running without waves from the sea levels simulated with waves. Results for the change in water levels due to waves are shown in Figure 6.3.

It is clear that there is a tendency of waves to contribute to water levels at the lowest tides. This is a preferable period in terms of the occurrence of large surges (e.g. *Fortunato et al.*, 2017; *Pinheiro et al.*, 2020; *Pugh*, 1996; *Zhang et al.*, 2010). For instance, *Fortunato et al.* (2017) indicated that the intensity of wave breaking increases in low tides, leading to the largest setup taking place during this period of the tidal cycle. In the first event shown in Figure 6.3 (February 2005), the largest wave setup occurred at about 07:00 PM on 2 February, when the tide had just slightly passed its lowest level (slack water). At this time, an estimate of the surge level up to above 0.8m was observed at Williamstown (northern region of PPB).

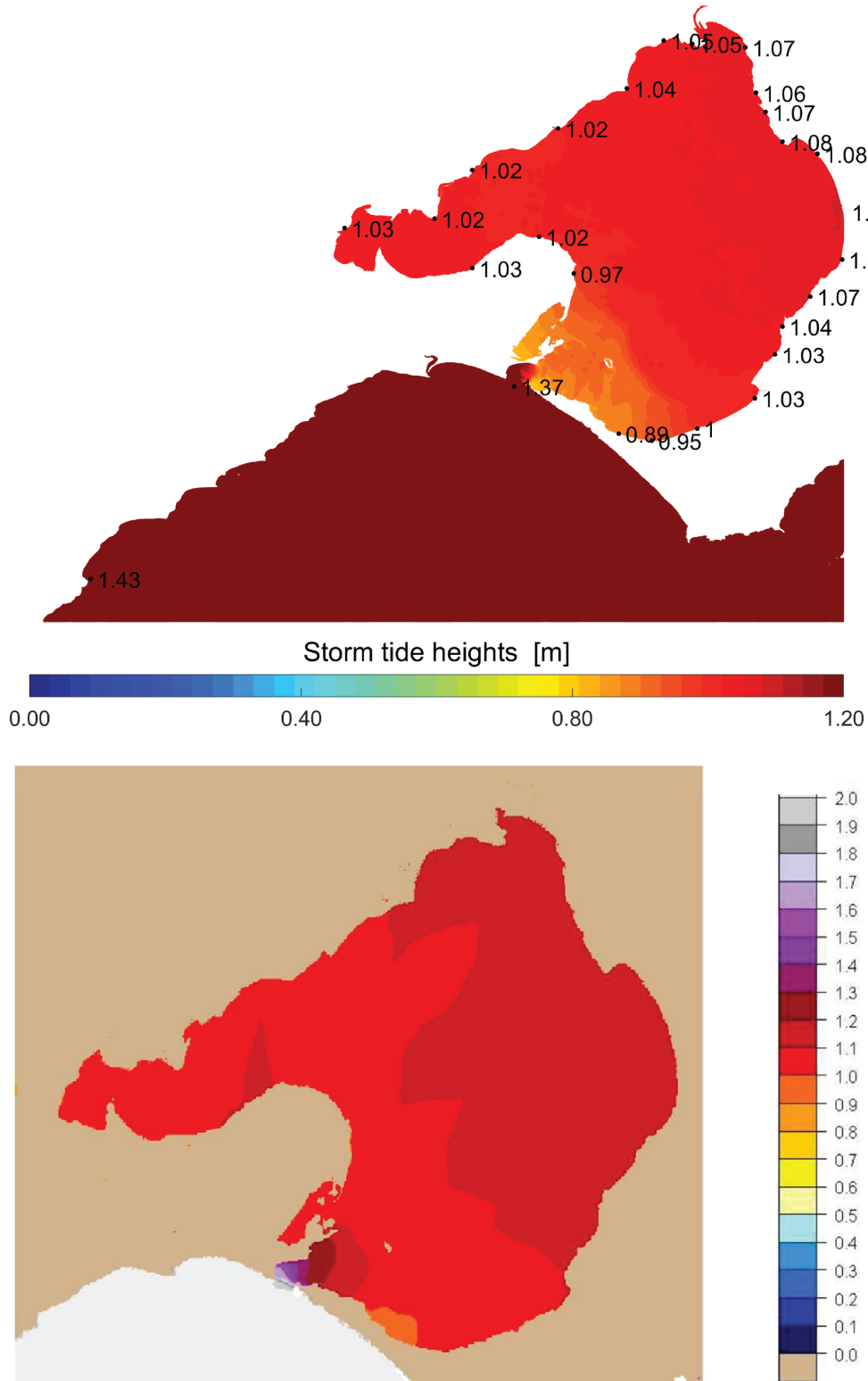


Figure 6.2: Maps of 100-year return values of storm tides from the present study (top panel) and those proposed by McInnes et al. (2009) (bottom panel). The latter did not consider the contribution of waves to extreme sea levels, and therefore, neglected non-linear effects. Ignoring wave setup could overestimate the extreme levels. For PPH, it was shown in Figure 3.17 (Chapter 3) that large waves events tend to reduce tidal anomaly (residuals), thus reduced extreme sea levels. Further, without wave affects, Arns et al. (2020) indicated that extreme sea levels could be overestimated up to 30%.

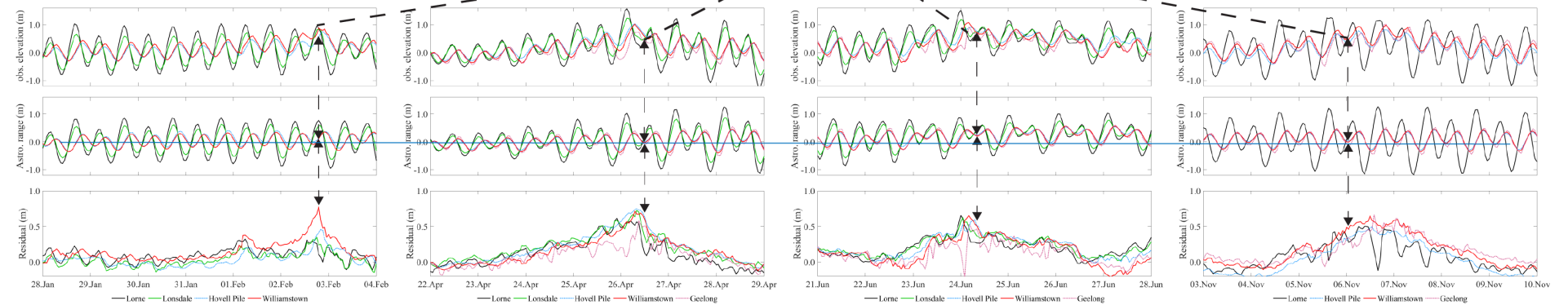
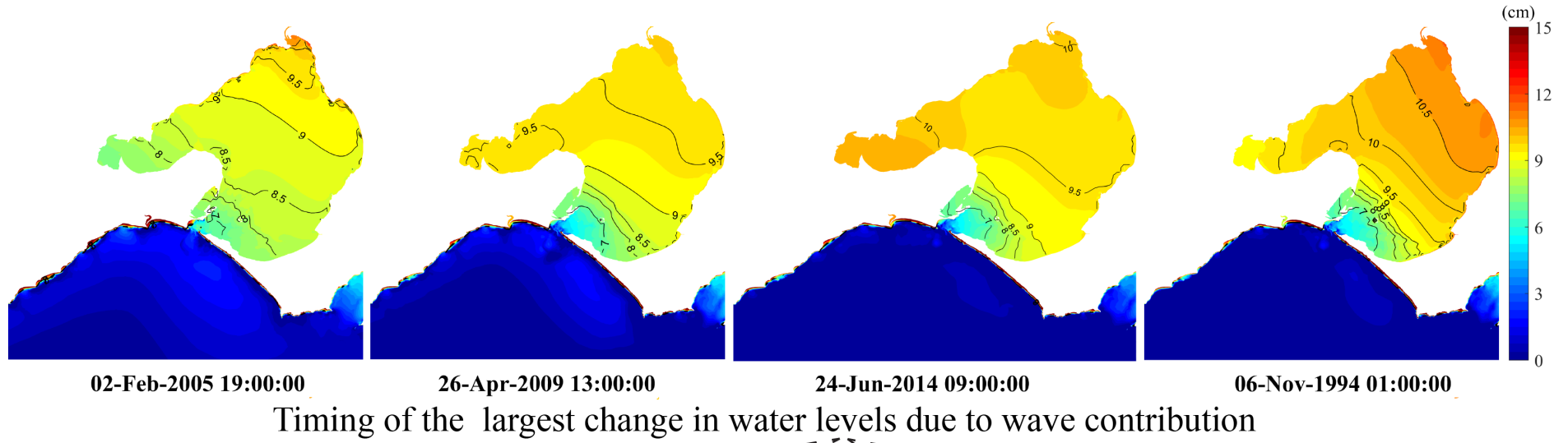


Figure 6.3: Contribution of waves to water levels during four storm events in PPB. The top panel show the largest change in water levels and timing in tidal cycles marked by arrows in time series observed from tidal gauges, which represent total observed elevations and astronomical tides only and non-residuals. One of the major patterns is that the largest wave setup occurred during the lowest tides of falling cycles. Another feature is that the largest setup can be observed in the northern portion of PPB (e.g. Williamstown). Of the four events, the greatest magnitude of wave setup in this region is up to 10.5cm (about 13% of the total residuals) in November 1994 as the offshore wind speed was constantly high over at least 8 hours.

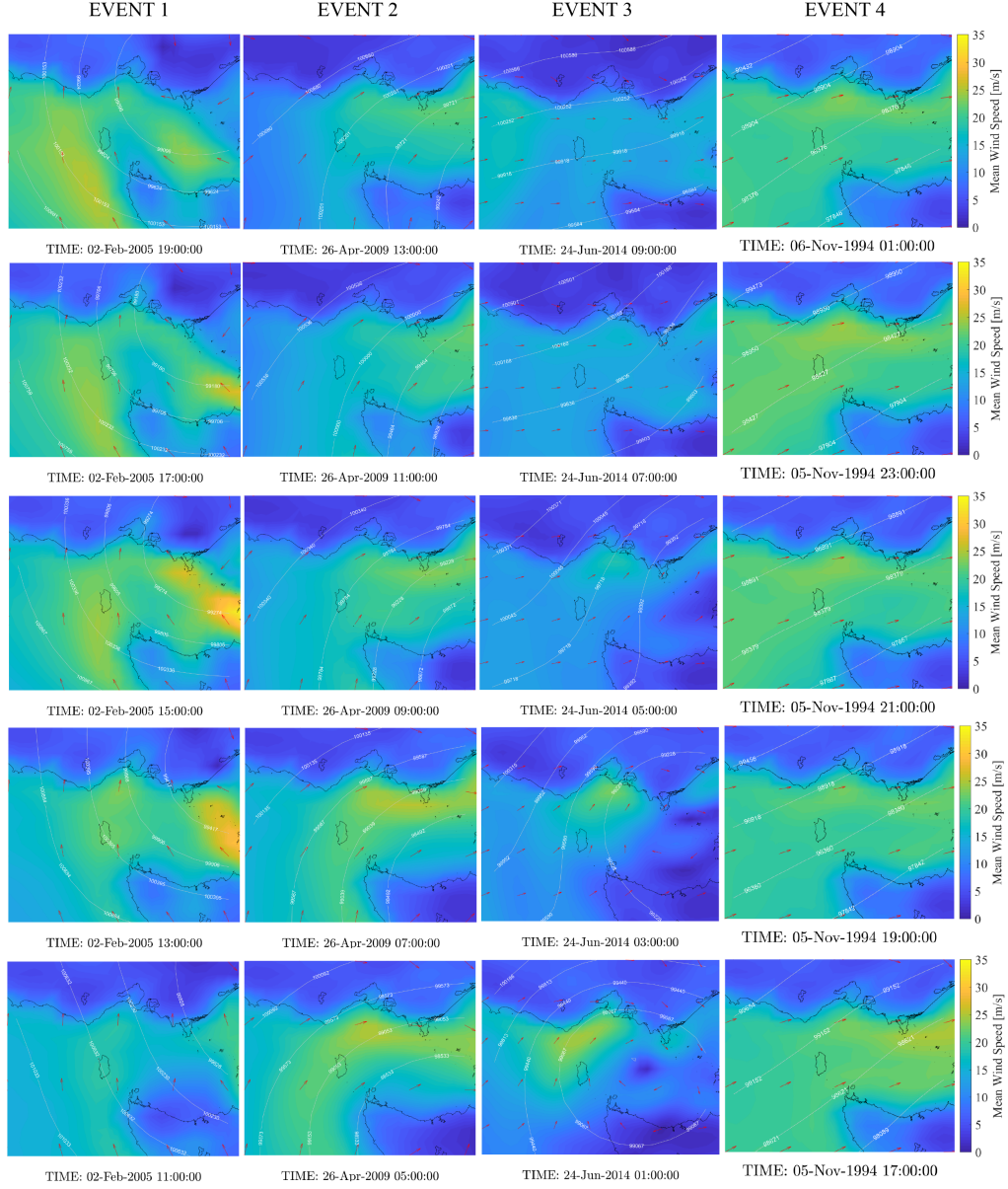


Figure 6.4: Weather conditions before the largest wave setup during four storm surge events. The first event (top), in summer, was due to the ‘Tasman low’ weather system moving from the southeast to the north. The other three events were caused by a cold front system moving mainly from west to east.

Referring to the wind and pressure patterns in Figure 6.4, the highest surge level occurs at the time of the lowest pressure. During this time, wind blows from the south to north as part of the Tasman low weather system. This means that waves have the longest fetches at the further points in the northern portion of PPB, explaining the fact that the largest increase in sea level (about 10cm) occurs at Williamstown. In the second event, the largest contribution of waves occurred on 26 April 2009 at 01:00 PM in response to a southwesterly wind.

Despite having a lower wind speed than the first event, the increase in water level of 9.5cm covered a larger area in the northern portion of PPB than in the previous event, as it happened at the slack tide with lower water depth. A reduction in

water depth can slow wave propagation and increase bottom friction and the wave setup. The third event on 24 June 2014 is interesting as the wind speed was not too high at the time of the largest increase in sea level, due to wave setup at 9:00 AM; the largest change, of up to 10cm, occurred at Geelong, perhaps because of the wind direction and presence of two wind systems at Geelong. Figure 6.4 shows that for the third event, the wind pattern in the northern portion of PPB between 7:00 AM and 9:00 AM changed from westerly to north-westerly, while the offshore wind remained the same. Further, Geelong water area is one of the shallowest and narrowest regions in PPB, so a westerly wind could increase the fetches, generating more waves towards the inner bay. These waves could even interact with flood tidal currents, which travel towards this region in the opposing direction to the wind. The last case, in November 1994, is the largest. Waves increased the water level up to 10.5cm over half of PPB. This occurred because of low pressure and a strong southwesterly wind, which constantly maintained a high speed of about 25 m/s over a period of at least 8 hours (e.g. see 6.4). Overall, waves can contribute to extreme levels in PPB by up to about 11cm in the north and east sections of the bay. The largest increase in sea levels due to wave setup is prone to occur during falling tides; in particular, at ebb tides, which occurred for three of the four events.

6.3.2 Waves

To date, there has been a lack of research defining the wave climate in PPB. The most relevant work was done by *Association of Bayside Municipalities* (2017). That study used the SWAN numerical model to quantify the wave climate within the bay based on the wind field at Point Wilson. Because the measured wind data were discontinuous, it was not possible to perform a long-term hindcast simulations. The wave climate was characterised by assuming the wind blowing on the surface water from prescribed directions. The 100-, 50- and 20-year return periods of the wave field were estimated from the maximum values of the modelled outputs simulated with 100-, 50- and 20-year return periods of wind data. A total of 248 locations were extracted to compute the maximum wave height, peak period and mean direction. Where relevant, *Association of Bayside Municipalities* (2017) will be used as a basis for comparison for the present study.

Seasonal variations

Mean Parameters:

Figure 6.5 presents the spatial patterns of the seasonal mean (e.g. averaging the data over a given season for the 27 years) of the mean wave direction, peak period and SWH. For the first parameter, the major wave direction in summer over half of PPB is between 180 and 210 degrees, which is caused by waves propagating from south towards north. This pattern is strongly linked with wind climate as it blows towards land in summer months (e.g. *Cardno Lawson Treloar*, 2007b). In winter, however, it is clear that more waves propagate from the north to the south towards the eastern coasts. In the southern portion, there is an interesting pattern in the vicinity of PPH.

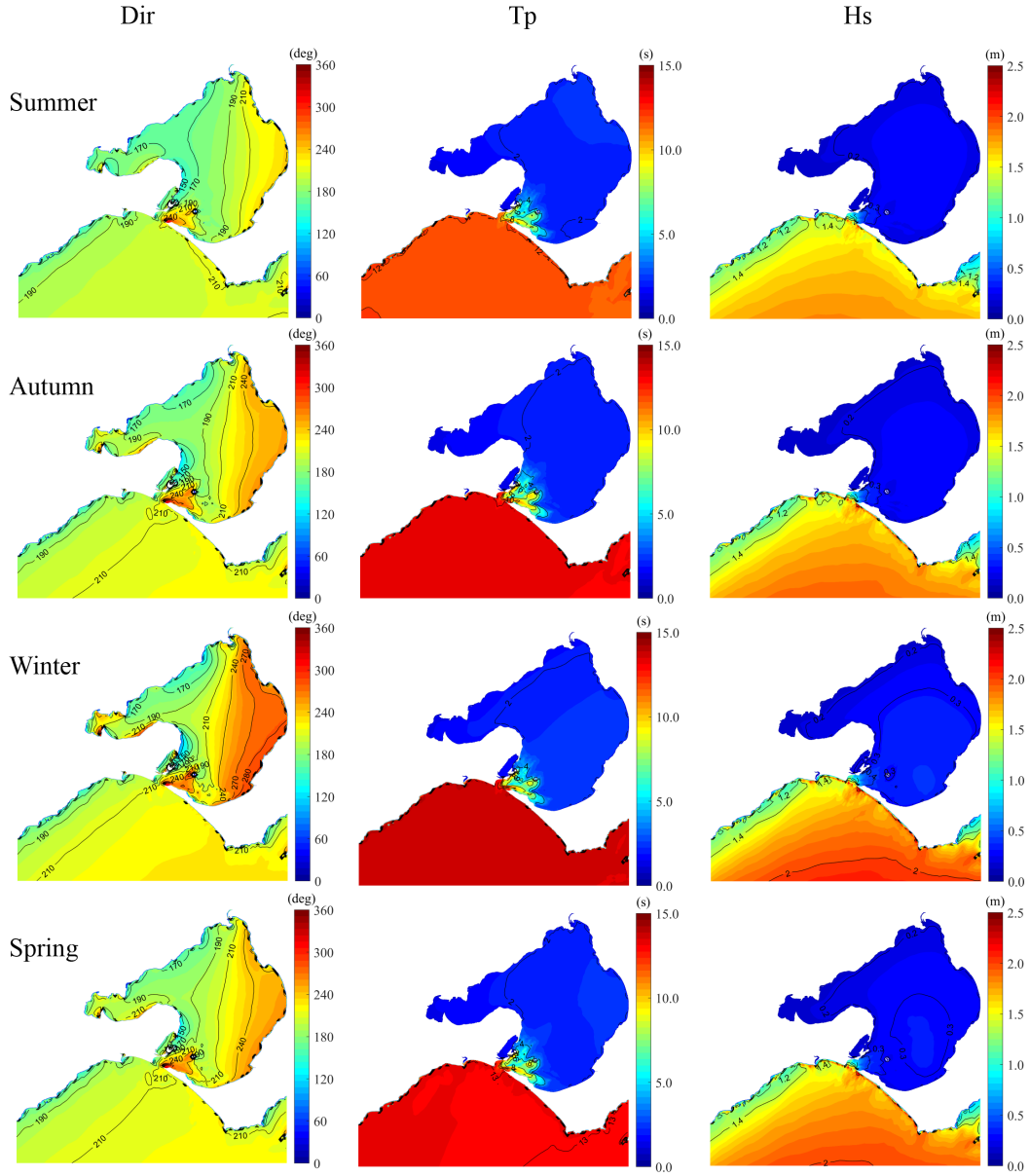


Figure 6.5: Spatial distribution of mean wave direction (Dir), wave peak period (Tp) and SWH (Hs) for the four seasons. For each season, the mean value of each parameter is averaged over 27 years of hindcast. For mean wave direction, dominant waves propagate towards the north in summers because the wind blows from the sea to the land. The opposite is true during winter. In terms of the peak wave period, the mean value inside PPB is relatively small as waves are generated by local winds only. Seasonality also affects the SWH inside PPB, with an increase of 0.1m during winter compared with summer. In winter months, these maps also show evidence of more refracted waves at the entrance of PPB (e.g. the area covered with the mean wave direction of 240° at the inlets is larger in winter).

Looking at the outside of PPB, the mean wave direction is about 210 degrees regardless of seasonality because it is dominated by swells from the Southern Ocean (Bass Strait). Just inside the entrance, however, the wave direction is mainly 240 degrees. This is because ocean waves are refracted when they pass the entrance. The area covered in this direction is larger in winter than in summer (e.g. around Sand Island). This suggests that swells may propagate further inside PPB in winter than summer. In terms of peak wave period, the mean value of about 2 seconds dominates a large portion of PPB as waves are generated by local winds only. The peak period slightly increases in winter compared with other seasons. The SWH is also affected by seasonality. Inside PPB, summer and winter months have an average of 0.2m, which is 0.1m lower than in spring and winter. The seasonal mean value of the SWH in the Bass Strait is much higher than that inside the lagoon because waves are dominated by swells propagating from deep waters. For example, the mean of value is about 1.4m in summer, and up to 2m at the offshore during winter. Because of swell attenuation through the narrow entrance, the mean of SWH is reduced to 0.4m in winter after passing PPB. However, it is still 0.1m higher than that in summer. Therefore, it is demonstrated that more swells are refracted through PPB in the winter than in the summer.

Extreme significant wave height

Figure 6.6 shows spatial and seasonal variations in SWH (H_s) on the basis of three extreme levels: the 90th and 99th percentiles and maximum values based on 27 years of hindcast data from 1990 to 2016. Clearly, H_s inside PPB is much smaller than that outside as waves in PPB are locally generated by surface winds. However, seasonality affects all extreme levels.

Inside PPB, winter extremes are systematically larger than those in summer. At the 90th percentile, for instance, H_s in winter months is about 0.5m, which is 0.2m higher than in summer. The values in spring are quite similar to those in autumn. The increment in extreme levels also results in different seasonal variations in H_s . The high extreme levels, the large differences occur. For instance, the difference between the 90th and 99th percentiles ranges from 0.2m in the summer to 0.5m in the winter. This range is much larger when the extreme level of H_s moves from the 99th percentile to its maximum value. This suggests that there is a rare event occurring between 1990 and 2016 that accounts for the maximum value. This event, as indicated in this study, occurred on 24 June 2014, during the winter. The maximum for the winter, as shown in Figure 6.6, is higher than 2m in the eastern side of PPB. On this side, H_s extremes are always higher than those on the western side of PPP because of the wind climate pattern. A strong seasonal dependence is clearly evident for the highest extremes. In summer, the largest H_s tends to move to the northern PPB, while the region of maximum H_s shifts to the east coast in winter. As indicated in *McInnes and Hubbert (1996)*; *McInnes et al. (2005)*, about 70% of storm events occurred in spring and winter months due to the cold fronts caused by westerly winds moving to the eastern coastline of PPB. It can be seen that the distance of ocean waves propagating into PPB is much larger in the highest extreme levels than in the lower percentiles. For instance, the map of maximum H_s in the winter shows that the H_s is up to 1.7m after passing the entrance towards West Channel Pile, a shallow channel system located between Queenscliff and Sand Island, while the H_s along the South Channel is

only 1.2m. This indicates that more ocean waves propagate straightforward to West Channel Pile before dissipating than in the main shipping channels. As a result, there are fewer refracted waves along South Channel. A similar pattern of

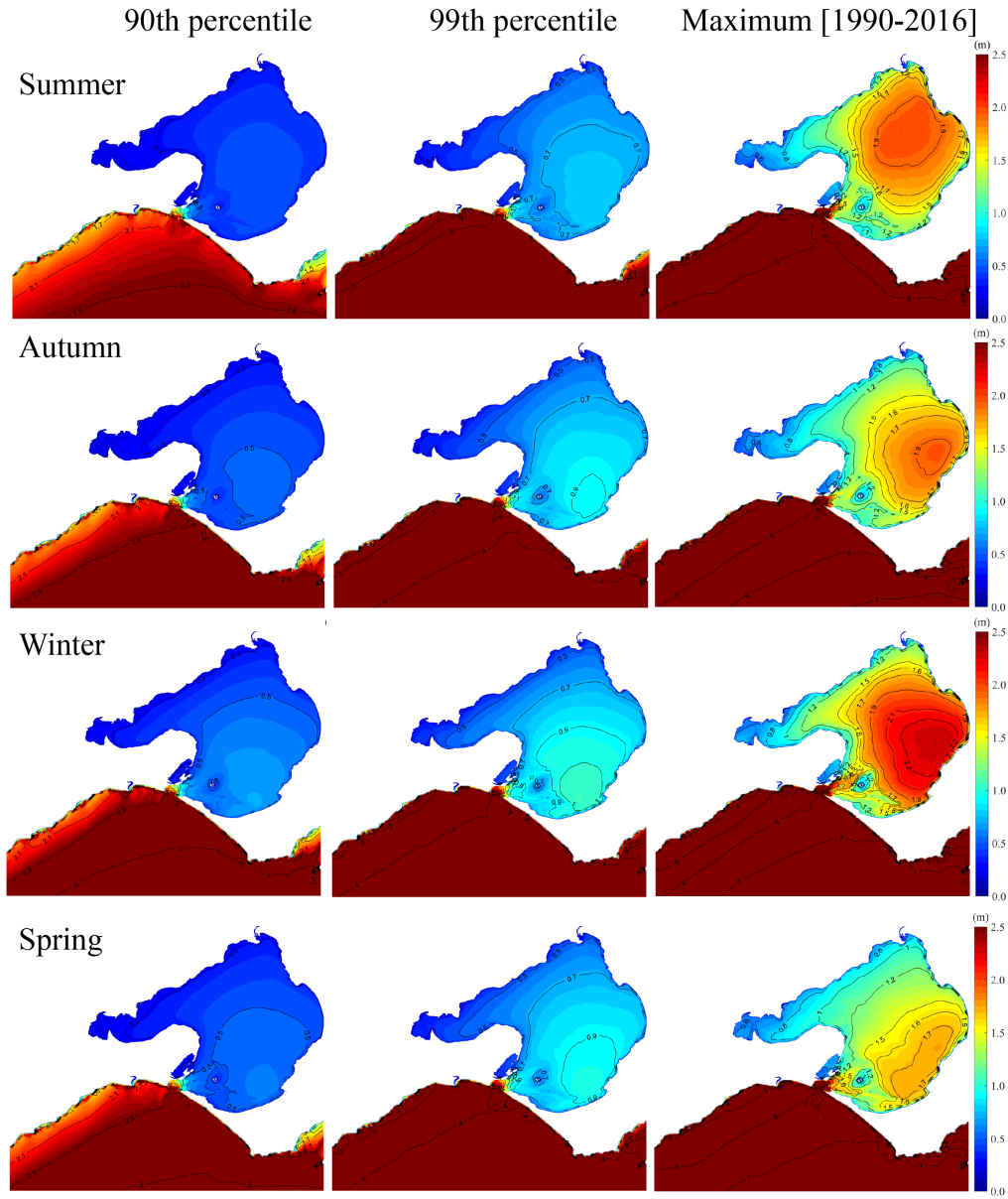


Figure 6.6: Maps of spatial and seasonal variations in H_s estimated for the 90th and 99th percentiles and maximum records over 27 years of hindcasts from 1990 to 2016. There is a large difference between the extreme values inside and outside PPB as ocean waves do not penetrate into the main body of PPB. A clear pattern is that seasonality affects all extreme levels. For example, winter extremes are systematically larger than those in summer. Inside PPB, the difference between the 90th and 90th percentiles ranges from 0.2m in summer to 0.5m in winter. The maximum values of H_s are much higher than those at the 90th and 90th percentiles. The most severe event in winter occurred on 24 June 2014.

seasonal variations of H_s also occurs in the offshore region of PPB located in the Bass Strait. In summer months, the 90th percentile of H_s is about 2.1m, which is 0.4m lower than in spring and winter.

For the 99th percentile, H_s is about 3m in summer and 4m in other seasons. In terms of extremes, H_s can be up to 7m in the winter and 6m in the warmer months. Such extremes in the Bass Strait are likely due to swells propagating from the Southern Ocean. The direction of swell propagation is approximately 210° all year around. Evidence of this can be seen from the contour lines of H_s in the Bass Strait, which are always in parallel to the shoreline along Lorne to PPH. Other evidence includes that H_s in the offshore Portsea and Sorrento regions is much higher than in the western offshore coastline.

Return periods

This section presents a data analysis of wave parameters with respect to return periods. First, it compares and contrasts the present findings with those in *Association of Bayside Municipalities* (2017) (hereafter ABM) at 24 stations around PPB, before analysing results for the entire domain of PPB. For ease of interpretation, these points are arranged anti-clockwise around PPB starting from Blairgowrie (Point 1), which is located between Rye and Sorrento (a few kms from the entrance of PPB), and ending at St Leonards (on the other side of PPB, close to Queenscliff). A map of these locations is provided . Further, it is noted that ABM computed the return levels of wave parameters by taking the maximum values from modelled results simulated with wind speed and direction options that were estimated for 1%, 2% and 10% of AEP (equivalent to 100-, 50- and 10-year return periods). In other words, ABM's approach is different from the POT approach used in the present study. In simple terms, only 1% AEP of H_{s100} , T_{p100} , Dir_{100} are compared with those maximum values from 27 years of hindcast data, which also provide these three parameters. To make a further comparison, the present study estimated 100-year return periods for H_{s100} based on several threshold levels: the 90th, 95th, 99th and 99.5th percentiles, which are also compared with 1% AEP of H_{s100} from ABM. Results are presented in Table 6.4.

Maximum wave parameters from 27 year-hindcasts vs ABM 1% AEP

One of the major features in Table 6.4 is that H_{smax} values found in the present study are similar to those of H_{s100} in ABM. These are located at Rye, Mt Martha, Fishermans, Mordialloc and Brighton (Points 2, 5, 6, 10, 13), with a difference of 0.1m only. These matched values are marked in square boxes in the table. In particular, ABM estimated the largest 100-year return value of H_s of 2.2m at Fishermans, which is the same as the present study. However, this study found the largest H_{smax} is located at Moondah (Point 7), with a value of 2.3m, higher than ABM's 0.4m. The second feature in Table 6.4 is that all maximum values of H_{smax} occurred during the two largest well-known storms. One of these events hit Victoria on 24 June 2014, causing much flooding including in the Melbourne CBD (<https://www.abc.net.au/news/2014-06-24/storm-damage-in-melbourne-june-242c-2014/5546568>). Another important point is the notable difference between results from this study and those provided by ABM for locations on the western side at Clifton Springs, Steeple and St Leonards (Points 22 to 24) and in the southern sections at Blairgowrie, Rye and Rosebud (Points 1, 2, 3).

Table 6.4: A comparison between return values based on Association of Bayside Municipalities (2017) and those estimated in the present study applied to various thresholds.

Point	Location	Long	Lat	Association of Bayside Municipalities (2017)									[This study based on 27 year hindcasts 1990-2016]							
				100-year			50-year			10-year			Max. values				H_{s100} with several thresholds			
				H_{s100}	Tp_{100}	Dir_{100}	H_{s50}	Tp_{50}	Dir_{50}	H_{s10}	Tp_{10}	Dir_{10}	H_{smax}	Tp_{max}	Dir_{max}	Time	90^{th}	95^{th}	99^{th}	99.5^{th}
1	Blairgowrie	144.7836	-38.3614	1.2	4.5	356	1.2	4.4	356	1.1	4.2	1.0	1.5	4.4	320	06-Nov-1994 01:00	1.58± 0.11	1.58±0.13	1.62±0.21	1.64±0.26
2	Rye	144.8331	-38.3702	<u>1.2</u>	5.8	6.0	1.2	5.8	6.0	1.1	5.2	6.0	<u>1.3</u>	3.6	301	24-Jun-14 01:00:00	1.34± 0.08	1.36±0.10	1.32±0.13	1.31±0.14
3	Rosebud	144.9011	-38.3548	1.7	6.2	356	1.6	6.1	356	1.4	5.5	356	1.9	4.9	263	24-Jun-14 03:00:00	1.94±0.13	1.97±0.17	2.02±0.28	1.98±0.29
4	Safety Beach	144.9884	-38.3167	1.5	5.7	274	1.5	5.6	284	1.4	5.2	274	1.8	4.9	297	24-Jun-14 01:00:00	1.82±0.12	1.85±0.16	1.90±0.25	1.93±0.32
5	Mt. Martha	145.0185	-38.2609	<u>2.0</u>	6.1	327	1.9	6.1	327	1.8	5.4	297	<u>2.1</u>	4.9	263	24-Jun-14 03:00:00	2.02±0.15	1.98±0.18	2.08±0.31	2.03±0.31
6	Fishermans	145.0297	-38.2255	<u>2.2</u>	6.5	297	2.1	6.2	297	1.9	5.9	298	<u>2.2</u>	5.4	260	24-Jun-14 03:00:00	2.09±0.18	2.04±0.19	2.10±0.31	2.09±0.33
7	Moondah	145.0713	-38.1876	1.8	6.1	278	1.7	5.9	291	1.6	5.7	291	2.3	5.4	261	24-Jun-14 03:00:00	2.06±0.17	2.01±0.19	2.07±0.30	2.10±0.36
8	Frankston	145.1192	-38.1408	1.8	6.4	277	1.8	6.1	266	1.6	5.8	265	2.2	5.4	261	24-Jun-14 03:00:00	1.92±0.16	1.91±0.19	2.01±0.32	2.03±0.38
9	Carrum	145.1234	-38.0811	1.8	6.4	365	1.8	6.0	266	1.6	5.8	265	2.1	5.4	255	24-Jun-14 03:00:00	2.03±0.19	2.01±0.23	2.09±0.32	2.11±0.46
10	Mordialloc	145.0821	-38.0068	1.8	6.2	257	1.8	5.9	243	1.7	5.5	243	1.9	5.4	220	02-Feb-05 17:00:00	2.18±0.24	2.01±0.25	2.13±0.44	2.18±0.56
11	Beaumaris.Y	145.0296	-37.9915	1.6	6.1	255	1.6	5.9	241	1.5	5.6	255	1.8	5.4	217	02-Feb-05 17:00:00	2.08±0.22	2.02±0.26	2.05±0.41	2.19±0.58
12	Sandringham	145.0043	-37.9539	1.2	5.9	254	1.2	5.8	242	1.1	5.4	255	1.9	5.4	212	02-Feb-05 17:00:00	1.89±0.19	1.84±0.22	2.01±0.42	1.94±0.43
13	Brighton	144.9901	-37.9295	<u>1.8</u>	5.7	262	1.8	5.7	245	1.6	5.2	225	<u>1.9</u>	5.4	209	02-Feb-05 17:00:00	1.90±0.20	1.85±0.22	2.02±0.42	2.00±0.47
14	St.Kilda M	144.9742	-37.8725	1.4	4.9	260	1.4	5.1	241	1.3	5.2	194	1.7	5.4	209	02-Feb-05 17:00:00	1.71±0.17	1.69±0.22	1.79±0.35	1.89±0.47
15	Williamstown	144.8937	-37.8680	1.4	5.6	194	1.4	5.6	180	1.3	5.2	194	1.6	5.4	197	02-Feb-05 17:00:00	1.79±0.20	1.67±0.22	1.58±0.28	1.63±0.35
16	Altonaburns	144.8510	-37.8639	<u>1.5</u>	5.5	177	1.5	5.6	177	1.4	5.2	178	<u>1.4</u>	5.4	180	02-Feb-05 17:00:00	1.55±0.18	1.48±0.20	1.31±0.22	1.35±0.27
17	Pointcook	144.7957	-37.9241	1.6	5.8	152	1.5	5.8	152	1.4	5.3	152	1.2	4.9	176	02-Feb-05 17:00:00	1.36±0.13	1.40±0.17	1.47±0.29	1.45±0.33
18	Werribee.S	144.6927	-37.9746	1.4	5.2	154	1.4	5.2	153	1.3	5.1	136	1.0	4	221	24-Jun-14 03:00:00	1.23±0.12	1.30±0.16	1.41±0.3	1.49±0.41
19	Beacon Point	144.5640	-38.0272	1.3	5.2	125	1.3	5.1	139	1.2	4.8	140	1.0	4	221	24-Jun-14 03:00:00	1.02±0.09	1.06±0.12	1.15±0.23	1.15±0.27
20	Pt.Wilson East	144.5077	-38.0893	1.6	5.8	152	1.5	5.8	152	1.4	5.3	152	0.8	3.6	200	02-Feb-05 17:00:00	0.85±0.06	0.86±0.08	0.84±0.11	0.84±0.13
21	Moorpanyal.P	144.3721	-38.1010	1.2	4.4	88	1.2	4.4	88	1.0	4.1	88	0.7	3	208	02-Feb-05 17:00:00	0.73±0.05	0.78±0.08	0.81±0.14	0.73±0.11
22	Clifton Springs	144.5642	-38.1516	<u>1.7</u>	5.2	317	1.7	5.1	317	1.5	4.7	317	0.9	3.3	272	24-Jun-14 03:00:00	0.97±0.06	1.00±0.08	1.06±0.16	1.04±0.18
23	Steeles	144.6643	-38.1122	1.6	5.5	4.0	1.6	5.4	4.0	1.5	4.9	4.0	1.2	4	267	24-Jun-14 03:00:00	1.17±0.08	1.20±0.10	1.23±0.18	1.23±0.20
24	St. Leonards	144.7163	-38.1585	1.5	5.6	39	1.5	5.7	40	1.4	5.3	4.2	1.3	4	267	24-Jun-14 03:00:00	1.17±0.08	1.20±0.10	1.23±0.18	1.23±0.20

For western locations, ABM found that H_{s100} values were mainly attributable to the northerly wind (e.g. wind blows from north). However, the H_{smax} values in the present study do not come from that direction. This might suggest an issue with predicting wind direction that comes from the mainland (in this case, north of PPB) in the global models, because wind direction and magnitude may be affected by the land. Such an effect cannot be perfectly represented in wind models. Still, there is a suspected value in ABM at Clifton Springs (Point 22, as underlined in Table 6.4) with $H_{s100} = 1.7\text{m}$ and $Dir_{100} = 317^\circ$. The reason is that the wind blows only a few kilometres over the surface from that direction, and as a result, waves are very fetch-limited.

Even given that, ABM provided lower return levels of H_{s100} in the southern section of PPB compared with the present study. For example, the maximum wave height computed from 27 years of hindcasts is 1.5m, at Blairgowrie beach, while ABM gives 1.2m for this figure only. This difference may be due to the fact that ABM simulated waves without a flow model and assumed that PPB is a closed basin, while the present study used a coupled model with a prescribed open boundary.

100-year return levels based on 27 year-hindcasts vs ABM 1% AEP

To investigate the sensitivity of thresholds on return values, this study provides estimated 100-year return SWHs with lower/upper-bound ($\pm$) limits (95% confidence interval, hereafter 95% CI) based on four threshold levels (see the last four columns in Table 6.4).

On the eastern side of PPB, the 100-year SWHs estimated from the four thresholds at Mt Martha, Fishermans, Moondah, Frankston and Carrum are lower than the maximum values found in the 27-year hindcasts. This suggests that the occurrence of the storm hitting PPB on 24 June 2014 is extremely rare, with such a low probability that it is not represented in 100-year events. At the (highest) 99.5th percentile threshold, the 100-year return SWH at Carrum is similar to the maximum value of the SWH from 27 years of hindcasts. This means that the 99.5th percentile threshold can be used to determine the 100-year event at this location. However, applying this threshold gives large lower- and upper-bound limits - up to 0.46m. At the (lowest) 90th percentile threshold, this study provides 100-year SWHs similar to those in ABM at Fishermans and Frankston, with a difference of 0.1m. For example, ABM reported values of 2.2m and 1.8m for the 1% EAP event at these two locations, while this study proposed 2.09m and 1.92m for the 100-year event, with a variation of 0.18m and 0.16m (95% CI). Examining the results in Table 6.4 more closely, this study confirms two important points. First, the 100-year event of SWH at points along the eastern side is some 2.2m, as both ABM and the present study found values that matched well, not only for H_{s100} but for Dir_{100} . The 100-year event is caused by wind blowing from the west to the east, as in the worst event, on 24 June 2014. The second point is that one threshold level cannot be used to determine the return values at points on the same eastern side of PPB; in particular, some points may require a higher threshold than others. This suggests that the distribution of wave height cannot be fitted by the same function with the same threshold.

A similar pattern as that mentioned in the analysis of maximum SWH is found for the return values in the southern and western portions of PPB. Specifically, this study provided higher return levels in the south but lower values in the west of the bay compared with those provided by ABM. On the one hand, as determined by ABM, 100-year events at the western points were due to southerly or westerly wind patterns. In these cases, the wind blows from land, which may not be well captured by the wind data derived from the global CFSR model because such a model cannot predict a change in wind direction over land. However, the issue related to southerly winds does not apply when predicting 100-year events in the southern section of PPB. At Blairgowrie, for instance, ABM and this study show the wind direction travelling from the north. However, ABM found 1.2m for the 1% AEP H_{s100} while this study proposed 1.5m SWH for the 100-year event. In general, an underestimation of the 1% AEP values may be associated with the prescribed southern boundary used in that study.

Mapping 100-year return periods of the significant wave height for the entire Port Phillip Bay

Figure 6.7 provides an overview of the 100-year return values of SWH in PPB estimated for four different thresholds. The histograms of tail data at one location (Sandringham; black point) are also presented. A clear pattern is that the increment

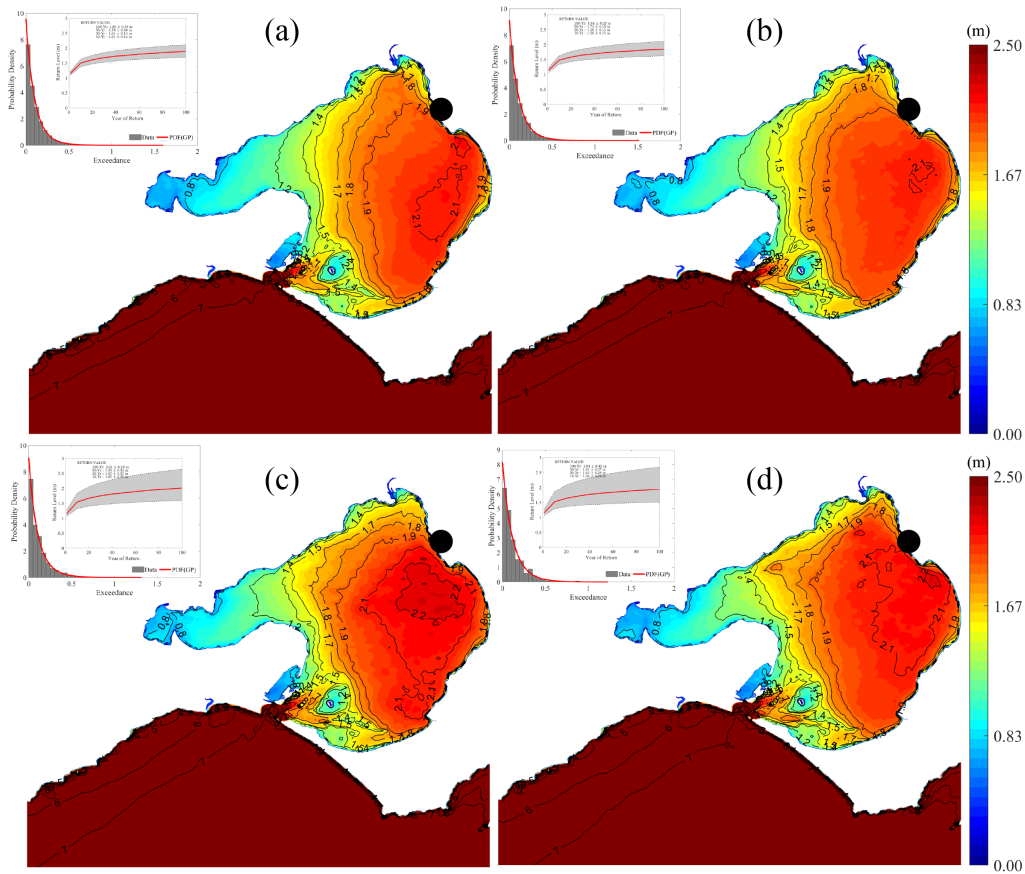


Figure 6.7: 100- year return values for SWH (H_{s100}).

of threshold level slightly increases the return values - changing the threshold from the 90th to the 99.5th percentile increased return levels by about 10%. A more

significant change occurs in the lower and upper limits (related to the confidence intervals). As indicated in the plots for Sandringham, the deviation from the estimated value is doubled if the threshold is adjusted from the 90th to the 99.5th percentile. In general, the largest return value of 100-year SWH is up to 2.2m along the eastern side of PPB. Conversely, the western region has much lower values - the lowest is in the Geelong area, with a 100-year event of only 0.8m.

6.3.3 Currents

Seasonal variations

Mean:

Figure 6.8 presents the mean of current speed calculated over 27 years of hindcasts without considering the effects of channel deepening. As can be seen from this figure, the strongest current speed, of up to 2 m/s, can be observed at the entrance of PPB. This magnitude gradually reduces when the flow moves further inside the South Channel towards the inner PPB. For the rest of the bay, the mean current is very weak - approximately 0.05 m/s. It can also be seen that the mean current in PPB is not affected by seasonality.

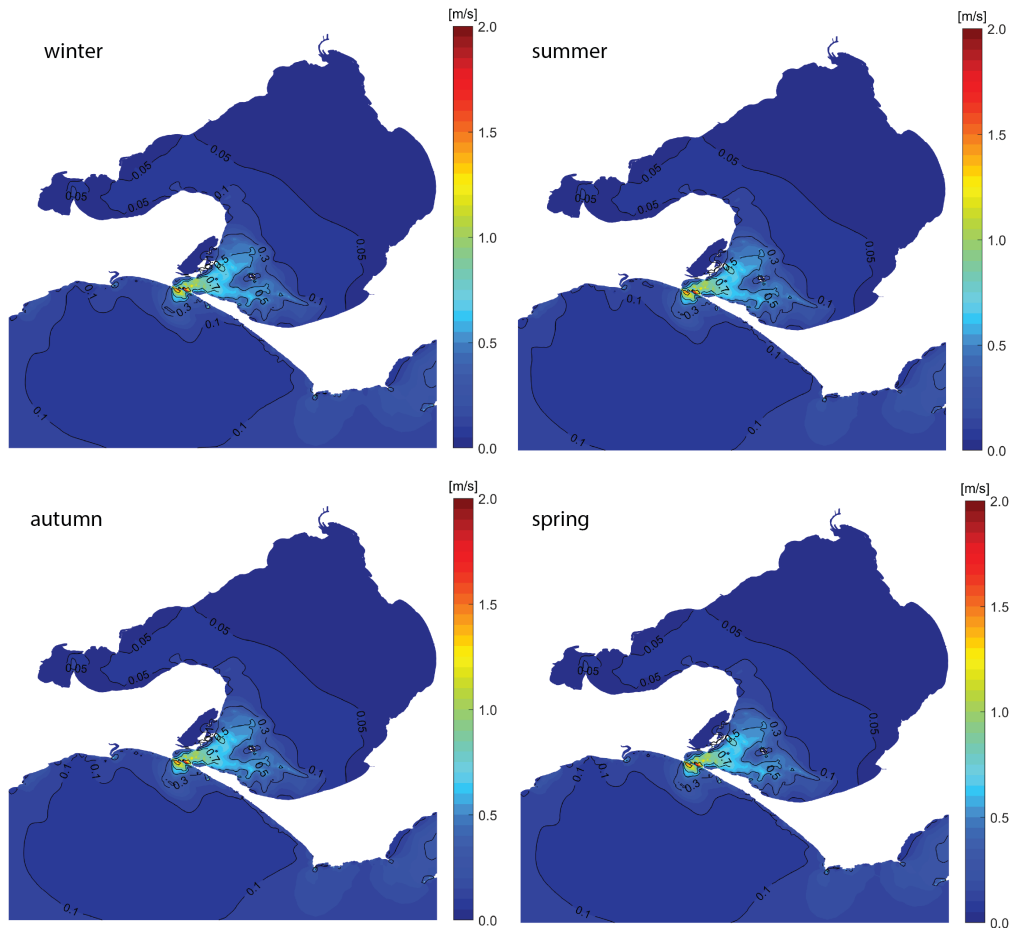


Figure 6.8: Maps of seasonal variation in current velocity over PPB. The mean magnitude can reach 2 m/s in the vicinity of PPH; after passing through the southern region, the current speed is very weak.

Extremes:

Figure 6.9 shows the contours of the 99th percentile current magnitude across all seasons. Like the mean, extreme currents do not seem to be affected by seasonality. However, in extreme conditions, the current speed in inner PPB is slightly increased by 0.05 m/s compared with the mean. A strong current speed is more pronounced in the vicinity of the entrance, where it can exceed 3 m/s.

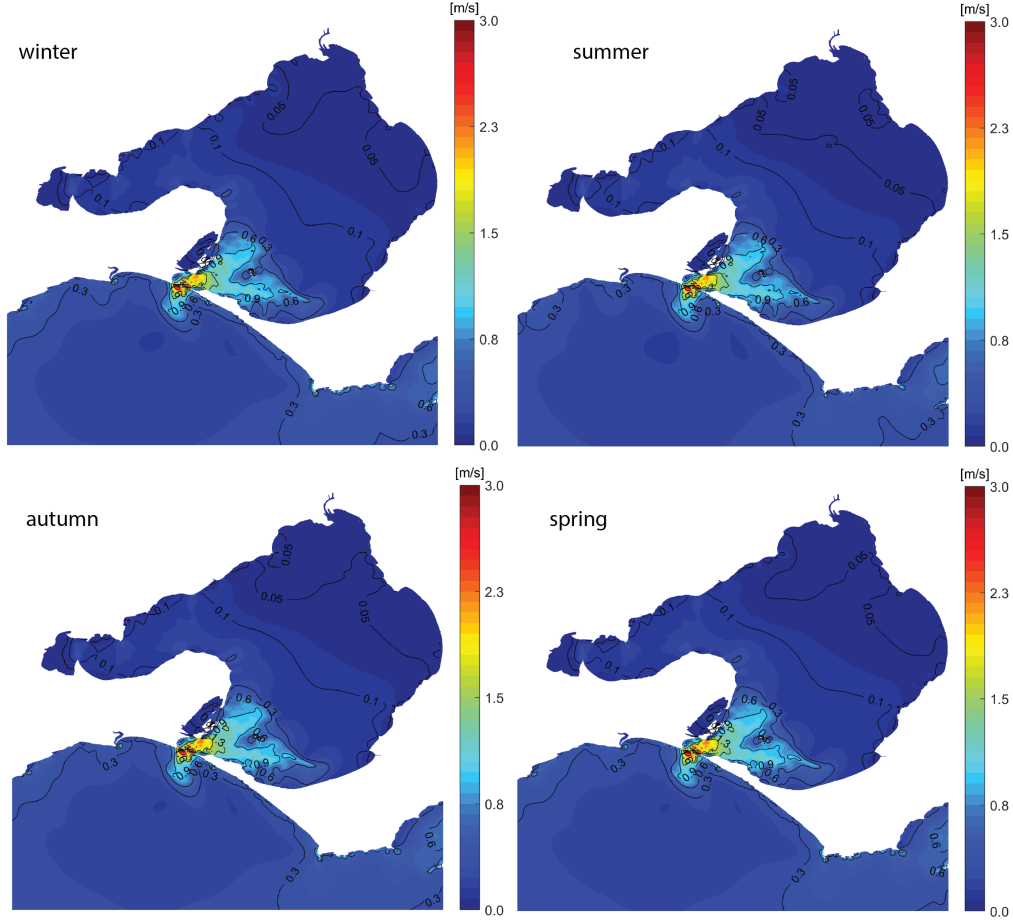


Figure 6.9: Maps of extreme currents over PPB. The mean magnitude goes beyond 3 m/s in the vicinity of PPH; however, after passing through the southern region, the current speed is no higher than 0.1 m/s.

6.3.4 Wind-driven circulations

Within PPB, wind-driven circulation is expected to follow the wind pattern. However, this is a complex issue and not yet fully explored and understood (e.g. *Cardno Lawson Treloar*, 2007b). To provide a better understanding of wind-driven circulation, this study undertook realistic simulations to examine seasonal variations in wind-circulation patterns in extreme conditions. Four storm events were configured for PPB. The model is forced with wind only (i.e. no tides). Results of wind-driven patterns together with the wind forcing fields are shown in Figure 6.10.

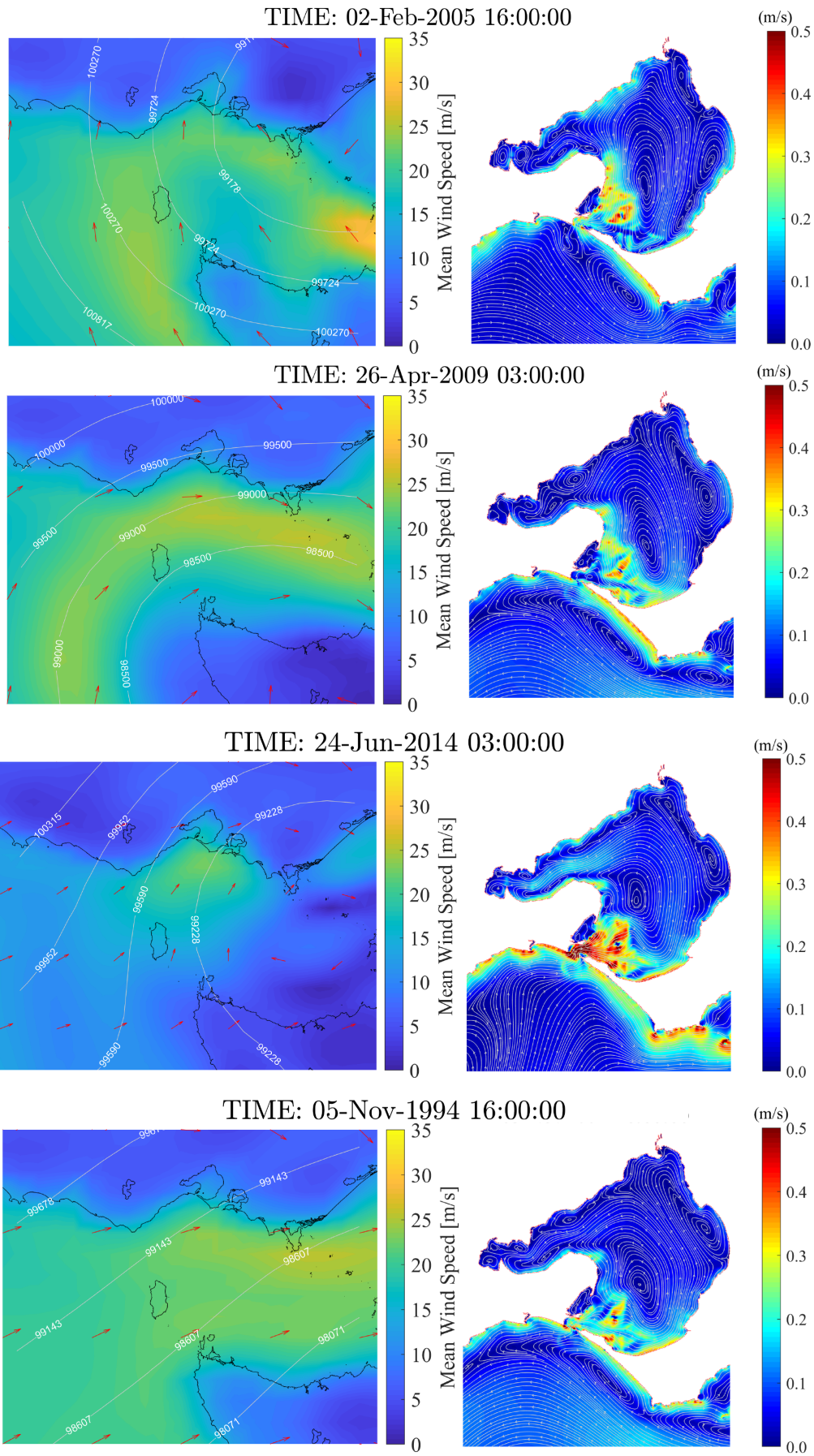


Figure 6.10: Wind-driven circulations during extreme events.

A similar pattern in wind-driven circulation across the four storms is evident in the two gyre systems. The major gyre is centred in the middle of the bay while the minor gyre depends on wind direction but is located near the shore. These two gyres always rotate in an opposite orientation to each other. In the first event, which occurred on 2 February 2005 at 04:00 PM (summer), the prevailing southerly wind is nearly perpendicular to the vertical axis of PPB. The wind-driven circulation inside PPB mainly rotates in a clockwise direction, creating the major gyre. The minor circulation system is located near the eastern coastline, and rotates in a counter-clockwise direction.

The other three events occurred on 26 April 2009 at 03:00 AM (autumn), on 24 June 2014 at 3:00 AM (winter) and on 5 November 1994 at 4:00 PM (spring), under a prevailing south-westerly wind. In these events, the major gyre system rotates in an counter-clockwise direction while the minor gyre travels in a clockwise direction. The location of this minor gyre system in the case of the south-westerly wind is always in the north-eastern side of PPB - the direction towards which the wind blows. The presence of two gyres may be attributable to the weather system because PPB is located on the boundary between the subtropical system from the north and the westerly wind belt from the southern hemisphere (e.g. *Cardno Lawson Treloar*, 2007b). Further, it is clear that wind-driven currents are also generated inside PPB, mainly concentrated in the shallow regions in the south of PPB around Great Sands. Obviously, the wind direction also influences the magnitude of these currents. For example, for the event on 24 June 2014 at 03:00 AM, the entrance of PPB was responding to the strong southwesterly wind, which travelled at a right angle to the entrance at about 210 degree. This wind created a strong wind-driven current, up to 0.5 m/s, covering a large region from PPH to Sand Island. Similarly to inner PPB, when two gyres are created outside in Bass Strait, the rotational convention of these systems are the same as inside PPB.

6.4 Conclusions

This chapter examined the climatology of hydrodynamics in PPB with the aid of numerical modelling and available field observational data. Modelled data were based on both long-term hindcasts and case scenarios. It focused on three major aspects of hydrodynamics: water levels, waves and currents. The first two aspects focused on seasonal variations and extremes. Estimates of return periods were also made for water levels (storm tides) and SWHs. Case scenarios were also conducted to examine the contribution of waves to extreme sea levels and wind-driven circulation.

For sea levels, this study shows that the tidal pattern and storm tide levels across a large part of PPB are nearly the same. Therefore, the sea level over a large portion of the bay is almost flat, including for the extreme sea levels. At the extremes, the highest storm tide level often occurs in the east coasts of the bay, in line with the wind pattern. For example, the frontal low weather system often results in the wind blowing from west to east. The wind pattern also affects the wave field. During summer months, wind often blows from south to north, while in winter, wind often travels from north to south. Consequently, the wave direction is strongly affected by seasonality and wind patterns. The SWH in extreme

conditions in PPB can be higher than 2m, and larger waves often occur on the eastern side of PPB. Within PPB, waves can contribute around 10%, but less than 15% of extreme sea levels. Current speed is only strong in the vicinity of PPH, and is very weak in the central and northern regions of PPB. In storms, wind can generate a current of up to 0.5 m/s in the southern region of PPB, mainly in the shallow area of Sand Island. For wind-driven circulation, there are often two different gyre systems occurring in PPB. This may be attributable to the weather system as PPB is situated near the boundary between the subtropical system from the north and the westerly wind belt from the south.

This study also estimated return periods for storm tides (extreme sea levels) and SWHs, and compared with previous studies. For 100-year return levels of storm tides, this study provided lower values than those proposed by *McInnes et al.* (2009), who did not consider waves in extreme sea levels - their results were based on simulated data derived from a circulation model alone, while the present study used a coupled wave-circulation model. Further, using different methods to estimate return levels can yield different results. For the return levels of SWHs, this study found that the 100-year return level is similar to the maximum SWH for the 27 years of hindcast simulations. It was also found that *Association of Bayside Municipalities* (2017) predicted 10% lower values of 100-year return values in the southern region of PPB. This may be due to the fact that *Association of Bayside Municipalities* (2017) did not consider offshore waves propagating into the bay.

In conclusion, this study undertook a comprehensive assessment of hydrodynamic climatology in PPB based on long-term simulations, which had not been done for these regions. Simulated data used for data analysis were derived from numerical models, which were carefully calibrated and validated against available field observations. In particular, the coupled numerical modelling system used in this study was integrated with the updated version of the spectral wave model WWMIII, using the observation-based physics ST6 with high resolution, rendering our results highly reliable.

Chapter 7

Effects of channel deepening on hydrodynamics and coastal erosion in Port Phillip Bay

7.1 Introduction

Coastal waters have economic, environmental and social significance for port navigation, fishing and recreational activities (*Mendes et al.*, 2016; *Ramesh et al.*, 2015). With population growth and global trade development, human-induced pressures have grown in these regions (*Mentaschi et al.*, 2018). Many ports around the globe have been expanded to serve large vessels (*Van Maren et al.*, 2015), resulting in channel deepening (*Bradford and Siebinga*, 2009; *Liria et al.*, 2009) and land reclamation (*Shao et al.*, 2018). These human-induced actions have disrupted and modified the pristine state of coastal environments, leading to significant changes in hydrodynamics. For example, in a case study for Oka estuary (Spain), *Liria et al.* (2009) showed that dredging channels can lead to a loss of tidal prisms, which can reduce ebb flows and modify the current profile. Similarly, *Lopes and Dias* (2015) investigated the impacts of channel modification on tidal properties, and concluded that tidal properties are significantly affected by the geometrical shapes. Similarly, *Zarzuelo et al.* (2015) pointed out that channel deepening can increase residual currents, enhancing erosion and deposition processes.

In PPB, the port authority plans to increase capacity to handle 8 million containers by 2035 with 7,000 TEU ships and a 14m draught at all tidal conditions. In line with this, a capital dredging project was carried out during 2008-2009 to accommodate larger vessels in the future. The major dredging works were located in the South Channel (see Figure 3.2E for approximate changes). This channel system was environmentally deepened up to 17.3m to achieve the designed water depth (*Bradford and Siebinga*, 2009), with the largest changes up to 3m, involving approximately 15 million m³ of sediment, mainly sand material (*Minister of Planning*, 2007).

7.2 Methodology

To investigate the impacts of channel deepening, long-term hindcast simulations from 1990 to 2016 were applied for two bathymetry options, which were mapped

before and after the channel deepening project with fixed shorelines and the same forcing model SCHISM-WWM configurations presented in Chapter 5. Additionally, several test cases were explored to investigate the change of swell propagation into PPB. To do so, simulations were carried out with only waves at the open boundary and no atmospheric forcing or currents. It is known that ocean waves

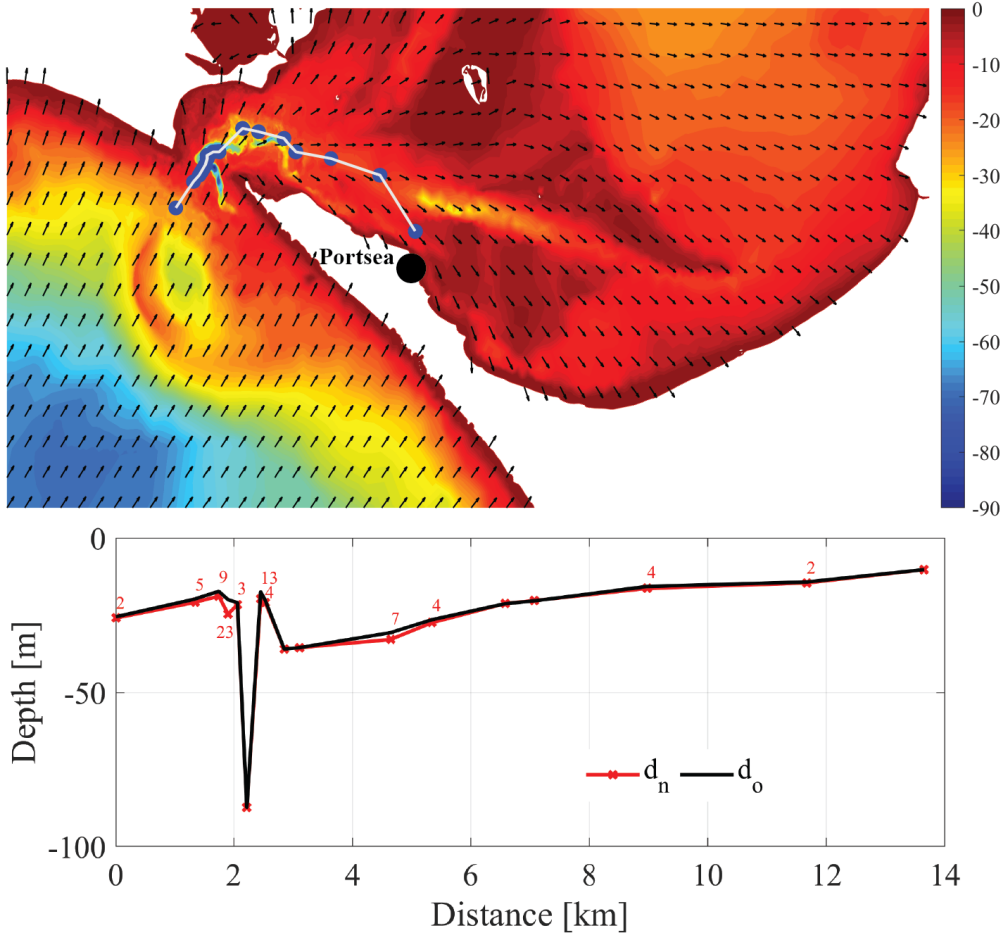


Figure 7.1: Top panel shows a typical pattern of mean wave direction (black arrows) in the southern portion of PPB where ocean waves (swells) propagate onto the entrance of the Bay with a regular direction. After passing the entrance, waves get refracted and change their direction toward the coasts (e.g. Portsea). This pattern of wave propagation even exists before channel deepening. Bottom panel plots the water depth prior to d_o and after d_n channel deepening along tracking points (shown in top panel; starting point is the furthest point offshore) with a relative change (%) in the water depth.

are refracted after passing the entrance of PPB towards the coasts (e.g. Portsea). This phenomenon occurred even before the channel deepening project. Therefore, data from a number of points along the shipping channel towards coastlines were extracted. The tracking path is shown in Figure 7.1. Data from these points were then analysed in more detail. To assess the impact of channel deepening on coastal erosion, a storm event was also simulated to assess the morphological change due to the difference in the bathymetry. This morphological assessment was conducted with the aid of the XBEACH numerical model (Roelvink *et al.*, 2009); the model's

features are presented in Appendix A2. The XBEACH model was forced with the hydrodynamic conditions produced by SCHISM-WWM.

7.3 Results

7.3.1 Swell propagation

The use of an extended open boundary further offshore ($\sim 200\text{km}$) has allowed this study to examine the contribution of swells alone to the wave profile inside the inlets of PPB and changes due to bathymetry modifications. Such research has not been conducted thoroughly in the past. For example, previous studies on waves in PPB either did not use open boundary forcing (e.g. *Association of Bayside Municipalities*, 2017) or only used wave buoys near the coast (e.g. *Rapizo et al.*, 2017) to simulate waves for the southern portion of the bay. However, wave buoys near PPH may contain much wind sea during storms as wind blowing over a large distance towards the entrance can add up to several metres of wave height to the wave buoys. Thus, the swell contribution to the wave profile within the southern region of PPB may be relatively underexamined. In this study, several test cases were conducted using the wave model alone (i.e. without wind/atmospheric forcing and tides). Simulations were run for bathymetry both prior to and after channel deepening. Figure 7.2 presents maps of SWH in the covered southern regions of PPB during mild (e.g. before the storm at 10AM on 23 June 2014) and extreme conditions (during the storm at 10:00AM on 24 June 2014). It is clear that swells

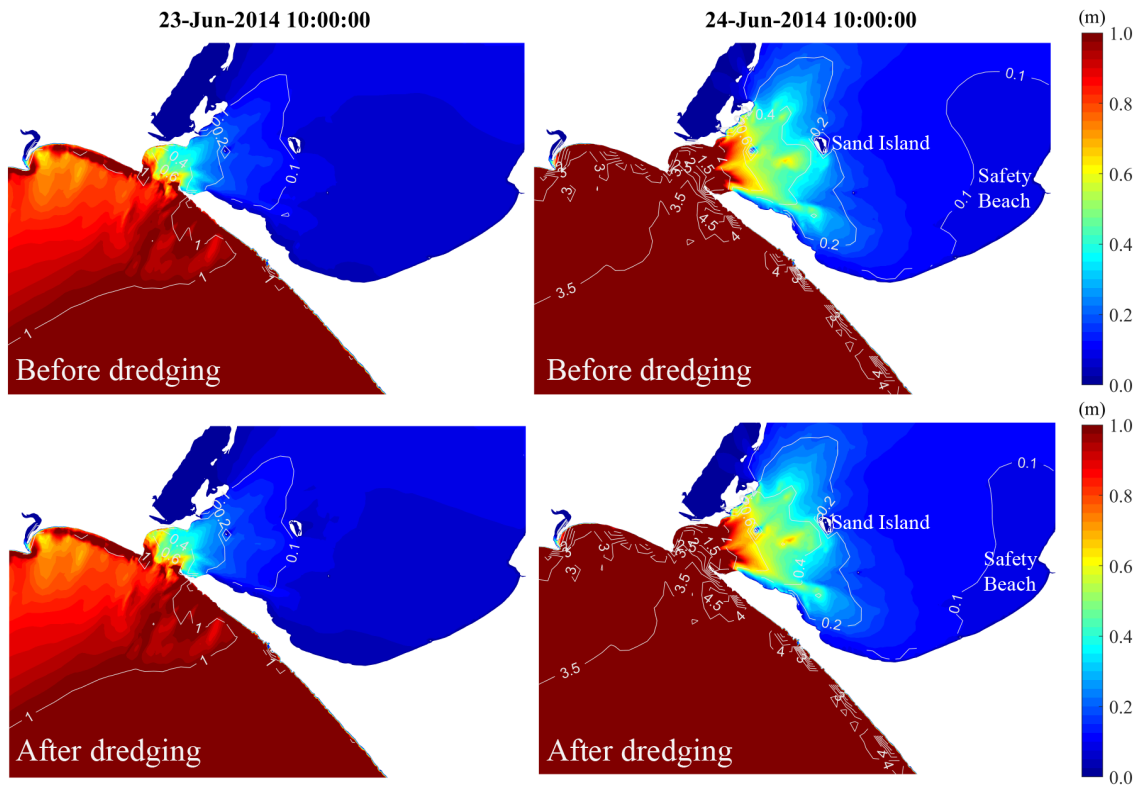


Figure 7.2: Contribution of swells to the waves in the southern portion of PPB before the storm (at 10 AM on 23 June 2014; figures in the first column) and after the storm (at 10 AM on 24 June 2014; figures in the second column) for bathymetry before (top row) and after (bottom row) channel deepening.

dissipate energy quickly after passing the head of PPB into the inlet. In a mild sea state condition, swells add up to 0.1m to the SWH covered up to Sand Island only. In this case, the important feature is that bathymetry modification does not cause any difference in the wave profile in this region. In extreme conditions, however, the area of SWH extends further into the body of PPB. As shown in the figure, the post-dredging bathymetry could allow more waves towards Safety Beach (e.g. the 0.1m contour line closer to the shoreline). Further, this figure also shows that the updated bathymetry after channel deepening slightly increases the wave height in front of Sand Island, resulting in a SWH of 0.4m propagating closer towards this island. It is important to stress that Figure 7.2 presents results for a test case before and during a storm that occurred in PPB on 24 June 2014, based on simulations that did not consider the influences of wind forcing, tides and currents. Results may differ when those effects are taken into account. However, Figure 7.2 can provide some important information. First, the deepening of the channel may modify the wave height in the southern region of PPB. Second, the wave climate inside PPB could be underestimated if the influence of swells or offshore waves is neglected because even 0.1m of SWH can contribute up to 10% to the total wave profile in this region during extremes. Therefore, this may be the reason why estimates of 1% AEP in *Association of Bayside Municipalities* (2017) for points in the southern area of PPB were lower than results from the present study (see Chapter 6), as *Association of Bayside Municipalities* (2017) assumed no waves propagating onto the southern area of PPB.

7.3.2 Hydrodynamic patterns along tracking points

To examine the influence of water depth due to dredging on hydrodynamic patterns in the inlet regions of PPB, first, data along a tracking path (Figure 7.1) were extracted for data analysis. In this regard, current speed (Cur.sp) and direction (Cur.di), water levels, SWH (H_s), mean direction (MeanDir), wave steepness in the form of $kH_s/2$ (e.g. *Longuet-Higgins*, 1969) and the growth amplitude U/C_g from before and after bathymetry data simulations were analysed and plotted. Several patterns, for a strong ebb tide, for a ‘medium’ ebb tide, for a strong flood tide, and for a flood tide near slack waters, are presented in Figures 7.3, 7.4, 7.5 and 7.6 respectively. In the case of strong ebb tides, results in Figure 7.3 show a hydrodynamic pattern at 8:00 AM on 5 November 1994, when the wave height is about 4m at PHH, and the current speed at PPH approaches 3m/s, which could lead to wave growth reaching strong breaking conditions $kH > 0.3$ and blocking limits $U/C_g \sim 0.5$ (*Banner et al.*, 2000; *Chawla and Kirby*, 2002; *Peregrine*, 1976; *Toffoli et al.*, 2015; *Zippel and Thomson*, 2015). Before entering the deep canyon, the H_s of the post-dredging bathymetry increased because of interaction with a strong current. For example, at points 3, 4 and 5, a 2% increase in current speed (see panel a) is likely to steepen the wave height by 2% (see panel d). An increase in the current speed at these points is due to the deeper channel, which may lead to an increased flow passing through this confined section. As a result, the post-dredging bathymetry would cause more wave breaking at points 8, 9 and 10, which can be clearly seen by looking at the SWH (see panel d), the wave steepness (see panel e) and the growth amplitude (U/C_g).

Especially at point 11, waves in both bathymetry options may reach the blocking

limit $U/C_g \sim 0.5$. The wave steepness significantly dropped from point 12 to point 15 ($kH_s/2 \sim 0.1$) as the wave height decreased. However, waves start steepening towards the coasts (e.g. Portsea) due to shoaling effects. H_s of post-dredging bathymetry may be higher than before dredging in the inlet region, because of the localised tidal modulation on the waves as water elevation (see panel b) modulated by tidal patterns in the case of post-dredging bathymetry decreased by up to 6% along these points. The effect of tides on wave steepness can be derived from the linear approximation since wave steepness depends on the wave number k , defined as $2\pi/L$, while the wavelength in shallow water: $L = T\sqrt{g.d}$ depends on water depth. Therefore, a drop in the depth would decrease the wavelength and increase the wave number affecting on the wave steepness. A similar pattern is observed -

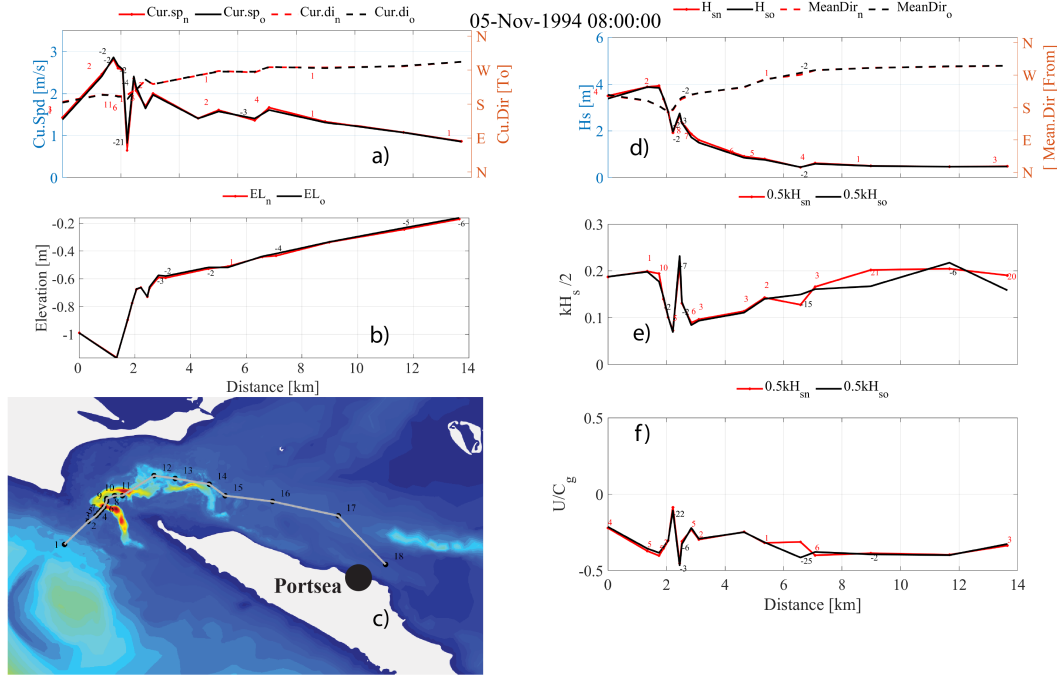


Figure 7.3: Hydrodynamic patterns at 08:00 AM 05-November 1994 during a strong ebb tide along tracking points (in panel c). The first point starts from the outside inlet and the last point is near Portsea. In this figure, subscripts 'o' and 'n' stand for modelled data from before and after channel deepening respectively. 'red' and 'black' numbers are increased and reduced percentages (%) due to channel deepening. A comparison between modelled current speed ($Cur.sp$) and direction ($Cur.di$) shown in panel a, water level in panel b, significant wave height and mean direction in d), wave steepness $0.5kH_s$ (k is wave number in panel e), the amplitude growth U/C_g in panel f. In this case, the H_s of post dredging bathymetry increased before entering the deep canyon but dropped inside it.

the H_s of post-bathymetry increased before entering the canyon but decreased inside it. Another event present in Figure 7.4 occurred at 12:00 PM on 17 April 2009 during a 'medium' ebb tide (current speed of around 2 m/s at the entrance and wave height of 2m). Further, the shoaling process in the post-dredging data in the inlet region also increased compared with prior to dredging. For example, H_s of the post-channel deepening data are small, but increase by 3-5% from point 15 to 16. At the same time, wave steepness also increases by 8-16% compared with

the before-dredging data. This may be explained by considering the influence of tidal patterns on the wave field. As can be seen in panel b, the water elevation drops by 8% at point 17 before reaching the end point. In the case of flood tides,

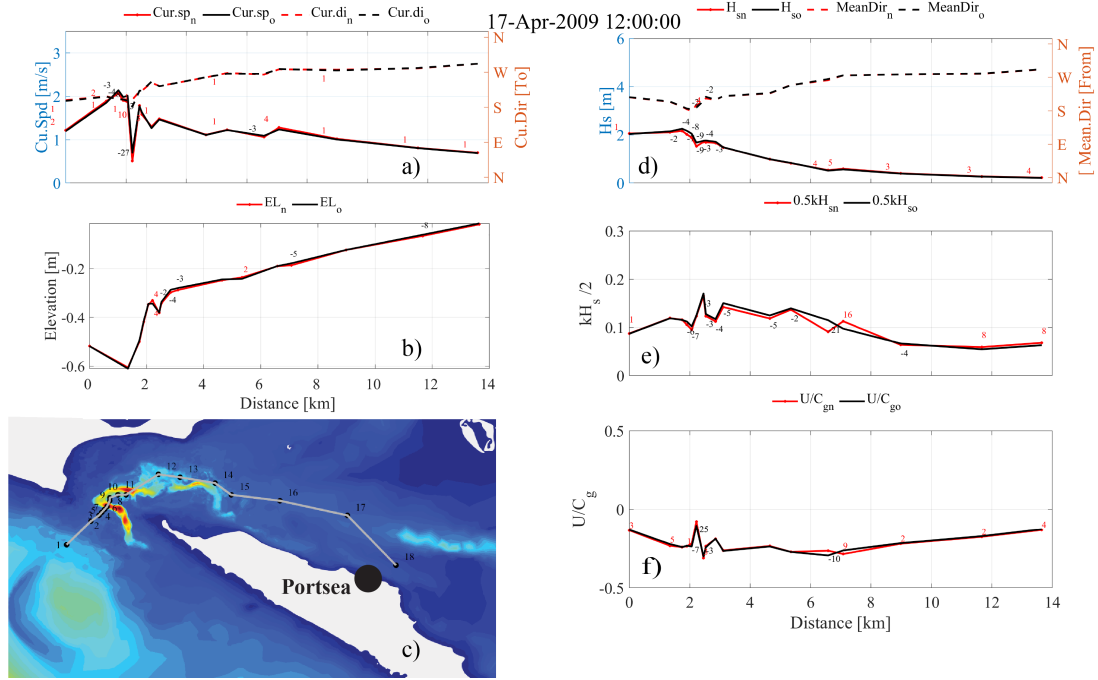


Figure 7.4: Hydrodynamic patterns at 17-April-2009 12:00 PM during a ‘medium’ ebb tide along tracking points in c. Similar to event in Figure 7.3, the ‘post-dredging’ significant wave height slightly increases (e.g. 1%) before entering onto the canyon but dropped inside it. A clear evidence can also be seen from panel e, where the wave steepness modelled by the post-dredging bathymetry is lower than before dredging.

Figure 7.5 shows the hydrodynamic pattern along the tracking path at 02:00 AM on 11 April 2009 during a strong flood tide when the difference between the water elevation outside and inside PPB was about 1m (see panel b). As indicated in panel a, the current speed modelled by the post-dredging bathymetry becomes stronger towards the inlets. This flow pattern could affect wave propagation in this region. Particularly compared with before dredging, panel d shows the SWH modelled by deeper bathymetry was flattened at the entrance (e.g. points 9, 10, 11, 12) of PPB due to stronger currents. This phenomenon is well explained in the literature (e.g. *Peregrine, 1976; Wolf and Prandle, 1999*). When the wave direction of propagation is in the same flow orientation, waves undergo a Doppler shift, adding to the speed of wave propagation. The largest flattening of the rate occurred at points 12 and 13, where the currents and waves are no longer confined to the narrow channels, as this is the turning area for ship navigation (e.g. see panel a in Figure 7.8 for the shape of the shipping channel). At this point, both currents and waves also start changing direction towards the South Channel (see panels a and d for direction of currents and waves). However, as indicated in panel d, the post-dredging H_s from points 15 to 18 are 7-9% higher than before dredging, which could be caused by the difference in water levels, current speed and direction. Figure 7.6 presents another event that happened at the end of a flood tidal cycle near the slack waters, when the current speed at the entrance of

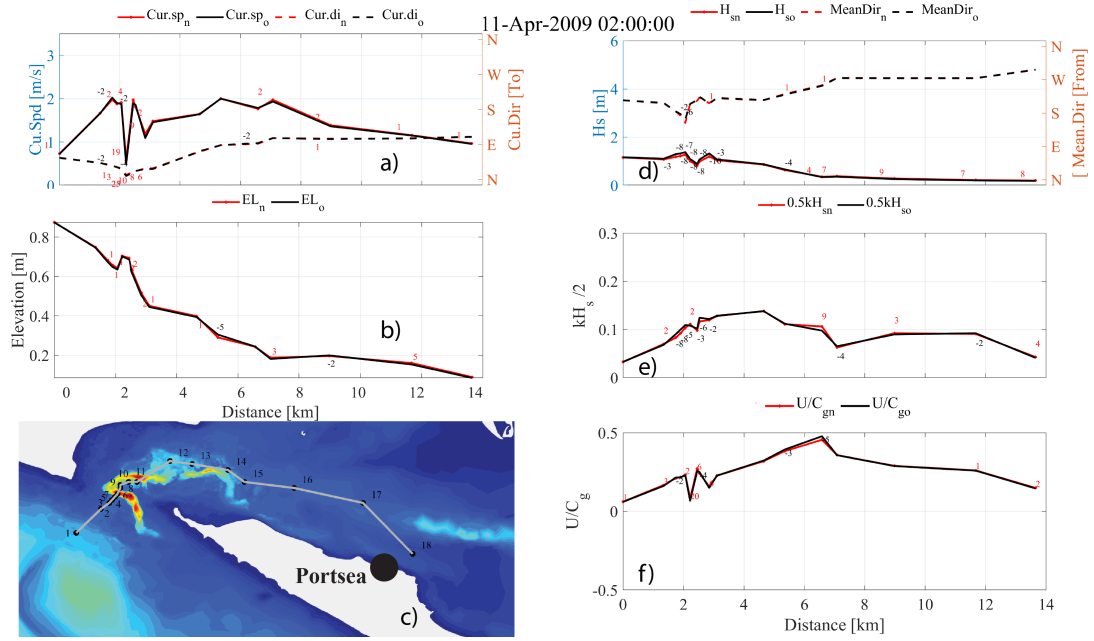


Figure 7.5: Hydrodynamic patterns at 02:00 AM 11-April-2009 during a strong flood tide along tracking points (panel c). The post-bathymetry features could produce stronger which could then flatten the wave height at the entrance of PPB.

PPB is considerably small. The difference in water elevation between inside and outside the inlets is about 0.3m (see panel b). In this case, water starts flowing out of the inlets, but the current speed is still weak. This is an extreme event as the wave height is higher than 4m at the entrance and about 1m in the inlets along the tracking path. Because of a weak current near the slack tide, waves are almost free to propagate into the inlet towards the coast. Therefore, it can be seen that more shoaling occurs at points 16, 17 and 18 in the case of the post-bathymetry data.

In summary, Figures 7.3, 7.4, 7.5 and 7.6 provide evidence based on hydrodynamic patterns along the tracking path that deeper channels caused by dredging could modify the hydrodynamics in the tidal inlet systems. For example, the deeper channel could increase the flow at the entrance of PPB. As a result, the flattening rate of the wave growth amplitude U/Cg would increase. Conversely, more wave breaking could occur due to stronger currents. Further, there is an increasing trend in shoaling in the vicinity of the coastline towards Portsea. In an ebb tide, it is found that the water level modelled in the post-dredging bathymetry scenarios decreased. This reduction in water depth would increase the shoaling process along the coast. More shoaling was also found during slack waters and extreme conditions.

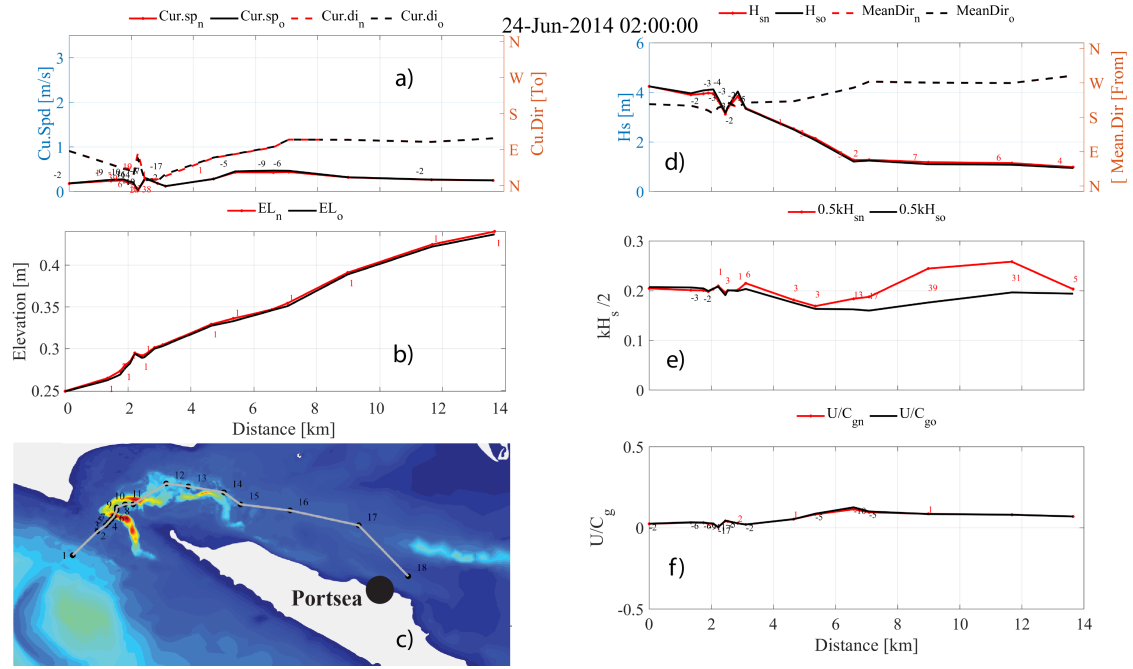


Figure 7.6: Hydrodynamic patterns at 02:00 AM on 24 June 2014 during a flood tidal cycle but near the slack tide along the tracking path (see panel c for locations of points). Evidence of near slack waters can be clearly seen in the wave growth amplitude in panel f, as U/C_g is positive but near zero at the entrance. The current speed inside the shipping channel modelled by post-dredging bathymetry data is weaker than before dredging.

7.3.3 Changes in sea levels

The change in sea levels due to channel deepening is investigated based on 27-year hindcast data from 1990 to 2016. First, it is noted that a comparison between the MSL before and after channel deepening did not detect a difference between these two bathymetries as the mean water level is generally small (e.g. only a few millimetres). Thus, the absolute difference between the mean value of each bathymetry is close to zero. Given this, only changes in high percentile levels are examined further.

Figure 8 shows the ‘relative difference’ in extreme sea level (ESL) caused by deeper channels. Accordingly, the largest change occurs in the vicinity of shipping channels where dredging works were undertaken, which now experience increased shoaling due to wave contributions, as indicated in previous sections. Inside PPB, the largest increase in ESL, of up to 1%, occurs in the western part (e.g. Geelong) of PPB during the autumn. This pattern also highlights that the impact of channel deepening also depends on seasonality. For example, the largest increase in ESL during the winter, which occurs in the western area, is about 0.8%, which is 0.2% lower than in the cold seasons.

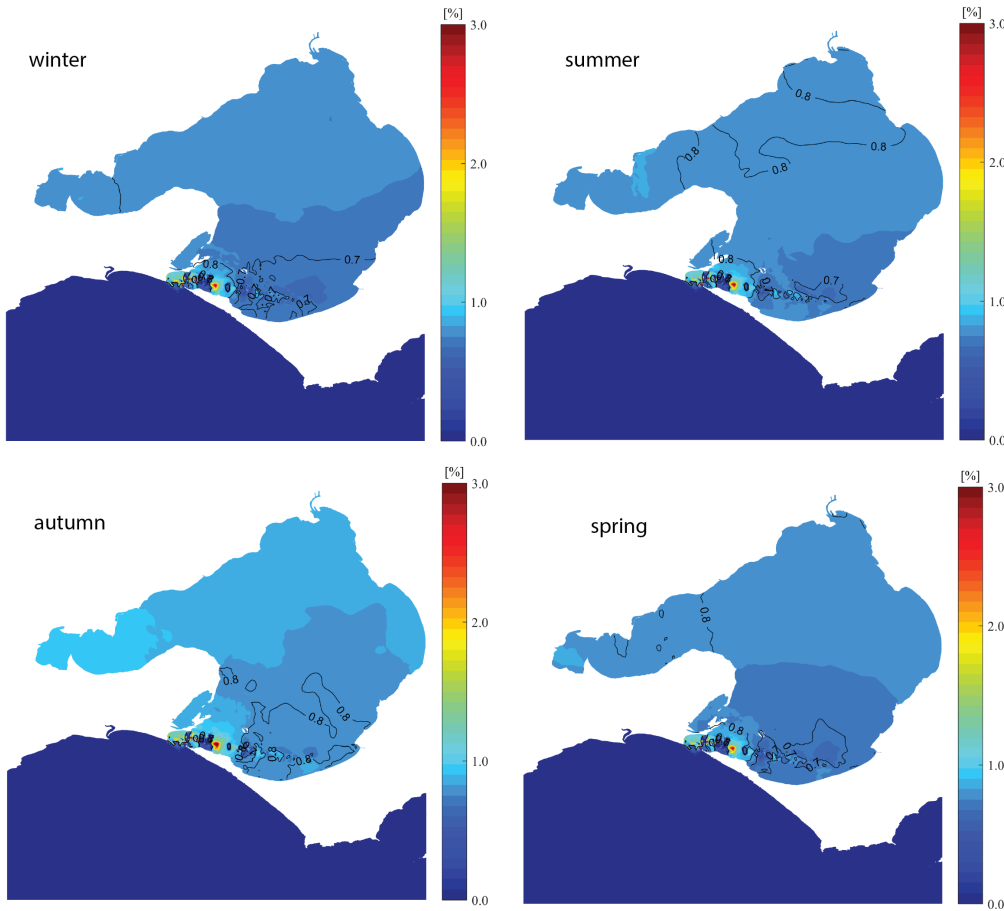


Figure 7.7: A relative change in the extreme (99th percentile) water level.

7.4 Changes in currents

Currents within PPB are dominated by tides (e.g. *Cardno Lawson Treloar*, 2007b). Figure 7.8 shows the most extreme current speed, calculated for the 99th percentile for all tidal cycles, ebb tides and flood tides from 27 years of current hindcast data. It is found that current speed significantly increases by 5-7% in the region (zone 1 as marked) curved by the offshore shallow region (c-shaped curve, refer to Figure 3.2 for the shape of the offshore bar). These significant changes occur during the ebb tides, as shown in panel C when the current goes towards the ocean (out of the bay). The same rate of change occurred during flood tides (panel D) at the end of the channels as marked in Zone 2. These changes are due to deeper channels, which can lead to increased flow passing through this section. These changes in current

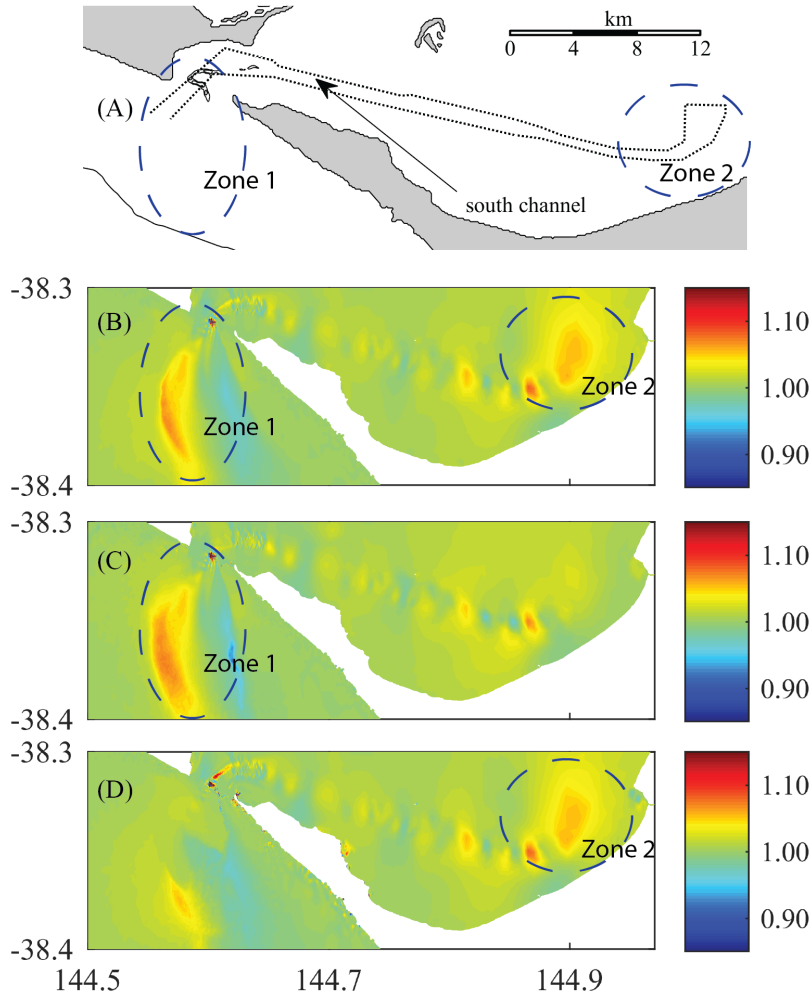


Figure 7.8: Plots for 99th percentile current speed. (A) A map marks two different zones of significant changes at the entrance of the Bay (Zone 1 as marked) and the end of shipping channels (Zone 2); For all tidal cycles (B) changes happened at both zones. Changes in Zone 1 correspond to ebb tides (C), and changes in Zone 2 correspond to the flood tidal periods (D).

speed are expected to influence the wave field. For instance, a stronger current speed at the entrance of PPB could result in more wave breaking as waves interact with currents in the ebb tide. Conversely, stronger currents during a flood tide can flatten wave height.

7.5 Changes in the wave pattern

This study found that channel modifications only change wave conditions in the vicinity of the entrance, over an area that extends 20km from the South Channel system.

Figure 7.9 shows the possible change (%) for SWH in the southern portion of PPB for mean values. There is a reduction in SWH in the vicinity of PPH for all tidal cycles, consistent with previous results, due to stronger currents as deeper channels allow more flow to pass through the restricted narrow entrance, resulting in more wave breaking during ebb tides and increasing the flattening rate of the wave height during floods. However, in the inlet region, the SWH may increase by up to 10% along the coastline nearby Portsea and Sand Island, as shoaling processes are more pronounced. Further, changes in tidal patterns may be attributed to changes in the wave field.

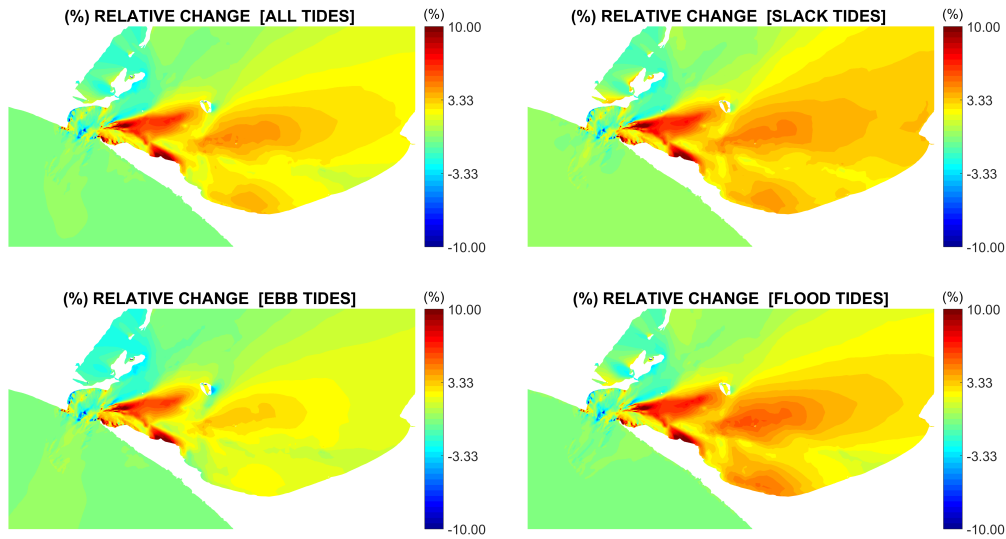


Figure 7.9: Possible changes in SWH (%) in the southern region due to deeper channels. At the entrance, there is a decreasing trend in SWH for all tidal cycles, as the deeper channel increases the flow magnitude in this region. As a result, more wave breaking occurs during an ebb tide in line with interaction with the strong current. Conversely, stronger flow may flatten the wave height during flood tidal cycles due to Doppler effects. The largest change may occur in the coastal regions due to an increase in shoaling processes caused by the deeper channel.

7.6 Impacts of channel deepening on coastal erosion

Impacts of channel deepening on coastal erosion were examined with the aid of the XBEACH model (Roelvink *et al.*, 2009). Realistic simulations under storm conditions were undertaken for the event of 24 June 2014. To reduce computational costs, XBEACH only covers the southern portion of PPB. The hydrodynamic conditions from SCHISM/WWM were used to force XBEACH. The forcing input data include water levels, wave spectra and wind forcing. Panel a of 7.10 shows hydrodynamic conditions at the entrance of PPB during the storm. The SWH at the entrance is about 7m during the ebb tidal cycle - this is considered a 100-year event for PPB, as indicated in previous chapters.

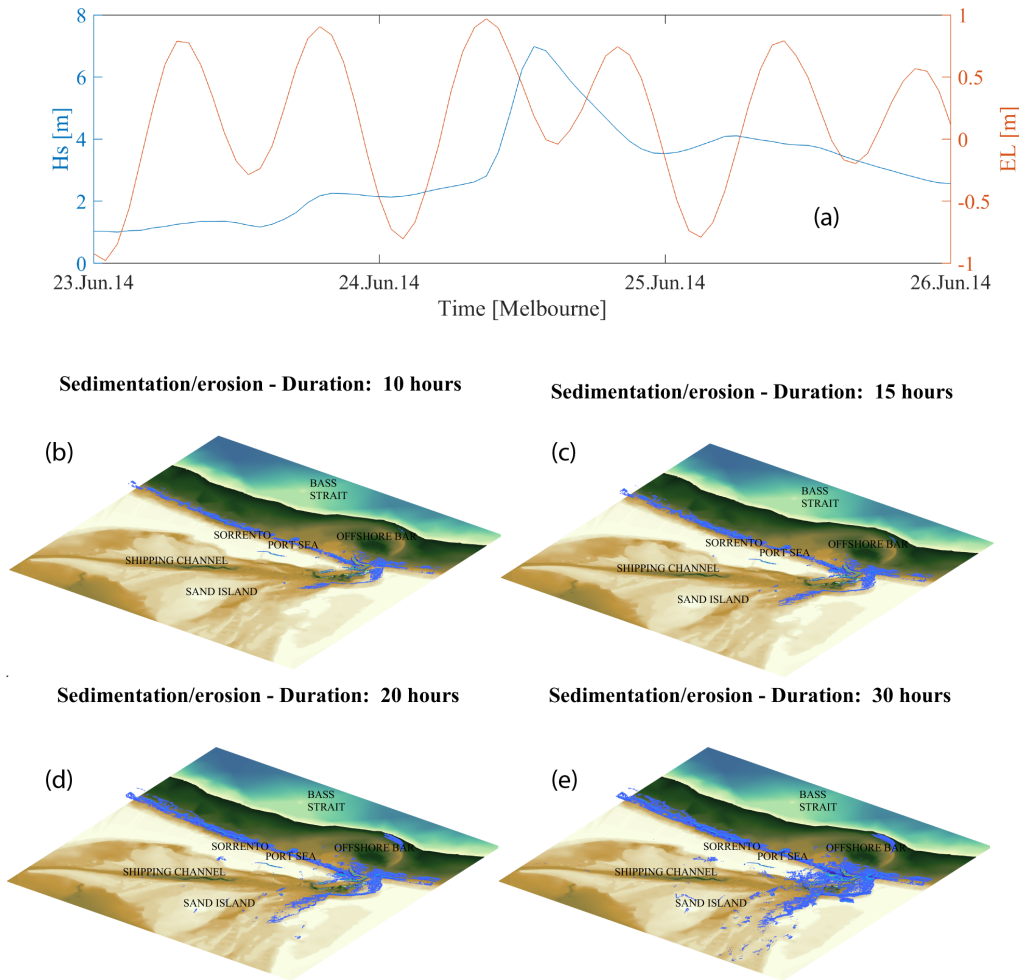


Figure 7.10: Hydrodynamic conditions during a storm event 24 June 2014 and coastal erosion pattern in the southern portion of PPB.

Figure 7.10 b–e present developments in erosion occurring along the coastline from 10 30-hour simulations for 23 June 2014. Outside PPB, high erosion occurred along offshore coastlines due to the exposure of these coasts to ocean waves. Inside PPB, after 10hrs of simulation, high erosion occurred along the coasts towards West Channel Pile due to the waves that propagated towards the bay through the entrance. At this time, no erosion around the Sand Island could be observed but a short section of coastline at Portsea was eroded. However, the erosion progressively developed over time with the arrival of the storm at noon on 24 June 2014. Panel e shows the results for 30 hours of simulation, matching the timing of the storm peak. As shown, erosion occurred not only at Portsea, but also in the region towards Sand Island and on the Sorrento coast. It is noted that there are no field measurement data available to validate these erosion results – this will be left for future studies.

Satellite images can provide more evidence to indicate a change in coastlines in the vicinity of coastal areas in the southern portion of PPB. Figure 7.11 compares several satellite images captured by Landsat 5, 7 and 8 from 1999 to 2019. The top panel shows a brighter colour (red arrow) in 2019 and 2013 compared with

previous years (2009 and 1999), implying that there is more sand exposure along this coastal region in recent times. From these images, it can also be seen that the new channel has become wider, while Sand Island is smaller than 20 years ago.



Figure 7.11: Changes in coastlines captured by Landsat 5, 7 and 8. The top panel (2019, 2013) is notably brighter in colour than the bottom panel (2009, 1999) in a region (red arrow) between Portsea and Sorrento, PPB.

7.7 Summary

This chapter is dedicated to examining the impacts of channel deepening on hydrodynamic fields including water levels, waves and currents and on coastal erosion based on numerical simulations. This study found a significant change of up to 10% for the current speed in the vicinity of shipping channels where the channel deepening was undertaken. Increasing flows at the entrance of PPB could modify the wave height in this region. On the one hand, more wave breaking could occur in the ebb tide because of an interaction with the current. On the other hand, waves could be flattened during the flood tides. In ebb tides, it is likely that channel deepening has modified water elevations, which could be attributed to tidal patterns. Particularly, more shoaling tends to occur in the coast regions (e.g. Portsea) because of a reduced water depth during these tidal cycles. Regarding possible impacts of deeper channels on coastal erosion, results from XBEACH simulations indicate that the Portsea coastline could be eroded in storm conditions. The evidence from satellite images also revealed more sand exposure and a smaller size in the Sand Island. However, there are no measured shoreline data to validate the XBEACH model results. This will be revisited in a future study.

Chapter 8

Conclusions and Outlook

8.1 Conclusions

This study aimed to improve our understanding of the hydrodynamics of tidal inlet and lagoon systems. This is an independent research effort, focused on PPB, which drew on the aid of numerical modelling and available observational data. PPB is a complex coastal water environment that plays an important role in many areas of socioeconomic and environmental significance linked to more than 4 million people living in Victoria.

One of the main tasks addressed in this study was to upgrade the WWMIII wave-forecast models employed in this study with new observation-based physics and an unstructured grid numeric. Once coding and programming was done, the new WWMIII version was comprehensively tested and validated: the test cases ranged from deep-water waves without swells (Lake Michigan), through large-scale simulations (global) with the presence of swells, to wind-seas in shallow water (PPB). This was an important task, aimed to include physical features in the spectral wave modelling and significantly reduce the number and effect of turning-knob parameters. This upgrade was then integrated with the high-resolution unstructured grid ocean model SCHISM, again following thorough calibration and validation procedures. The second task undertaken was to investigate the climatology of hydrodynamics for PPB, with a focus on three main aspects: water levels, currents and surface waves. Long-term hindcasts were carried out for 27 years, from 1990 to 2016. Modelled data were then analysed to investigate the seasonal variations and extremes of hydrodynamic fields including their return periods. The final task of this research was focused on assessment of the effects of channel deepening on PPB hydrodynamics. For this reason, long-term simulations were conducted for two bathymetry options, mapped before and after the deepening. These long-term hindcast data can provide a unique opportunity to explore changes due to dredging from different angles.

In the first stage, the implementation of ST6 (i.e. the observation-based) physics was undertaken through source-code updates. After that, simulations were conducted to test the model performance for Lake Michigan, where waves are purely generated by wind (no swells). CFSR wind was used to force the WWM testing, with two different physical packages: ST4 and ST6. Two months were

selected for calibration and validation. Modelled SWHs were then compared with available buoy data. Model results revealed that ST6 performs slightly better than ST4 at high wind speeds. At the global scale, simulations were conducted for a one-year hindcast of 2011. Modelled results were then validated through the deep-water buoy measurements from around the US coasts and with satellite altimeter data from five different missions available for that year. A further step was also conducted to test ST6 for shallow water waves at PPB, where waves are generated by local winds only. For this domain, ST6 again outperforms ST4, especially at high wind speeds: ST4 could underestimate SWH inside PPB by as much as 0.4m.

Further, based on the results of tests with 4m offshore swells propagating into PPB under extreme conditions, it was found that up to 0.1m is possible near the Safety Beach coastline. Therefore, it is important to take this wave component into account when considering the wave climate in the southern region of PPB - a 0.1m wave height can add up to 10% to total wave heights inside the bay.

Overall, waves can contribute up to 10-15% of the total surge in extreme sea levels (ESLs) inside PPB. The largest wave setup is likely to occur during the low tide near the slack water. This timing has also been reported in other studies (e.g *Arns et al.*, 2020; *Pinheiro et al.*, 2020; *Zhang et al.*, 2010). The underlying reason for this is that a reduction in water depth can lead to a drop in wave propagation velocity, resulting in greater influence of nonlinear-bottom friction. Thus, it is important to consider tide-surge interactions when predicting ESLs in coastal regions.

Channel deepening in shallow tidal inlet systems can modify the hydrodynamic conditions in adjacent areas of the inlets. This study undertook an assessment for such an engineering modification in PPB. Results revealed that current speed is likely to increase at the entrance of the bay by up to 10%. The underlying reason for this is that the deeper channel can allow a larger water flow to pass through the narrow restricted and confined channels. As a result, more wave breaking may occur during ebb tides due to interaction of waves with a stronger opposing current. Conversely, wave height could be flattened by a stronger current during flood tidal cycles.

This study also revealed that the deeper channel is likely to modify water elevations in the inlet regions. For example, modelled water levels taken from post-dredging bathymetry along the shipping channels are lower than those before the dredging during ebb tides. This does not mean, however, that the channel deepening has led to reduced water depth inside inlets or in the bay as a whole. It only suggests that the channel deepening may have altered the tidal patterns in the inlet regions by causing a phase lag. Consequently, such modifications could affect wave propagation by increasing shoaling effects when waves propagate towards coastlines.

8.2 Outlook

Findings of this research study are not exhaustive because it was based on 2D hydrodynamic models. Model configurations were based on barotropic modes which did not consider baroclinic effects (e.g. rain). The impact of channel deepening on hydrodynamics was not detailed (e.g. change in current residuals, regimes). Further, coastal erosion modelling done in this study was based on sensitivity tests. No shoreline measurements were available to validate XBEACH. Therefore, further work could be focused on the following aspects:

- 3D effects on storm surges and ESLs. There is a number of 3D effects that may be important. Nonlinear bottom friction, which often plays an important role in modelling ESLs, is one. 3D models can provide different bottom stress vectors, which may produce different results. Another effect is wave-induced mixing (unrelated to breaking), which can lead to sediment suspension all the way to the surface in response to storms.
- Compound flooding in coastal inundation (e.g. *Ye et al.*, 2020; *Zhang et al.*, 2020) where the influence of rain and precipitation is considered in storm surge modelling. This consideration is likely to be important in predicting ESLs in the vicinity of rivers or estuary entrances, as rain or precipitation may increase sea levels in these regions. At the same time, the influence of hydrodynamic fields may alter the results in hydrological simulations, which are often decoupled from hydrodynamic models.
- Long-term effects of channel deepening could be explored to investigate the coastal evolution induced by such engineering modifications. This could be done by running long-term morphological simulations – this was not done in this study due to computational expense.
- Modifications of tidal patterns due to channel deepening could be examined in more detail. This may be an important issue for sediment transport and morphological applications. The underlying reason is that a change in tidal patterns could result in tidal asymmetry, which in turn can create an imbalance in influx and outflux of sediment volumes, resulting in more coastal erosion or closure to tidal inlet systems. This issue is also important for engineering applications, including maintaining dredging works, channel design and optimisation.
- The influence of channel modifications could be investigated with regard to biological applications. For example, if the hydrodynamic conditions were modified by channel deepening, this would also change temperature and salinity in the inlet systems, affecting marine species.

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Appendix A

A description of linear wave theory

The most common method to describe surface gravity waves is the small-amplitude approximation or linear wave theory first introduced by *Airy* (1845). This theory describes the random wave field as a summation of an infinite number of harmonic wave components and considers small wave amplitudes compared with water depth. However, additional terms are required to be added to represent waves in infinite wave depth, as in the coastal region. The following sections describe linear wave theory with the added terms to capture behaviours of the wave field in the coastal zone.

Small-amplitude approximation

Consider an arbitrary property D , which could be replaced by mass or momentum per a unit volume. This property travels through a three-dimensional control volume $\Delta x, \Delta y, \Delta z$ within a coordinate system in which x, y components are in a horizontal plane and the z -axis is orientated vertically upwards. During a period of Δt time, the balance equation of this property can be written as follows (*Holthuijsen*, 2007):

$$\boxed{\text{STORAGED QUANTITY} = \text{NET IMPORT} + \text{LOCAL PRODUCTION}} \quad (\text{A-1})$$

On the left of Eq. (A-1), the stored quantity measures the rate of change of quantity D during a time Δt ,

$$\text{STORAGED QUANTITY} = \frac{\partial D}{\partial t} \Delta V \Delta t \quad (\text{A-2})$$

On the right, the first term of Eq. (A-1) represents the import quantity through the control volume. This term arises from particle motion transporting with u_x, u_y, u_z velocity components in u_x, u_y, u_z directions. It is written as:

$$\text{NET IMPORT} = - \left[\frac{\partial D u_x}{\partial x} + \frac{\partial D u_y}{\partial y} + \frac{\partial D u_z}{\partial z} \right] \Delta V \Delta t \quad (\text{A-3})$$

The second term of Eq. (A-1) is local production due to external force F , written as:

$$\text{LOCAL PRODUCTION} = F\Delta V\Delta t \quad (\text{A-4})$$

By combining Eq. (A-2), (A-3) and (A-4), the balance equation in (A-1) becomes:

$$\frac{\partial D}{\partial t} + \frac{\partial Du_x}{\partial x} + \frac{\partial Du_y}{\partial y} + \frac{\partial Du_z}{\partial z} = F \quad (\text{A-5})$$

Now if the arbitrary term D is replaced by the mass term, which is assumed to be constant, then the balance equation in Eq. A-5 becomes the continuity equation, written as:

$$\boxed{\frac{\partial u_x}{\partial x} + \frac{\partial u_y}{\partial y} + \frac{\partial u_z}{\partial z} = 0} \quad (\text{A-6})$$

The next step is to derive the momentum equation of the water density by replacing the arbitrary term D in Eq. (A-4) with the velocity components, which are vector equations. The general form of the momentum equation is:

$$\frac{\partial \rho u_i}{\partial t} + \frac{\partial \rho u_i u_x}{\partial x} + \frac{\partial \rho u_i u_y}{\partial y} + \frac{\partial \rho u_i u_z}{\partial z} = F_i \quad (\text{A-7})$$

where u_i is the velocity in the i -direction, which can be replaced by x,y,z. However, in any (x,y,z) plane, it is important to note that in Eq. A-7, only the first term is kept to obey linear theory because the others are nonlinear terms (e.g. quadratic). Additionally, this theory only considers pressure P and the gravitational term F_g as external forces. Therefore, the momentum equations in x, y, z directions respectively are:

$$F_x = -\frac{\partial P}{\partial x}, \quad F_y = -\frac{\partial P}{\partial y}, \quad \text{and} \quad F_z = -\frac{\partial P}{\partial z} - F_g \quad (\text{A-8})$$

or

$$\boxed{\frac{\partial u_x}{\partial t} = -\frac{1}{\rho} \frac{\partial P}{\partial x}, \quad \frac{\partial u_y}{\partial t} = -\frac{1}{\rho} \frac{\partial P}{\partial y}, \quad \text{and} \quad \frac{\partial u_z}{\partial t} = -\frac{1}{\rho} \frac{\partial P}{\partial z} - g,} \quad (\text{A-9})$$

In Eq. A-9, all terms have a positive sign because the defined convention is positive upwards. The gravitational force only affects the vertical component.

To solve Eq. (A-6) and (A-9), the next task is to specify kinematic and dynamic boundary conditions. For kinematic conditions, it is assumed that, on the surface, the particle may not go beyond the surface. This means that the velocity normal to the surface in the z-direction at points on the surface is the same as the speed of the surface. Further, it is assumed that the water motions do not penetrate at the bottom. These assumptions lead to:

$$\text{at } z = 0 \quad \rightarrow \quad u_z = \frac{\partial \zeta}{\partial t} \quad \text{and,} \quad \text{at } z = -h \quad \rightarrow \quad u_z = 0 \quad (\text{A-10})$$

For the dynamic boundary conditions, it is considered that the wave is free at the surface, leading to:

$$\text{at } z = 0 \quad \rightarrow \quad P = 0 \quad (\text{A-11})$$

To solve the boundary conditions above, the velocity potential function $\Phi(x, y, z, t)$ is introduced. This function considers the partial derivatives of velocity components in x, y, z space. However, it only exists when the water motion is irrotational. Once this condition is met, the velocity components of water particles can be defined as:

$$u_x = \frac{\partial \Phi}{\partial x}, u_y = \frac{\partial \Phi}{\partial y}, u_z = \frac{\partial \Phi}{\partial z} \quad \text{with} \quad \Phi = \Phi(x, y, z, t) \quad (\text{A-12})$$

Replacing these velocity components in the continuity equation (Eq. A-6) gives the Laplace equation:

$$\boxed{\frac{\partial^2 \Phi}{\partial x^2} + \frac{\partial^2 \Phi}{\partial y^2} + \frac{\partial^2 \Phi}{\partial z^2} = 0} \quad (\text{A-13})$$

This equation gives an analytical solution for the kinematic boundary condition in the form of a harmonic wave:

$$\zeta(x, t) = A \sin \omega t + kx \quad (\text{A-14})$$

for which the velocity potential function is:

$$\Phi = \left[\frac{\omega A \cosh[k(h+d)]}{k \sin(kd)} \right] \cosh(\omega t - kx) \quad (\text{A-15})$$

From the kinematic perspective, the derivation of this function (Eq. A-15) does not require any aspect of wave dynamics. This means all kinematic parameters can be derived independently without worrying about forces. In terms of dynamics, because Eq. A-9 can be rewritten by substituting Eq. A-12, it becomes:

$$\frac{\partial}{\partial x} \left(\frac{\partial \Phi}{\partial t} \right) = -\frac{1}{\rho} \frac{\partial P}{\partial x}, \quad \frac{\partial}{\partial y} \left(\frac{\partial \Phi}{\partial t} \right) = -\frac{1}{\rho} \frac{\partial P}{\partial y}, \quad \frac{\partial}{\partial z} \left(\frac{\partial \Phi}{\partial t} \right) = -\frac{1}{\rho} \frac{\partial P}{\partial z} - g \quad (\text{A-16})$$

or

$$\frac{\partial}{\partial x} \left(\frac{\partial \Phi}{\partial t} + \frac{P}{\rho} \right) = 0; \quad \frac{\partial}{\partial y} \left(\frac{\partial \Phi}{\partial t} + \frac{P}{\rho} \right) = 0; \quad \frac{\partial}{\partial z} \left(\frac{\partial \Phi}{\partial t} + \frac{P}{\rho} + g \right) = 0; \quad (\text{A-17})$$

In Eq. A-17, it does not matter if gz is added inside the brackets first and second terms, and g in the third term is replaced by gz because when taking derivatives (in x, y, z directions) gz would become 0 in the first two terms and g in the last term. Therefore, all terms have the same expression inside the brackets. This expression is an arbitrary function. Therefore, it can be simplified to:

$$\frac{\partial \Phi}{\partial t} + \frac{P}{\rho} + gz = 0 \quad (\text{A-18})$$

To meet the dynamic boundary conditions, it is required that $P = 0$ on the surface, which means:

$$\frac{\partial \Phi}{\partial t} + g\zeta = 0 \quad (\text{A-19})$$

Replacing ζ and Φ from Eq. (A-14) and (A-15) into Eq.(A-19) gives the dispersion relationship:

$$\boxed{\omega^2 = gk \tanh(kd)} \quad (\text{A-20})$$

This is the most important expression in the approximation of small-amplitude waves or the linear theory.

Propagation (deep water)

The phase speed can be directly obtained from the dispersion relationship $c = \frac{L}{T} = \frac{\omega}{k}$. The dispersion relationship shows a dependence of the phase speed on the wave number and the wave frequency. For instance, long waves propagate faster than short waves. It is also seen that the speed of propagation of these waves also depends on the wavelength and frequency. In an initial wave field generating from a storm, waves are disintegrated when moving out of the storm, creating a more regular wave field. Among these waves, low-frequency waves travel ahead, leaving the high-frequency waves travelling on the edge. This phenomenon is called ‘frequency dispersion’ (*Holthuijsen*, 2007; *Young*, 1999). Additionally, short-crest waves also transform into long-crest waves as the direction of wave propagation is arbitrary. Therefore, the initial wave field is also disintegrated in various directions - referred to as ‘direction dispersion’. Dispersed waves from an initial place (e.g. the origin of a storm) propagating over the ocean are called ‘swells’.

Wave propagation in a limited water depth

Wave direction changes due to depth-induced variations in the phase speed and the lateral direction (along the crest). Refraction leads to waves propagating towards the coasts. The wave crest travels faster in deep water than in shallow water. Waves always propagate towards the region of lower speed. Refraction can result in an increase or decrease in wave height depending on the actual changes in the wave direction (e.g. *Nielsen*, 2009). The refraction process is often assumed to be slow. Therefore, it can be described via the linear theory. If the variation of the wave amplitude changes rapidly, an expansion of linear theory is required. Further, in the presence of a current, the wave speed can be modified. The well-known phenomenon on the effect of a current on wave propagation is the Doppler, which can alter the wave phase speed. For instance, the angular frequency ω of the wave field would become under a Doppler effect: $\omega = \sigma + k.U$. Therefore, phase velocity becomes:

$$C = \frac{\omega}{k} = \sqrt{\frac{g}{k} \tanh(kd)} + \frac{1}{k}k.U \quad (\text{A-21})$$

The change in wave speed would also cause wave setup and setdown *Longuet-Higgins and Stewart* (1964, 1962). These phenomena are important in the surf-zone *Nielsen* (2009). If waves are not steep, these setup and setdown phenomena can be described by the linear theory. If it changes quickly, Bussiness model or phase-resolving models can be used to replicate these phenomena (e.g. *Roelvink and Reniers*, 2011). If the change is relatively weak, the spectra (e.g. fine resolution) can be computed at a scale of several wavelengths.

Appendix B

A description of XBEACH model

An exciting enhancement of coastal erosion modelling in the last decade was brought by *Roelvink et al.* (2009) with the introduction of the open-source code numerical model XBEACH. This was first developed to study hydro-morphological processes on sandy beaches. Since then, there have been numerous studies applying this model to deal with morphological impacts of storms on sandy and gravel coasts. The model can solve coupled 2D horizontal equations for flow, wave propagation, sediment transport and bottom modification with inputs from varying wave spectral inputs and flow boundary forcing (see https://xbeach.readthedocs.io/en/latest/user_manual.html for more detail). Thus, it can be used to predict various aspects of the hydrodynamic fields including short (wind) and long (infra-gravity) wave transformation, propagation and dissipation such as breaking, refraction, shoaling, wave-induced currents, inundation and overwash.

Morphological processes in XBEACH can be predicted through the computation of bed load and suspended sediment transport, avalanching, breaching and changes in the bed level. Depending on the user's interest, this model can be configured in hydrostatic and non-hydrostatic (resolving) modes. The former has low computational cost because the computation of short-wave amplitude is separated from that of long waves, and morphological changes. This mode does compute the phase of short waves. In the latter, short-wave fields are fully resolved, increasing computational time. Indeed, the non-hydrostatic model was initially developed based on the SWASH model. The purpose of using XBEACH in this research is to simulate the effects of channel deepening on coastal erosion during storm events.

Model descriptions

XBEACH uses the wave action balance equation (e.g. *Holthuijsen*, 2007), written as:

$$\frac{\partial N}{\partial t} + \frac{\partial C_{gx}}{\partial x} N + \frac{\partial C_{gy}}{\partial y} N + \frac{\partial C_{g\theta}}{\partial \theta} N = -\frac{S_{br} + S_{bo} + S_{ve}}{\sigma} \quad (\text{B-1})$$

All three terms on the right of Eq. B-1 account for dissipation processes due to breaking S_{br} , bottom friction S_{bo} and vegetation S_{ve} , which will be explained in detail in the next section. On the left, the first term represents the local rate of change of action density N with respect to time t . The other terms are the net

transport of actions in x,y-space and directional space θ respectively:

$$N(x, y, t, \theta) = \frac{E(x, y, t, \theta)}{\sigma(x, y, t)} \quad (\text{B-2})$$

$$\begin{aligned} C_{gx}(x, y, t, \theta) &= Cg \cos \theta; & C_{gy}(x, y, t, \theta) &= Cg \sin \theta \\ C_{\theta}(x, y, t, \theta) &= \frac{\sigma}{\sin 2kD} \left[\frac{\partial D}{\partial x} \sin \theta - \frac{\partial D}{\partial y} \cos \theta \right] \end{aligned} \quad (\text{B-3})$$

where E is the energy density of each directional bin, D is local depth, k is wave number, C_g is group velocity and σ is intrinsic wave frequency, which follows the dispersion relation:

$$\sigma = [gk \tanh kD]^{0.5} \quad (\text{B-4})$$

The intrinsic frequency in the absence of a current and the absolute frequency ω are the same.

Wave-current interaction

The interaction between waves and currents can affect both wave and current fields. XBEACH predicts wave-current interaction by correcting the wave number using the Eikonal equations. The wave number is divided into two components along the x and y axes:

$$\begin{aligned} \text{x-axis: (cross-shore)} : k_x^n &= k_x^{n-1} + k_x^c \\ \text{y-axis: (long-shore)} : k_y^n &= k_y^{n-1} + k_y^c \end{aligned} \quad (\text{B-5})$$

where k_x^n, k_y^n are the wave numbers at step n and wave numbers with subscript $n-1$ represent the previous step. k_x^c, k_y^c are the wave number corrections due to the presence of a current. These correcting elements are calculated from the Eikonal equations:

$$\frac{\partial k_x}{\partial t} + \frac{\omega}{\partial x} = 0; \quad \frac{\partial k_y}{\partial t} + \frac{\omega}{\partial y} = 0 \quad (\text{B-6})$$

Then the wave number is calculated by:

$$k = (k_x^2 + k_y^2)^{0.5} \quad (\text{B-7})$$

The absolute frequency is now changed to:

$$\omega = \sigma + k_x U + k_y V \quad (\text{B-8})$$

where U, V are the depth-averaged Lagrangian currents in the x, y plane. It is noted that wave-current interaction also affects the wave group velocity in x, y, θ domains. In other words, C_g in x, y, θ domains from Eq. B-1 now becomes:

$$\begin{aligned} C_{gx}(x, y, t, \theta) &= Cg \cos \theta + U; & C_{gy}(x, y, t, \theta) &= Cg \sin \theta + V \\ C_{\theta}(x, y, t, \theta) &= \frac{\sigma}{\sin 2kD} \left[\frac{\partial D}{\partial x} \sin \theta - \frac{\partial D}{\partial y} \cos \theta \right] + \\ &\cos \theta \left[\sin \theta \frac{\partial u}{\partial x} - \cos \theta \frac{\partial u}{\partial y} \right] + \sin \theta \left[\sin \theta \frac{\partial v}{\partial x} - \cos \theta \frac{\partial v}{\partial y} \right] \end{aligned} \quad (\text{B-9})$$

Dissipation processes (S_{br})

Wave breaking: XBEACH considers the wave breaking energy dissipation in several ways depending on type of wave. For instationary waves, the wave energy due to breaking events can be calculated based on *Roelvink* (1993) and *Daly et al.* (2012). The approach based on *Daly et al.* (2012) is presented below. First, the total wave energy is estimated by integrating over each frequency bin:

$$E_{tot} = \int_0^{2\pi} E(x, y, t, \theta) d\theta \quad (B-10)$$

From E_{tot} , the root-mean-square wave height H_{rms} and the maximum wave height H_{max} are calculated. These help to determine the fraction of wave breaking B_f :

$$\begin{aligned} H_{rms} &= [8E_{tot}/\rho g]^{0.5}; H_{max} = (D + \delta H_{rms})\gamma; \\ B_f &= 1 - \exp[-(H_{rms}/H_{max})^n] \end{aligned} \quad (B-11)$$

where δ is the fraction of wave height, γ is the breaking index, ρ is the density of water and g is the gravitational force. Finally, the wave energy dissipation due to breaking is predicted as follows:

$$S_{br} = 2 \frac{\alpha}{T_w} B_f E_{tot} \quad (B-12)$$

$$S_{br} = 2 \frac{\alpha}{T_w} B_f E_{tot} \frac{H_{rms}}{D} \quad (B-13)$$

where α is the wave dissipation coefficient and T_w represents the wave period. The major difference between Eq. B-13 and B-12 is that the wave breaking is dependent on the water depth. Recently, another option to account for wave breaking proposed by *Daly et al.* (2010) was added to XBEACH for non-stationary waves. The main feature is that users can define two breaking limits, defined as γ and γ_2 . The former parameter is the threshold of fully breaking; the latter is the limit point where waves stop breaking:

$$\begin{aligned} \text{Fully breaking} \quad S_{br} &= 1 \quad \text{when:} \quad H_{rms} > \gamma \cdot h \\ \text{Stop breaking:} \quad S_{br} &= 0 \quad \text{when:} \quad H_{rms} < \gamma_2 \cdot h \end{aligned} \quad (B-14)$$

For stationary waves, XBEACH employs two formulas based on *Baldock et al.* (1998) and *Janssen and Battjes* (2007).

Dissipation due to bottom (S_{bo})

XBEACH estimates the bottom friction affecting short-wave motions. A coefficient is required to determine the model inputs. The value is often larger than that defined for the flows:

$$S_{bo} = \frac{1}{2} \rho f_b [\ddot{u}]^3 \quad (B-15)$$

The term $[\ddot{u}]^3$ is defined as the ‘even velocity moment’. There are two approximations for this expression. First, the monochromatic approximation is applied if regular waves are considered. In that case, it is required to average the slowly varying

dissipation over a wave period, and $[\ddot{u}]^3$ is calculated as:

$$[\ddot{u}]^3 = 0.42u_{orb}^3 \quad \text{with:} \quad u_{orb} = \frac{\pi H_{rms}}{T_{peak} \sinh kD} \quad (\text{B-16})$$

The second approximation, considered optimal, is based on the linear Gaussian approximation:

$$[\ddot{u}]^3 = 0.57u_{orb}^3 \quad (\text{B-17})$$

Substituting $[\ddot{u}]^3$ into (B-15), S_{bo} now becomes:

$$S_{bo} = 0.28\rho f_b u_{orb}^3 \quad (\text{B-18})$$

The original version of XBEACH was developed to model the short-wave variations on the wave envelope and associated long waves.

Sediment transport

Governing equation: XBeach employs the depth-averaged advection-diffusion model based on *Galappatti* (1983):

$$\begin{aligned} \frac{\partial D\Psi}{\partial t} + \frac{\partial D\Psi u_e}{\partial x} + \frac{\partial D\Psi v_e}{\partial y} + \frac{\partial}{\partial x}\dot{X} + \frac{\partial}{\partial y}\dot{Y} &= \frac{h(\Psi_{eq} - \Psi)}{\mathfrak{S}_i} \\ \dot{X} = \Gamma D \frac{\partial \Psi}{\partial x}; \quad \dot{Y} = \Gamma D \frac{\partial \Psi}{\partial y}; \quad \mathfrak{S} &= \max \left[f_{\mathfrak{S}} \frac{D}{w_l} \mathfrak{S}_{s,min} \right]; \end{aligned} \quad (\text{B-19})$$

where Γ is the sediment diffusion factor, D is the local water depth, Ψ represents the concentration of sediment, which is dependent on the wave group and $\mathfrak{S}_i$ accounts for the instantaneous sediment response. The calibration parameter $f_{\mathfrak{S}}$ is used to respond to the adjustment of settling velocity w_l on the depth-averaged form. The degree of entranced or deposited sediment is computed by the difference between two parameters Ψ and Ψ_{eq} , which represent the actual and equilibrium sediment concentration.

Computational parameters: In Eq. B-19, information on both bed and suspended load is required to compute the equilibrium sediment concentration Ψ_{eq} . These loads are linked to the orbital motions u_{orb} , the Eulerian velocity and the falling velocity. Using linear wave theory, XBEACH computes u_{orb} as follows:

$$u_{rms} = \frac{0.7071\pi H_{rms}}{T_w \sinh [k(D + \sigma H_{rms})]} \quad (\text{B-20})$$

If the impact of broken waves on turbulence is considered, the wave-induced turbulent factor k_{β} is added to the orbital velocity:

$$u_{rms}^{2*} = u_{rms}^2 + 1.45k_{\beta} \quad (\text{B-21})$$

For the Eulerian velocity, XBEACH provides two computational options, which depend on the determination of the stirring factor due to long waves. With wave stirring:

$$v_{tot} = [u_e^2 + v_e^2]^{0.5} \quad (\text{B-22})$$

No wave stirring:

$$v_{tot} = \left(1 - \frac{\Delta t}{m_1 T_w}\right) v_{tot}^{i-1} + \frac{\Delta t}{m_1 T_w} [u_e^2 + v_e^2]^{0.5} \quad (\text{B-23})$$

where v_{tot}^{i-1} is the velocity in the previous step, m_1 is a factor to represent the current averaged time scale of the wave period T_w and Δt is the difference in time between the current and previous step. In the last term, the falling velocity is computed based on the framework proposed by *Ahrens* (2000) and *Hallermeier* (1981):

$$w_l = \underbrace{K_1 \left[\frac{\rho_s - \rho_w}{\rho_w} g d_{50} \right]^{0.5}}_{\text{turbulent regime}} + \underbrace{K_2 \frac{\rho_s - \rho_w}{\rho_w \nu} g d_{50}^2}_{\text{laminar regime}} \quad (\text{B-24})$$

where ρ_s, ρ_w are the sediment and water density respectively, g is the gravitational force, d_{50} denotes the median grain size, which is related to the characteristics of sediment diameter, ν is the kinematic viscosity, and K_1 and K_2 are coefficients related to the Archimedes buoyancy index I_b :

$$\begin{aligned} K_1 &= 1.06 \tanh(0.016 I_b^{0.05} \exp(-120/I_b)) \\ K_2 &= 1.06 \tanh(12 I_b^{-0.59} \exp(-0.0004 I_b)) \end{aligned} \quad (\text{B-25})$$

with:

$$I_b = \frac{\rho_s - \rho_w}{\rho_w \nu^2} g d_{50}^3 \quad (\text{B-26})$$

In case of high sediment concentrations, w_l is simplified:

$$w_{s, sim} = (1 - C)^r w_l, \quad r = 2.35 \frac{2 + 0.175 R^{0.75}}{1 + 0.175 R^{0.75}}, \quad R = \frac{w_l d_{50}}{\nu} \quad (\text{B-27})$$

Transport methods As mentioned earlier, the equilibrium sediment concentration term Ψ_{eq} in Eq. B-19 requires information on the bed and suspended load, which can be computed separately or combined in total bulk transport. The general approach to compute Ψ_{eq} is to find the maximum sediment concentration by comparing the equilibrium state with the maximum allowance:

$$\Psi_{eq} = \max \left[\underbrace{\min(\Psi_{eq}^b, 0.5 \Psi_{max})}_{\text{due to bed load}} + \underbrace{\min(\Psi_{eq}^s, 0.5 \Psi_{max}, 0)}_{\text{due to suspended load}} \right] \quad (\text{B-28})$$

Two different approaches to compute Ψ_{eq} can be applied in XBEACH, as follows:

Computational approaches

Stationary: This computational mode allows the computation of variations in the wave envelope, but neglects long-wave motions (e.g. infra-gravity waves). Despite offering simplicity, this model is mostly resolved dominant coastal processes including wave propagation, shoaling, fraction, dissipation due to bottom friction and wave breaking. The wave setup and setdown are computed from the radiation

Van Thiel-Van Rijn	Soulsby-Van Rijn
* Due to bed load: (Ψ_{eq}^b):	
$\frac{S_b}{h} \left([v_{tot}^2 + 0.64u_{rms}^2]^{0.5} - U_c \right)^{1.5}$	$\frac{S_b}{h} \left([v_{tot}^2 + 0.18u_{rms}^2/C_d]^{0.5} - U_c \right)^{2.4}$
* Due to suspended load: (Ψ_{eq}^s):	
$\frac{S_s}{h} \left([v_{tot}^2 + 0.64u_{rms}^2]^{0.5} - U_c \right)^{2.4}$	$\frac{S_s}{h} \left([v_{tot}^2 + 0.18u_{rms}^2/C_d]^{0.5} - U_c \right)^{2.4}$
* Where S_b and S_s are:	
$S_b = 0.015D \frac{(d_{50}/D)^{1.5}}{(\frac{\rho_s - \rho_w}{\rho_w} d_{50})^{0.75}}$	$S_b = 0.005D \left[\frac{d_{50}}{D \frac{\rho_s - \rho_w}{\rho_w} d_{50}} \right]^{1.2}$
$S_s = 0.012d_{50} \frac{d_{50} d_1^{-0.6}}{\left(\frac{\rho_s - \rho_w}{\rho_w} d_{50} \right)^{1.2}}$	$S_s = 0.012d_{50} \frac{d_{50} d_1^{-0.6}}{\left(\frac{\rho_s - \rho_w}{\rho_w} d_{50} \right)^{1.2}}$
$d_1 = d_{50} \sqrt[3]{\frac{\rho_s - \rho_w}{\nu^2 \rho_w}}$	
$U_c = 0.19d_{50}^{0.1} \log 10(4D/d_{90})d_{50} > 10^{-4}$	
$8.5d_{50}^{0.6} \log 10(4D/d_{90}) : d_{50} \leq 0.05$	

Table B.1: Different Ψ_{eq} formulations

stress gradients. This model also accounts for the wind and tidal forcing.

Surf beat: Variations in short-wave envelopes are predicted by the wave action equations. These variations are presented in the form of radiation stress gradients that can force the water column and motions of infra-gravity waves. The nonlinear shallow water equations are used to predict unsteady currents, which can be driven by waves and wind.

Non-hydrostatic: This was developed initially from a prototype of the SWASH model. This model does not require the computation of the action of short waves. However, wave-resolving model requires finer resolution and smaller time steps, which can increase computational costs.

Hydrostatic setting: The variation in short-wave amplitude is computed separately from the long waves, the current field and morphological processes. This separation reduces computational costs because it does not compute the phase of the short waves.

Coordinate conventions

XBEACH employs a coordinate system where the x-axis of the modelled grid faces towards the shoreline at an approximation of 90 degrees (perpendicular to the shores). The y-axis of the modelled grid is parallel to the shoreline. The positive direction is counterclockwise.

Computational grid

The computational grid can be rectangular or curvilinear. Both options use a staggered grid convention. Accordingly, the water elevations and water depths are computed in central cells, while the velocity and sediment transport are defined at

the cell interfaces. For waves, the energy balance is calculated at the centre of each cell, and the gradient of radiation stress terms is computed at U and V points.

Model Forcing and boundary conditions

XBEACH allows several options for wave conditions at the open boundary. This model can be forced with water levels, wind forcing and waves at the boundary. Wave information can be prescribed in parametric format or spectral form (e.g. 2D SWAN spectral outputs). In the present study, the water elevations and wave information (2D spectra) were taken from the validated coupled SCHISM-WWM model.

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