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Maria Creek Sustainable Infrastructure Project

Concept Study

Prepared for

KINGSTON DISTRICT COUNCIL
Gateway to the South East Coast

17 June 2020

Executive Summary

Study Purpose

Kingston District Council (KDC) engaged Wavelength to identify and consider concepts related to coastal infrastructure and management at the southeast Kingston foreshore, focusing on the area bounded by the Kingston Jetty and the Maria Creek boat launching facility.

The purpose of this study is to provide KDC with sufficient information to select a concept to progress further. Concepts have been evaluated against the following criteria nominated by KDC:

1. Provide a boat ramp during peak times (October to May), that is financially sustainable (low maintenance) through an affordable capital solution.
2. Provide a jetty that services the needs of community and visitors.
3. To create an opportunity to activate open spaces and facilities, specifically the area between the jetty and breakwaters.
4. Consider the effects of the natural processes and the coastal environment.

For the purpose of this study, the criteria above have been assumed to have equal weighting/importance.

In addition to the above, Wavelength has provided other general recommendations pertaining to ongoing coastal management best practices relevant to the study area.

Structure

This report is organised as follows:

- General background and contextual information: Sections 1 to 4
- Evaluation of concepts: Section 5-7
- Recommendations: Sections 8-9

Study outcomes and recommendations

The following are the key findings of the Maria Creek Sustainable Infrastructure Project concept study:

- No concept has been identified that meets all four of the criteria nominated by KDC.
- None of the concepts meet Criteria 1. All concepts that provide a boat launching facility with high levels of service come with significant capital expenditure (approximately $3M-$10M) and do not substantially reduce maintenance costs compared to the current annual spend required at Maria Creek.
- A number of concepts meet the remaining Criteria 2-4, and come with reasonable capital and reduced ongoing maintenance expenditure. However, these concepts likely result in the loss of Maria Creek as a usable boat launching facility. These concepts are collectively termed 'Pathway 2'.
- Considering KDC's criteria, and applying some judgement to further refine the options within Pathway 2, Wavelength recommends the following for further consideration by KDC:
    - Remove the seaward extension of the Maria Creek southern breakwater to reduce southern beach widths and improve jetty and foreshore amenity.
    - Develop an informal 'over the beach' boat ramp at Johnston Street. A temporary ramp should be trialed if residents are having difficulty with beach conditions over the peak use period.
    - Keep Maria Creek open for environmental purposes and to manage flood levels in the creek.
- The works above have a combined cost of ~$1.5M (NPV).

The following key recommendations are provided for KDC's consideration:

- In light of KDC's evaluation criteria, it is recommended that the refined version of Pathway 2 above be progressed further by KDC.

- The results of the evaluation are sensitive to the evaluation criteria, before proceeding with any works KDC should consider and reconfirm the criteria and their relative weighting/importance.
- This study focuses specifically on the management of coastal infrastructure in the area bounded by the Kingston Jetty and Maria Creek. Pathway 2 results in the likely loss of Maria Creek as a usable boat launching facility, currently leaving Cape Jaffa as the remaining level 4 boat launching facility to service the Kingston Area.
- It is recommended that KDC implements the ongoing coastal management best practices relevant to the study area as summarised in this report.

Rev	Date	Description	Prepared	Reviewed/submitted to
A	8/6/2020	Draft for discussion	B Smith	A Sandery
B	10/6/2020	Draft for external review	B Smith	KDC
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1 Introduction

1.1. Background

Kingston District Council (KDC) is located approximately 300km to the south-east of Adelaide, South Australia. KDC operates and maintains a number of coastal assets adjacent to the Kingston SE townsite, including the Maria Creek recreational boating facility (boat ramp) and the Kingston Jetty (jetty) shown in Figure 1.

Figure 1: Site location

In the last 5 years, KDC have noticed an increase in the severity of storm events leading to several coastal management issues, including:

- Significant sand accumulation to the south of the Maria Creek breakwaters, allowing sand to naturally bypass the southern breakwater and accumulate within the Maria Creek entrance channel and adjacent to the boat ramp.
- The sand accumulation extends at least 1km south of the Maria Creek breakwaters underneath the jetty, reducing the promenading, fishing and swimming function of the jetty.
- Large volumes of seagrass wrack are trapped along the coastline between the jetty and southern breakwater, which causes a public nuisance in the foreshore area. These wrack accumulations have also exacerbated the need for Council to regularly intervene to remove blockages to the Maria Creek entrance channel to maintain navigational access and allow creek flows.
- Recent storm events have resulted in increased damage to the Maria Creek breakwaters, which require significant repairs to maintain their function.

Given the significant costs associated with dredging and maintenance works for the Maria Creek facility over the past financial years, KDC acknowledge that the current short-term solutions to maintain the boat ramp are not sustainable.

Two studies have recently been completed for the Maria Creek boat ramp:

- The Maria Creek Seaweed Infiltration Improved Management Options study involved a desktop review of seagrass dynamics in the area and investigation of concepts for managing the seagrass accumulations impacting on boat ramp navigation (GHD, 2013).
- The Maria Creek Breakwater and Boat Launching Facility Options Study explored concept options to re-open the Maria Creek boat ramp following significant sand and seagrass wrack accumulations in early 2018 as a result of saturation of the southern breakwater (Tonkin, 2018). This also included a structural condition inspection of the Maria Creek breakwaters and recommended repairs (Tonkin, 2017a).

Wavelength Consulting Pty Ltd (Wavelength) has been engaged by KDC to build upon these previous studies undertaking a more detailed assessment and modelling of the Maria Creek area, considering the impacts of recent wrack and sand accumulations on key assets identified through community consultation within the study area.

This report summarises the methods, findings and recommendations of the concept study.

1.2. Study purpose

The purpose of the Maria Creek sustainable infrastructure project is to identify and consider concepts with the aim of providing Council sufficient information to select an option to progress further. These concepts have been assessed against the following criteria nominated by KDC:

1. Provide a boat ramp during peak times (October to May), that is financially sustainable (low maintenance) through an affordable capital solution.
2. Provide a jetty that services the needs of community and visitors.
3. To create an opportunity to activate open spaces and facilities, specifically the area between the jetty and breakwaters.
4. Consider the effects of the natural processes and the coastal environment.

1.3. Approach

The approach employed for this study is outlined below:

1. Identify existing data and review previous studies investigating the boat ramp and coastal processes in the study area.
2. Undertake community consultation to identify key asset values and impacts, as well as potential concept options for investigation.
3. Develop conceptual understanding of key coastal processes and drivers through wave and hydrodynamic modelling.
4. Develop concept options, and review them against the criteria outlined in Section 1.2.
5. Consider and recommend best practice coastal management for the study area and pathway forward for Maria Creek.

This concept study including the subsequent findings and recommendations assume that there is no change to existing coastal management practices (such as dredging operations at Cape Jaffa and erosion protection strategies at Wyomi Beach). Any changes to the coastline or coastal management would have a subsequent impact at Maria Creek and the findings and costings presented here may need to be revisited in the future.

2 Coastal assets

KDC is responsible for the management of several coastal assets and facilities in the Kingston area. The following section summarises the key coastal assets within the study area related to the criteria set out in Section 1.2.

2.1. Maria Creek boat ramp

The Maria Creek boat ramp is located within the downstream reaches of Maria Creek, as previously shown in Figure 1.

The existing facility consists of a 'four lane' boat ramp with floating pontoons and the entrance is protected by two rock armoured breakwater structures. The original facility was constructed in 1997 and upgraded in 2002. Upgrades since the original construction have included widening and deepening of the entrance channel, a new boat ramp and pontoons and extensions to the breakwater structures.

Typical breakwater cross-sections are presented in Appendix A. In July 2017, Tonkin completed an inspection of the breakwater structures to identify their condition and recommended rectification works (Tonkin, 2017). The study highlighted that the southern breakwater required significant repairs in the immediate to short term. KDC confirmed they completed the immediate repairs to the navigation light footing and minor repairs to the breakwater head but the short-term repairs have not been completed at the time of this report (Pers. Comm. Dave Worthley KDC, 13/5/20).

Based on the breakwater design drawings (Appendix A), the entrance channel navigable depth is approximately -2.0 m Australian Chart Datum (CD). The entrance channel navigable width is not shown on the drawings but was noted as 20m by GHD (2013), which was used in this concept study.

2.2. Kingston jetty and foreshore

The Kingston Jetty (jetty), shown in Figure 1, was constructed in 1876 and has been rebuilt a number of times after large storm events have damaged the structure (SMH, 2020). The jetty once served the fishing fleet but today is primarily used by the public as a promenading jetty, as well as a fishing and swimming platform. This study has not considered the asset condition of the jetty or associated costs for maintenance and repair. It is understood that this is an asset held under a lease agreement with the State Government. It is acknowledged that the jetty is considered an important coastal asset to the community and Council.

The Kingston Foreshore (foreshore) extending from the southern side of the jetty to the Maria Creek breakwaters is also an important coastal asset. Assessment criteria 3 of the project (Section 1.2) relates to activating open spaces and facilities, specifically the area between the jetty and breakwaters.

2.3. Cape Jaffa

Cape Jaffa Anchorage is a small boat harbour located approximately 20km south-west of Maria Creek and was constructed between 2007 and 2008. The harbour consists of two breakwater structures protecting an entrance channel and dredged harbour area. Approximately 290,000 m^3 of sediment has been bypassed from the western beach and channel onto the eastern beach at Cape Jaffa between 2008 and the end of 2019, including large bypassing campaigns each more than 100,000 m^3 in 2016 and 2018/19 (Magryn, 2020). A recent review of sand accumulation and bypassing volumes suggests sand volumes in the order of 47,000 m^3 have accumulated on the western beach each year since 2008 (Magryn, 2020). KDC are actively managing the sand accumulation at Cape Jaffa with a new Damen CSD350 dredge.

2.4. Wyomi Beach erosion

Over the last two decades, storm erosion at Wyomi Beach, located approximately 4km south-west of Maria Creek, has resulted in the loss of approximately 10 to 15m of dune width, damaging paths and threatening Marine Parade (Wavelength, 2020a). Department of Environment and Water (DEW) have been monitoring the shoreline at Wyomi at approximately 1 to 3 year intervals since 2003, with approximately 10 to 15m of dune width lost between March-2016 and May-2017.

Most of this erosion is likely to have occurred during a large storm event between 10th and 13th July 2016. This storm was reported to have damaged the Kingston Jetty and Maria Creek breakwaters (Pers. Comm. KDC, 21 February 2020), as well as causing significant beach erosion at Wyomi, as shown in Figure 2 (ABC News, 2016).

Figure 2: 15th July 2016 storm erosion photograph (ABC News, 2016)

The July 2016 event was predicted to have an Average Recurrence Interval (ARI) in the order of 20 to 50 years (Wavelength, 2020a).

A 380m granite rock seawall was constructed at the rear of Wyomi Beach in April 2018, following the severe storm events and subsequent dune erosion through 2016. The seawall is flanked by temporary geotextile sand container seawalls for approximately 90m in both directions. KDC recently placed sand nourishment in front of the sand container seawalls to provide short term protection, whilst longer term adaptation options are developed as part of the Kingston District Coastal Adaptation Plan (Wavelength, 2020b).

3 Stakeholder engagement

3.1. Approach

Stakeholder engagement was undertaken to inform the Maria Creek concept study, as well as the Kingston Coastal Adaptation Strategy also being undertaken by Wavelength. Engagement consisted of the following:

- Twenty-two 30 min one-on-one consults held with community members at Council offices over Thursday 13 and Friday 14 February 2020.
- A session for Council's elected members.
- Three written responses were received and two phone call sessions were held for community members who were unable to attend in person.

The intent and aim of the sessions were to:

- Allow an opportunity for community members to have their say.
- Add value to the project by collating local residents' anecdotal evidence and observations of the coast over seasons, years and (in some cases) many decades.
- Provide an opportunity for the community to understand and ask questions about the project methods (what's involved in numerical modelling, hazard mapping etc).
- Build on the existing understanding of what the community values regarding this section of coast.

The following provides a summary of the key findings of the stakeholder engagement. The full stakeholder engagement summary report is contained in Appendix B.

3.2. Key findings

Open two-way dialogue focused on two key areas, with key findings outlined below:

- Great concern regarding the longer-term impact closing Maria Creek boat ramp will have on the town:
    - Town depends on tourism, many noting that a number of local businesses, commercial and residential properties for sale since the boat ramp has been closed;
    - One member noting that whilst the cost of keeping the facility open is being investigated also need to look at the financial impact to the town whilst the facility has been closed. (E.g. the impact on local businesses of not having the annual fishing competition);
    - President of the Upper SE Rec Fishing group noting that memberships are down from 300 to 130 as a result of Maria Creek being closed;
    - Community members who have chosen to retire in Kingston for recreational fishing no longer can benefit from safe and convenient launching facilities;
    - One community member noting a concern if businesses continue to close and people move away there is a potential risk of the hospital closing;
    - "*Rate payers are happy to pay higher boat ramp fees for a facility that works*";
    - The facility is also missing a cleaning station, lighting, toilets and channel markers
- Whilst beach launch is an option, beaches are quite soft and present a safety risk, particularly for unexperienced users and those with larger boats;

- Maria Creek is preferred over Cape Jaffa due to the less 'friendly' waters, fuel costs and general inconvenience associated with Cape Jaffa;

- Some community members saw priorities beyond the function of Maria Creek as a boating facility:

    - The jetty is important to the town and needs to be maintained. More specifically, sand should be removed to restore its function and the structural condition of the Jetty is an issue that needs attention;

    - If Maria Creek is not maintained this presents an environmental and flooding risk upstream;

    - Financial considerations are important *e.g. "Really important to seek a low maintenance cost solution so ratepayers aren't impacted, priority is a financially sustainable solution"*;

    - Council should address environmental management and rezoning opportunities between Maria Creek and Blackford Drain;

    - Some believe that Maria Creek is not an optimal location for a boating facility e.g. *"The Boat ramp was built in the wrong spot, should never have been built in the creek"*

    - Other tourism opportunities exist outside of boating, if shorebirds and migrating seabirds were prioritised this would create tourist interest.

- Stakeholders would like to be notified and kept in the loop so families and businesses can plan accordingly.

- **Preferred options for Maria Creek** put forward by community members in order of frequency of reference or comment:

    1. Remove southern breakwater
    2. Reduce the width of the channel
    3. Alternate location of boat ramp
    4. Extend and alter direction of southern breakwater
    5. Pump seawater out of Maria Creek
    6. Weir upstream of boat ramp
    7. Redesign of breakwaters with culverts
    8. Capital dredging program

This information was incorporated into the conceptual understanding of coastal processes, as well as the concept options assessment for Maria Creek. The preferred options put forward by community members as listed above were considered through the 'First Pass Assessment' (refer to Section 5.3, Table 3).

4 Conceptual understanding

4.1. Approach

The coastal assets at Maria Creek and surrounds are impacted by two distinct coastal processes, which are considered throughout this study:

- Sand accumulations:
  - Impacting navigation within Maria Creek and the entrance channel.
  - Impacting jetty amenity due to reduced depths as the beach widens.
- Wrack accumulations:
  - Impacting navigation and creek flows within Maria Creek.
  - Impacting foreshore amenity between the southern breakwater and the jetty.

An understanding of these coastal processes and their key drivers is required to develop concept options and to assess their effectiveness. This understanding of coastal processes was developed through:

- Review of previous regional and local coastal studies.
- Review of available data, including:
  - Aerial and satellite photographs.
  - DEW coastal monitoring profiles.
  - April 2018 terrestrial LiDAR survey.
- Site visits by Wavelength coastal engineers on 19 December 2019 and 20 February 2020.
- Collection of nearshore hydrographic and beach survey data by Precision Hydrographic Surveys (PHS) on 30 January 2020 presented in Appendix C.
- Results of modelling completed by Port and Coastal Solutions (PCS), with the modelling results presented in the Modelling Summary Report presented in Appendix D. The PHS detailed hydrographic survey noted above was used as input to the PCS wave and hydrodynamic modelling (Appendix D).

4.2. Bathymetry & coastal geomorphology

Maria Creek is located in Lacepede Bay, at the southern end of a long sandy beach extending from Victor Harbour in the north to Cape Jaffa in the south. Lacepede Bay has a wide and shallow offshore shelf (the 10m depth contour is 18km offshore) and shallow nearshore reefs (PCS, 2020).

The PHS nearshore hydrographic and beach survey of the study area shows the following features at Maria Creek:

- The beach to the south of the southern breakwater (southern beach) is up to 200m wide and extends to the full length of the breakwater.
- A shallow sand bar has formed across the Maria Creek entrance, with a small creek flow area on the eastern side of the entrance channel.
- The entrance channel and creek has shallowed significantly to approximately 0m Australian Height Datum (AHD).

The southern beach is backed by a low dune system and significant wrack was also observed on the beach and within the Creek during the site visits, which is outlined further in Section 4.5.

4.3. Key metocean conditions

The Modelling Summary Report prepared by PCS (2020) and presented in Appendix D contains a detailed description of the metocean conditions within the study area and the wider Lacepede Bay. The following presents a summary of the key metocean conditions based on this work and review of other studies:

- **Waves:** waves at Maria Creek approach the coast from a narrow offshore band, predominately from the west (PCS, 2020). The area is generally sheltered from large waves, given the wide and flat nearshore area, resulting in depth limited wave breaking (Short and Hesp, 1980). During storms, with elevated water levels, larger waves in excess of 1.5m are expected to reach the shoreline.
- **Water levels:** The study area lies within a semi-diurnal, mixed micro-tidal environment with tidal ranges less than 1m on spring tides. Of importance in this region is the occurrence of 'dodge tides' periodically throughout the year when there is very little change in tidal level for a significant portion of the daily tidal cycle.
- **Storminess:** A review of severe wave events at Maria Creek (PCS, 2020) suggests that there does not appear to be an increase in storm activity over recent years, however 2016 was a notably stormy year. Between July and September 2016 there were three storm events where the significant wave height (Hs) exceeded the 1 in 1 year Average Recurrence Interval (ARI) wave height at Maria Creek.
- **Nearshore currents:** The nearshore area experiences low tidal flows orientated parallel to the coast (PCS, 2020). There is a dominance in tidal currents to the north during calm conditions due to a large-scale eddy which forms in Lacepede Bay, resulting in northward currents throughout the tidal cycle during spring tides. Wind induced currents can occur during periods of strong winds and under certain conditions the resultant currents can dominate the nearshore currents in the Maria Creek region. For example, under strong northerly winds, the nearshore currents can flow in a southerly direction directly into Maria Creek entrance.
- **Creek currents**: In the mouth of Maria Creek there is a spring tide flood dominance in the currents which indicates that the Creek will typically act as a net importer of both sediment and wrack. When combined with wave sheltering provided by the breakwater structures, any sediment or wrack which is transported into the creek entrance by waves and tidal/wind-driven currents during the flood stage of the tide is expected to be deposited and is unlikely to be remobilised.
- **Creek discharge**: There are no creek discharge records available for Maria Creek, but freshwater discharge rates will follow the same trend as the rainfall records, peaking in July and August (PCS, 2020). Prior to July and after August, peak flows in the Creek are reported to be insufficient to naturally flush out any deposits brought into the Creek by storm events (GHD, 2013).

4.4. Sand accumulations

Sediment is transported under the combined actions of waves, currents and wind. Longshore transport is movement of sand along the beach typically by the action of waves approaching the coast at an angle (i.e. not perpendicular to the shore) and/or by the action of tidal or wind driven flows. Structures such as breakwaters and groynes can intercept the longshore sediment transport, leading to sand accumulation on one side and erosion on the other.

Cross-shore transport is the movement of sand perpendicular to the beach alignment and is often associated with erosion from severe storm events or accretion from swell and wind-blown sand processes.

4.4.1. Longshore sediment transport

Review of the historical sediment transport within the study area can provide insights into the causes of recent sand accumulations and potential management options. Figure 3 presents the 1987 (pre-construction) and January 2020 shoreline alignments at Maria Creek.

Figure 3: Historical shoreline positions

Figure 3 shows significant volumes of sediment have accumulated to the south of Maria Creek since the breakwaters were constructed in 1997. It is understood from the community engagement sessions, that dredging of sediment accumulation in the Maria Creek area (through volunteer operations on a local sourced dredge) stopped sometime in 2015 or 2016, however, the details of sand volumes and management effectiveness were unable to determined (either anecdotally nor factually).

A significant proportion of the sediment is believed to have accumulated to the south of the breakwaters since 2016. Using the limited available survey and profile data, Wavelength developed a sediment budget for the 2 ½ year period from March 2016 to October 2018 presented in Figure 4.

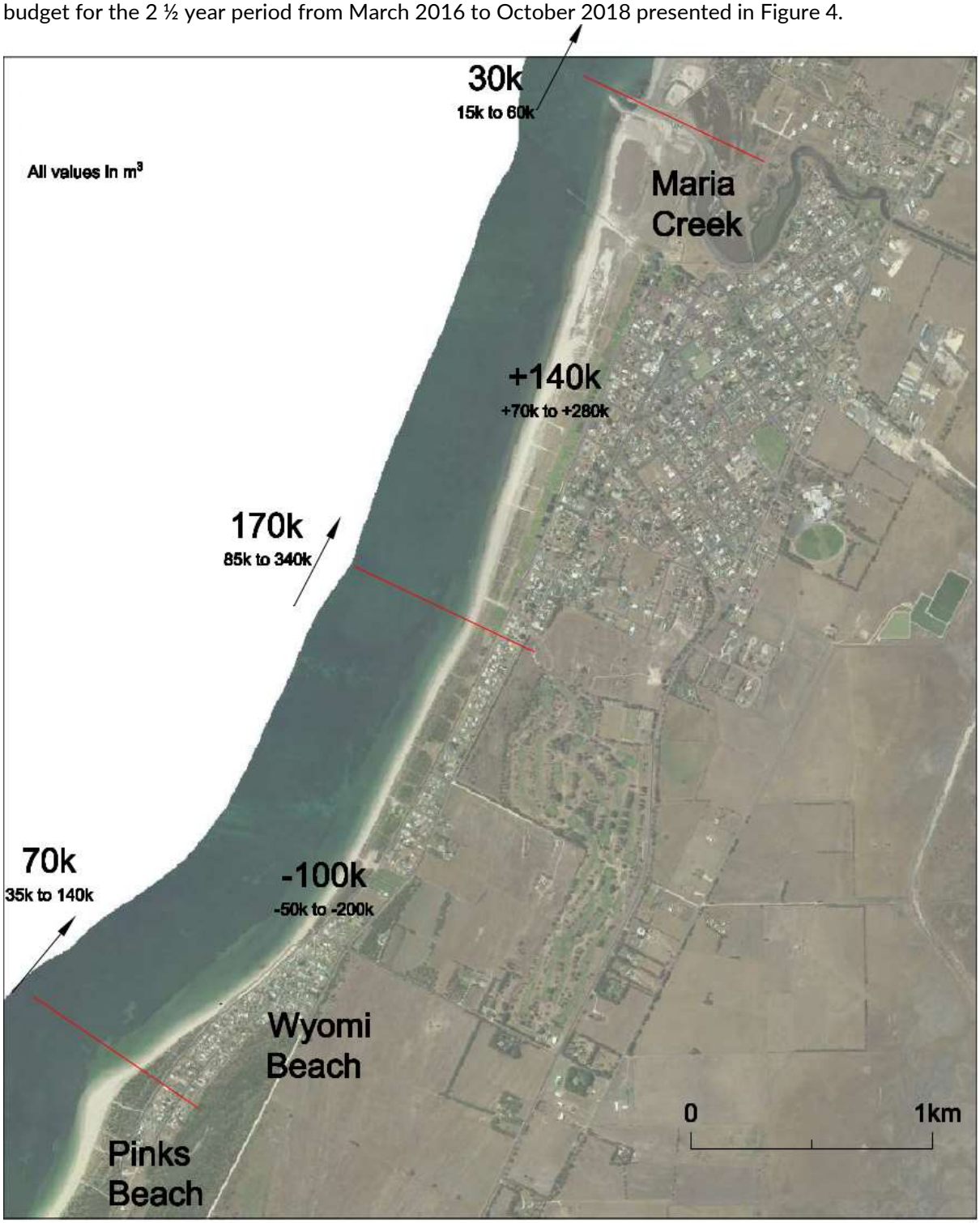

Figure 4: March 2016 to October 2018 Sediment Budget

Figure 4 shows the volumes of accretion (positive values) and erosion (negative values) at key coastal segments (red lines) between Pinks Beach and Maria Creek. Longshore transport rates between segments (at the red lines) were estimated from the change in segment volumes.

Given the lack of consistent profile and survey data adjacent to Maria Creek and the range of assumptions made in developing the sediment budget, the sediment accumulation and longshore

transport estimates are indicative only. Sensitivity testing using ±100% of segment volumes was undertaken (shown as smaller values in Figure 4) to test the sensitivity of the longshore transport rates to different accretion and erosion volumes.

Key findings of the sediment budget analysis are outlined below:

- Through 2016, a number of significant storm events eroded the beach and dune by up to 15 m at Wyomi Beach. The event in July 2016 was considered to be in the order of the 20 to 50 year ARI erosion event for the area. This caused cross-shore erosion, releasing a large 'slug' of highly mobile sand into the intertidal region (~ 100,000 m^3), which was out of alignment with the typical longshore transport processes. The storm induced sand slug was transported to the southern side of the Maria Creek southern training wall over approximately a 6-month period following the initial erosion event (May 2016). This type of sand slug transport has been recorded at other locations around Australia, including at Dawesville (WA), Tweed River (NSW), and Noosa (QLD) (DPI WA, 2006; Jacobs, 2017; Coastal CRC, 2006).
- Typical longshore transport rates at the southern edge of Wyomi Beach were in the order of 28,000 m^3/yr over the 2 ½ year period (sensitivity testing shows -100%=14,000 m^3/yr to +100%=56,000 m^3/yr). These rates are generally higher than the estimated longshore transport rates of approximately 15,000 m^3/yr calculated in the Cape Jaffa Sand Management Strategy from data prior to 2004 (WBM Oceanics Australia, 2005). Recent review of sand bypassing estimates at Cape Jaffa suggests longshore transport rates of approximately 47,000m^3/yr since 2008 (Magryn, 2020).
- The Maria Creek southern breakwater became saturated in late 2017, allowing sand to bypass the southern breakwater and fill the entrance channel. This required management by KDC in late 2017 through to July 2019, with approximately 75,000 m^3 (disturbed) of sand and wrack removed from the creek over this period (Pers. Comm. David Worthley, KDC, 17 April 2020.

Results of sediment transport calculations by PCS (2020) presented in Appendix D, suggests the dominant sediment transport process in the Maria Creek region is wave action, with waves typically approaching slightly south of shore normal driving the northwards longshore transport. Analysis has also shown that the nearshore tidal and wind-induced currents can also influence the longshore transport rates during periods with larger wave conditions, having the potential to result in more than a 50% increase in the potential longshore transport rates.

PCS calculated longshore transport rates using model output over the 19-year period from 2001 to 2019. These were output at the four locations P1 to P4 shown in Figure 5 (within 500m north and south of Maria Creek), with the gross calculated sediment transport rates presented in Figure 6.

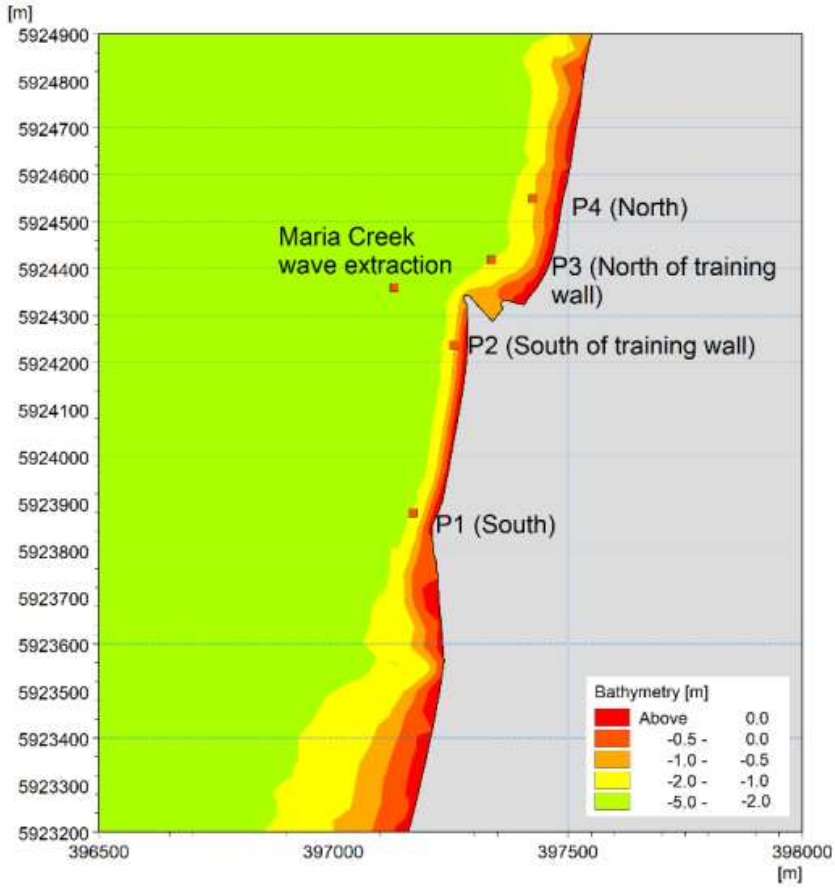

Figure 5: Longshore transport calculation locations

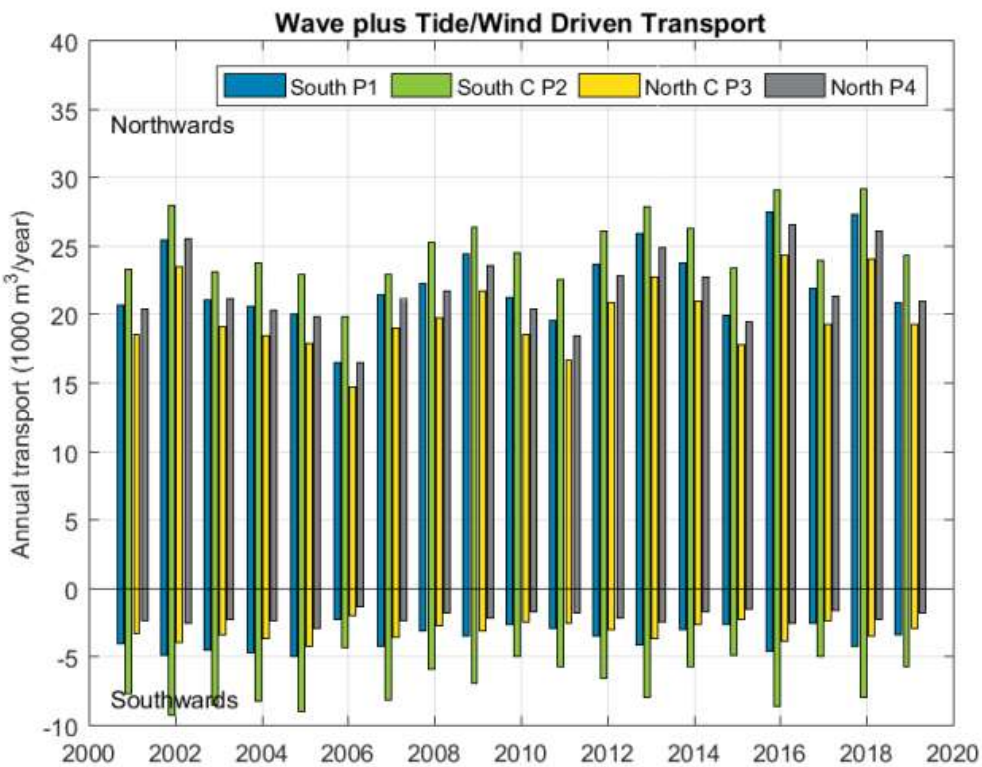

Figure 6: Calculated gross longshore sediment transport in the study area

This diagram shows the following:

- The dominant (net) transport direction is to the north. However, there is a component of southerly sand transport, which needs to be considered when developing concept options.
- The longshore transport rates prior to 2008 were generally lower than transport rates in recent years. The transport rate estimates for the Cape Jaffa development were based on this pre-2008 data, which may explain the lower values used in these earlier studies.
- The two years with the highest longshore transport rates estimated in the last 19 years were in 2016 and 2018.
- The net northerly transport rates of ~25,000m^3 in 2016 and 2018 match the rates estimated in the sediment budget (Figure 4).

A longshore transport rate of 30,000 m^3/yr has been adopted for use in this study based on the sediment budget and longshore transport calculations. Longshore transport rates presented in this report and PCS (2020) are approximate only and provide an indication of the relative differences in longshore transport rates within the study area for different years. On-going shoreline monitoring of beach and nearshore profiles at regular intervals along the beach is recommended to improve the understanding of sediment transport rates within the study area, as discussed further in Section 8.1.

4.4.2. Sediment transport summary

Review of previous studies, sediment budget development and modelling by PCS (2020) suggests the following key processes have led to the boat ramp and jetty impacts:

- Boat Ramp:
  - Sand is typically transported to the north by waves and currents, which has accumulated to the south of the breakwaters, saturating the southern breakwater in late 2017. Sand is able to naturally bypass the southern breakwater forming a sand bar across the entrance channel, limiting navigable depths and creating a longshore transport connection from the south to the north of the creek.
  - Creek discharges and tidal currents are generally insufficient to flush the sand bar from the entrance channel. Calculations suggest the out-going tidal flows would need to be increased by an order of magnitude (10 times) to maintain a stable entrance.
  - Given the spring flood tide dominance, the creek is a net importer of sediment. With a readily available source of sediment in the sand bar, the creek quickly fills with sand.
- Jetty:
  - Sand has continued to accumulate to the south of the breakwater since 2017, with the beach underneath the jetty widening thus reducing its amenity.

Figure 7 presents a conceptual diagram of the sediment transport processes.

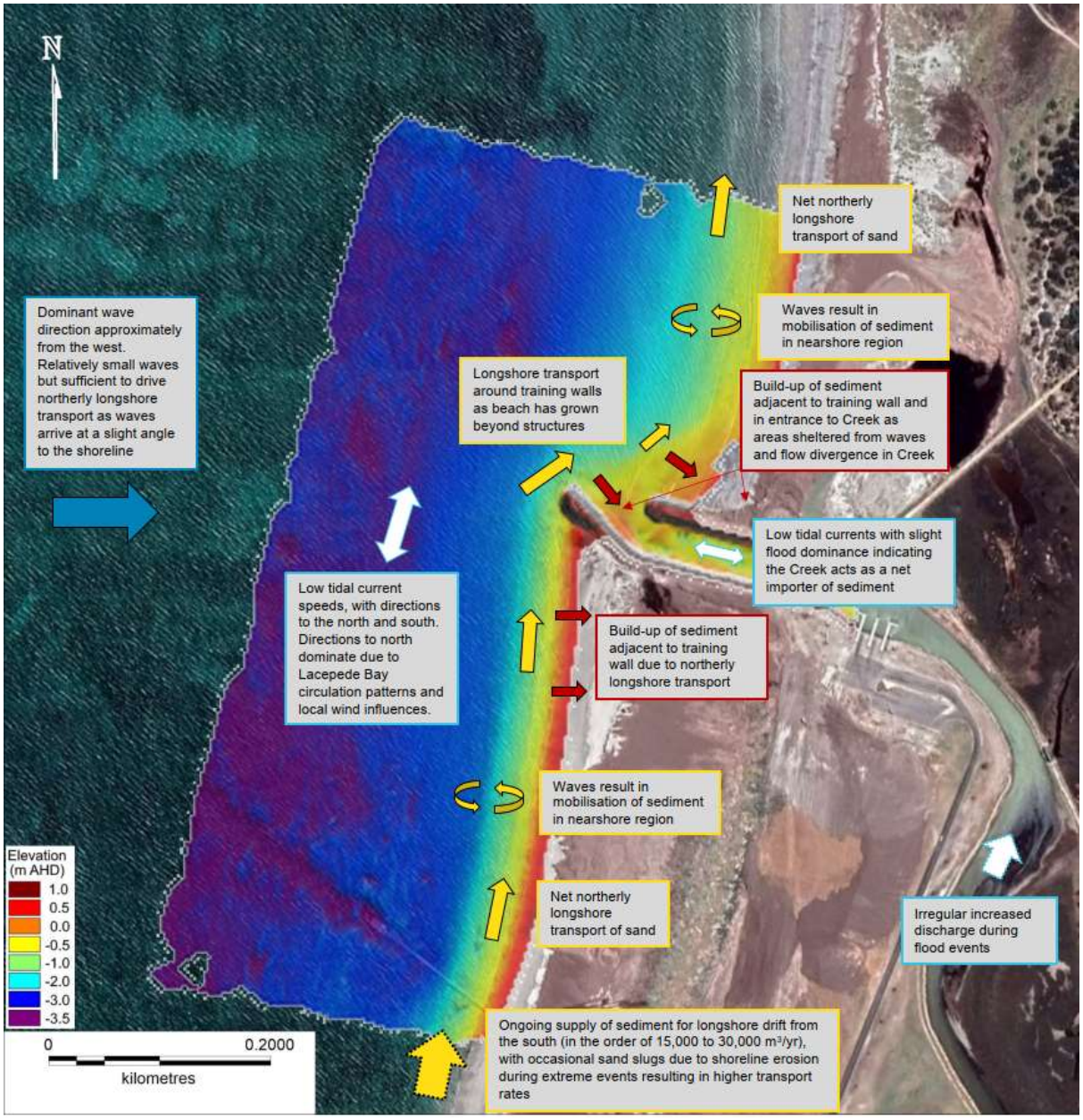

Figure 7: Sediment transport conceptual understanding diagram (PCS, 2020)

4.5. Wrack accumulations

4.5.1. Description and sources of wrack

Wrack is the term used for detached marine macroalgae, seagrass and other marine detritus that may be found floating, mobile on the seabed, or accumulated in sheltered areas. Wrack observed onsite during the site visit was predominantly seagrass species, as shown in Figure 8.

Figure 8: Seagrass wrack observed during site visit (20 February 2020)

Seagrass species typically shed their leaves in late autumn and early winter, which represents the peak availability of wrack material at Maria Creek (GHD, 2013). Large winter storms tend to mobilise the wrack, depositing it in sheltered areas, such as the wide beach berm adjacent to the southern breakwater and within Maria Creek.

Seagrass wrack accumulating within the study area is likely to have come from the dense seagrass meadows found within the protected waters of Lacepede Bay (Figure 9), of which the meadow-forming species *Posidonia angustifolia*, *Posidonia sinuosa*, and *Posidonia australis* are most abundant (PIRSA, 2014). Posidonia generally requires stable, non-mobile sediments on which to grow and establish dense meadows.

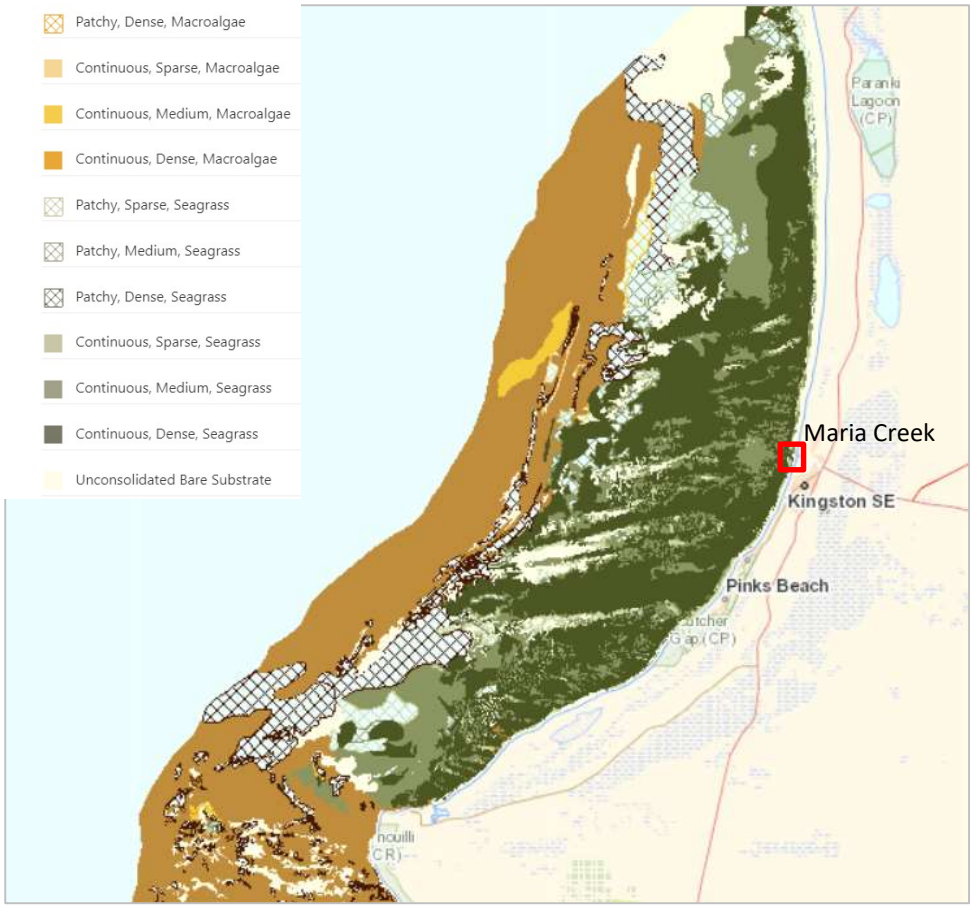

Figure 9: Benthic habitat mapping of Lacepede Bay (DEW, 2020)

Seagrass coverage is thought to have been affected by outflows from various drains along the Kingston coastline, including Maria Creek, with increased loss of coverage due to poor water quality from these outflows (Wear, R. J. et al, 2006). It should be noted that detailed benthic mapping was not undertaken as part of this study.

Wrack has accumulated in the Maria Creek area prior to construction of the breakwaters, as shown in Figure 10.

Figure 10: Historical wrack accumulation in 1987

Commercial wrack harvesting has historically taken place on beaches around the shoreline of Lacepede Bays, mainly along the Kingston foreshore (PIRSA, 2014). In recent years, commercial harvesting and industrial use of the wrack has ceased, with the accumulations on the beach becoming an amenity issue. In 2018/2019, KDC undertook the following wrack management within the study area:

- Removal of approximately 8,500 m^3 of wrack from the beach south of the jetty.
- Redistribution of approximately 55,000 m^3 of wrack from the beach between the jetty and the southern breakwater. This material was placed at the rear of the beach.
- Removal of approximately 40,000 m^3 of sand and wrack from within Maria Creek, where it was placed to the north of the breakwaters. The sand and wrack were mixed, so the proportion of wrack is unknown.

4.5.2. Wrack transport and accumulation dynamics

Wrack transport and accumulation dynamics are complex with the age and species of the wrack affecting its behaviour in the water (Oldham et al, 2014). Most seagrass wrack species are slightly negatively buoyant (Oldham et al, 2014) and will tend to sink to the seabed under calm conditions (i.e. areas with low currents or waves). However, wrack is readily resuspended and mobilised with wave and current action (Oldham et al, 2014). Work by Oldham et al (2014) suggests currents in excess of 0.06 m/s are likely to remobilise and transport *Posidonia* wrack species.

GHD completed a study into wrack accumulations in Maria Creek in 2013. Key findings from this study are outlined below:

- Anecdotal reports suggest that storm events rather than ambient winds tend to bring the wrack into Maria Creek in large volumes.
- Wave action is believed to be the main driver of transportation into the creek mouth, with winds pushing the wrack mass up into the creek.
- Tidal currents contribute to the process but in the absence of storm events are unlikely to be strong enough to transport large quantities of wrack.

Wrack typically accumulates on the beach berm during storm events with high water levels and waves. The wrack is often deposited above the ambient wave and water levels, where it can become compacted and decay over time (Oldham et al, 2014). Beach wrack remobilisation events are highly complex (Oldham et al, 2014) but wrack can remobilise rapidly, as seen at Maria Creek in November 2019, when an estimated 100,000 m^3 of wrack was washed from the beach within a number of days (Pers comm. KDC, 13 May 2020).

4.5.3. Wrack summary

The following summarises the key aspects of wrack transport and accumulations within the study area:

- Seagrass wrack availability is highest in early Autumn and Winter when significant seagrass meadows within Lacepede Bay shed their leaves.
- Winter storms mobilise the wrack onshore and deposit this within Maria Creek and on the wide beach berm.
- Wave action is believed to be the main driver of wrack transportation into the creek entrance, with wind and tidal currents moving the wrack deeper into the creek.
- Significant volumes of wrack are deposited on the beach berm above the typical ambient tidal water levels, where it compacts and becomes difficult to remobilize.

4.6. Summary

The key drivers of the sand and wrack accumulations within the study area are presented in Table 1.

Table 1: Summary of key drivers of coastal processes

Asset	Coastal process	
	Process	**Key metocean and physical drivers**
Maria Creek boat ramp	Sand accumulation Saturation level of southern beach, resulting in formation of sand bar across entrance channel and import of sediment into creek	Wave and current driven sediment transport: -ambient net northerly transport -storm induced 'slug type' transport -Increased sediment availability at entrance Currents: -High flood current speeds and low ebb current speeds (net importer of sediment) -Slack tide duration Creek discharge: -Low creek discharge for majority of year does not flush sand bar
	Wrack accumulation Settlement of wrack within Maria Creek	Storm conditions: -Storm waves increase wrack availability -Storm waves transport wrack into creek entrance -Storm winds and tidal currents move wrack up creek Currents: -High flood current speeds and low ebb current speeds (net importer of wrack) -Slack tide duration Creek discharge: -Creek discharge does not flush wrack
Kingston Jetty	Sand accumulation Accumulation of sand underneath jetty due to saturation levels of southern breakwater	Wave and current driven sediment transport: -ambient net northerly transport -storm induced 'slug type' transport
Kingston Foreshore	Wrack accumulation Settlement of wrack on beach between Jetty and Maria Creek	Storm conditions: -Storm waves increase wrack availability -Elevated storm water levels deposit wrack on beach berm above ambient wave and water level action

5 Maria Creek boat ramp

5.1. Approach

The presentation of concept options have been separated for the boat ramp (Section 5) and the jetty and foreshore amenity (Section 6) given the different coastal processes and effectiveness related to the study criteria (Section 1.2).

The following section focusses on reinstating and maintaining a navigable boat ramp within Maria Creek (Criteria 1). This section includes the following:

- Success criteria- what makes a successful concept to meet Criteria 1?
- First pass assessment of boat ramp concepts.
- Selected concepts for modelling.
- Concept design and cost estimates.

5.2. Success criteria

Criteria 1 relates to provision of a boat ramp during peak times (October to May), that is financially sustainable (low maintenance) through an affordable capital solution.

Success criteria for the boat ramp concepts have been developed based on the conceptual understanding of the key drivers for sand and wrack accumulation as summarised in Section 4.6, the success criteria are presented in Table 2 on the following page.

Table 2: Maria Creek boat ramp concepts success criteria

Asset	Coastal Process		Asset impact	
	Process	**Key driver**	**Impact description**	**Success criteria** Options that achieve the following are considered more effective
Maria Creek recreational boating facility	Sand accumulation Saturation level of southern beach, resulting in formation of sand bar across entrance channel and import of sediment into creek	Wave and current driven sediment transport: -Ambient net northerly transport -Storm induced 'slug type' transport -Formation of sand bar across entrance -Increased sediment availability at entrance Currents: -High flood current speeds and low ebb current speeds (net importer of sediment) -Slack tide duration Creek discharge: -Low creek discharge for majority of year does not flush sand bar	Boat Ramp Navigation Requires reactive management as entrance channel navigable depths are constantly impacted by longshore transport connection Settlement of sand within Maria Creek channel and at boat ramp reduces navigable depths inside creek	Wave and current driven sediment transport: Provide at least 100m buffer on southern beach to halt formation of sand bar at entrance Modify currents: -Decrease/eliminate net import of sediment (increase outgoing currents, decrease incoming currents) -Reduce duration of slack tide Modify creek discharge: -Increase creek discharge
	Wrack accumulation Settlement of wrack within Maria Creek	Storm conditions: -Storm waves increase wrack availability -Storm waves transport wrack into creek entrance -Storm winds and tidal currents move wrack up creek Currents: -High flood current speeds and low ebb current speeds (net importer of wrack) -Slack tide duration Creek discharge: -Creek discharge does not flush wrack	Boat Ramp Navigation Blockage of boat ramp and channel inside breakwaters Environmental Creek Discharge Blockage of creek discharge through rapid accumulation of wrack in creek	Storm conditions: -Minimise creek entrance exposure to west and north-west storm waves and winds (vertical currents) Modify currents: -Decrease potential for remobilisation of wrack into creek, currents less than 0.06 m/s during storm and ambient conditions -Decrease/eliminate net import of wrack (increase outgoing currents, decrease incoming currents) -Reduce duration of slack tide Modify creek discharge: -Increase creek discharge

5.3. First pass assessment

A first pass assessment was undertaken of all boat ramp options identified in the initial stages of the study, including those identified in the stakeholder consultation, to provide an initial screening and removal of unfeasible options to be disregarded for further assessment. Certain options may be rejected through an initial screening approach because they contravene certain requirements or are considered ineffective against the success criteria. This approach is taken to focus the more detailed assessment on the viable concepts.

Table 3 presents the first pass assessment of concepts for the Maria Creek boat ramp.

- Red crosses and items are expected to have limited positive effect or a negative effect on the relevant driver of sand and wrack accumulation.
- Yellow RFI (Requires Further Investigation) are likely to have a positive effect but this would need to be confirmed through modelling or further assessment.
- Green ticks and items are expected to have a positive effect.

Table 3: Maria Creek boat ramp first pass assessment

Concept Option	Anticipated effectiveness						Other considerations	Outcome
	Sand accumulation			Wrack accumulation				
	Provide 100m southern beach buffer	Stop net import sand	Increase creek discharge	Decrease creek entrance exposure to west and south-west storm waves and winds	Stop net import sand	Increase creek discharge		
Remove southern breakwater	X	X	X	X	X	X	Reduced wave protection at ramp in westerly storm conditions Reduced queuing distance at ramp	Net negative impact, therefore not considered further. Removal of breakwaters would be accepting Maria Creek facility is no longer operable, alternate location of boat ramp investigated in Section 6.
Reduce the width of the channel	X	RFI	X	RFI	RFI	X	Allows easier excavator access for wrack removal Narrower entrance for navigability	Potential to improve wrack and sand accumulations in creek. To be modelled in Section 5.4
Extend and alter direction of breakwaters	X	RFI	X	✓	RFI	X	More complex entrance for navigation	Potential to improve wrack and sand accumulations in creek. To be modelled in Section 5.4
Pump seawater out of Maria Creek	X	X	✓	X	X	✓	Water volume of approx. 700,000m^3 would need to be pumped out of the creek each ebb tide (twice daily) to keep entrance open at an annual operating cost of approx. $1.2M. Limited control of ebb shoal formation, which is likely to require dredging to provide a navigable entrance	Annual sand bypassing costs would to be lower than pumping costs, therefore not considered further.
Weir upstream of boat ramp	X	X	X	X	X	X	Potential for increased flooding upstream of weir, requiring additional management in flood events.	Net negative impact, therefore not considered further.
Redesign breakwaters with culverts	X	X	X	X	X	X		Net negative impact therefore not considered further.
Capital dredging campaign	✓	X	X	X	X	X	Likely to require environmental approval and management	To be modelled in Section 5.4 A capital dredging campaign should be considered for all options to reduce saturation level of southern beach
Alternate location of boat ramp	n/a							To be considered in Section 7

The first pass assessment suggests that no capital solution exists to maintain navigability at Maria Creek boat ramp without significant capital and operational expenditure related to sand management at the southern beach and entrance channel. This sand management includes:

- A significant capital dredging campaign to provide at least a 100m buffer at the southern breakwater. This aims to disconnect the formation of the sand bar, which is the key driver of sediment accumulation in the entrance channel and boat ramp.
- A dredge campaign within the creek to return navigable depths to the entrance channel and boat ramp.
    - Annual sand bypassing of at least 30,000 m^3/yr to maintain the 100m buffer and prevent reformation of the sand bar across the creek entrance.

These requirements have been included in the concept options pursued for input to the numerical models below.

5.4. Modelled options

The first pass assessment identified the following three options for further consideration through numerical model input:

- Concept 1 – On-going management with large capital dredging campaign
- Concept 2 – Extend breakwaters
- Concept 3 – Narrow entrance channel

Concept design drawings for the three options are shown in Figure 11, with descriptions outlined in Sections 5.4.1 to 5.4.3.

5.4.1. Concept 1

Concept 1 aims to test the hydrodynamics of the creek at design depth (-2.7mAHD) and with a suitable buffer between the southern shoreline and the end of the southern breakwater. This would return the Maria Creek to its original design function.

Concept 1 includes the following:

- A large capital dredging campaign extending from the southern breakwater to south of the jetty, approximate volume of 300,000 m^3 (in-situ).
- A dredging campaign to -2.7mAHD within the creek and adjacent entrance channel (width of 20m), approximate volume of 22,500 m^3 (in-situ).
- A small dredging campaign to the north of the creek entrance to -2.7mAHD, approximate volume of 10,000 m^3 (in-situ).
- Placement of the dredged/excavated material on the northern side of the northern breakwater.
- Approximately 250m of southern breakwater repairs to provide a 25 year design life (refer Section 5.6.2 for details).

5.4.2. Concept 2

Concept 2 aims to reduce the exposure of the entrance channel to waves and currents from the west and north-west, as well as modifying the tidal and wind induced current dynamics. Concept 2 includes the following:

- Removal of approximately 80m of the southern breakwater.
- An approximate 250m extension of the southern breakwater to the west and north-west.

- An approximate 60m extension of the northern breakwater to the west.
- A large capital dredging campaign extending from the southern breakwater to south of the jetty, approximate volume of 100,000 m^3 (in-situ).
- A dredging campaign to -2.7mAHD within the creek and adjacent entrance channel (width of 20m), with an approximate volume of 30,000 m^3 (in-situ).
- A small dredging campaign to the north of the creek entrance to -2.7mAHD, with approximate volume of 5,000 m^3 (in-situ).
- Placement of the dredged/excavated material on the northern side of the northern breakwater.
- Approximately 150m of southern breakwater repairs.

5.4.3. Concept 3

Concept 3 aims to narrow the entrance channel to investigate if the tidal currents can be modified to reduce the net import of material into the creek. This may also allow long-reach excavators to reach the majority of the creek entrance from the widened northern breakwater to reduce maintenance costs. Concept 3 includes the following:

- Widening the northern breakwater by 5-10m, reducing the entrance channel to 15m width (at navigable depth of -2.7mAHD).
- A large capital dredging campaign extending from the southern breakwater to south of the jetty, approximate volume of 300,000 m^3 (in-situ).
- A dredging campaign to -2.7mAHD within the creek and adjacent entrance channel, with an approximate volume of 21,500 m^3 (in-situ).
- A small dredging campaign to the north of the creek entrance to -2.7mAHD, with approximate volume of 10,000 m^3 (in-situ).
- Placement of the dredged/excavated material on the northern side of the northern breakwater.
- Approximately 250m of southern breakwater repairs.

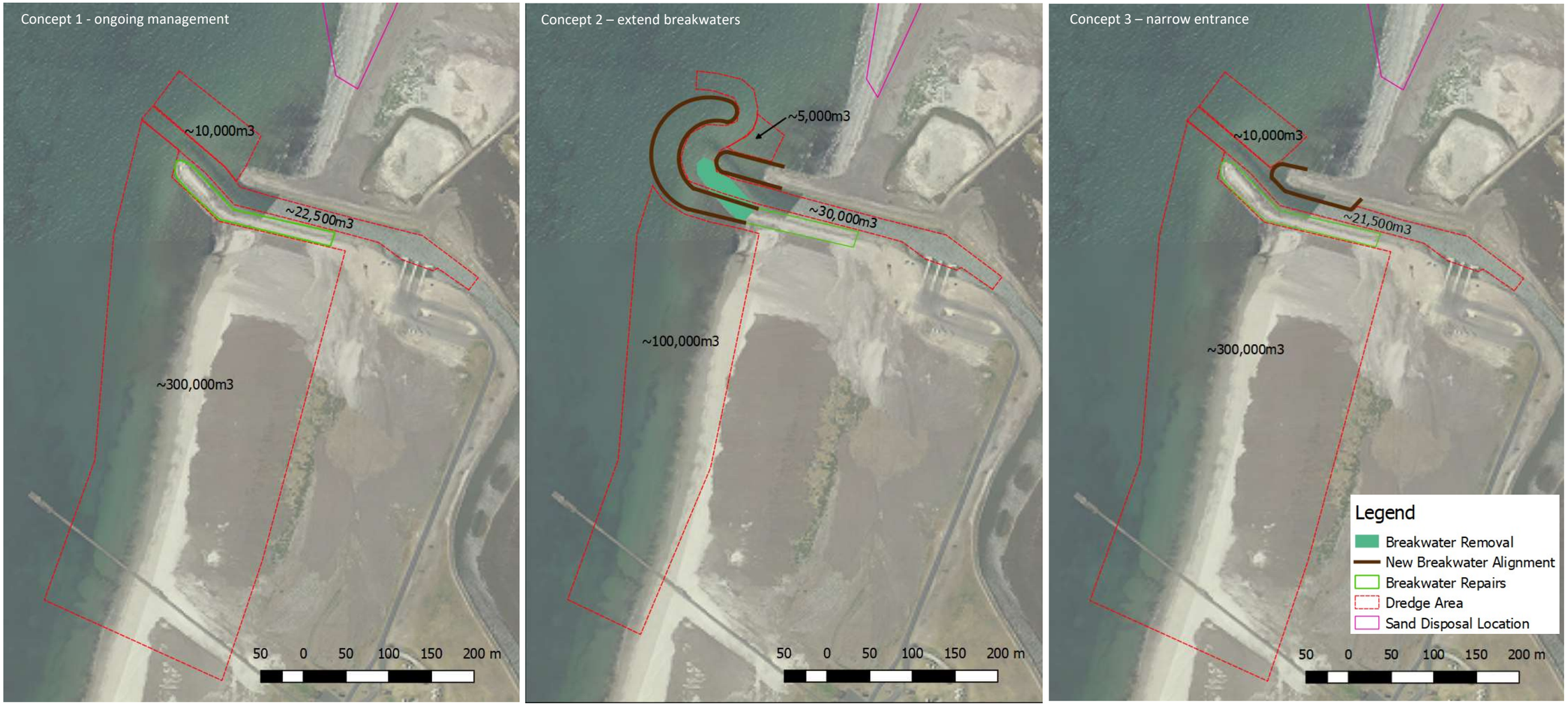

Figure 11: Boat ramp concept diagrams

5.4.4. Effectiveness review

The following section reviews the modelling results against the boat ramp success criteria presented in Table 2. Model outputs of the key wave event (Figure 12) and wind and tidal currents (Figure 13) are presented below. Full details of the modelling results for the three concepts are presented in the Modelling Summary Report (Appendix D).

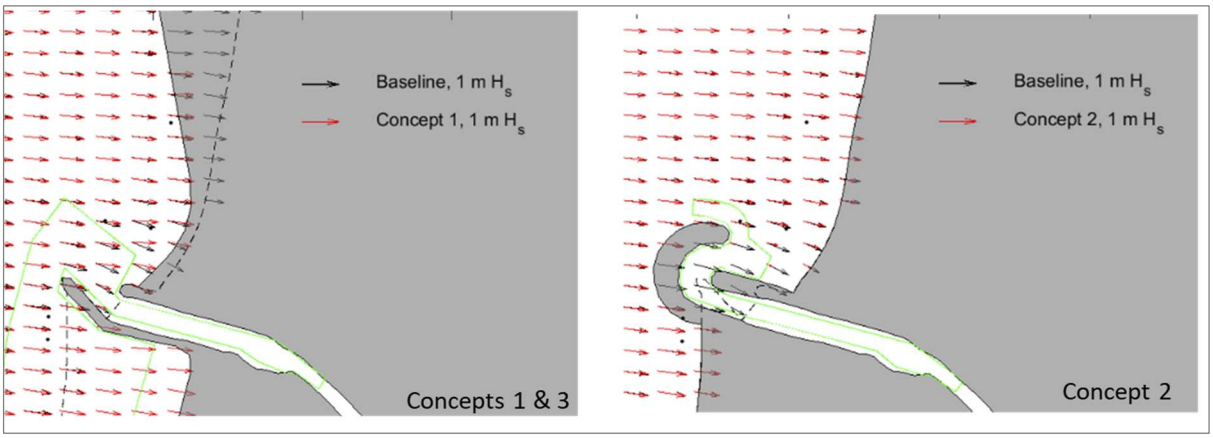

Figure 12: Wave model output during storm event for Concepts 1 and 3 (left) and Concept 2 (right)

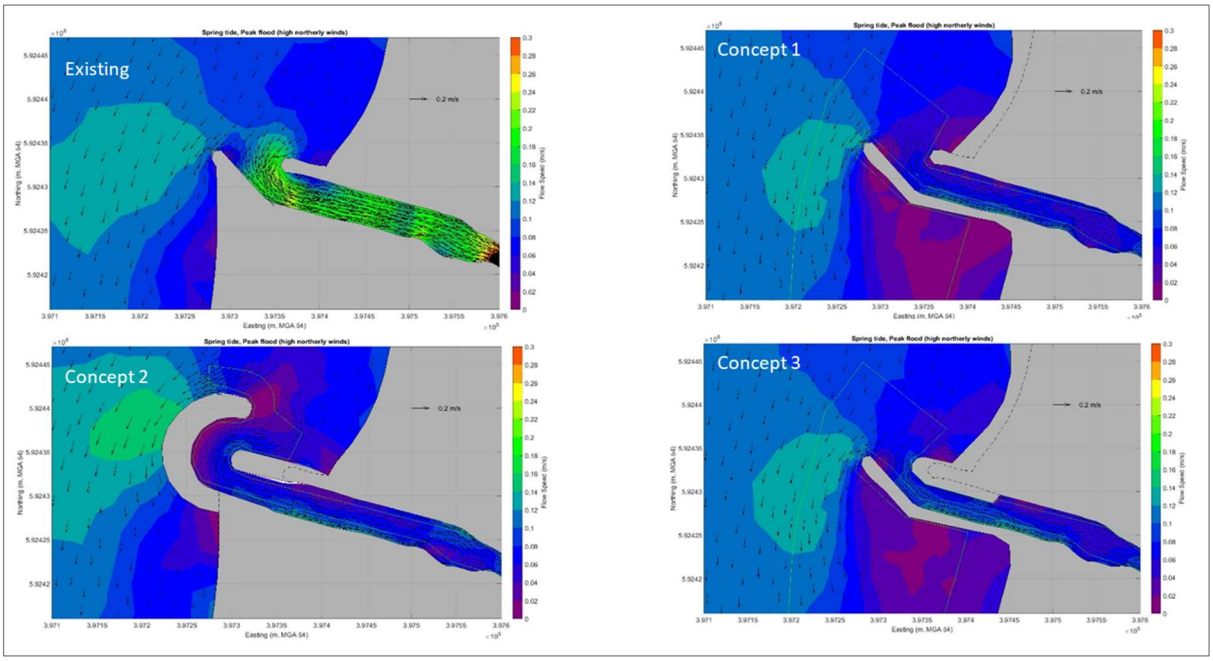

Figure 13: Hydrodynamic model output during northerly storm event combined with incoming spring tide

Sand management effectiveness:

- No structural change will be effective at stopping sand management requirements at the boat ramp. A large capital dredging campaign and on-going sand bypassing of at least 30,000m^3/yr is required to reduce the sand accumulations in the entrance channel and at the boat ramp.
- No structural changes can stop the creek being a net importer of sand, on-going dredging within the creek will be required for all options. However, these volumes will be

significantly lower than those seen in 2018 and 2019 when the southern breakwater was saturated.

- Concept 2 is predicted to create a larger wave shadow on the northern beach, which is likely to trap sediment transported into the area from the north. This will require additional management of approximately 2,000m^3/yr, moving sand from near the entrance channel to the north.
- Sand management effectiveness results are summarized in Table 4.

Wrack management effectiveness:

- The breakwater extensions under Concept 2 are likely to reduce the storm waves and incoming storm currents sufficiently to stop large wrack accumulations forming at the boat ramp. Some wrack is still expected to accumulate at the entrance, but these volumes are likely to be much smaller than under the existing breakwater alignment.
- Concepts 1 and 3 would not reduce the wave climate in the entrance channel sufficiently to prevent wrack transport into the creek and to the boat ramp.
- Wrack management effectiveness results are summarized in Table 4.

Table 4: Maria Creek boat ramp concepts sand and wrack management effectiveness

Concept	Effectiveness of structural change			Indicative annual volumes in creek	
	Sand management		Wrack management	Sand (m^3 in situ)[1]	Wrack (m^3)
	without capital dredging and annual sand bypassing	**with** capital dredging and annual sand bypassing[1]			
1. On-going Management	No structural change	No structural change	No structural change	5,000	30,000
2. Extend Breakwaters	X	✓	✓	8,000	5,000
3. Narrow Entrance	X	✓	X	5,000	30,000

Notes: 1. Effectiveness and volumes based on large capital dredge campaign and on-going sand bypassing of at least 30,000m^3/yr.

5.5. Potential environmental impact

All three concepts require a significant dredging and bypassing campaign to maintain navigability at Maria Creek boat ramp. Careful consideration and planning are required to avoid the following environmental impacts during dredging campaigns:

- Monitoring and management of any turbid plumes on seagrass communities.
- Management of return water flows for water quality.
- Consideration of de-oxygenation of the water column and resultant fish kills through release of nutrients from seagrass wrack accumulations disturbed by dredging activities.

Careful planning is also required within the Creek to avoid unnecessary dredging delays from tidal constraints. A specific Dredge Management Plan (DMP) would need to be developed for any dredging campaign, as was required for the 2017/2018 dredge works at Maria Creek (Tonkin, 2017b). This would also include significant consultation with the Coast Protection Board and DEW in the development of the DMP. Costs associated with the DMP development, consultation and subsequent environmental approvals have been included in the capital costs for Concepts 1 to 3 (Section 5.7).

5.6. Breakwater concept design

5.6.1. Concept 2 Breakwater extension

The following section summarises the conceptual breakwater structural design for the Concept 2 southern breakwater extension. The breakwaters were designed for a design life of 25 years based on Australian Standards AS4997-2005 and a 100-year ARI design event.

Given the potential exposure of the breakwaters to different storm directions and wave periods, two design wave scenarios were investigated for concept design as noted below:

- Scenario 1 – Offshore long period wave conditions propagating from the west and south-west, as modelled in CPS (2020).
- Scenario 2 – Locally generated short period wave conditions across north-west fetch using 100yr ARI wind speeds and water levels.

Table 5 presents the breakwater concept design parameters, an indicative breakwater cross-section is shown in Figure 14.

Table 5: Maria Creek Concept 2 breakwater structural design parameters

	Value	Method/reference
Steady water level at structure 100-year ARI	+1.9 mAHD	CPB steady water level + 0.3m sea level rise to 2050
Wave conditions at structure Wave height (Hs) Mean period (Tm)	2.75m 7s	Scenario 2 Locally generated waves from north-west fetch Depth limited breaking with factor of 0.55 based on (Nelson, 1994)
Rock SSDD	2.5 t/m^3	Granite quarry SSDD rock testing
External southern armour rock size (M_{50})	6t	Hudson & Van der Meer (CIRIA 2007)
Internal armour rock size (M_{50})	2t	Rear-side stability formula (CIRIA 2007) Hudson & Van der Meer (CIRIA 2007)
Filter size (Ø)	Core 0.1 to 0.8m	Filter stability (CIRIA 2007)
Crest Level	+4.5 mAHD	Overtopping discharge < 50 L/s/m for 100-year ARI design case

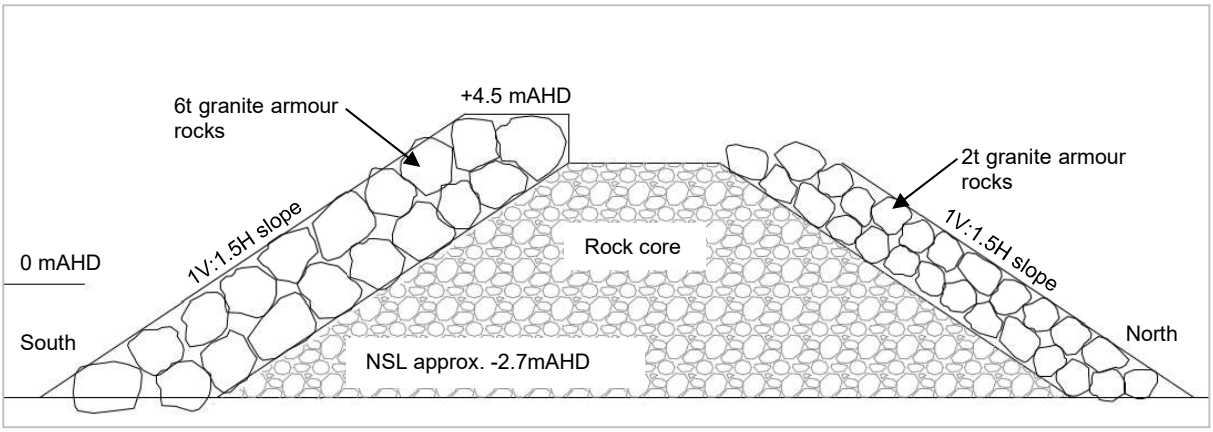

Figure 14: Indicative southern breakwater cross-section

The design section shown in Figure 14 includes a sea level rise allowance of 0.3m to 2050 in line with recommendations of the CPB for coastal development.

Should a boat ramp concept be pursued for detailed design, a local wave measurement campaign in the Maria Creek area is recommended to improve estimates of extreme design waves for detailed breakwater design. Wave measurements would need to be taken over at least a 1 month period through winter to capture a range of ambient and storm wave conditions for validation of the wave model.

5.6.2. Breakwater repairs

Repairs to the southern breakwater aim to return the breakwater to a condition suitable for a 25 year design life. The existing condition and extent of breakwater requiring repairs has been based on the southern breakwater condition inspection undertaken by Tonkin (Tonkin, 2017a).

The proposed repair works would need to be undertaken in short sections, utilising the existing breakwater as a base, working from the head, as outlined below:

- Remove external armour layers at head and along trunk. This material is stockpiled, sorted and re-used where appropriate.
- Supply and place rock core to widen the crest and increase permeability of the structure. If required, a geotextile filter could be placed between the existing core and the new core to prevent loss of fines.
- Supply and place 2 layers of 6t granite armour rocks on the southern side of the breakwater within 180m of breakwater head.
- Supply and place 3 layers of 1.5t armour rocks on the southern side of the breakwater for remaining 50m. Re-use suitable stockpiled armour.
- Supply and place 1 layer of 1.5t armour rocks over the existing internal armour layers. Re-use suitable stockpiled armour.

An indicative breakwater repair section is shown in Figure 15.

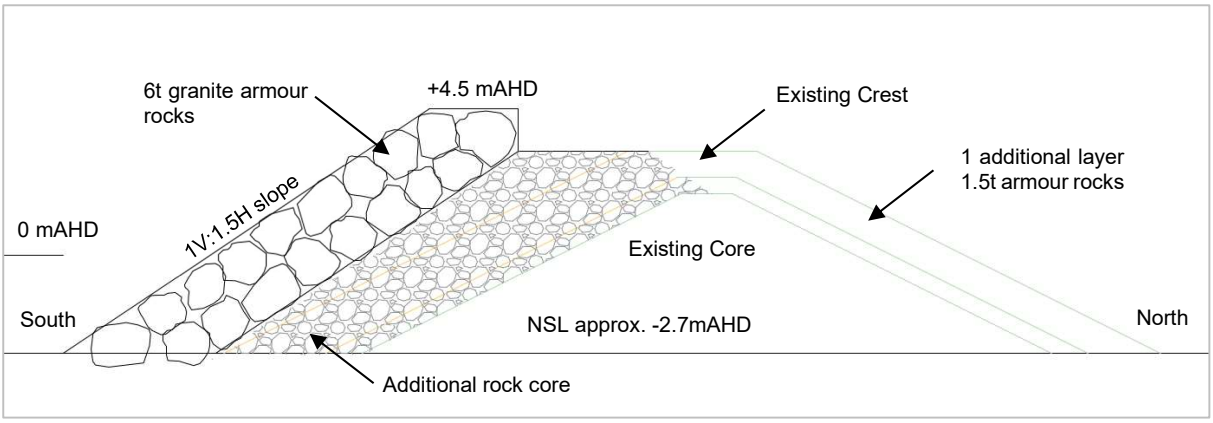

Figure 15: Indicative southern breakwater repairs section

5.7. Cost estimates

Order of magnitude capital and recurrent maintenance cost estimates for the boat ramp concepts have been prepared using a Net Present Value (NPV) analysis as presented in Table 6 below. NPV analysis provides an indication of the relative costs of the boat ramp concepts over the design life, taking into account capital and on-going costs. The cost estimates presented are to be used as a guide only, detailed costings would be developed following selection of option to be progressed to detailed design. NPV cost breakdowns for each concept are presented in Appendix E.

Table 6: Concept Cost Estimates

Concept	Capital costs		Annual Management			25 year NPV
	Structural changes/ repairs	Dredging *(Volume)*	Sand	Wrack	TOTAL	
1. On-going Mgmt	$3.3M	$2.7M *(335k m³)*	$370K	$135K	$505K	**$13.8M**
2. Extend Breakwaters	$9.8M	$1.1M *(135k m³)*	$400K	$30K	$430K	**$17.6M**
3. Narrow Entrance	$3.9M	$2.7M *(335k m³)*	$370K	$135K	$505K	**$14.4M**

NPV analysis and cost estimates were developed under the following assumptions and limitations:

- **NPV analysis**
    - A discount rate of 5% was used in the NPV calculations based on long term inflation rates from 1950 to 2020 (RBA, 2020).
    - NPV calculations were prepared over a 25 year period, the design life of the facility and breakwaters. This provides an indication of the relative costs of the boat ramp concepts over the design life.

- Costings are based on 2020 value and costs. These costings are reflective of a point in time and given the timeframes for breakwater construction are unknown, costings will need to be revised prior to commencing works.

- **Capital costs**
  - Capital dredging works costed at a rate of $8 per m^3 based on recent commercial tender rates for dredge works (received from reputable dredging company), as provided by KDC (Pers. Comm. Chelsea Burns, KDC, 18 May 2020). This includes mobilization costs from within South Australia. It is noted that the Damen CSD350 dredge is commercially and contractually obligated to be stationed at Cape Jaffa for the foreseeable future and is not available for use in the Maria Creek.
  - Supply, cartage and placement rates for armour and core were sourced from Clarke Brothers from a local granite quarry. These were confirmed against other recent supply and placement rates for similar breakwater projects in SA.
  - Existing breakwater armour and core removal rate of $30 per m^3.
  - Given conceptual nature of estimates, preliminaries of 10%, contingency of 20% and management (design, approvals and construction) of 5% were included in the total project cost.

- **On-going costs**
  - Annual sand dredging rate of $10.50 per m^3 based on recent commercial tender rates for dredge works provided by KDC (Pers. Comm. Chelsea Burns, KDC, 18 May 2020). The overall rate is higher than the large scale capital dredging given a smaller campaign volume with the same mobilization costs.
  - Wrack removal rates from within Maria Creek of $4.50 per m^3 based on existing contract rates using long-reach excavators provided by KDC (Pers. Comm. Chelsea Burns, KDC, 2 March 2020). Rates would be higher should a dredge be required to manage wrack.
  - It is assumed that breakwater repairs would be undertaken twice over the 25-year timeframe (once every 15 years – nominally at year 10 and 20), with 5% of rock armour replaced during each repair campaign. This allows for factors including armour voids created by movement/settlement of the rock armour layers and damage from significant storm(s).

5.8. Maria Creek boat ramp summary

The following are the key findings related to the Maria Creek boat ramp concept review:

- No capital solution exists to maintain navigability at Maria Creek boat ramp without significant capital and operational expenditure.
- All concepts require sand management at the southern beach and entrance channel, which includes:
  - A significant capital dredging campaign to provide at least a 100m buffer at the southern breakwater. This aims to disconnect the formation of the sand bar, which is the key driver of sediment accumulation in the entrance channel and boat ramp.
  - A dredge campaign within the creek to return navigable depths to the entrance channel and boat ramp.
  - Annual sand bypassing of at least 30,000 m^3/yr to maintain the 100m buffer and prevent reformation of the sand bar across the creek entrance.
- No concept is expected to stop the creek being a net importer of sand, on-going dredging within the creek will be required for all options. Within creek dredge volumes expected to

be significantly lower than those seen in 2018 and 2019 when the southern breakwater was saturated.

- The breakwater extensions under Concept 2 are expected to stop the large wrack accumulations historically seen in the creek, with wrack accumulations expected to remain close to the entrance in relatively small volumes.

6 Kingston jetty and foreshore

6.1. Approach

Criteria 2 and 3 focus on reducing sand and wrack accumulation on the southern beach to improve amenity of the Kingston jetty and foreshore. The following section focusses on these assets using the approach below:

- Success criteria- what makes a successful concept to meet Criteria 2 and 3?
- Review of concept options.
- Concept cost estimates and effectiveness.

6.2. Success criteria

The success criteria for Criteria 2 (Provide a jetty that services the needs of community and visitors) and 3 (Activate open spaces and facilities between the jetty and breakwaters) have been developed based on the understanding of the key drivers for sand and wrack accumulation as set out in Section 4.6. These success criteria are presented in Table 7 below.

Table 7: Kingston Jetty and Foreshore concepts success criteria

Asset	Coastal process		Asset impact	
	Process	Key driver	Impact description	Success criteria
Kingston Jetty	Sand accumulation underneath jetty due to saturation levels of southern breakwater	Wave and current driven sediment transport: -Ambient net northerly transport and storm induced 'slug type' transport trapped by Maria Creek breakwaters	Jetty Amenity Reduced amenity of Jetty due to sand accumulations. Results in loss of promenading, swimming and fishing potential	Reduce beach berm width on southern side of breakwaters
Kingston Foreshore	Wrack accumulation on beach between Jetty and Maria Creek	Storm conditions: -Storm waves increase wrack availability -Elevated storm water levels deposit wrack on beach berm above ambient wave and water level action -Increased wrack trapping potential due to Maria Creek breakwaters	Foreshore Amenity Large beach wrack accumulations are a public nuisance along foreshore	

Table 7 identifies that concepts which reduce the beach width to the south of the Maria Creek breakwaters are likely to have dual benefits to the foreshore and jetty amenity:

- a reduced beach berm width is likely to accumulate less seagrass wrack, reducing the overall volume of wrack requiring management and/or removal; and
- a reduced beach width will reduce the seabed depths under the jetty, thus improving it's amenity value.

6.3. Jetty and Foreshore concepts

The success criteria review highlighted that a reduction in beach berm width of the southern beach is likely to lead to improve jetty and foreshore amenity. As such, concepts that lead to a reduced beach berm width are generally considered to be more effective.

The three concepts outlined in Section 5.4 were investigated to improve the jetty and foreshore amenity comparable to a Do Nothing approach and an additional concept option of removing the breakwaters (Concept 4).

6.3.1. Do Nothing

The Do Nothing concept involves the following:

- Leave the breakwaters in their current form.
- Recent review of flood mapping undertaken for the Kingston Coastal Adaptation Strategy (Wavelength, 2020b) has identified a number of assets at imminent risk of inundation adjacent to Maria Creek drain. Under a Do Nothing scenario, Maria Creek would need to be kept open for environmental and flood risk mitigation purposes, with the assumption that the boat ramp would not be used. Assume removal of approximately 5,000 m^3 of sand and wrack from the creek entrance each year.
- Seek an alternate boat launching site (detailed in Section 7).

Under this concept, the beach width at the jetty is likely to continue to increase over time, as the accretion front moves south from the saturated southern breakwater. Whilst the accretion front is close to equilibrium at the jetty, empirical calculations and review of the longshore transport calculations suggest the jetty beach width will change as below:

- widen up to 15m over the coming 20 years; and
- widen up to 25m over the coming 40 years, after which the jetty shoreline is expected to reach equilibrium.

The potential stable shoreline alignment (in 40 years' time) for the Do Nothing concept is shown in Figure 16.

Figure 16: Potential shoreline position under Do Nothing concept

6.3.2. Concepts 1 to 3

The Maria Creek boat ramp Concepts 1 to 3, previously presented in Section 5.4, include a recommended capital dredging campaign on the southern beach. The capital dredging campaigns would reduce the beach widths at the jetty and foreshore up to 60m for Concepts 1 and 3 and up to 20m for Concept 2, as shown in Figure 17.

Additional dredging presented in Table 8 would be required to reduce the jetty beach width to the approximate 2012 alignment, a width reduction of 120m from the 2020 shoreline alignment shown in Figure 17.

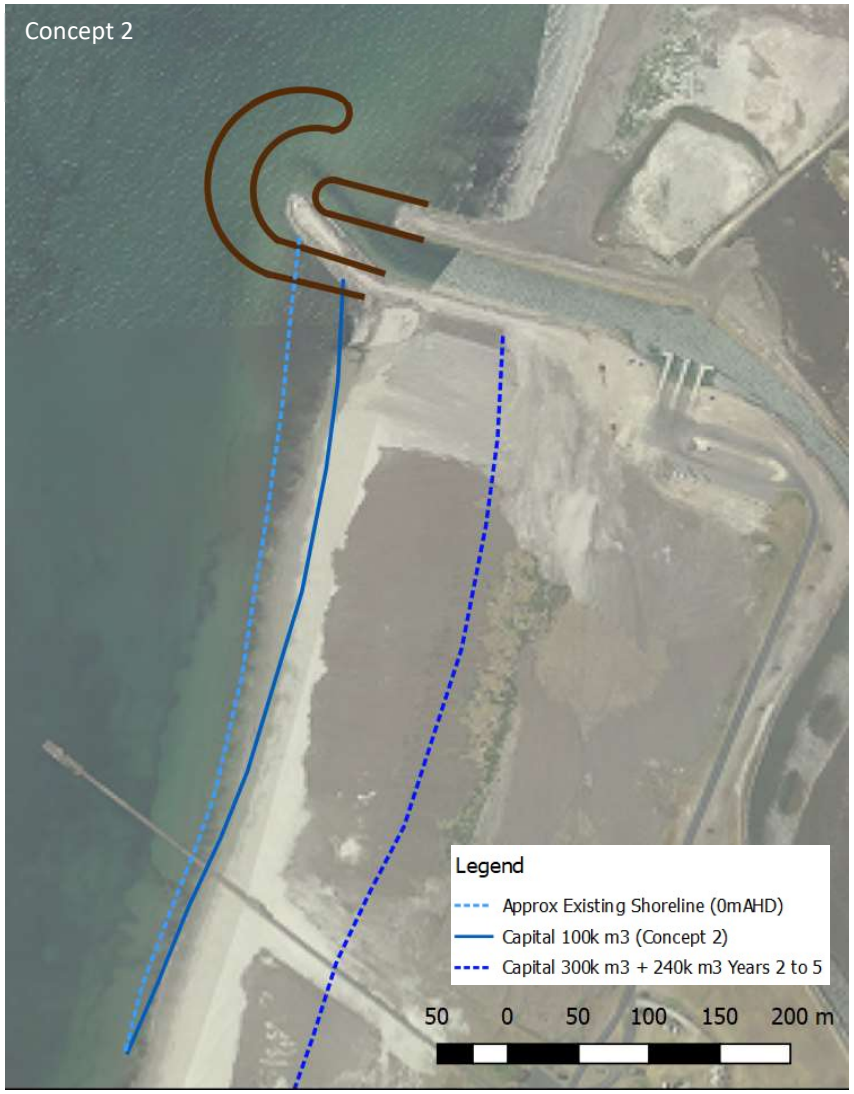

Figure 17: Potential shoreline position for boat ramp Concepts 1 to 3

Table 8: Concepts 1 to 3 additional capital dredging volumes to reduce jetty beach width by 120m

Concept	Volume (m^3)[1]					
	Year 1	Year 2	Year 3	Year 4	Year 5	Total
1	-	120k	60k	30k	30k	**240k**
2	200k	120k	60k	30k	30k	**440k**
3	-	120k	60k	30k	30k	**240k**

Notes: 1. The volumes presented do not include on-going sand bypassing of at least 30,000m^3/yr required to maintain beach width at the southern breakwater and Jetty.

6.3.3. Concept 4

Concept 4 involves removing a portion of the southern and northern breakwaters, with the aim of returning the shoreline to an approximate pre-construction alignment. This concept involves the following:

- Remove approximately 240m of the southern breakwater.
- Remove approximately 150m of the northern breakwater.
- As stated in Section 6.3.1, Maria Creek would need to be kept open for environmental and flood mitigation risk purposes only, with the assumption that the boat ramp would not be used. Assume removal of approximately 5,000 m^3 of sand and wrack from the creek entrance each year.
- Seek an alternate boat launching site (detailed in Section 7)

Wave modelling and longshore transport calculations of the option were undertaken for this concept, which are detailed in the Modelling Summary Report (Appendix D). Review of this modelling and conceptual understanding of the longshore sediment transport suggests the following shoreline changes over time:

- Immediate term (0-6 months) rapid loss of sand immediately south of the breakwaters to the Jetty, as the sediment trapped by the offset southern breakwater is released.
- Short term (1-2 years) on-going loss of sand to the south of the Jetty feeding into the southern beach area.
- Medium term (2-10 years) relatively slow retreat of shoreline given low transport fluxes (<4k m^3/yr net difference) predicted across creek following breakwater removal.
- Long term (10-40 years) the shoreline is expected to continue to erode to a similar alignment as the pre-construction shoreline.

This indicative progression is shown in Figure 18.

Figure 18: Potential shoreline positions for Concept 4

Detailed shoreline evolution modelling in conjunction with a 1-month wave measurement campaign would be recommended to confirm the findings of these conceptual investigations if this concept is pursued for detailed design. This modelling and wave measurement is likely to cost in the order of $50,000, which has been included in the capital cost presented for Concept 4.

A shorter section of the southern and northern breakwater/s could also be removed, with associated reductions in beach width at the Jetty. For example, removal of the southern breakwater extension (seaward 80m of length), would result in a shoreline similar to the green dashed line shown in Figure 18.

6.4. Concept cost estimates and effectiveness

Order of magnitude capital and recurrent maintenance cost estimates for the jetty and foreshore concepts have been prepared using a Net Present Value (NPV) analysis, as presented in Table 9. For Concepts 1 to 3, the costs presented are the total additional dredging costs that would be incurred, assuming that the Year 1 capital dredging campaign is already accounted for in the Maria Creek cost estimates.

The Do Nothing and Concept 4 costs assume the Maria Creek boat ramp is no longer operational but wrack and sand management is undertaken annually to maintain environmental creek discharges.

Table 9 also presents the anticipated beach width reduction at the Jetty location.

Table 9 presents the costs of Concept 4 to return the shoreline to the approximate pre-construction shoreline alignment. The length of breakwater removal could be fine-tuned to achieve a desired balance between jetty amenity and capital costs of breakwater removal. For example, the seaward 80m of southern breakwater could be removed to reduce jetty beach widths by approximately 50m at a relatively low capital cost of **$0.7M**.

Table 9: Jetty and foreshore concept cost estimates

Option	Capital dredging southern beach (Year 1)[1]		Additional dredging southern beach (Years 2 to 5)[1]		Structure costs (Year 1)	Annual sand and wrack management to maintain creek flows	Jetty beach width reduction (m)	25 year NPV additional costs[1]
	Volume (m^3)	Additional Cost[1]	Volume (m^3)	Additional Cost[1]				
1. On-going Management	300K	n/a	n/a	n/a	n/a	n/a	60	**n/a**
	300K	n/a	240K	$1.8M			120	**$1.8M**
2. Extend Breakwaters	100K	n/a	n/a	n/a			20	**n/a**
	300K	$1.6M	240K	$1.8M			120	**$3.4M**
3. Narrow Entrance	300K	n/a	n/a	n/a			60	**n/a**
	300K	n/a	240K	$1.8M			120	**$1.8M**
4. Remove Breakwaters	n/a				$2.4M	$22.5K	160	**$2.7M**
Do Nothing	n/a					$22.5K	-25	**$0.3M**

Notes: 1. The costs presented for Concepts 1 to 3 are in addition to the Maria Creek concept cost estimates.

The cost estimates presented are to be used as a guide only, detailed costings would be developed following selection of option to be progressed to detailed design. Cost breakdowns for each concept are presented in Appendix F.

The NPV, capital and on-going cost estimates were developed based on the assumptions previously presented in Section 5.7.

Beach wrack management rates and costs are difficult to predict at this stage, as KDC is currently reviewing beneficial use options for the wrack material, including industrial uses. As such, potential future beach wrack management costs have not been presented in this report. Nevertheless, the beach width reduction results presented in Table 9 provides an indication of the likely effectiveness of options at reducing beach berm width and thus associated beach wrack volumes between the jetty and Maria Creek.

6.5. Summary

The following summarises the key findings of the jetty and foreshore concepts review:

- Under the Do Nothing approach, the Jetty beach is expected to continue to widen up to 25 m over the coming 40 years.
- The shoreline is expected to return to the approximate pre-construction alignment under the Concept 4 (remove the breakwaters) approach. However, this change is likely to take a significant amount of time, given estimated longshore transport fluxes of less than 4,000m^3/yr across the creek entrance with the breakwaters removed.
- Concepts 1 and 3 are expected to result in a beach width reduction of 60m under the proposed 300,000m^3 capital dredging campaign. Additional dredging of 240,000 m^3 over a 4-year period would be required to return the shoreline to an approximate 2012 alignment at an additional cost of $1.8M.
- Concept 2 is expected to result in a beach width reduction of 20m under the proposed 100,000 m^3 capital dredging campaign. Additional dredging of 440,000 m^3 over a 5-year period would be required to return the shoreline to an approximate 2012 alignment at an additional cost of $3.4M.

7 Alternate boat ramp options

Concept 4 (remove the breakwaters) and the Do Nothing concept outlined in the previous section were developed on the assumption that the current location of the Maria Creek boat ramp is not acceptable and that an alternate boat launching site is required. The following section contains a high-level review of potential alternate boat ramps within the Kingston area, considering the following:

- Boat ramp service level.
- Potential locations, considering factors such as shoreline movements and wave climate.
- Indicative capital and on-going costs.

7.1. Boat ramp service levels

Department of Planning, Transport and Infrastructure (DPTI) SA have developed a list of functional service levels for boat ramps in South Australia (Appendix G). The following boat ramp service levels have been considered within the Kingston area related to Criteria 1 of the study, providing a usable ramp through the peak season from October to May:

- Service Level 1 - Informal over the beach boat ramp.
- Service Level 3 - Formal piled boat ramp with finger jetty.

A formal over the beach boat ramp with concrete panels (Service Level 2 ramp) is not recommended, as it is likely to become saturated with sand, requiring frequent removal of sand accumulation at the toe of the ramp.

A newly constructed Service Level 4 or 5 boat ramp, which includes breakwater structures, is not recommended within the Kingston area. Any breakwater structures would block sediment transport in the same way as the Maria Creek breakwaters, requiring significant on-going sand management costs. KDC also manage the Cape Jaffa boat ramp, approximately 20km south-west of Kingston SE townsite, which is a Service Level 4 or 5 ramp offering protected boat launching and retrieval.

7.1.1. Service Level 1 - Informal over the beach

A Service Level 1 ramp provides limited infrastructure to assist ramp users, requiring launching and retrieval over the beach. Historically, several locations have been used for informal launching of boats within the Kingston area, including at Johnston Street and Pinks Beach (Figure 22). An informal ramp has the benefit of being relatively flexible to on-going changes to shoreline movement.

These types of ramp are typically only suitable for small boats and experienced users, as they can be subject to a range of difficult conditions, including:

- Sandy beach surface, which can bog heavy trailers or inexperienced beach drivers and are generally unsuitable for 2WD vehicles.
- Wave and surge action on the beach face with no jetty to hold the boat during launch and retrieval.

One option to reduce the potential for bogged vehicles is to install a temporary ramp across the beach during the peak months of October to May. District Council of Yankalilla (DCY) have managed a temporary ramp at Normanville using a proprietary solution (Versadeck panels from Envirex), presented in Figure 19 and Figure 20. DCY Chief Operating Officer Andy Baker was contacted regarding their experience with the temporary ramp, as summarised below (Pers. Comm. Andy Baker, DCY, 25 May 2020):

- The ramp pieces should be kept out of the ocean, as they cannot withstand any wave action.
- The smaller sized Versadeck panels were often removed by members of the public to assist with bogged vehicles on other parts of the beach.

- The ramp pieces abrade over time due to sand and tyre wear, which can create sharp edges and a pedestrian safety hazard.
- In general, DCY would not recommend installing the smaller sized Versadeck panels (2.4mx1.2m).

Envirex offer a larger ramp panel option in the order of 2m x 4m, with each panel weighing approximately 130kg. Whilst more difficult to setup and remove, they are less likely to be taken by the public. The manufacturer noted two methods of installation for these larger panels:

- Hiab loader crane on the back of a 4WD truck/ute lifting the panels into place in series.
- A bobcat with forks longer than 1m with the panels slung with a chain.

The mats are pinned down with long, stainless steel screws driven in by a large drill. As noted previously, the panels can not withstand wave action and would need to be removed prior to any significant storm events. As such, it's recommended that the panels are removed at the end of each peak season prior to any winter storms.

Figure 19: Normanville example temporary boat ramp using Versadeck panels (Envirex, 2020)

Figure 20: Example Versadeck panels on storage container (Envirex, 2020)

7.1.2. Service Level 3 - Formal piled ramp and finger jetty

Any boat ramp structure built across the beach is likely to trap longshore sediment, requiring on-going management of siltation at the toe of the ramp. To limit the sediment trapping potential of the ramp structure, a piled ramp could be constructed over the beach to allow boats to launch in deeper water. A finger jetty structure is also recommended to improve public safety of boat launching and retrieval during poor weather conditions.

An example of a piled ramp is shown at Port Kennedy, Western Australia in Figure 21. The Port Kennedy ramp has two lanes and a finger jetty and would fit within DPTI's Service Level 3 category.

Figure 21: Port Kennedy piled boat ramp and finger jetty

City of Rockingham manage the ramp and were contacted regarding their management experience, as summarised below (Pers. Comm. Matt Donaldson, City of Rockingham, 29 April 2020):

- Frequent and pro-active wrack removal is required underneath the ramp structure during winter months to prevent wrack accumulations from blocking sediment transport and thus creating a siltation issue at the end of the ramp.
- The ramp is subjected to significant swell and surge effects, particularly during winter months and requires signage to warn users of the potential risks from waves.

This type of ramp would need to be built on a relatively stable section of shoreline to prevent erosion or siltation of the ramp toe. Further discussion of suitable ramp locations is outlined in the following section.

7.2. Review of feasible locations

Alternate boat ramp locations have been considered from Butcher's Gap drain in the south to Toops Rd in the north within close proximity of Kingston SE townsite (Figure 22). Formal, piled boat ramps should be sited at locations with minimal long-term shoreline movement often referred to as a null point. These null points can be identified through review of historical shoreline movements, as shown in Figure 22.

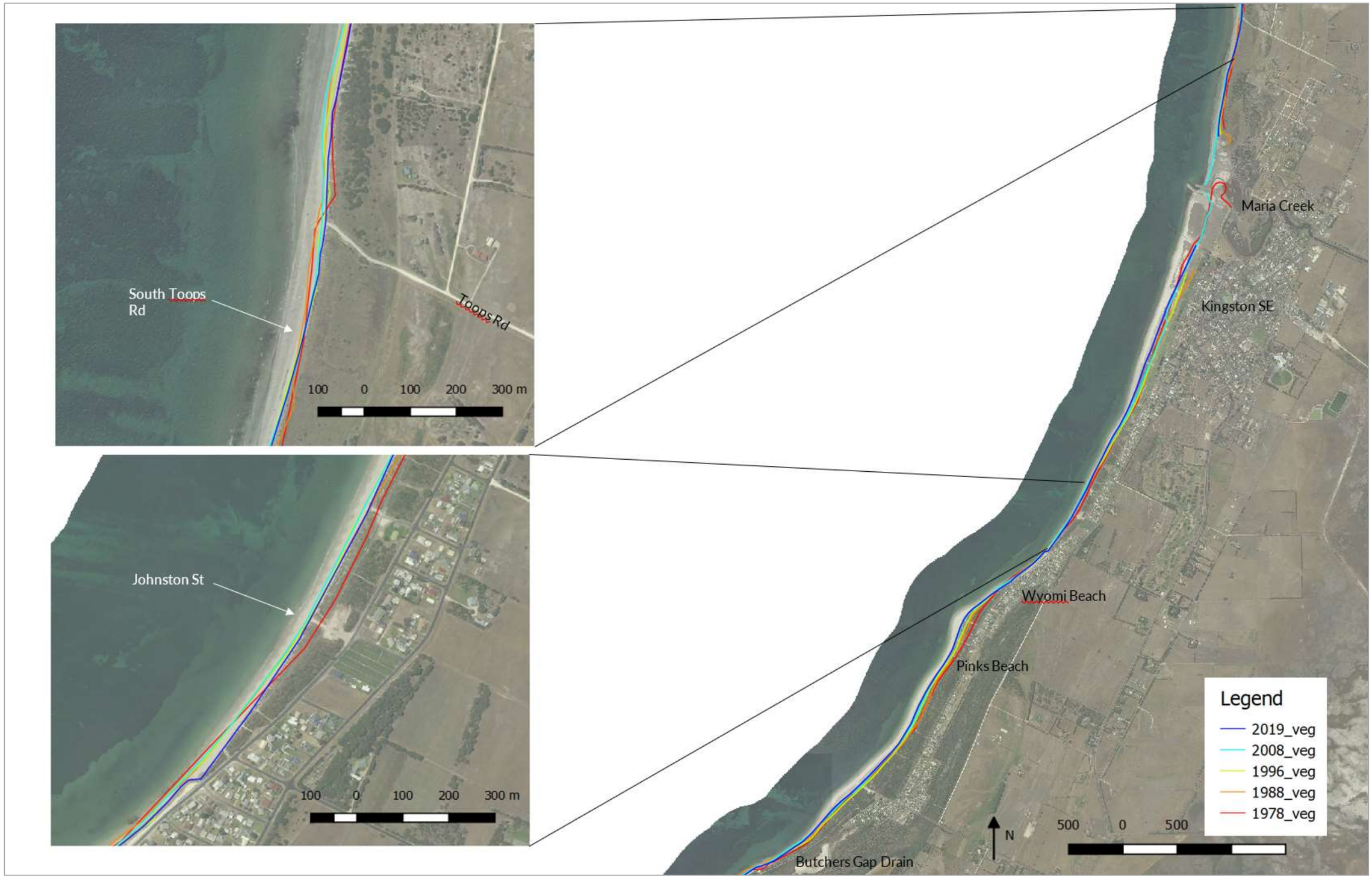

Figure 22: Historical shoreline movements and null points

Two null points were identified within the search area:

- Johnston Street, approximately 3.3km south of Maria Creek. Shoreline has been relatively stable over last 30 years since 1988. A broken concrete ramp currently exists at Johnston Street, which would need to be removed for any ramp options at this site.
- South Toops Road, approximately 1.3km north of Maria Creek. Shoreline has been relatively stable over last 40 years since 1978.

A null point would also exist at the end of the Maria Creek southern breakwater for the Do Nothing approach, as the breakwater is saturated and the shoreline is no longer accreting. A piled boat ramp could be constructed off the straight section of southern breakwater into deeper water, as shown in Figure 23. However, this is unlikely to be a viable option given the high risk nature of an approximate 300m reverse along the breakwater. Breakwater repairs would also be required to ensure the access road is in a suitable condition for on-going use. Providing a turnaround and reversing area for boat trailers would involve significant additional costs. On balance, given the realistic accessibility during peak season and high risk safety concerns, this option is not recommended.

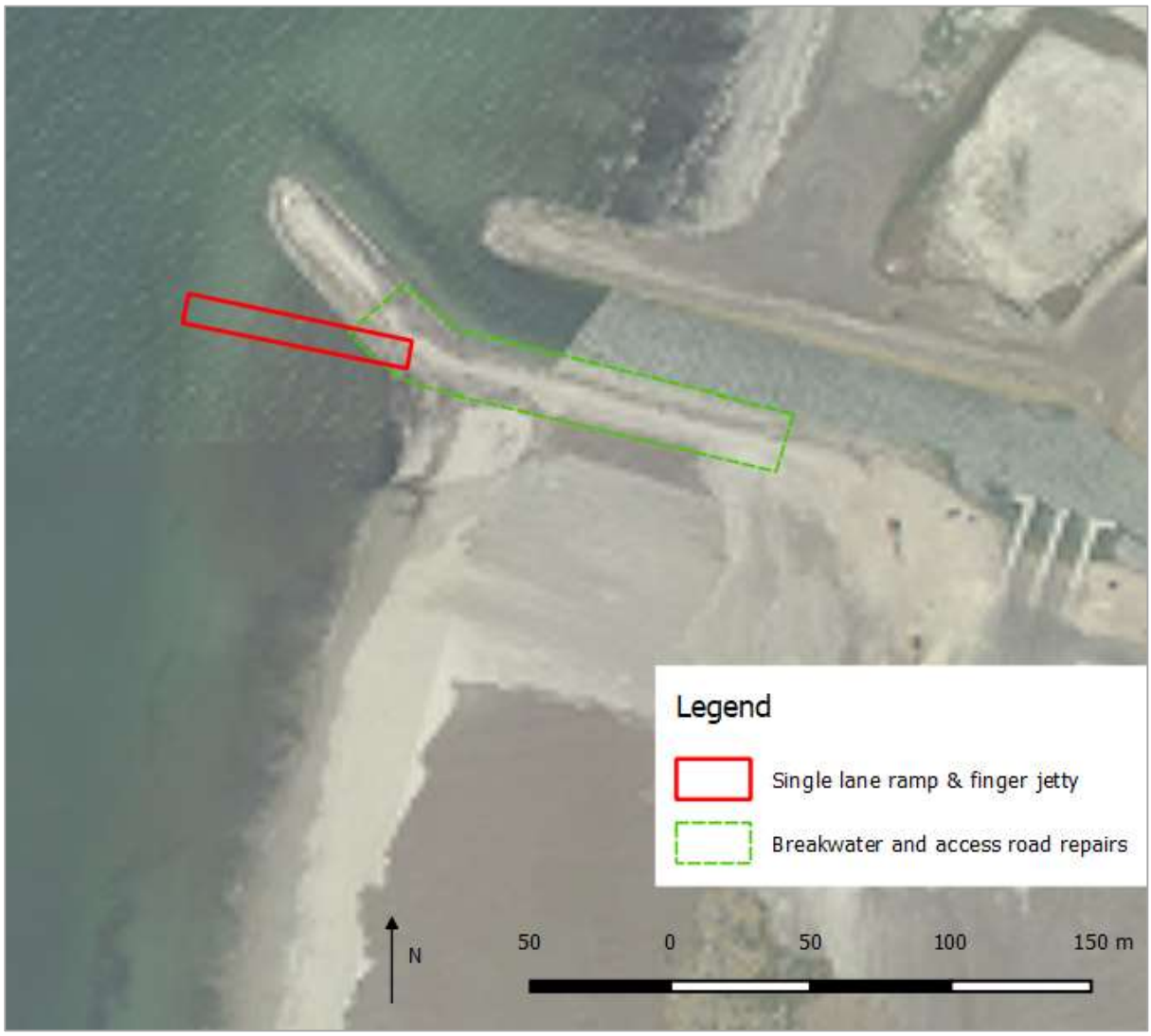

Figure 23: Single lane ramp and finger jetty concept at end of southern breakwater

Future shoreline movements and piled ramp feasibility at the two feasible locations will be significantly impacted by the removal or otherwise of the Maria Creek breakwaters. As such, different ramp locations have been considered for the likely future shoreline movements for the Do Nothing and Concept 4 (remove the breakwaters) concepts below.

7.2.1. Do Nothing

The Maria Creek breakwaters are kept in place under the Do Nothing concept. As such, the leading edge of the sand accretion front is expected to continue moving to the south of the breakwaters towards Johnston Street. Parametric equations suggest that the Johnston Street shoreline may accrete up to 50m in the coming 20 years and up to 75m over the coming 40 years. Therefore, a formal piled ramp would not be recommended at Johnston Street under the Do Nothing approach given likely future shoreline accretion.

The remaining feasible options have been presented in Table 10, along with an indicative capital cost and on-going management cost.

Wave conditions were also extracted from the PCS wave model at the three locations and are presented in Table 10 to give an indication of how frequently significant wave heights between October and May exceed the 0.2m wave height limit set out in the Australian Standards (AS3962:2001). This suggests that throughout summer the wave conditions frequently exceed 0.2m at both ramp locations, which may become unsafe at times. This may require management through signage or potentially closure of the ramps at times of increased wave heights or surging.

Indicative capital costs are based on the following:

- Removal of existing, damaged concrete slab at Johnston Street, using 2 excavators and side tipper at an estimated cost of $20,000 (Pers comm. John Clarke, 15 June 2020).
- Temporary over the beach ramp panels at $130 per m^2 provided by supplier.
- Formal piled ramp costs based on construction costs of Port Kennedy boat ramp modified for the site conditions (i.e. beach width and number of ramps) and present day costs.

Indicative on-going maintenance rates are outlined below:

- Temporary ramp annual installation and removal cost of $20,000 per year.
- Temporary ramp replacement of 2 panels per year and replacement of all panels every 10 years for abrasion.
- Formal piled ramp wrack management costs at a rate of $4.50 per m^3.
- Formal piled ramp repairs of 5% of capital cost every 10 years.

NPV calculation assumptions were previously presented in Section 5.7.

7.2.1. Concept 4 Remove Breakwaters

Under Concept 4, the two Maria Creek breakwaters are removed and sand transport across the entrance channel is expected to increase. The shoreline to the north of Maria Creek is likely to be affected for several decades while the shoreline stabilises. Therefore, formal ramp at South Toops Road would not be recommended under this concept, leaving Johnston Street as the recommended location.

The indicative capital cost and on-going management costs for an informal and formal boat ramp at Johnston Street are presented in Table 10. The last column in Table 10 also presents the total cost for Concept 4 including breakwater removal costs and on-going management of environmental discharge at Maria Creek for both concepts. Cost estimate assumptions were presented in the previous section.

Table 10: Alternate boat ramp review

Concept	Location	Service Level	Description	Wave Climate % exceedance of Hs=0.2m Oct - May	Alternate boat ramp costs			Total 25 year NPV[2]
					Indicative Capital Cost	Wrack management	25 year NPV	
Do Nothing	Johnston St	1	No infrastructure	26%	$20K	n/a	**$0.02M**	**$0.3M**
		1	Single lane ramp temporary over the beach ramp panels	26%	$55K[1]	n/a	**$0.5M**	**$0.8M**
	South Toops Rd	3	Piled two lane ramp Finger jetty	34%	$6.6M	$135k	**$9.3M**	**$9.6M**
Concept 4 Remove Breakwaters	Johnston St	1	No infrastructure	26%	$20K	n/a	**$0.02M**	**$2.7M**
		1	Single lane ramp temporary over the beach ramp panels	26%	$55K[1]	n/a	**$0.5M**	**$3.2M**
		3	Piled two lane ramp Finger jetty	26%	$5.4M	$135k	**$7.9M**	**$10.6M**

Notes: 1. Does not include installation or removal of ramp panels each year. These have been included in total 25 year NPV calculations.
2. Including capital costs associated with removal of breakwaters at Maria Creek and per annum dredging costs for environmental and flood mitigation purposes only.

7.3. Summary

Three sites were identified as potential alternative boat ramp locations in the Kingston area based on minimal long-term shoreline movements:

- Johnston Street.
- South Toops Road.
- The end of the Maria Creek southern breakwater.

A single lane ramp at the end of Maria Creek southern breakwater is not recommended given the realistic accessibility during peak season and high risk safety concerns.

Of the two feasible alternate sites, Johnston Street has the smallest wave climate and is a historical site for over the beach boat launching and has a small car and trailer parking area in the vicinity. Should an informal ramp (Service Level 1) be pursued, Johnston Street is the recommended location. The existing, damaged concrete ramp at Johnston Street would need to be removed for any ramp options at this site. A temporary ramp could be installed and removed over the peak months from October to May for a 25 year cost of approximately $0.8M, including environment creek flow management costs.

Formal, piled ramps are likely to have significant capital and on-going costs with 25 year NPV estimates in the order of $10M to $11M. All piled ramp options would still require rapid management of any wrack accumulations to prevent blocking sediment transport and subsequent siltation at the end of the ramp and to minimise wrack amenity issues associated with large accumulations on the beach.

For the Do Nothing approach, Johnston Street is not suitable for a formal piled ramp structure due to the anticipated on-going accretion front to the south of Maria Creek. The remaining option at South Toops Road has a much wider beach and minimal existing infrastructure and access, resulting in a significantly higher capital cost for the piled ramp facility compared to Johnston Street. This ramp location also has significantly higher wave exposure and would require active management with signage and potentially closure during large wave events even in summer months.

Should the breakwaters be removed, Johnston Street is the recommended location for a formal piled ramp with an estimated 25 year NPV of $10.6M.

8 Best practice management

There are a number of actions that represent good coastal management practice, which can be pursued by KDC without the need for compromise or significant capital-raising. Such actions can improve resilience and preparedness for coastal processes impacts without limiting the ability to change a management approach in the future. These actions are discussed in more detail below and summarised as recommendations in Section 9.

8.1. Shoreline monitoring

DEW have established several beach and nearshore profiles across the KDC coastline. Whilst providing a useful reference point, a number of these profiles have not been surveyed in recent years and they are relatively sparse across the coast.

Establishment of a series of beach and nearshore monitoring profiles is recommended to improve the understanding of shoreline movement and sediment transport adjacent to critical infrastructure such as Maria Creek, Kingston Jetty, Wyomi seawall and Cape Jaffa. These profiles should be monitored on at least an annual, preferably a twice annual basis to observe any seasonal changes in shoreline position. The profiles should extend from the rear of the dune to the depth of closure (at least -4 mAHD) and should be spaced at most 200m apart. This monitoring would be particularly important should one of the Maria Creek boat ramp concepts be implemented to direct the on-going sand bypassing requirements.

8.2. Works documentation

In conjunction with the shoreline monitoring above, on-going reporting and detailed record keeping of coastal management works is recommended to improve the understanding of shoreline movement and sediment transport within the study area. This is particularly important given the complexity of the coastal management issues at Kingston. A close-out report should be prepared for each of the following management works:

- Dredging and bypassing works, including details of:
  - Volumes
  - Placement areas
  - Timing
  - Estimated proportions of wrack vs sand quantities
- Creek and beach wrack removal works:
  - Volumes
  - Placement areas
  - Timing
- Nourishment works
  - Volumes
  - Source sites
  - Placement sites
  - Timing

Should it not be possible to prepare a close-out report for each set of works, an annual summary report of coastal works is recommended for future reference.

8.3. Wave measurements

Should a boat ramp concept be pursued for detailed design, a local wave measurement campaign in the Maria Creek area is recommended to improve estimates of extreme design waves for breakwater and ramp design. Wave modelling undertaken by PCS (Appendix D) suggests that the storm wave heights

at Maria Creek are highly sensitive to the input friction factor. Whilst the use of wave measurements at Cape Jaffa and Rivoli Bay are considered reasonable for this concept design phase, a local measurement campaign would help to refine the wave modelling results at Maria Creek for detailed design. Wave measurements would need to be taken over at least a 1 month period through winter to capture a range of ambient and storm wave conditions for validation of the extreme wave model results.

9 Summary of key findings & recommendations

The following summarises the key findings and recommendations of the concept study:

- No concept has been identified that meets all four of the criteria nominated by KDC.
- None of the concepts meet Criteria 1. All concepts that provide a boat launching facility with high levels of service come with significant capital expenditure (approximately $3M-$10M) and do not substantially reduce maintenance costs compared to the current annual spend required at Maria Creek. On-going sand management for all Maria Creek concepts includes:
    - A significant capital dredging campaign to provide at least a 100m buffer at the southern breakwater. This aims to disconnect the formation of the sand bar, which is the key driver of sediment accumulation in the entrance channel and boat ramp.
    - A dredge campaign within the creek to return navigable depths to the entrance channel and boat ramp.
    - Annual sand bypassing of at least 30,000 m3/yr to maintain the 100m buffer and prevent reformation of the sand bar across the creek entrance.
- Based on the understanding of the key drivers of sand and wrack accumulation, three concepts were investigated for further modelling and cost development, as summarised in Table 11.
    - Of the three concepts to maintain the existing facility, no structural change to the design of the breakwaters will reduce the sand management requirements within Maria Creek.
    - Of the three concepts to maintain the existing facility, only Concept 2 (breakwater extensions) has the potential to reduce the wrack management requirements within Maria Creek.
- A Dredge Management Plan would be required to gain environmental approval for dredging associated with all Maria Creek concepts. This would need to consider potential dredge plumes, return water quality and anoxic condition creation from released seagrass wrack nutrients within the Creek. Significant consultation with DEW would be required to gain environmental approval for the dredging works.
- The following summarises the key findings of the jetty and foreshore concepts review:
    - Under the Do Nothing approach, the jetty beach is expected to continue to widen up to 25 m over the coming 40 years.
    - The shoreline is expected to return to the approximate pre-construction alignment under Concept 4 (remove the breakwaters) approach. However, this change is likely to take several decades, given estimated longshore transport fluxes of less than 4,000m^3/yr across the creek entrance with the breakwaters removed.
    - Concepts 1 and 3 are expected to result in a beach width reduction of 60m at the jetty under the proposed 300,000m^3 capital dredging campaign. Additional dredging of 240,000 m^3 over a 4-year period would be required to reduce the beach width at the jetty by a further 60m (120m total) at an additional cost of $1.8M.
    - Concept 2 is expected to result in a beach width reduction of 20m at the jetty under the proposed 100,000 m^3 capital dredging campaign. Additional dredging of 440,000 m^3 over a 5-year period would be required to reduce the beach width at the jetty by a further 60m (120m total) at an additional cost of $3.4M.
    - The anticipated jetty and foreshore concept costs are presented in Table 11.

Table 11: Summary of boat ramp and jetty concepts

Concept	Effectiveness of structural change		Maria Creek boat ramp Maintain navigable boat ramp Minor improvement at Jetty			Alternate boat ramp location Develop alternate boat ramp for Do Nothing and Concept 4			Jetty Return shoreline to 2012 alignment	Total 25year NPV[2]
	Sand in creek	Wrack in creek	CAPEX	OPEX	25 year NPV	Location	Service Level	25 year NPV	25 year NPV[1]	
1. On-going Mgmt	n/a	n/a	$6.0M	$505K	**$13.8M**	n/a – Maria Creek boat ramp operational			$1.8M	**$15.6M**
2. Extension	X	✓	$10.9M	$430K	**$17.6M**				$3.4M	**$21.0M**
3. Narrow Entrance	X	X	$6.6M	$505K	**$14.4M**				$1.8M	**$16.2M**
Do Nothing	n/a – Maria Creek boat ramp not operational					Johnston St	1 Over the beach ramp	**$0.02M**	n/a Jetty shoreline predicted to accrete 25m over coming 40 years then stabilise	**$0.3M**
							1 Over the beach ramp with temporary ramp panels	**$0.5M**		**$0.8M**
						South Toops Rd	3 Two lane ramp	**$9.3M**		**$9.6M**
4. Remove Breakwaters						Johnston St	1 Over the beach ramp	**$0.02M**	$2.4M Breakwater removal	**$2.7M**
							1 Over the beach ramp with temporary ramp panels	**$0.5M**		**$3.2M**
							3 Two lane ramp	**$7.9M**		**$10.6M**

Notes: 1. The costs presented for jetty Concepts 1 to 3 are in addition to the Maria Creek concept cost estimates.
2. Includes costs for wrack and sand management within the creek, including to maintain environmental and flood mitigation flows (Do Nothing and Concept 4) 25 year NPV of $0.3M. Does not include costs to manage wrack on the beach.

- Alternate boat ramp locations were investigated, as outlined below:
    - A formal over the beach boat ramp with concrete panels (Service Level 2) is not recommended, as it is likely to become saturated with sand, requiring frequent management of sand accumulation at the toe of the ramp. Additionally, this service level does not provide a finger jetty, which is recommended to improve public safety given the wave exposure in the area.
    - A newly constructed Service Level 4 or 5 boat ramp is not recommended as any breakwater structures would block sediment transport in the same way as the Maria Creek breakwaters, requiring significant on-going sand bypassing costs. It is acknowledged that the existing facility at Cape Jaffa is a Service Level 4 or 5 ramp offering protected boat launching and retrieval.
    - Should an informal ramp (Service Level 1) be pursued, Johnston Street is the recommended location. A temporary ramp could be installed and removed over the peak months from October to May for a 25 year cost of approximately $0.8M, including environment creek flow management costs for Maria Creek. The existing, damaged concrete ramp at Johnston Street would need to be removed for any ramp options at this site.
    - A piled ramp at the end of the southern breakwater is not recommended due to the reduced accessibility of a one lane ramp during peak season and high risk safety concerns.
    - A piled ramp and finger jetty at Johnston Street in conjunction with Maria Creek breakwater removal is the recommended approach for a Service Level 3 ramp at a 25 year cost of approximately $10.6M. Whilst more expensive than the South Toops Rd alternative, this approach has several benefits, including:
        - Improved jetty and foreshore amenity, through removal of the breakwater structures.
        - A lower wave climate than South Toops Rd.
    - All piled ramp options would still require rapid management of any wrack accumulations to prevent blocking sediment transport and to minimize wrack amenity issues.
    - Indicative alternate ramp costs and wave climate are summarized in Table 11.
- Under a Do Nothing or Concept 4 (remove the breakwaters) approach, Maria Creek would still need to be kept open for environmental and flood risk mitigation purposes, with the assumption that the boat ramp would not be used. Assume removal of approximately 5,000 m^3 of sand and wrack from the creek entrance each year.
- A shoreline monitoring program and improved reporting of coastal management works is recommended to improve understanding of shoreline movement and sediment transport rates in areas of critical infrastructure. Should an option be pursued for detailed design a local wave measurement campaign is recommended to refine the wave height estimates.
- A shoreline monitoring program and on-going reporting of coastal management works is recommended to improve understanding of shoreline movement and sediment transport rates in areas of critical infrastructure. Should an option be pursued for detailed design a local wave measurement campaign is recommended to refine the wave height estimates.
- In light of community expectations, and financial pressures the following viable pathways are presented:
    - **Pathway 1:** Concepts that **provide a formal boat ramp and reduced beach widths at jetty**, however **significant capital and on-going costs**:
        - Concept 4 (remove the breakwaters) with a formal, piled two-lane boat ramp at Johnston Street has the lowest long-term cost of any option to provide a formal boat ramp and reduced jetty beach widths, with a 25

year NPV of approximately **$10.6M**. This option is expected to reduce beach widths and wrack volumes at the jetty and foreshore and would provide a two-lane boat ramp with finger jetty during peak season. However, the wave conditions at the ramp are expected to frequently exceed 0.2m during summer and would require signage and potential closures under excessive wave conditions. Pro-active wrack management would be required at the ramp to prevent siltation issues and to minimize wrack amenity issues.

        - Maria Creek Concept 1 (on-going management) is the recommended option should KDC wish to provide a protected four-lane boat ramp (waves less than 0.2m) at a total cost of approximately **$15.6M** over 25 years. This would require on-going sand bypassing and removal of large wrack accumulations from within the creek.

    - **Pathway 2:** Concepts with **minimal capital and on-going costs**:
        - The lowest cost option (**$0.3M)** is to Do Nothing, with an informal over the beach ramp at Johnston Street. The jetty shoreline is predicted to accrete an additional 25m over the coming 40 years under this concept.
        - A temporary boat ramp could be trialed to improve usability at a 25 year cost of **$0.8M**, including environmental management costs at Maria Creek. Larger boats could use Cape Jaffa, which is being maintained as a protected boat launching facility.
        - The seaward 80m extension of the southern breakwater could be removed to reduce jetty beach widths by approximately 50m at a relatively low additional cost of **$0.7M**.

- Considering KDC's criteria, and applying some judgement to further refine the options within Pathway 2, Wavelength recommends the following for further consideration by KDC:
    - Remove the seaward extension of the Maria Creek southern breakwater to reduce southern beach widths and improve jetty and foreshore amenity.
    - Develop an informal 'over the beach' boat ramp at Johnston Street. A temporary ramp should be trialed if residents are having difficulty with beach conditions over the peak use period.
    - Keep Maria Creek open for environmental purposes and to manage flood levels in the creek.

The following key recommendations are provided for KDC's consideration:

- In light of KDC's evaluation criteria, it is recommended that the refined version of Pathway 2 above be progressed further by KDC.
- The results of the evaluation are sensitive to the evaluation criteria, before proceeding with any works KDC should consider and reconfirm the criteria and their relative weighting/importance.
- This study focuses specifically on the management of coastal infrastructure in the area bounded by the Kingston Jetty and Maria Creek. Pathway 2 results in the likely loss of Maria Creek as a usable boat launching facility, leaving Cape Jaffa as the remaining boat launching facility to service the Kingston Area. Prior to further works at Maria Creek, it is recommended that KDC confirm that Cape Jaffa is the preferred facility to service the needs of the Kingston area for the long term.
- It is recommended that KDC implements the ongoing coastal management best practices relevant to the study area as summarised in this report.

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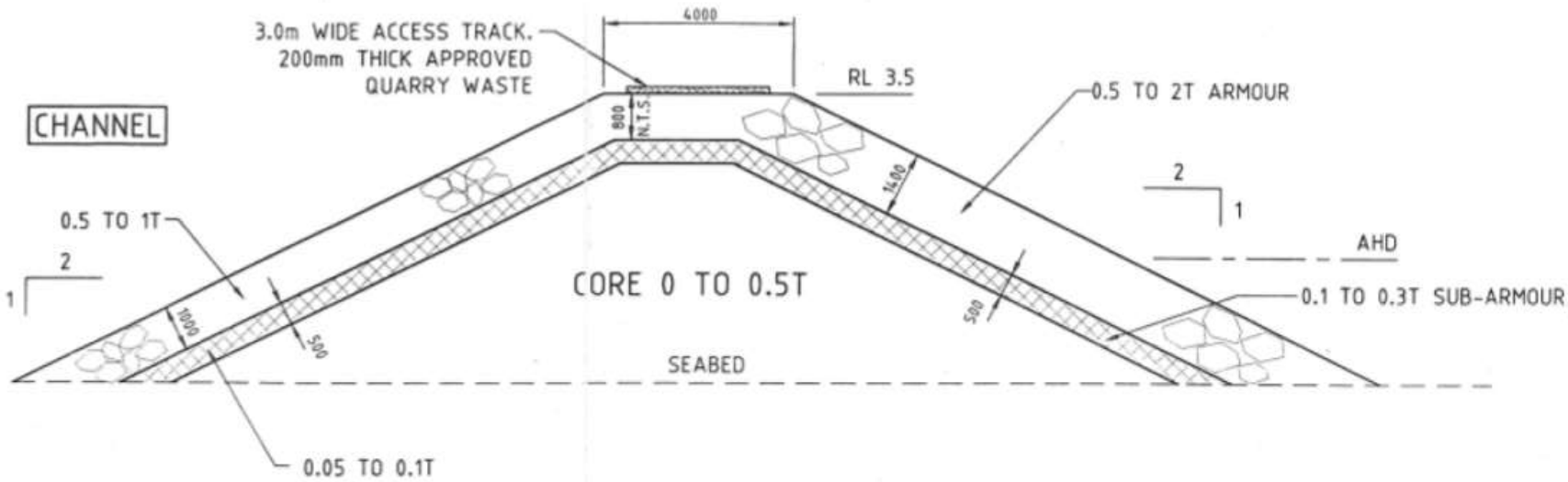
Appendix A Maria Creek breakwater design drawings (from Tonkins, 2017)

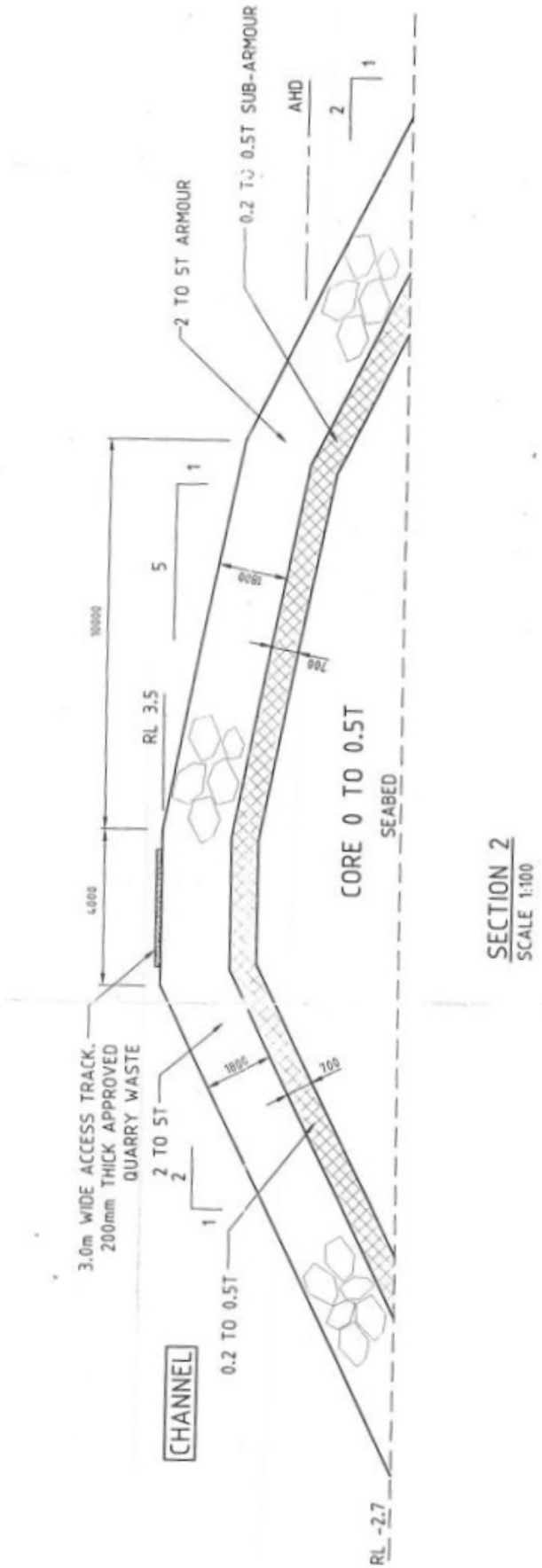


SECTION 1
SCALE 1:100

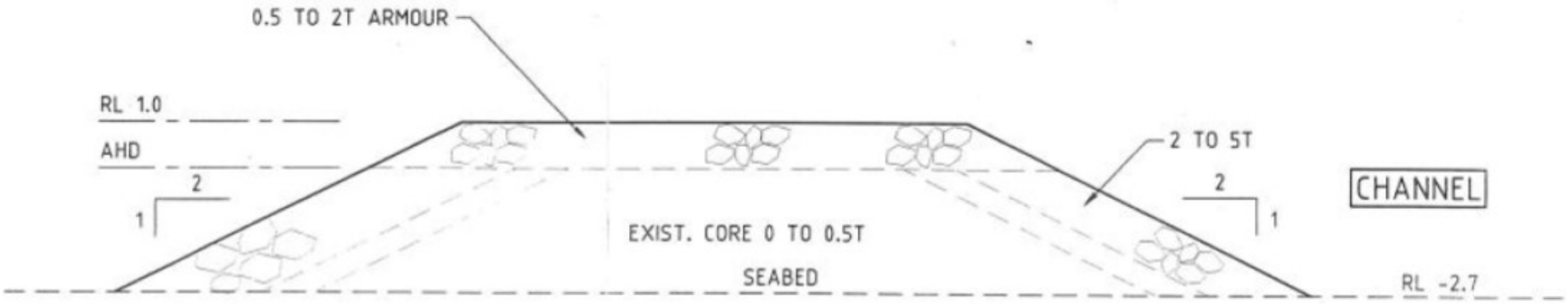
GROYNE MATERIALS:

1. ARMOUR AND SUB-ARMOUR ROCK:
   QUALITY AS PER EXISTING BREAKWATER.
2. CORE ROCK:
   GRADED MATERIAL WITH SUFFICIENT FINES
   TO FORM IMPERVIOUS CORE.

50
ORIGINAL



SECTION 2
SCALE 1:100



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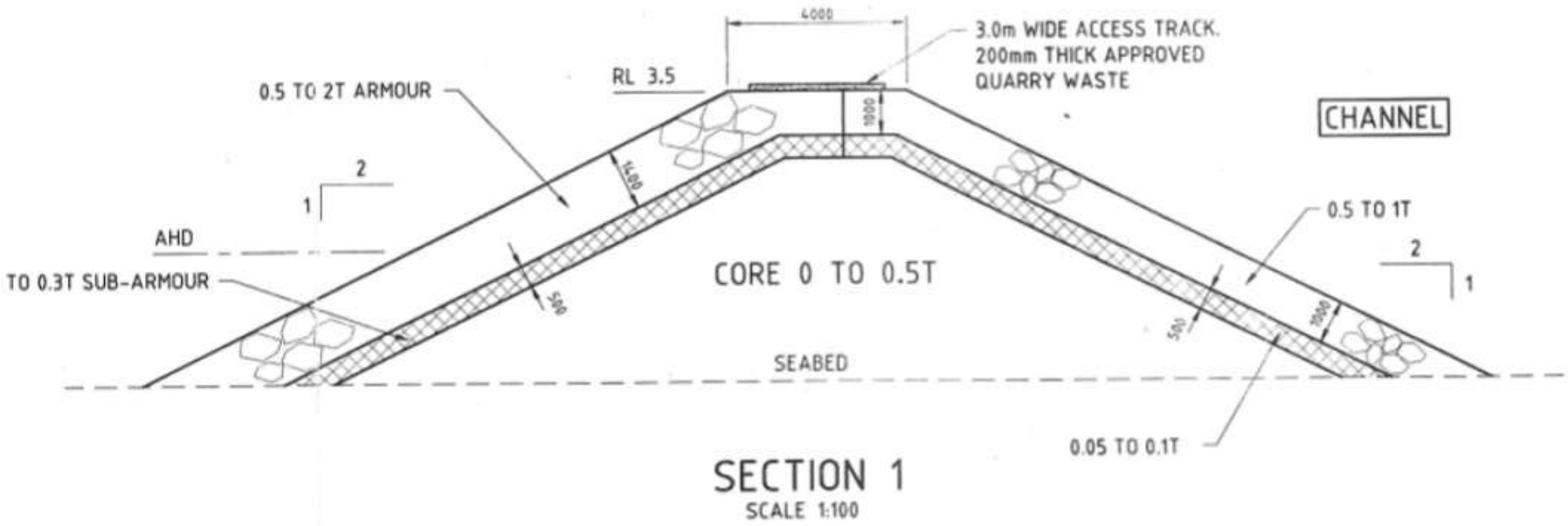

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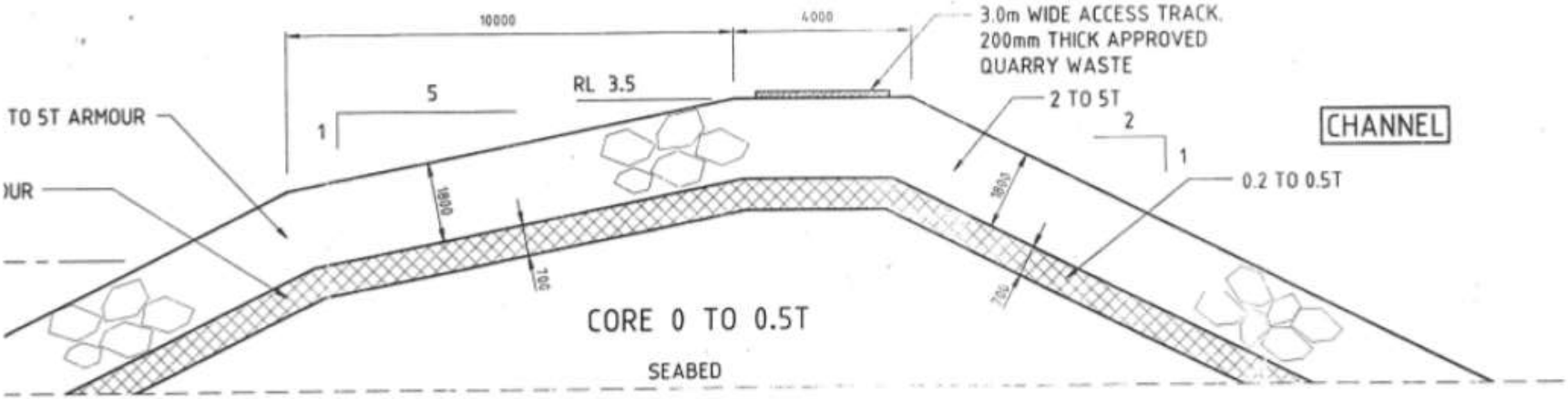
SCALE 1:100



SECTION 2

SCALE 1:100

NORTHERN GROYNE SECTIONS

(SOUTERN GROYNE REVERSED)

MATERIALS:

1. ARMOUR AND SUB-ARMOUR ROCK:
   QUALITY AS PER EXISTING BREAKWATER.
2. CORE ROCK:
   GRADED MATERIAL WITH SUFFICIENT FINES TO FORM IMPERVIOUS CORE.

Appendix B Stakeholder engagement summary report

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Maria Creek Sustainable Infrastructure Project and Coastal Adaptation Strategy

Date: 26/02/2020

Client: Kingston District Council

Subject: Stakeholder engagement sessions – collation of responses

Introduction

In line with the proposed stakeholder engagement strategy to support both the Maria Creek Sustainable Infrastructure Project and the Coastal Adaptation Strategy (CAS), one-on-one consults were held at Council offices over Thursday 13 and Friday 14 February. Whilst the focus of the engagement was on the Maria Creek Project, it also provided valuable insights for the CAS. Twenty-two individual 30 min sessions were held along with a session for Council's elected members. In additional to this, three written responses were received from community members who were unable to attend these have been attached to this summary. The intent and aim of the sessions were to:

1. Allow an opportunity for community members to have their say.
2. Add value to the project by collating local residents' anecdotal evidence and observations of the coast over seasons, years and (in some cases) decades.
3. Provide an opportunity for the community to understand and ask questions about the project methods (what's involved in numerical modelling, hazard mapping etc).
4. Build on the existing understanding of what the community values regarding this section of coast.

The sessions were designed to provide an open two-way dialogue. Commentary from these session are collated under the following key themes:

- Preferred options for Maria Creek.
- Coastal management (for the extent of the Kingston coastline).
- Coastal processes (in the vicinity of Maria Creek and Kingston more broadly).

It should be noted that this commentary is not (necessarily) Wavelength or Council opinion. 'Preferred options' are options preferred by respondents, and does not reflect any assessment, priority or ranking by Wavelength or Council. Further to this, comments and opinions are captured faithfully, without fact checking.

Where a comment or opinion was repeated by multiple attendees, the number of respondents offering this comment or opinion is denoted by a number in brackets, e.g. "(4)". Any anecdotal information or opinion relevant to the volunteer operated Kingston Dredge (which is no longer in survey) has not been included in this summary. Council have advised that the ongoing use of this dredge is not a viable option and therefore will not be considered further in this study.

Preferred Options for Maria Creek

1. **Remove southern groyne (9).**

   Related comments/opinion:

   - Will assist with realignment of coastline, reduce sand under Jetty (5)
   - Sediment likely to move north quickly (5)
   - If groyne removed, seaweed would need to be loosened with an excavator. Otherwise seaweed will take years to mobilise as it doesn't shift easily
   - Consider using the rock from southern groyne for protection along sections of the coast which are likely to be impacted this winter (e.g. In the vicinity of 62 Marine Parade)

2. **Reduce the width of the channel** (7)

   Related comments/opinion:

   - Could use the rock from the southern groyne (if removed) (5)
   - Could be achieved by bringing the northern groyne inwards (5)
   - If shifted inwards 5m or more, the tidal flow will increase
   - This approach will allow ongoing maintenance of the channel with long arm excavator (4)
   - The width of the channel was constructed wider than planned against the advice of the drainage board and locals
   - There is a relatively flat limestone rock base, 1.5m deep at low tide suitable for placement of rock to narrow the channel
   - Adequate limestone present in the northern breakwater to construct a new solid rock northern breakwater
   - "Walking tracks" exist within channel for excavator which could be use as foundation for sheet piled walls for mooring
   - Channel would still need to be maintained, Council could consider buying long arm excavator, undertake maintenance from northern side and place sand north of northern groyne to reduce carting costs.

3. **Alternate location for boat ramp (6**)

   Related comments/opinion:

   - This would only need to be a fair weather facility, not all weather (only required Oct – April)
   - Take ramps out of Maria Creek and put at Johnson's landing (old pro ramp)
   - Suggest timber piled, floating pontoon design (see attached), north of Jetty or at Johnson boat landing (2)
   - Launching facility between Jetty and Maria Creek breakwater (2)
   - Consider ramp directly south of southern groyne

4. **Extend and alter the direction of the southern groyne (5)**
   - Intended purpose to stop weather coming in from the N – NW (3)
   - Needs to be extended by 10m to divert swell away from the channel
   - Could create a gap in the southern groyne to allow sediment and water to flow through (2)
   - Original plan was for the breakwaters to head further north however it is understood that lack of funding during construction impeded this

   Comments in opposition of extending the southern groyne:
   - Eventually the coastline will extend out further along the new Groyne – certainly building the coast out further under the jetty (5)
   - This is an expensive reprieve, but no long-term solution.

5. **Pump seawater out of Maria Creek** (5)
   - The intake pump could be located at the Jetty, attached to a pile of the Jetty
   - Land based infrastructure could be based at Safcol
   - The outlet pump could be located closest point to Maria Creek upstream of boat ramp
   - Solar pump could be used to reduce ongoing maintenance costs
   - Lift pumps used in Kununurra were given as a workable example
   - Wouldn't need to be used in winter, could start up in October each year
   - Approach to be used in conjunction with an upstream weir (See Item 8 below)

6. **Capital dredging program** bring beach alignment back to pre-groyne construction extent between Jetty and southern breakwater (4)

7. **Redesign of breakwaters with culvert design** to allow flushing and natural flow of sediment through breakwaters. (e.g. Modulars 10 x 6m is seen in Europe) (4)

8. **McInnes Weir** presented in GHD 2013 report represented (drawing and detailed comments provided in attachment) (4)
   - To be constructed 50m east of the boat launching ramp with one-way flaps to reduce flows inwards by at least 80% on inclining tides.

9. Fence type wave restrictor offshore which controls wave motion but allows sand movement.

10. Close access to Maria Creek in winter with boom gate (example given of Mt barker Inlet (St Kilda)).

11. Dredge a sediment trap at Channel entrance allowing sediment to settle out.

12. Block off a section of Maria Creek upstream (north of the boat ramp). Allow outflow to divert to where it naturally used to flow (north of the entrance).

Coastal management (for the extend of the Kingston coastline)

1. Comments in relation to the impact of the closure of Maria Creek:
    - Closure of the boat ramp has had a direct impact on the town given relies heavily on tourism (11)
    - Local businesses noting a considerable drop in business since facility closed (3)
    - President of the Upper SE Rec Fishing group noting that memberships are down from 300 to 130
    - A number of people have returned to Kingston or retired here purely for access to easy and safe launching facilities for their boats (4)
    - An increase in houses and commercial properties for sale since facility closure (5)
    - Likely to have a flow on effect to the town (e.g. the hospital could close if population declines in the area)
    - It is believed that tourist numbers weren't down as much this year as a result of the bushfires in other area (3)
    - 130 trailers noted at one time at Maria Creek, evidence is of high demand in peak periods
    - Whilst ongoing maintenance costs need to be considered, it should also be considered what the financial impact to the town is for not holding the fishing contest
    - Kingston always had the reputation as a safe fishing area, noted as one of the safest facilities in the south east (2)
    - All things considered, access to boat ramp is a priority, have a reasonable Jetty

2. Whilst beach launch is an option, beaches are quite soft and unless experienced presents a safety risk (7)
    - Especially in the case of emergency rescue at sea (3)
    - Not suitable for bigger boats (4)

3. Cape Jaffa isn't preferred launching due to:
    - Unfriendly waters (7)
    - Fuel costs (50kms round trip from Kingston) and additional distance from Cape Jaffa to better fishing spots north of Maria Creek boat ramp (5)
    - General inconvenience (additional time) (7)

4. Damen dredge at Cape Jaffa should be used at Maria Creek (5)

5. The jetty is important to the town and needs to be maintained (6):
    - Sand should be removed from under the Jetty
    - Structural condition of the Jetty is an issue, Council could apply to State Jetty Fund to address needed repairs

6. Access to launching facilities only required during peak season (Oct – April) (7)
    - Could be restricted to between Christmas – Easter (2)

7. If Maria Creek is not maintained this presents an environmental and flooding risk upstream (4)

8. Council to address environmental management and rezoning opportunities between Maria Creek and Blackford Drain (detailed comments provided in attachment).

9. Really important to seek a low maintenance cost solution so ratepayers aren't impacted, priority is a financially sustainable solution (2)

10. Flows haven't been high enough (downstream) to naturally flush Maria Creek (2)

11. Rate payers are happy to pay higher boat ramp fees for a facility that works (2)

12. The facility is also missing a cleaning station, lighting, toilets and channel markers

13. Concern raised for the potential impact to Kingston coastline and Maria Creek if Cape Jaffa forecloses (i.e. sand settles out in Marina, no bypassing – further impact to the coast).

14. Dog policy needs to be reviewed at Butchers Gap, dogs that are off leash are having an impact on bird life

15. Boat ramp was built in the wrong spot, should never have been built in the creek

16. Other tourism opportunities exist outside of boating, if shorebirds and migrating seabirds were prioritised this would create tourist interest.

17. Desire to see less money spent on streetscape and beatification and the money redirected to maintaining Maria Creek.

18. EPA restrictions presented limitations to previous dredging programs (i.e. hours the dredge was able to operate).

19. Need to consider looking after tapeworm for shorebird migration, need to stop harvesting of seaweed.

20. Revegetation approach needs greater consideration, cheaper 'quick fix' plants chosen over more expensive labour intense plants (that require seeding) that are more resilient longer term. Skilled technicians for seeding and planting labour intensive plants are located in Meningie.

21. Stakeholders would like to be notified and kept in the loop so families and businesses can plan accordingly.

Coastal processes (in the vicinity of Maria Creek and Kingston more broadly)

1. Cape Jaffa is holding up a lot of sediment on the western groyne impacting the remainder of the Kingston coastline, similar to the southern groyne at Maria Creek. (9)

2. Seaweed build up on the coastline has always acted as a buffer for erosion during Winter (8):
    - Fresh seaweed occurs at the local beaches from late March which forms a natural coastal barrier
    - There used to be 6 – 10 ft of seaweed build up on the beaches in winter
    - Since Council began to remove the seaweed from the beach 4 years ago erosion of sand has been an issue (5)
    - Significant decrease in volume of seaward depositing on the beach noted in more recent years

3. Significant increase in the number of 'breakers' within Lacepede Bay in recent years, only used to be one, now several are visible. (5)

4. Considerable seagrass loss has been noted along the coast:
    - Offshore seagrasses (in the vicinity of Maria Creek) no longer visible, understood to be covered with sediment now. (4)
    - Twelve years ago, at Pinks Beach there used to be one sand hole, now there are 32 that are visible.
    - Impact of drain discharge on seagrass has been investigated and drainage board now prioritises flows to wetlands over outflows.

5. There has been significant changes to the coastline in more recent years, and significant change in weather patterns in the past 4 years (5) :
    - Increased NW winds in the last few years
    - Wave strength coming into Kingston significantly greater
    - Increased wave motion over the reefs and increase penetration into the shoreline
    - the rate of sediment moving north up the coast has significantly increased

6. Significant erosion noted at Wyomi seawall. Houses currently at risk at Wyomi are a result of the original foredune being bulldozed many years ago removing the sand 'buffer' required to maintain this section of coastline. Further to this, coastal vegetation has been removed by residence (even mowed) to clear the view.

7. The dominate sediment transport pathway in summer is south to north, which is reversed in winter.

8. Clockwise eddies of sediment driven by SW swell between southern groyne and Jetty. This build-up of sand becomes the supply source from the continual blockage of the entrance and should be removed on a regular basis. (2)

9. Coastline north of Maria Creek (towards Blackford drain) has seen huge seasonal patterns, large accretion extents in summer and significant cut backs in winter due to storms. This occurs as the offshore profile is very shallow.

10. Increase erosion (scouring out) noted at Thredgold beach in more recent years.

11. Storm is likely to breach out sections of the dune along Marine Parade this year (62 Marine Parade).

12. Water currents are wind induced and stronger during winter months.

SUBMISSION TO COASTAL ADAPTATION STRATEGY Wavelength Consulting 14 Feb 2020

Annabel@wavelengthconsulting.com.au

Our Representation: Steve and Carolyn Adams as custodians of the largest parcels of land in the focus area, primarily S43, S44, S45, S46 (zoned Coastal Open Space) currently used for grazing and hosting up to 25 kangaroos at any one time.

Our Interest : Supporting in a practical, strategic and cost effective way, the objectives of the Coastal Open Space by reducing the area of its zone and redefining the area.

Our Focus : Maria Creek to Blackford Drain and specifically the areas west of the northern route of current Catherine Gibson Way.

Our Relationship to land and foreshore : family farmed for generations, with the "beach land" on lease for controlled grazing but some of which changed zone in the late 1980s.

Our concerns:

The Coastal Open Space Zone in the focus area has never had the ability or the support to achieve its objectives.

1. Coastal Protection Board area has evolved into an environmental disaster since controlled grazing of cattle was removed in the late 1970s. It is now a host for millipedes, snails, rabbits, weeds and an over-population of kangaroos and invasive coastal wattle that chokes out any balance of greenery).
2. The efforts of private owners has not been recognised or supported. Private ownership of lots S 43 to S54 has resulted in controlled non-compliant developments on at least 7 of the lots. The owners of all the lots have provided reasonable to good oversight of the land in their care. (fencing, planting, rabbit controls, snail baiting and weed control). Whether building, developing or farming, these owners have the passion and the resources to preserve the scenic character and the environmental needs of the area. Developing the said land has provided the only improvement or preservation of the land (so please don't diminish the power of development as a protector of environment as stated in Objective 1 of the Coastal Open Space Zone document).
3. The Kingston Council "fobs off" responsibility when it suits. It provides some occasional weed controls when grants are available, but otherwise leaves the responsibility of management to landowners or rabbits.
4. Maria Creek Marina and changes to the outlet have caused some changes to foreshore. The ibis rookery and Aboriginal Grounds are strengths.
5. Population growth north of Maria has motivated the need for beach access adjacent to 11th Street Rosetown. Along with the "new" Rosetown access, the "drying out" of the Blackford drain now means that cars move along the beach without barrier from 11th street to Salt Creek. The once quiet "First Beach" between the Maria and Blackford outlets is now a traffic hazard for those who walk themselves, their dogs and their horses along a beach that was once only accessible for vehicles from Toops Road.

6. Some people who use the area choose to abuse the roos and desecrate the middens and dunes.
7. The area S484 and part of S604 has been effectively controlled by Indigenous workers in collaboration with Dept of Environment. Tree planting has been relatively successful. The fencing at Toops Road and at the extension of 11th St. has almost stopped intrusion by 4wd enthusiasts and motor-bike riders who delight in destroying dunes, middens and flora and scaring the day-lights out of native wildlife. This success is only partial and needs on-going support.
8. S43-S46 have features of high and low ground. They are privately owned but zoned Coastal Open Space rendering them unavailable for compliant development, when development is the only strategy that has protected the environment on similar country.
9. The sandhills and high ground require protection, the highest hills being on Catherine Gibson Way adjacent to S43, S44 and S45.
10. The low ground on S44 and S46 is only wet in "good years". The low ground features a couple of freshwater swamps that are fed by run-off and rising fresh ground water. They host frogs and birds in good seasons. In our family's long history of farming, the swamps and ponds have never been inundated by sea water although we realise that they could be. If they were inundated though, the high ground will remain intact while the rest of Kingston would be underwater. Looking at the current trends, I think that the likelihood of inundation is low considering the amount of sand that is already being deposited along our beach.

Our advice:

1. Review the zoning and reduce the size of the Coastal Open Space to make it manageable. The area controlled by Council would be increased without significant costs as most are borne by land-owners.
2. Concurrently, re-zone S43 to S54 to reflect the current and future use of the area and acknowledge the contribution of private development to coastal protection. They could be rezoned as Primary production, Rural Living or Residential, thus sharing the responsibility for protection and development between the land-owners and Kingston Council. In addition, any future development would result in increased rate revenue for Council as well as further reducing Council's responsibility for land management.
3. Control the millipede invasion, the noxious weeds and the rabbit explosion on the Coastal Open Space.
4. Re-route Catherine Gibson Way at North Terrace or Bay Street to terminate at Bullocky Town Road. Close the section of Catherine Gibson Way that was West Terrace (east of S43 – S46) to protect the largest sandhills between the Maria Creek and Blackford Drain. There should never be a road along that route. The area could be a native reserve.
5. To counter for the closing of that part of Catherine Gibson Way, allow for a raised easement or access road from Toops Road along the western boundary of lots S43 to S46. A raised road, bank, levee or dyke would provide a barrier for potential seawater inundation that would protect the freshwater ponds and swamps, protect the sandhills as well as enable

some future private development that would increase Council rates and protect the integrity of the area.

6. Government departments and Kingston Council should collaborate to support the management of S484 along with the Aboriginal Reserve. This area could be developed into public walking trails with indigenous interpretations. As such, it would attract native animals and tourists alike to the area. Council and Coastal protection Board should support the fencing and the greening projects.
7. Close the 11th Street vehicle access (where the recent fires were started) to the beach to protect the Aboriginal Grounds, the beach close to the original Maria mouth, and the people using the beach (by reducing vehicles and controlling speeds).
8. Initiate a count of the resident wildlife, particularly kangaroos, and use the data to establish protections and limitations.
9. Check the southward movement of wombats towards and across the Blackford Drain, and feed the data into an EIS to precede protections for the environment.

Many thanks,

Steve Adams.

Rob England
"Shepherds Hill"
PMB 47
Kingston 5275

To: Mr Nick Brown and Councillors
Kingston District Council
Holland St
Kingston 5275

Subject: Maria Creek/Boat Ramp Sand and Sea Weed

Dear Nick and Councillors,

I noticed an article on "Seaweed Solution" in the Oct 23rd 2013 issue of the Leader and feel I have the experience to make worthwhile comments on the topic.

I am a long time resident (69yrs) and landholder in the Kingston district – "Shepherds Hill" – and served on the SE Water Conservation and Drainage Board for 16 years, and also the Upper SE Dryland Salinity Project, having designed the concept for the "Floodway/Catch drain system" that allows the transmission of major flood flows while still affecting drainage from adjacent land, and which has been implemented in a major part of the USE Scheme.

During this time, the SEWCDB had problems with the outlet at Beachport from Lake George to the sea, with a build up of sand in the channel coming in on storm tides and during periods of rough weather and low outflows. Numerous site inspections showed that the sand was stirred up by wave action, and the suspended material carried into the channel and lake with the large amount of inflowing water that results at high tide as the sea water fills the channel and lake basin. The sand settled out prior to the reversal of the tide, and remained in the channel once settled, despite the velocity of the following outflow of water at the next low tide, because there was no action within the lake and channel to agitate the sand into suspension to facilitate the reverse action. Estimates and rough survey indicate up to 3million tonnes of sand may have lodged close to the Lake George outlet, and out into the lake. Further sand inflows have been prevented by the operation of the control weir in the channel, and the water levels in the Lake are also manipulated successfully.

A very similar situation exists at Kingston, with both sea grass and sand being the major offending parties, and for brevity, I make the following dot points.

- As a child I played on the huge sea grass banks along the fore-shore, and remember seeing Drainage Board and Council staff manually pitch-forking a hole through to allow the drain to flow, once water started to back-up and threatened to flood the town.
- As a sailing member of the Lacepede Bay Sailing Club for a number of years, and also an occasional boat fisherman from the Kingston facility, I am well aware of the vast area of sea grass growing in the Lacepede Bay, and the amount of detached material these sea-grass beds are capable of generating each year, which inevitably finds its way onto the fore-shore area.
- Having walked out onto the jetty both during and after storms, noticed how much weed and sand is suspended in the sea water, often extending beyond the first landing on the jetty, and this is progressively deposited along the coast.
- Initially the Groynes worked well at keeping the channel fairly open.
- Progressively, since the Groynes were built, a mass of weed and sand has stacked up and reshaped the coastline, largely filling up the area between the jetty and a considerable distance out the Southern Groyne of the Boat Ramp Channel due to the northerly coastal drift.
- As with all Groynes – once they have reached a "steady state condition", the weed and sand in the water, along with that dislodged from the shore-bound heaps, starts passing around the seaward end of the Groynes,

- During storms and high tides this sand and weed, plus all that which is already suspended in the water, is carried into the Channel and upstream areas by the strong flow created as the sea runs in to fill the large “sump” of the tidal wetlands and drainage channel of the Maria Creek and Kingston Main Drain.
- These flows have been depositing banks and layers of sand and sea weed (later quite odorous) not only in the channel, but also far inland, and lodged in the Kingston Main Drain adjacent to the Mt Gambier road, and along the channel towards Flints Road.
- During periods of such major sea water incursions so far into the wetlands and drainage network during high tide, all outflow of drainage water from the adjacent farm land is prevented, causing flooding and salinity damage to considerable areas – often with a potent shandy of sea-water mixed with the local run-off.
- Drainage of large areas of agricultural land is impeded until the extra inflow of sea-water can be cleared from the system at following low tides.
- During the period of neutral tide, and then initial slow outflows as a storm abates or the tide starts to recede, the sand and weed settles out onto the floor of the channel and waterways.
- By the time the out-flows start to increase in rate due to the tide reversal, most of the weed and sand has settled out, and without any wave action to cause it to be re-suspended, it remains where it settled within the system.
- As successive high tides and storms fill the channel by more weed and sand depositions, less material is brought in, but the rate of drainage is also reduced, and the channel becomes un-useable for boat operators.

I am strongly of the view – despite “the consultant expressing their view that a weir concept wouldn’t be successful” - that a weir concept as variously proposed some time ago by Leigh Von Bertouch, Lynton McInness, John Kuhl and me, is the only realistic solution to keeping the channel open and useable for boat owners, to provide a drainage service, and for environmental and beautification purposes for the Maria Creek Estuary.

Points in Favour of the Establishment of a Weir just prior to the bend upstream from the Boat Ramps.

- A Weir would stop any undesirable sand and weed laden tidal inflows, and hence stop the movement of sea grass and sand into the Groyne Channel, Maria Creek Estuary, Wetlands, and the Kingston Main Drain.
- While the weir is closed during a storm or high tide, inland drainage waters can continue to flow towards the exit of the drainage system, and will slow as the total available pondage fills – but this time it fills with drainage water, not a cocktail of sea water, sea weed and sand, and no sea water is able to flood out onto adjacent lands.
- Better drainage is ensured, as only stored drainage water is discharged quickly through an open channel once the weir is opened again after storms and at lower tides.
- As the current supply of sea grass rots down within the Maria system, the odour will slowly abate.
- A pool level – at a level to be determined - can be maintained within the wetlands and Maria Estuary when not fed by drainage water, by allowing inflows of sea water from high tide events that occur during periods of calm weather so no sand or weed is mobilised. Regular flushing and re-filling could also be effected with weir manipulation during low and high tides. This will greatly improve the visual and environmental appeal of the area for our visitors and adjoining landowners.
- Weir systems are now available that can be operated remotely, automatically, or manually, or a combination of all three systems, and should be seriously considered as a preferred means of a long term solution to the current problem.

While I have not seen the 13 options GHD has proposed, I will comment on why I feel the apparently preferred Options One, Two and Six referred in the front page article in the Leader will eventually fail – again in dot point form for brevity.

Option One - Extending the Southern Groyne into deeper water.

- Extending the Southern Groyne into deeper water will be hugely expensive due to the ever increasing amount of stacked material required for each metre of length gained, but it will give some temporary relief to the problem.
- Eventually the coast line will extend out further along the new Groyne – certainly building the coast out further under the jetty - possibly as far as the first landing, depending on the length of addition to the Groyne.
- There will be dramatic erosion again on the beach and dune to the north of the Channel – more-so than happened when the first Groynes were built - as the natural continuous replenishment northward sand and weed movement will be interrupted until another "steady-state condition" of continuous removal and replacement is established along the shore either side of the new extended Groyne.
- Success will be claimed during this initial period, as only the lesser amounts of sand and weed suspended in the water will be carried up the channel and into the wetlands and drains.
- Once steady state resumes, mobilised sand and large chunks of weed dislodged by storm and tide action will again start to in-fill the channel – ie, an expensive reprieve, but no long term solution.

Option Two - Extending both Groynes into deeper water.

- Twice as expensive as option One, more sand will be deposited south of the Groyne until "steady state" is achieved, and it will only achieve the same temporary benefit, but again, no permanent solution.

Option Six – Reducing the Channel Width at the mouth.

- Also an expensive option – even if just repositioning the materials already present in the current Groynes.
- Any reduction in the area of the opening will have negligible effect on the ingress of sand and sea weed to the channel as an even stronger weed carrying tidal current will pass through a narrower channel "choke" at the sea-ward end, as sea water rushes to fill the large receiving areas of the wetlands, Maria Creek, and Kingston Main drain.
- A narrow channel may have an undercutting scouring and destabilising effect at the base of the groynes.
- A reduced channel width will become a navigation hazard for boats – particularly in windy conditions.

I have had a long held interest in the Maria Creek Estuary beautification process, having drawn up the later plans adopted by the Apex Club and Council, and enjoyed many hours of "working B" involvement with club members and other volunteers to cart in, level and compact the fill, and to install the stone pitching and other facilities in the Apex Park. An affordable and workable weed and water management system for this area and its sea connection would be a great finale to this long term project.

In conclusion, I urge Council to give strong consideration to the points I have made relating to using a weir to alleviate the sand and weed management issue. A weir would also allow better drainage and year round environmental benefits to be enjoyed by our rate payers and visitors, including a permanent access loop walkway across to Rosetown - elements which don't appear to be forthcoming from any of the other options seen (by me) to date.

With regard to this 2020 proposal, I make the following closing remarks.

The current build up of sand along the fore-shore will be of great benefit to the town in future years as sea levels continue to rise, and if a protecting fore-dune is allowed to develop again, it will provide a level of wave energy abatement that has been lost by previous "beatification levelling works." It would be a grave mistake to decided to dismantle a section of the groyne to allow the sand and weed bank to resume its former northward movement, as it is obviously depicted in your aerial photo that it has just reached that steady state stage required to enable longshore drift to resume and continue as it did prior to Groyne construction. Provided infra-structure is constructed in the form of a weir, to ensure that

inflow into the creek and drain and wetland basins only occurs during calm periods, the ongoing system should work efficiently. While this weir will also stop sea-grass also being carried into these inland basins, winter storms are probably always going to overtop the sea wall in high seas, and fill the boat channel to the weir, and this will need to be mechanically cleared to allow the boating facility to continue to be used.

I am happy to provide more detail should it be required at a later date, but am not available to attend the one on one discussion sessions..

Yours sincerely

Rob England

The River Murray Valley Trade.

Biggest Scheme in Commonwealth.

Outlet to Markets of the World via Lacepede Bay or Goolwa.

From Hume Reservoir to Murray Mouth

Both Schemes to be Investigated

Compiled by Editor as Reported on by "Narracoorte Herald" Under Data Supplied by Mr. Edward Goode, Barooka and Kingston, South-East, Furnished by Admiral Sir. Wm. Creswell, of Royal Australian Navy, and Others.

hed by A. Caldwell, Smith-Street,
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Everyone is entitled to my opinion

MAY BE INTERESTING READ

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1/

Experiences with Ports in the South-East.

The question of an efficient port for the South-East has long been a question which has occupied the attention of those who take an interest in its development. There is no doubt that water carriage is the cheapest means of transport, if a safe and efficient port is at hand. The people of the South-East have had under consideration for a number of years such a port along the coastline. At various times four ports have been opened in the South-East, viz., Robe, Port MacDonnell, Kingston, and Beachport, and it may be said that at present only one survives for sea transport purposes, and that is Kingston. Robe, or Guichen Bay, held the sway in the early days of settlement in the South-East as the principal port, and had merits as such, but the people of Mount Gambier very properly wanted a handy outlet for their trading, and Port MacDonnell came into vogue as a port for a large proportion of the South-East, but it was feared that it had not the necessary conditions to make an efficient and safe port, and a railway was constructed from Mount Gambier to Beachport to see how that would shape as a port. It also had merits, but it did not attract a large trade, though it was a good outlet for the trade of the Millicent district. Kingston, or Lacepede Bay, did considerable trade for a number of years in the northern portion of the South-East, and from across the Victorian Border, but railway communication linking up to Adelaide and Melbourne practically closed up all the ports by competition freight rates of the railways. It was felt that what was required to obtain regular and cheap transport by sea was a central and safe port, instead of hauling our products and goods over rail for hundreds of miles to a metropolis.

The Government of South Australia has from time to time had the establishment of a central port for the South-East on the coastline under review. It obtained inspection and reports from leading harbor and ports' engineers of the world as an efficient deep-sea port, and a contest for suitability narrowed itself down to Kingston and Robe.

According to seamen who have had experience of Lacepede Bay, it has natural conditions about it which renders it a safe port. Its merits in this respect are described in the British Admiralty of Directions for sailing along our coast from knowledge gained by careful surveys. Seamen who know the port have absolute confidence in the safety of it, and during stormy weather run their vessels into the bay as a refuge from rough weather in the open sea.

Another question that has been under consideration for many years in this State is the Murray Mouth, which is not navigable, and prominent harbor engineers, as well as shipping commanders, have enquired as to the best means of dealing with it in constructing a port for exporting the products of the Murray Valley overseas. The bulk of our products must be shipped overseas by the cheapest means in order to compete in the markets of the world.

TWO SCHEMES FOR PROVIDING MURRAY RIVER WITH OCEAN PORT.

There are two schemes for supplying a port to the Murray Mouth. It was naturally thought that a suitable place for a port could be found somewhere in the vicinity of where the river ran into the sea. It was found

to be impossible to make use of the Murray mouth for that purpose. Our early public men saw the necessity of providing an outlet for the traffic on the Murray River, the waters of which traverse a vast expanse of fertile country throughout Australia, comprising South Australia, Victoria, and New South Wales; and also tracts of pastoral country in Queensland through the River Darling, which junctions with the Murray. In fact, the Murray junctions a number of rivers.

The leading early public men at first directed their attention to Port Victor for fulfilling teh misiosn of a port for the Murray, and a quantity of money was spent in making it a port, but it has proved a better seaside resort than a port; and this plan for an outlet to the river had to be abandoned. Then came the Goolwa proposition of a canal. It was found that the water from the sea was going too far up the river, impregnating the fresh water with too much salt, and it was decided to erect a barrage at Goolwa to prevent it, with a view of also making it in time a port for the mouth of the river. It is the position that a large amount of money has been spent on the barrage, and we may say there is no active contention in the matter.

Goolwa and Kingston are not opposing schemes, but are content so long as the BEST scheme is chosen. Admiral Creswell, in his scheme states that the Coorong is the natural mouth of the Murray, and that it leads to Lacepede Bay, which is peculiarly adapted, according to nautical experts who have seen it, for a safe and commodious port for ocean vessels.

Survey of Admiral Creswell's Scheme Asked for.

There are thus two outlets to the ocean under review. One is the canal direct to Goolwa and the other is Admiral Creswell's scheme via the Coorong to Lacepede Bay. The first has been surveyed, and reported on. Admiral Sir William Creswell's latest expressed wish was for a survey of his scheme. He said that "it was simply rediculous to suppose that when all that was needed was a canal only a little bigger than your drainage channels to join a splendid port to a novigable river system of 3,000 miles, that it will be allowed to pass."

The rival claims of Goolwa and Lacepede Bay should be looked into. No other river in the world of the size and importance of the Murray is without a port. A Murray port would help primary producers to compete in the world's market without the assistance of bonuses, bounties, etc. The Commonwealth insisted in the preservation of navigation through all the upper river locks. Sir David Gordon (President of the Legislative Council of South Australia), who designated the Murray "the Nile of Australia," and delivered a number of lectures on its national importance, said— "This is not only the biggest scheme in South Australia, but in the Commonwealth."

RIGHT HON. S.M. BRUCE, HIGH COMMISSIONER OF AUSTRALIA, ON UTILIZATION OF MURRAY RIVER.

OVERSEAS' PORT OF SOME DESCRIPTION REQUIRED AT MOUTH.

The Hon. S. M. Bruce, when Prime Minister of Australia, on a visit to Victor Harbor, and speaking on the question of locking the Murray and its value to Australia in transport in general, said:—

The problems of the Murray were very important to the future of Australia. The original conception was a dual one—irrigation for the settlement of the fertile valley of the Murray, Murrumbidgee, and Hume, and a locking system to bring the produce from these areas to the sea. The irrigation side was now almost completed. **It was no use locking the River unless they had a port to which to bring the produce. That question has never been properly dealt with.**

The States had been building a network of railways to the River, and if

their railways were not to be a burden on the community they must carry the produce. A new system that had revolutionized transport in Australia was that by motor. **Transport was vital to the country.** He did not agree that the whole of the produce should go where vested interests existed in capital cities. The produce should be taken to the natural ports, and very little progress had been made in the development of these ports. He would support any proposal which could be shown to be the most suitable for dealing with the traffic, whether it was by motor, road, or rail. In Queensland there were three great railways running east and west. If these railways had been running north and south the present drought in that state would hardly have been felt, as the stock could have been transported to non-drought areas. The coming of the new six-wheel motor vehicles would open up great areas in the interior, and revolutionize transport by being used as feeders to existing lines. The TRANSPORT PROBLEM was national and the best methods would have to be found to solve it. He had not the slightest misconception about the matter of a Murray Port, an dhad no hesitation in saying that it would have to be seriously considered. If they locked the River, and made it passable for navigation, then the logical conclusion was that **an overseas' port of some description would have to be established.**

HOW TRADE WAS DIVERTED FROM KINGSTON BY CONSTRUCTION OF INTERSTATE RAILWAYS.

The port of Kingston and the railway running therefrom commands an extensive area of land in the South-East and in Victoria near the border land awaiting development for closer settlement. When the line to Kinggston was first constructed to the Tatiara district it did a great deal for the developemnt of the land on the Victorian side of the border for agricultural settlement, notwithstanding that the border was furnished with duty collectors. The Victorian Government was alive to its interests in retaining the trade from Melbourne and Geelong, and extended the railway line to the border. Victoria found it necessary, to prevent the trade from going to Kingston, to put on preferential and penal freight rates. There is an interesting history in connection with the border prior to Federation. Notwithstanding much opposition from the Victorian railway authorities, a good deal of Victorian trade crossed the border at various points between Penola and the Tatiara, and went to KINGSTON, particularly from the Wimmera district of Victoria when agricultural settlement took place and a good deal of Victorian border trade found its way to Naracoorte by road, and thence to Kingston by rail, to be shipped to Melbourne and Adelaide. A large quantity of wool went this way to Kingston to be shipped direct to London. The institution of wool sales in the various metropolitan centres of the States killed the trade. The woolgrowers in the South-East and across the border found their rates for shipping direct more than ordinarily cheap. Now, Britain, the Continent of Europe, and elsewhere send buyers to Australia to get the wool they require, and it is shipped from the metropolitan ports. The South Australian railway department in time collared the trade that went to Kingston and other ports of the South-East by special rates between the South-East and Adelaide, and the ports were virtually closed. Kingston is reviving again as a shipping port, with vessels regularly trading between Kingston and Adelaide and Melbourne.

WHAT EXPERT MARINERS THINK ABOUT KINGSTON AS A PORT.

A number of people have for some years advocated Kingston as a suitable and safe port, and capable of dealing with the shipping trade of the South-East. Among them is Mr. Edward Goode, of Kingston, who, with his sons, follow pastoral pursuits. He has taken a leading part for a number of years in advocating the claims of

Kingston as a port. He has gone to no end of trouble in consulting authorities on its safety and capabilities, and has collected a great deal of data as to its merits. He found in the late Admiral Sir W. R. Creswell, a strong advocate for the merits of Lacepede Bay as a port, who furnished Mr. Goode with important data, which more than strengthened his opinion that it was outstanding as a port on the South-Eastern coast.

AN INTERESTING REVELATION ABOUT KINGSTON AS A PORT.

Admiral Creswell, as a skilled navigator, went further than Mr. Goode as to the value of the port of Kingston, and was well prepared to show that it could be made into a port of national importance to Australia. At present we are dealing with its claims as a port for the South-East. A further claim put forward by Admiral Creswell as to being made a port to serve an immense area of highly productive land, will come to most people as an interesting revelation.

Through the courtesy of Lady Creswell, widow of the late Admiral Creswell, Mr. Goode received complete plans of the late Admiral's scheme for fitting Kingston as a port for not only the South-East, but for the Murray Valley, and the navigable rivers running into it, such as the Darling and the Murrumbidgee from Victoria and New South Wales. These plans, with carefully compiled data, have been forwarded to us by Mr. Goode, with the view of making the people of the South-East acquainted with the valuable trading possession they have on their coastline in Kingston as a port, and which could be improved at very small cost to serve an immense area of productive land in portions of South Australia, Victoria, and New South Wales, which has to depend upon railways taking circuitous and expensive routes in forwarding products to the seabord for export.

The following notes give a brief outline of the scheme showing on the plan, with accompanying data giving a full explanation of it:—

1. A direct connection from producers to overseas ships.

2. A vast saving in freight—water transport has been universally proved to be always cheaper than rail or road.

3. The dual purpose in view is the immense saving in carriage, enabling Australia to compete on the most favorable terms with other nations.

4. Sufficient already exists to warrant a close investigation of our favoring conditions.

5. It is recommended that the first step in the investigation should be a thorough and complete survey.

6. If survey data confirms previous reports it is recommended that the initiatory canal works to be put in hand forwith.

7. The prosperity of the Murray Valley depends on a thorough search to the sea.

There are two points of view in connection with this matter. Admiral Creswell's plans and data deals with Kingston as a port for the South-East, and also for the Murray Valley. The Admiral's plans should be kept separate in looking at them from a South-Eastern aspect, in making it a central port for the district at large as urgent for its development.

WHAT ADMIRAL CRESWELL THOUGHT OF KINGSTON AS A PORT.

Dealing with Kingston as a port generally, Admiral Creswell, writing in a letter to Mr. Goode, dated 9/12/33, stated:—"I don't think you need in the least be concerned or doubt as to Kingston or its immediate neighborhood being the future selected port. It is not even a question of selection, for there is no other to compare with Kingston. To land folk one place seems as good as another along the coast, but not so with sea folk. Shipping people who use the port must be confident of its safety and favorable conditions for loading, etc. Perhaps you will understand this better for some little explanation. With very good reason did Sir Day Bosanquet (Admiral), Governor of South Australia, call Lacepede Bay one of the maritime marvels of the world. What is required in a port as a fundamental need is smooth water, so that ships are without motion, and can load and unload at ease, and without interruption. There must be no rough water—no swell or 'scend—as at anchorage exposed to open sea conditions.

This is an authoritative statement from the Admiralty of the safety of Lacepede Bay.

REMARKABLE LOCALITY OF LACEPEDE BAY.

Furthermore, concerning the safety of Lacepede Bay as a port the Admiral writes:—Evidence of the smoothness of the waters of Lacepede Bay we have in the fact that in pre-railway days wheat was loaded by ship at the anchorage. It was taken to them from the shore in open sailing vessels. Small coasters of that time left and returned for shelter, meeting the normal westerly seas some miles off the coast. Although the gale blows home to the shore there is NO SWELL for miles to seaward. Although doubtless the very gradual slope of the seaboard lessens the swell, its complete disappearance is, I think, due to vast forests of kelp. I have seen it reach the surface off the coast in 20 fathoms of water. The water is actually much smoother than in Port Phillip. Of this I have had proof in the very small boats moored with light lines off the shore in a way that would not be risked in Port Phillip. Even such light sea—a boat sea—as is raised by a gale, I am told by fishermen, GOES DOWN DIRECTLY THE WIND DROPS. The extraordinary phenomenon of a perfectly calm sea with apparently nothing to account for it, is one which I have not met with in any other part of the world.

THE GREAT TRANSPORT DEVELOPMENTAL SCHEME BEQUEATHED TO AUSTRALIA BY ADMIRAL SIR WILLIAM CRESWELL.

COORONG CANAL SCHEME.

Making River Murray Navigable to the Sea with a Safe Port.

Admiral Sir William Creswell, in communicating with Mr. E. Goode, forwarded him details of an important scheme he had worked out for benefiting and assisting in the development of an immense area of Australia in connection with Kingston as a port. His object was to solve the problem of the Murray mouth by making the River navigable to the seas. Enormous difficulties have hindered the navigation of the mouth in highly favorable conditions, and Admiral Creswell held the opinion that even with the creation at great cost of a proper entrance way, captains of liners might refrain from venturing into any harbors made near its mouth, owing to the terrific range of the seas in Encounter Bay. His scheme is based on the view that the outlet of the Murray was formerly at Kingston—that the combined effect of ocean winds and waves was to slue or twist the river from a straight course, and cause it to run down the Coorong Channel to Lacepede Bay.

The chief argument for establishing a river port at Kingston is that Lacepede Bay provides a calm sea, in which ocean liners can find a safe anchorage in all weathers. The scheme includes the dredging of the Coorong (which is already navigable for 40 miles) for a further distance of 50 miles, and the conveyance of produce from the river boats to deep-sea vessels.

PORT FOR MURRAY RIVER IRRIGATION SETTLEMENTS.

In this matter the pressing claims of the producers in the numerous river irrigation settlements must be

paramount over those of any particular marine locality in this State. The heavy cost of getting irrigation products to market from the Murray has been and is still one of the severest handicaps to the success of productive industries in the valley. Water carriage is notoriously cheap as compared with land transport by road and rail, and the Murray is not doing its duty.

The solving of the problem of an outlet for the Murray would certainly go far to ensure the rapid expansion of the population in the irrigable districts, the greater safety of Australia, and the capture of satisfactory markets for Australia's dried fruits.

THE PLANS OF THE SCHEME SHOWING IMMENSE AREA IT WILL SERVE FOR TRANSPORT.

The data explaining the canal scheme is accompanied by four plans drawn with much neatness and expert engineering skill, and are of much assistance in giving a lucid explanation of it in connection with the linking of the Coorong Canal with the port of Kingston.

One shows the junction of the Murray with the Canal at Pelican Point, through Lake Alexandrina, and the situation of Hindmarsh Island, Port Victor, and Goolwa to the Murray Mouth. The prominent places on the Coorong, such as Lake Albert, Meningie, McGrath's Flat, Wood's Well, Salt Creek, Chinaman's Well, Cantara, and Dalkeith, are shown in the tracing to Kingston. The Coorong water tapers out after leaving Dalkeith to Lacepede Bay. There is a plan giving a bird's-eye view of the country which the port of Kingston will serve for transport purposes if the canal were constructed through the Murray and its tributaries—the Darling, the Lachlan, and the Murrumbidgee.

The data accompanying the plans describes the scheme in detail, in a clear manner. It will be 32 miles long, 60 ft. wide on surface, 10 ft. deep, and dredged by two dredges, and each one would do 200 ft. per day, moving 3,703 cubic yards per day. The dredging is worked out in detail, showing that it would cost £71,000. The total cost of the canal is worked out as follows:—Allow for actual cutting of canal, £120,000; cutting approach from sea, £100,000; dredging portion of Coorong, £100,000; total cost of canal, £320,000.

ADVANTAGES OF CANAL DESCRIBED.

The advantages of the canal are thus described:—For the outlay of about £300,000 the entire River Murray and its tributaries covering a navigable distance of 3,212 miles on the third largest river in the world, would be opened to the sea. When one compares the thousands of tons of products which can be conveyed direct to the sea, at a very cheap freight compared with that of the railways, one can see whereby there are possibilities for the producers to have two great benefits—one, more profit; the other, lower costs for oversea customers. The cutting of the canal would be a fitting and sensible climax to the vast expenditure already laid out on the erection of weirs and cutting irrigation canals. Without the River Murray being open to the sea, the whole existing scheme is that of a HOPELESSLY CONFINED MONSTER WITHOUT AN OUTLET OTHER THAN EXPENSIVE RAILWAY CARRIAGE AND ADDITIONAL HANDLING. In the paper which I (Admiral Creswell) wrote for the Economic Conference in Brisbane in 1930, I stated:—"THE RIVER MURRAY SHOULD BE TO THAT PART OF AUSTRALIA WHAT THE CANADIAN PACIFIC RAILWAY IS TO CANADA."

FREIGHT, Etc.

The cheapest means of conveying the freight would be by barges. One barge would be self-propelled, and capable of carrying 400 tons of cargo. There would also be three dumb barges, each carrying 450 tons of cargo. The dumb barges would be towed by the self-propelled barge. The barges would be built of steel, and so designed as to offer the minimum resistance when being towed. The group of four barges could be increased as the trade increased. To commence with, it is proposed to have two groups of barges—3 of 450 tons, carrying 1,350 tons; and towing barge —1 at 400 tons; making a total of one group carrying 1,750 tons.

Take 1,750 tons over a distance of 2,000 miles, travelling at a speed of four miles per hour. The group of four barges would take 500 hours travelling time. Barges would be travelling for 16 hours out of 24 therefore the group would move at the rate of 64 miles per day, thereby taking 31 days to convey 1,750 tons over 2,000 miles. Take the return journey empty at the same speed. One group should easily complete a 4,200 mile trip per year; or one group would convey 1,750 tons of cargo 8,000 miles per year. The data on working out details shows that the cost of the barges per group would be £50,000. The pay of barges in ten years means £5,000 per year—four trips per year; then add £1,250 to cost per trip; and then add for annual repair of £1,000 per year. The cost to convey 1,750 tons for 2,000 miles would cost per trip £2,440. Thus, the 1,750 tons can be towed for £1 5/ per mile; and the freight altogether would be .500 penny per ton per year. Profit and administration per group per year equal £7,525 to discharge 7,000 tons. Canal toll per group for one year equals £11,666 to discharge 7,000 tons. A scale of freight can be arranged to cover distances under 500 miles. It can be seen where good profits can be made from freight only.

A table is supplied showing cost of railway freight from Renmark to Adelaide for transport of wool, fruit and wheat—214 miles; also from Swan Hill to Melbourne.

We give some of these interesting details in connection with the cost of transport by the scheme to show the thoroughness with which the Admiral sought out his scheme for opening up the Murray River for transport via a canal to Kingston to tap all fertile areas of AUSTRALIA converging on the Murray and its tributaries.

The following is extracted from the report of Captain Johnston, an American expert of harbour construction, on his general scheme for the improvement of the River Murray, furnished in 1913:—Length of Murray from Albury to mouth, 1,366 miles; navigable principal tributaries of the Darling, 1,180 miles; Murrumbidgee has been given as 666 miles; total navigable distance, 3,212 miles.

The Coorong is already navigable for about 40 miles. This leaves 50 miles at the most to be made navigable.

The Admiral, in his data, stated:—Producers can convey their wheat and fruit to alongside the oversea ship at a cost less than one half-penny per ton per mile—in other words, one ton can be carried 500 miles for £1.

If the various State Governments, or if the Commonwealth could spend anything from £300,000 to £400,000 to render the River Murray navigable to the sea, this could be paid back gradually by the payment of a toll for every ton of cargo which passed up and down the canal.

The opening of the canal would present a concrete achievement by those interested which would be the means of encouraging people to settle in a part of the country where there are irrigation and water transport.

People on the land had been very greatly handicapped by the heavy railway freight. In this scheme there is room for all, even supposing a few hundred miles of railway line lay idle and rusting—what is that to be compared with a profitable scheme which is going to be a benefit to so many people.

WHY NOT HAVE AN ENQUIRY INTO THE SCHEME.

This scheme of Admiral Sir William Creswell, the father of the Australian navy, has been before the public of Australia for some years, and when it was first published it was favorably criticised by the press of Australia. It was first mooted in 1926, but politicians have been slow in comprehending its worth. It is what may be termed a NATIONAL SCHEME so far as Australia is con-

cerned. No doubt politicians view it as something to benefit one of the small States, and Kingston is looked upon as a small port on the coast of the South-East district of South Australia. South Australia has not been aroused to the value of Kingston as a safe port for even a portion of its own State, let alone a port which could be made an important overseas port for exporting at a cheap cost in freights of primary products of a big area of the most fertile land in Australia. How long will it take South Australia to rouse itself up to the advantages of the late Admiral Creswell's scheme? It being a matter in which the Federation is greatly concerned the Federal Government should at least make an enquiry into the scheme.

The Admiral devoted much thought and time to the necessity of bringing into a useful existence the great possibilities whereby with great ease the River Murray can at a small cost be opened to the sea. The Adelaide "Register" of May 5, 1926, and "The Observer" of the same date quote in glowing terms the advantages of the Creswell scheme. Favorable comment on the scheme was made in the "Sydney Morning Herald" by Mr. Brunsdon Fletcher. Many correspondents in the leading newspapers have commented very favorably on the scheme.

OPENING OF RIVER MURRAY TO THE SEA.

The opening of the River Murray to the sea is summed up as follows in support of Admiral Creswell's scheme:—

(1.) What is the average cost of railway freights per ton per mile between the River Murray and the nearest seaport for loading oversea cargo.

(2.) The cost of the freight from the producer to the overseas ship has to be paid for by somebody.

(3.) If the transport is charged over from railway to river and canal at freight per ton per mile, which is only a fraction of one penny, what would be the natural result?

(4.) Due to the loss of freight the producer can make a reasonable profit, and the goods can be sold overseas at a fair competition price with other nations.

(5). Can any person give any reasonable reason why Australia should not spend a few hundred thousand pounds when a return of millions is positively assured.

(6.) Under the existing high railway freight, our excellent agricultural machinery is unable to assure its proper value unless great skill is used towards the ingenuity necessary for introducing water transport.

(7). From private enquiries made it is found that the producers all along the River Murray are in a bad way financially. They have done their best within their power, but are tied up from the serious lack of economical transport.

(8). The produce from the River Murray finds its way to many countries, but the quantities have been limited, because the producers have been unable to show sufficient profits whereby they can expand and increase their production.

(9). Under the existing conditions what prospects are offering for the rising generation?

(10). In every country throughout the world right back to B.C., every opportunity was taken to convey goods by river or canal, and for what reason? Simply because it was faster and cheaper.

(11). Can any reason be offered why the River Murray cannot be opened to the sea? To use Sir William Creswell's own words, "Why has this area so remarkably favoured by nature for water transport been so neglected?"

Vested Interests.—The Government is not asked to spend money on the extension of Kingston jetty. What is required is better steam lighterage to the roadstead.

We have given sufficient evidence by quotations from reports of experts who are free from bias of any kind and have no personal interests to serve, to show the merits of Kingston as a safe port. Kingston has been developed as a port from the earliest settlement in the South-East, and has sustained its reputation for safe

ty. It could be gradually improved so far as appliances for shipping are concerned at very little expense as its trade increased. It has all the natural conditions for an efficient port, but it must have no artificial conditions to hinder its development. The port has a railway to it from the centre of the South-East, and it extends to the northern and southern portions of the South-East. Before it comes into its own as a port it must have the goodwill and support of the people. In our advocacy for a central port for the South-East for many years as an absolute necessity for the development of the district as a whole, we have endeavoured to take a broad view of the matter with the evidence at hand. All ports in the South-East have been tried, and they have all virtually closed down, so far as shipping is concerned, except the one in Lacepede Bay. The port of Portland has had a railway specially constructed to it from the southern portion of the South-East, but it has failed to attract the general trade of the district.

SETTLEMENT OF EMPTY LANDS AND DRAINAGE.

There are other matters pertaining to the settlement of the South-East that require consideration, which we have not touched upon. There is a large area of light land which will be taken up, and, in course of time, it will be so improved that it will carry a large population. A good deal of this land has been leased by the Government, and much of it is within easy distance of the railway to Kingston, which will be served by it as a port, providing cheap transport to convey its products to the markets. There is a large area of scrub land between the portion of the overland railway line from the Tatiara to the River Murray and the railway line from Narracoorte to Kingston. This land was recently inspected by the Commissioner of Crown Lands (Mr. McIntosh) and the chairman of the Land Board (Mr. Colebatch). There is no population in this country, and a limited number of sheep. It is almost an unknown country and they were of an opinion that much of this land could be considerably improved by modern methods disclosed by reseach, such as superphosphates, to carry a fair number of stock. There are swamps here and there through it and water can be obtained at shallow depths. To the south-west and south of the Narracoorte and Kingston railway line there is a large quantity of light land which could be considerably improved, and also good land fitted for closer settlement which Kingston would serve admirably as a port. The scrub and light land could be allotted in fair sized blocks on conditions of improvements, which could be taken into account in fixing rents over a period of years. The settlement of this country is a matter that should be studied out carefully. There is no doubt that there are thousands of acres of land in the South-East awaiting development.

The drainage of the land will come in time as settlement increases. A number of the main drains have been constructed, and those with wet country who desire to improve it can drain into them. It would appear that the country would be all the better if some main drains were made in the Hundreds of Ross and Bray, where there is fairly good land. Kingston would be the outlet for the products of this country.

SCENIC HUME RESERVOIR.

INLAND SEA OF THE MURRAY VALLEY.

Situated ten miles from Albury (New South Wales) at the confluence of the Murray and Mitta Mitta Rivers, the Hume Reservoir, a practical expression of the River Murray Water Conservation Scheme, is a monument to man's ingenuity in imprisoning a vast mass of water for irrigation purposes. This inland sea, which covers an area of 52 square miles, holds untold wealth in store for the Murray Valley, a tract of fertile country whose potentialities have been multiplied enormously by the regulated water supply from the Reservoir.

10

Fed by the melting snows which flew in great volume from the dam's catchment area of 6,000 square miles of mountainous country in Victoria and New South Wales, the immense reservoir disgorges its pent-up power over the fertile Murray Valley, plentifully irrigating a huge area of 2,000,000 acres. With this regular and controlled water supply to supplement a steady rainfall, the Murray Valley is destined to become one of the richest stretches of land in the world. Besides its irrigational benefits, the Hume Reservoir has provision for outlet works for the generation of electricity by hydro-electric power.

With a capacity for 1,250,000 acre feet of water, and provision for extensions enabling it to impound ultimately 2,000,000 acre feet, the vast dam is 5,280 feet long, and is composed of an earthern section of 4,238 feet, with a core wall of concrete touching bedrock and a concrete spillway (including turbine wells and outlet works) 1,042 feet in length. One hundred and fifty feet above the river bed, a first-class roadway runs along the 32-feet wide crest of the dam.

Standing on the brim of that mighty bulwark and surveying the expansive sea of imprisoned water, one experiences a sensation of puniness in the presence of the potent forces of nature, and, at the same time, a feeling of satisfaction that this immense reservoir is a man made contrivance, successfully preserving for civic use a huge water supply that was formerly a wasted asset.

"Narracoorte Herald" Print.

This Scheme is now under Government Consideration.

THE MURRAY PROBLEM

NEW SCHEME FOR SEA COMMUNICATION.

UTILIZING THE COORONG.

By Vice-Admiral Sir William Creswell, K.C.M.G., K.B.E.

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Direct communication by water carriage to the sea is an urgent need for the Murray Valley. The grea cost, together with uncertainty of ultimatate success, have delayed any measure of relief being undertaken up to the present. This can be effected at least cost and in the shortest time by the improvement and extension of the Coorong Channel to Lacepede Bay.

An extract from the Admiralty Sailing Directions shows that Lacepede Bay is perfectly suited for this purpose. The anchorage in Lacepede Bay is 1½ to 2 miles from the shore. The gap from the south terminus of the Coorong Channel to the steamers hold can be best and most economically bridged by a "Bi-cable Aerial Rope Way." Manufacturers of these rope ways guaranteed a capacity of up to 500 tons per hour with individual loads of up to 5 tons. This will mee present requirements.

The sea terminal of the aerial ropeway would be an island wharf. Produce of all kinds on the Murray would then have direct carriage to the steamer's h The aerial ropeway would in time be superseded by the harbour works usual in any considerable port, which doubtless this would become. Should the volume of trade exceed the capacity of the aerial ropeway, I can suggest a ready method of supplementing it pending the completion of the docks and wharfs of an ordinary seaport. The aerial ropeway is proposed as the quickest and most economical means of immediate relief of the present situation.

The Coorong is already navigable for about 40 miles. This leaves 50 miles at the most to be made navigable With out a survey and borings to ascertain the nature of the ground to be dredged or excavated, I cannot give an estimate of the cost, but considering the light draft of Murray River craft I cannot think that the three items necessary for direct sea communication, viz.:—

1. Improving the Coorong Channel and extending it.
2. Bi-cable Aerial Rope Way.
3. Island wharf in Lacepede Bay,

would exceed £500,000.

No Interference With Natural Conditions.

Other advantages of this scheme should be stressed. The Coorong-Lacepede Bay scheme deals with conditions that are settled and permanent. It involves no disturbance of natural conditions. On the other hand, long breakwater piers to resist a surf caused by a sea that ranges uninterrupted from Cape Horn, must cause violent changes, which in a sandy locality it is not easy to determine. This uncertainty is further complicated by the river itself. The ultimate resultant such a swirl of forces—surf, breakwater, and river current might be an obstruction that would block navigation and be costly to remove.

The Murray Mouth Scheme.

The basic idea of the Murray Mouth proposed works is twin piers projecting some way seaward. In the gales and mountainous seas so prevalent in the winter months blowing across the entrance piers, I think it doubtful whether the risk of entering by the great liners of to-day would be accepted. It must be remembered that individual tonnage of ships is increasing at an accelerated rate. The 20,000 ton ship of to-day is becoming as common as the 5,000 ton ship of a generation ago. An artificial harbour designed for ships to-day may be unusable by ships a generation hence.

The Coorong-Lacepede Bay scheme is completely free of all such risks and uncertainties. The smooth water which would cost millions to provide at the Murray Mouth is provided by Nature at Lacepede Bay. Ships may grow to any imaginable dimensions without affecting their accommodation. There are no pilotage difficulties Ships can arrive and leave at any time of the day or night. Large vessels of to-day would not be delayed in their loading by ordinary gales.

Safe Anchorage.

Lacepede Bay is formed by the bight in the coast between Cape Jaffa and the Granite Rocks, which are 19 miles north-north-eastward from the cape. It is a remarkable fact that this bay, although apparently exposed to the ocean swell, affords safe anchorage in all weathers, there being tolerably smooth water even in the height of a westerly gale. Two reasons account for the smoothness of the water—the force of the prevailing swell from the south-west is broken by the reefs off Cape Jaffa, and that from the north-west and west by traversing (before it arrives near the anchorage) a long extent of undulating ground with shallow water over it, there being only 20 fathoms 16 miles westward of Kingston jetty.

From Kingston to Cape Jaffa the land is very low; the coast is a sandy beach, with a bank behind covered with trees, the tops of which are visible seven to eight miles to seaward.

A Remarkable Locality.

Evidence of the smoothness of the waters of Lacepede Bay we have in the fact that in pre-railway days wheat was loaded by ships at the anchorage. It was taken to them from the shore in open sailing lighters. Small coasters of that time left and returned for "shelter," meeting the normal westerly sea some miles off the coast. Although the gale blows home to the shore there is no swell for miles to seaward. Although, doubtless, the very gradual slope of the seabed lessens the swell, its complete disappearance is, I think, due to vast forests of kelp. I have frequently seen it reach the surface off that coast in 20 fathoms of water.

The water is actually much smoother than in Port Phillip. Of this I have had proof in the very small boats moored with light lines off the shore in a way that would not be risked in Port Phillip. Even such slight sea—a boat sea—as is raised by a gale, I am told by fishermen, goes down directly the wind drops. The extraordinary phenomenon of a perfectly calm sea with apparently nothing to account for it, is one which I have not met with in any other part of the world.

A Murray Seaport.

Leading article in The S.A. Register of May 5, 1926.

The distinguished attainments and naval career of Vice-Admiral Sir William Creswell, K.C.M.G., and his intimate acquaintance with the sea coast of Australia, will ensure serious public attention for the novel and extremely interesting proposals for the solution of the principal River Murray problem which he has outlined in *The Observer* to-day. There is excellent reason for supposing that an experienced navigator who has studied the oceanic conditions which prevail along the southern coast of Australia is no less competent than an eminent harbour engineer to proffer advice concerning the provision of a seaport for the River Murray. The navigator is in the position to become apprised of facts which may profoundly influence a considered judgment on the feasibleness or otherwise of creating an artificial port. In any case, Admiral Creswell's carefully reached conclusion respecting a suitable site for a Murray outport makes a strong appeal to common sense and the imagination; and it is curious, indeed, that the conception should have

been so long delayed. Admiral Creswell is, of course, fully informed regarding the schemes for the opening of the existing Murray mouth and the construction of a canal from Goolwa to Victor Harbour; and he has applied to the problem much practical and technical knowledge which has not been readily available to engineers who have reported on the subject. The originality of the Admiral's ideas should not, therefore, be allowed to weigh against them. Probably the Murray mouth is not an insoluble problem to the modern engineer, but Major Johnston, the American authority who recommended the formation of a river port at Goolwa, recorded that since 1839 the mouth has shifted 1,500 feet westward under the influence of prevailing winds from the south-east and south-west. Another matter for consideration is the terrific range of the seas in Encounter Bay.

Enormous difficulties have hindered the nagivation of the mouth in highly favourable conditions, and Admiral Creswell is of opinion that even with the creation at great cost of a proper entrance-way, captains of liners might refrain from venturing into the harbour. His scheme is based on the view that the outlet of the Murray was formerly at Kingston—that the combined effect of ocean winds and waves was to slue or twist the river from a straight course and cause it to run down the Coorong channel to Lacepede Bay. The chief argument for establishing a river port at Kingston is that Lacepede Bay provides a calm sea, in which ocean liners can find safe anchorage in all weathers. The scheme includes the dredging of the Coorong (which is already navigable for 40 miles) for a further distance of 50 miles, and the conveyance of produce from the river boats to deep-sea vessels by a bi-cable aerial ropeway, the sea-terminal of which would be an island wharf. Commodities of all kinds would by this means have direct carriage from the Murray to the steamer's hold. In course of time the ropeway would be superseded by the harbour works usual in any considerable port. Both the Commonwealth Government and the State Government should be alive to the wisdom of submitting the scheme forthwith to thorough examination and test. Its merits should be impartially compared with those of rival proposals. In this matter the pressing claims of the producers in the numerous river irrigation settlements must be paramount over those of any particular marine locality in this State. The heavy cost of getting irrigation products to market from the Murray has been and is still one of the severest handicaps to the success of productive industries in the valley. "Water carriage is notoriously cheap as compared with land transport by road and rail, and the Murray is not doing its duty," observed Mr. Brunsdon Fletcher in *The Sydney Morning Herald* recently. Dried fruit for export should be sent downstream by barges and river boats to the mouth of the Murray, and there be transhipped to ocean-going steamers ready to take cargoes at once to the other side of the world. The solving of the problem of an outlet for the Murray would certainly go far to ensure the rapid expansion of the population in the irrigable districts, the greater safety of Australia, and the capture of satisfactory markets for Australia's dried fruits.

Murray Problems.

In the course of a leading article under the above heading emphasizing the need for improving the marketing arrangements in connection with the Australian fruit industry the Adelaide Advertiser of 11th May, 1926, thus calls attention to the Creswell scheme:—

"So far as the Murray settlements are concerned, the need for better and cheaper facilities for overseas transport has long been felt. Whether or not a solution of the perplexing problem has been supplied by Sir William Creswell in the scheme for linking up the traffic of the Murray with Lacepede Bay, which appeared in our columns last week, he has rendered a service to the State by again directing attention to the whole subject. Throughout the entire period of the negotiations between the riparian States in connection with the storage and use of the waters, South Australia held to the view that a navigable river should be maintained, and although under the agreement, as finally accepted, the main purpose of the storage of the waters was irrigation, the requirements of navigation were not overlooked. So strongly did this aspect of the river's potentialities appeal to the authorities that they obtained a report on the question of a harbour for the river from Colonel Johnston, the eminent American engineer, who had already gained a considerable knowledge of the conditions of the river near the mouth. After a painstaking investigation, the Colonel reported on several suggested schemes. These included the improvement of the Murray mouth and the construction of a harbour within the mouth and a large canal from Goolwa to Victor Harbour. There was, also, a proposal for the exclusion of salt water from the river above Lake Alexandrina. This was not the first time that serious attention had been given to the question of making navigation between the river stations and some seaport possible. As far back as 1874 Mr. H. C. Mais, then Engineer-in-Chief for the State, submitted a plan for a canal to connect the Goolwa channel with the sea. Details regarding width and depth of water were given, and the approximate estimate of the cost was £355,000. Of course, such an undertaking could not be carried out now for that amount.

"The problem is confessedly a difficult one, but sooner or later it will have to be solved, unless it is found that in modern circumstances other forms of transit can compete successfully with water-borne freight. As Australian settlement develops the Murray Valley will become more and more populous. The vast sums of money which have been expended in improving the river and providing for storage and reticulation will make it imperative that the best use should be made of the areas within the watershed. Years ago it was thought in the United States that railway transport offered better facilities than many of the rivers for the carriage of produce, especially produce that rapidly perishes, but, according to some authorities, later opinions have been in favour of dredging deep channels in the streams almost from the sources to the mouths of the rivers, so that there may be regular navigation. What effect motor transport may yet have on the question remains to be seen. Sir William Creswell's scheme for providing direct communication from the river to ocean-going steamers looks attractive on the face of it, but it is a matter for the consideration of experts. It seems remarkable, however, that nothing of the kind should have suggested itself to previous students of the river problems, especially as the advantages offered by Lacepede Bay as a harbour for vessels of all sizes have long been recognized. Sir William's proposition includes three points—improving the Coorong channel and extending it, a bicable aerial ropeway, and an island wharf in Lacepede Bay—and he believes the cost would not exceed £500,000. He admits, however, that before a trustworthy estimate can be given surveys and other preliminary work will be necessary. Better and cheaper transport is a crying need, and any practical suggestion which promises to meet these requirements is sure to receive careful consideration."

The Murray Problem.

IS IT SOLVED AT LAST?

By a Correspondent of The Register, May 5, 1926.

In the scheme of Vice-Admiral Sir William Creswell for direct sea communication for the Murray Valley there is displayed a profound belief in its possibilities which cannot fail to strike the reader. Its apparent simplicity at first raises wonder that such a self-evident method has only now discovered itself, until on second thoughts a natural doubt is obtruded whether engineering and financial difficulties may not be the lions which have frightened from the path all the minds which have already attempted to solve the problem. But no fears of this kind assail Admiral Creswell.

While ruminating over the question of Empire migration and the necessary relation thereto of the great Australian waterways of the interior, he was led to an examination of the Coorong and the two lakes as the Murray nears the coastline, and the well-known fact that the harbour of Kingston in Lacepede Bay provides calm water at all times of the year suggested the wish that was father to the thought of continuing that waterway through the lakes, the Coorong, and a system of canals direct to the sea. Feeling certain from his knowledge of the locality and from data ready in the publications of the Admiralty that the idea was worth testing, he lost no time in laying the scheme before the Prime Minister of the Commonwealth and the Premier of South Australia, in which State the work necessary would be carried out. Both are now seized of the daring originality of the conception, and must be struck by the advantages which would accrue if it could materialize for anything like reasonable cost. The Harbours Board have the scheme under consideration, and make no secret of the fact that their preliminary—though, of course, but cursory—view of the proposal has not caused them to doubt its feasibility.

Before a survey and borings are made it is impossible to give any estimate of the cost of excavation and dredging, but of one thing Admiral Creswell has no doubt; viz., that there would be no great difference between that and what would be required in the suggested Goolwa Canal, which, it must be remembered would not connect with a harbour safe in all weathers, but with one that would require the expenditure of a very large sum in addition to the cost of the canal. To investigate the details of the new proposal would involve but a trifling outlay, which would be quite insignificant in comparison with the expected benefits. At this time of difficulty in the Empire, while migration is a paramount necessity to both Great Britain and ourselves, and while the problem of the employment of cheap money available to Australia on loan for that purpose is engaging close attention, such initial expenditure would surely recommend itself to all patriotic South Australians, to whom every sound effort for the development of their own State bears the appealing mark of high endeavour. Without any great exercise of imagination, the reader will grip something of the extent and importance of the scheme, if and when it comes into being. It would completely change the aspect and conditions of the whole Murray Valley. For the settler it would mean direct communication from his wharf on the river to the markets of the world—that is, practically extending the sea route from the oceans to the heart of Australia, and serving its richest portion. It would make Lacepede Bay one of the principal ports of Australia. Indeed, Admiral Creswell believes that at no very distant date it would become the chief port. Serving such a highway of trade, which would carry the merchandise to and from all the settlements that in du time will grow along the course of the great river and spread on either side, it seems not unlikely to fulfil even so splendid a prediction. Even in the immediate future it would be the port for the hinterland of the three impinging States, which would command the heavy tolls of entry and departure arising in the gateway of trade, opened by a curious provision of Nature. All about this remarkable bay is known to the British Admiralty, and Admiral Creswell speaks by the book when he says that this great port of the future will not be affected by the increasing size of ships, excepting in a manner that can be easily and cheaply remedied. There are many

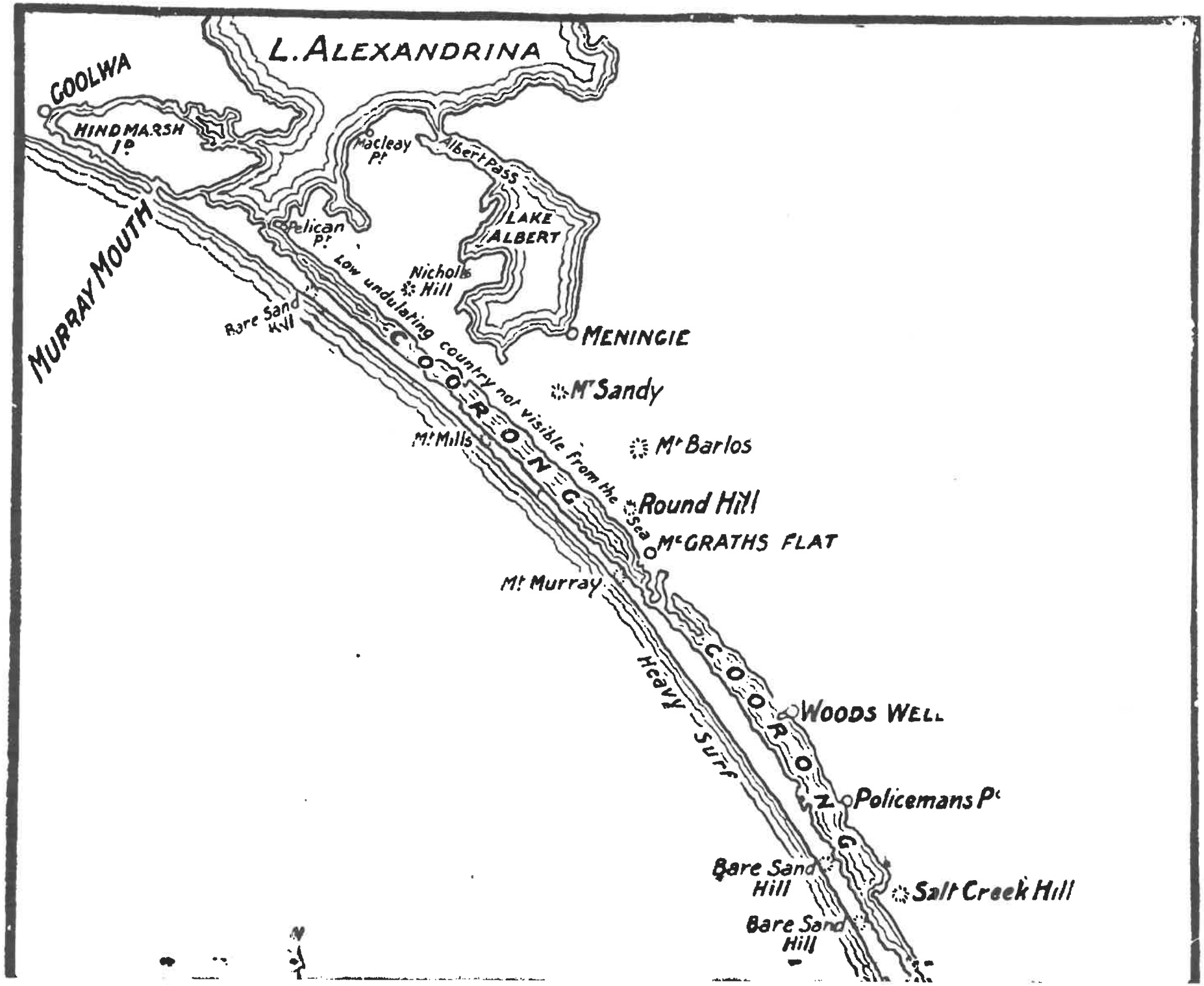
other points that arrest one's attention in the scheme, as the imagination looks into the future. With freshened waters in the Coorong, Lake Alexandrina, and Lake Albert, which would result from a barrage at the present Murray mouth and a regulating lock, we can foresee fruitful irrigated lands right down to Kingston. And in the mind's eye we can see an expanse of country drained by the lock, stretching from the southern terminal of the Coorong Canal, to-day barren and forbidding land, then to become rich with the burden of harvest.

It is no new thing for South Australia to undertake and complete a work good for itself, and of equal benefit to the rest of the continent, for even in the face of the greatest difficulties and discouragements, the overland telegraph line was carried through. No great obstacle of finance blocks the way of this proposal. If it passes the test of investigation, the complete scheme should be in operation and returning revenue, according to its author, within two years.

A SUGGESTED SHORT CUT.

From the S.A. Register, 17th May, 1926.

Regarding Admiral Sir William Creswell's Coorong-Lacepede Bay scheme, may I point out that the distance from the Murray, say, at Wellington, to Lacepede Bay, could be shortened by at least 12 miles; also that the rough weather often encountered west of Point McLeay could be avoided, and, further, the problem of Pelican Reef eliminated by taking a course through "The Narrows" into Lake Albert, thence across that lake and via what is known as Dodd's Creek to the Coorong. This would necessitate cutting a canal from the end of Dodd's Creek into the Coorong, a distance of half a mile. Most of this route is already navigable for River Murray steamers at ordinary levels.—FRANCIS GOODE, Meningie.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Mt Barlos
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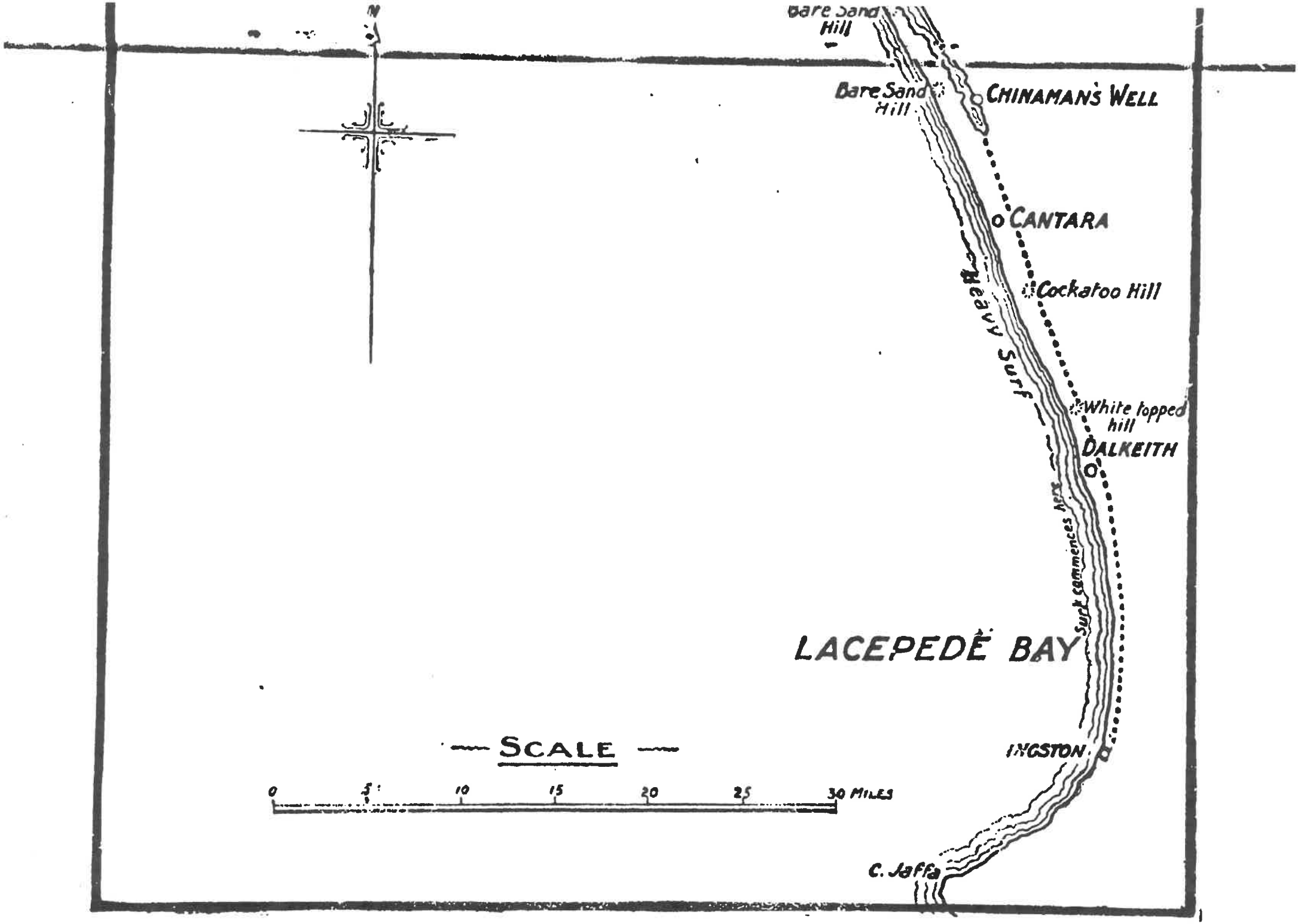
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AUSTRALIAN NAVY FOUNDATION DAY "CRESWELL ORATION"

102nd ANNIVERSARY CELEBRATION

Address by Commodore Brian G. Gibbs AM RAN Ret'd on 1st March 2003 - 102nd Anniversary of the Australian Navy's Foundation Day (1st March 1901)

WILLIAM ROOKE CRESWELL
1852 - 1933

A Remarkable life.
Australian Navy Foundation Day
Creswell Oration
28 February, 2003
by

Commodore B. G. Gibbs AM, RAN (Ret'd.)

Acknowledgments
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. Memoranda
. Captain Creswell's plea for local Naval Force (2nd September 1905)
. Captain Creswell on Australia's defence needs (11th October 1905)
. Captain Creswell "The Case for an Australian Naval Force" (1st January 1906)
. Historic Navy Orders (1911)
. Commander J. M Wilkins, RFD*, RANR (Ret'd) - verification of historical data.
. "The Commanders" Edited by D.M.Horner (1984)
. Contributions by Dr Stephen D Webster

Ladies and Gentlemen,

In my submission, any analysis of the performance of our Australian military leaders in the past should include an assessment of their performance under the stress of action and policy making.

Indeed, this has occurred in a number of analyses undertaken by several recognised military historians in fairly recent years.

The analyses to which I refer were in respect of a number of First World War and Second World War leaders, the names of whom will be familiar to most, if not all of you here today:

· Chauvel
· White
· Sturdee
· Monash
· Bridges
· Morshead

· Lavarack
· Robertson
· Scherger
· Wilton
· Vasey
· Herring
· Rowell
· Blamey
· Henry Gordon Bennett
· Creswell

No Australian naval officers reached positions of senior command in the First World War, and in the Second World War, only Collins and Farncomb commanded the Australian Squadron in action.

Admiral Sir John Crace RN commanded the Australian Squadron during the Battle of the Coral Sea, but although he was born in Australia, he was a Royal Navy Officer detached to the Royal Australian Navy and he returned to Britain soon after the Battle.

The three Chiefs of Naval Staff during the Second World War were all British Officers.

During the Second World War Collins was Commodore Commanding China Force in 1942 and in 1944 and 1945 he was Commodore Commanding the Australian Squadron in important battles in the Pacific.

As Australia's senior naval commander during the Second World War it seems unfortunate that the performance of Collins has received somewhat less historical attention than those leaders to whom I have referred. However, as the military historian, D.M.Horner, points out, we do have Collins' autobiography, "*As Luck Would Have It*".

Vice Admiral Sir William Creswell, as First Naval Member of the Australian Naval Board during the First World War, had little opportunity to command Australian Naval Forces in war.

That being so, one could be excused for wondering why his performance has, like that of the leaders to whom I have referred, been the subject of such close scrutiny. The answer, again according to Homer, is because the history of Creswell demonstrates the problems of senior policy-making faced by Australia's top naval officers during the First World War.

In his Doctoral thesis, the historian Stephen Webster in 1976 described **Creswell** in the following terms:

"As First Naval Member of the Australian Naval Board, Rear Admiral Sir William Rooke Creswell (1852-1933) played a key role in the direction of local naval operations and policy during the First World War. Creswell's long and colourful career- beginning in the Royal Navy of Queen Victoria, then in Australia's motley colonial naval forces, and finally as the senior officer of the Australian Navy - mark him as the single most important figure in the gestation, early development and first testing of the Royal Australian Navy. By virtue of his long, continuous involvement in the debate surrounding Australian naval defence, no other figure, politician or naval officer, played such an influential part."

Over the years, much has been written about Creswell's achievements while holding high office, but rather less about his earlier life.

Known as the "*Father of the Australian Navy*", **William Rooke Creswell** was born in Gibraltar on 20th July 1852.

He was the third son of Edmund Creswell, the colony's Deputy Postmaster General.

Although there is no evidence to support such an association, it nevertheless seems likely that Creswell was given the second Christian name of Rooke in memory of Admiral Sir George Rooke, who, in 1704, led a combined force of seamen and marines, resulting in the capture of Gibraltar from the Spaniards.

While of no particular relevance, yet nonetheless of some interest, is the fact that the action, which became known as the Battle of Gibraltar, was later selected as the only Battle Honour to appear on the Royal Marine Colours.

Admiral Sir George Rooke died in 1709, five years after the capture of Gibraltar.

Creswell received his early education at Gibraltar and it is not unreasonable to assume that the naval and military environment prevailing at the "Rock" at that time, would have played some part in his inclination to pursue a career in one of the Services.
Indeed, in 1864, when **Creswell** was 12 years of age, his father made the decision to send him home to England to be coached for service in the Royal Navy.
In December 1865, at the age of 13, Creswell joined the Training Ship Britannia, from which he graduated as a midshipman, 18 months later. Creswell's first ship was the Phoebe, a 35 gun screw frigate, which at the time, was deployed to the North American Station, as part of Admiral Sir Phipps-Hornsby's Flying Squadron. It was while serving in the Phoebe that Creswell, in 1869, first visited Australia. Creswell later served in the Manotaur in the Channel Fleet, and then as a Sub-Lieutenant, he was appointed to the Thalia on the China Station.
It seems that the young Creswell was exceptional as an athlete, and indeed, research reveals that while serving in the Thalia, he won a 440 yards hurdle race, an event open to the whole of the combined Fleet on the China Station.
Of some incidental interest is that, as the winner, Creswell was presented with a cup donated by the Grand Duke Alexis, who at the time was serving in the Russian Navy.
Service on the China Station occasionally required that action be taken against Chinese pirates. In one such action Creswell, who at the time was temporarily serving in the gunboat Midge, was severely wounded and in recognition of his distinguished conduct, he was specially promoted to the rank of Lieutenant.
Upon recovery from his wounds Creswell was appointed to the Royal Naval College and later to HMS Topaze, as part of the Squadron then deployed to India for the visit by the Prince of Wales, later King Edward VII.
Creswell next served in HMS Undaunted, Flag Ship of the East Indies Squadron. He was then appointed to HMS London, a Depot Ship stationed at Zanzibar during operations against East African slave traders.
Operations against the slave traders often required that naval parties operate in small boats with frequent absences from their parent ships, for upwards of a month at a time.
During the three years 1875 to 1878, Creswell was frequently involved in these activities, experiencing many brushes with Arab dhows. Indeed, one vessel he intercepted was found to be carrying a record cargo of slaves.
On another occasion, during which he and his crew landed ashore with the intention of liberating a number of slaves, Creswell and his party came under armed attack. One member of the party who was severely wounded, was saved by Creswell who helped him swim back to their boat.
Unfortunately, Creswell's health deteriorated during his service in the East Indies, to the point where it became necessary for him to be invalided back home to England. It so happened that the Royal Navy was at this time undergoing significant reductions and the prospects of young naval officers was not bright.
As a result, Creswell retired from the Navy in 1878, and in 1879, along with one of his brothers, he decided to migrate to Australia with a view to pursuing life as a farmer, which he did in Queensland for some years. It is said that he was a member of the first party of drovers to take cattle overland to the Northern Territory.
However, the call of the sea reasserted itself and, in October 1885, Creswell joined the newly formed naval service in South Australia, in the rank of Lieutenant Commander.
Thus began what can only be described as a truly distinguished Australian naval career, spanning as it did, some 34 years.
Upon joining the South Australian naval service, Creswell was appointed First Lieutenant of the small but heavily armed cruiser *Protector*.
In 1893 Creswell was appointed Naval Commandant, South Australia, serving in that capacity until 1900, at which time he transferred to the position of Naval Commandant, Queensland. During 1902 he also acted for a brief period as Captain Commanding the New South Wales Naval Forces.

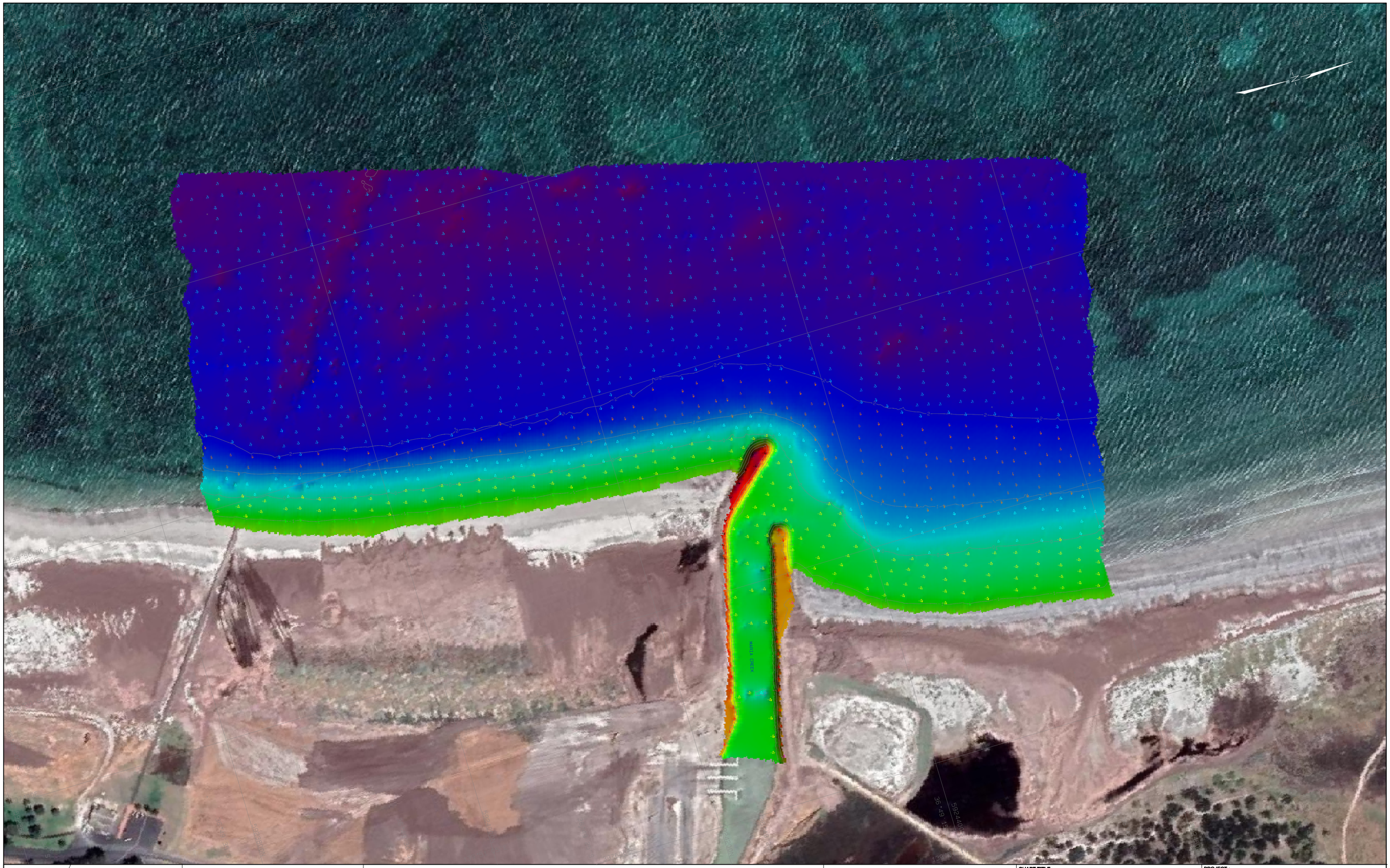
It is noted that at the time of his appointment as Naval Commandant, Queensland, Creswell was also appointed in command of his old ship *Protector*. Indeed, while under his command, the *Protector* was deployed to the China Station, in order to assist in suppressing the Boxer Rebellion. Upon return of the *Protector* to Australia, Creswell resumed his duties in Queensland.
On 1st March 1901, two months after the creation of the Australian Commonwealth, the Australian States transferred their local naval and military forces to the Federal Government.
While the amalgamation and development of the military establishments presented no major difficulties, and were immediately proceeded with, this was not so in respect of the naval units. This appears to have been due to the considerable confusion which existed in respect of naval doctrine among various Ministers of the Crown, and Members of the new Federal Parliament.
Victoria had, for half a century, developed what has been described as a "formidable" flotilla, and South Australia and Queensland had also supported the principle, apparently first enunciated in 1885 by Admiral Sir George Tryon of the Royal Navy, who commanded the British Naval force in Australian waters in 1886-87, that a separate Australian Auxiliary Squadron be formed and manned to the greatest extent possible by locally trained Australian personnel.
New South Wales, on the other hand, had maintained no permanent naval force, a situation which appears to have been largely due to the fact that Sydney was the base of the British Imperial Squadron; that the Commander-in-Chief's residence was there; and because strong economic and social interests combined in opposing transfer of the naval administration to the temporary seat of Federal Parliament in Victoria.
Indeed, locally, New South Wales did nothing of any significance in respect of the new Commonwealth Navy, until the United Kingdom gifted its Sydney base to Australia in 1913.
In 1887 a Colonial Conference was held in London. While, as a result of that Conference an Australian Auxiliary Squadron was formed, this did not realize Admiral Tryon's ideal of a locally manned and locally controlled Squadron. That this was so was hardly surprising so long as the Australian Colonies remained under mutually independent governments.
Creswell has been variously described as essentially a seaman, trained in the Navy when masts and yards and sails were in use, and boatwork was the order of the day. He was a practical sailor who believed in maintaining in seagoing order all of the ships and craft coming under his orders.
It has been further said that Creswell was characterised by indomitable I courage and the persistent pursuit of his objectives. There is no doubt I that he was very much a "man's man", but above all else he was a man of exceptional vision and the possessor of infinite patience.
For three years after the creation of the Australian Commonwealth, the Government used existing State Acts and Regulations to administer its defence forces with, in the naval sphere, a Naval Commandant in each State exercising control over the forces in his area, but with no officer I appointed in overall command.
This interim period came to an end on 1st March 1904, when the Commonwealth Defence Act 1903 was proclaimed, bringing into force legislation necessary to administer the defence forces. At the same time the position of Naval Officer Commanding Commonwealth Naval Forces was created.
On 9th December 1904 an amendment to the 1903 Act came into force which, among other things, provided for a change in the administration of the Naval Forces, by the replacement of the Naval Officer Commanding Commonwealth Naval Forces as the administering authority, with a Naval Board of Administration of three regular members, headed by the Minister of State for Defence.
The position of Naval Officer Commanding Commonwealth Naval Forces was abolished on 24th December 1904 and the position of Director of Naval Forces was created, in which was vested both administrative and inspecting duties.
On 12th January 1905 the Board of Naval Administration was constituted for the first time. It consisted of the Minister for Defence (Hon. J. W. McKay), as

President, the Director of Naval Forces (Captain W. R. Creswell, CMG), and a civil member named as the Finance Member (J. A. Thompson Esq.). Commander F.H.G. Brownlow, Officer Commanding the Naval Forces, New South Wales, was named as Consultative Member.
At the time of Creswell's appointment as Director of Naval Forces, the local naval forces consisted of about 1000 men (nine-tenths of whom were engaged on a militia basis) and a few hundred cadets. The ships available were the *Cerberus, Protector, Gayundah Paluma, Countess of Hopetoun, Childers, Nepean, Lonsdale* and *Mosquito*.
The replacement of these vessels, all of which were launched between 1883 and 1891, was repeatedly urged by Creswell, who unfortunately found himself having to do so in the face of an almost bi-annual change of Ministers of Defence.
In urging the Government to take replacement action, Creswell suggested a program of construction over a period of seven years that would provide three 3000 ton cruiser destroyers, sixteen torpedo boat destroyers and fifteen torpedo boats.
In 1906 the Australian Government sent Creswell to England to study naval developments.
While his aspirations for a distinctly Australian element to the sea power of the Empire received sympathetic understanding from the First Sea Lord, Admiral of the Fleet, Sir John Fisher, they received scant consideration from the Committee of Imperial Defence.
Undaunted by lack of enthusiasm in England, Creswell persisted, and supported by the Australian Government, continued to further his plan to establish a strong local navy. Meanwhile, pending a final decision, Prime Minister Deakin set aside the sum of £250,000 for expenditure on harbour and coast defence.
In November 1908 Andrew Fisher succeeded Deakin as Prime Minister. Using Deakin's savings and taking advantage of the Colonial Naval Defence Act (1865), which empowered the colonies to maintain men-o'-war, he immediately ordered the building of two 700 ton, 27 knot torpedo-boat destroyers.
In 1909 Britain became alarmed by the rapid increase of German naval power. It was a challenge which could not be ignored and the Admiralty requested Parliament to take exceptional measures to secure the safety of the Empire. It was decided to convene an Imperial Conference in London, and in advance Australia offered a Dreadnought or any other form of help recommended by the Conference.
The Conference met on 28th July 1909, and for the British Dominions of Australia and Canada it proved a momentous occasion. It led to those countries forming independent navies, over which they exercised full control, but it was agreed that they should operate as an integral part of the Royal Navy in time of war.
In discussion, it was recommended that the whole system of Pacific Ocean defence should be remodelled by the creation of three Fleet units, one on the Australia Station, one on the East Indies Station and a third on the China Station. Each unit was to consist of a battle cruiser, three second-class cruisers, six destroyers and three submarines. The Dreadnought offered by Australia was to be flag-ship of the Australian unit and that offered by New Zealand was to be flag-ship of the China unit. The East Indies and China units would remain under direct Admiralty control as squadrons of the Royal Navy, but the Australian unit would be paid for and controlled by Australia and eventually *fully manned* by Australians.
Thus, at long last, the formation of a purely Australian navy was agreed upon and Admiral Tryon's principle, that 'personal service' was an essential part of any colonial naval force, was acknowledged. The era of payment in cash for naval protection was ended.
The following year a further Imperial Conference reached final agreement on the status of the Australian fleet, and on 10 July 1911 His Majesty King George V granted the title of 'Royal Australian Navy' to the Permanent Commonwealth Naval Forces. Creswell was promoted to Rear Admiral in 1911 and Vice Admiral (Retired) in 1922. In 1911 he was created a Knight Commander of the Most Distinguished Order of St Michael and St George (KCMG), and then, in 1919, the

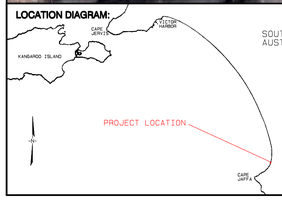
year I in which he retired, he was created a Knight Commander of the Most Excellent Order of the British Empire (KBE). In a reconstituted Naval Board Creswell was named First Naval Member, Captain B. M. Chambers, RN, Second Naval Member, Engineer Captain W. Clarkson, Third Naval Member, Staff Paymaster H. W. E. Manisty, Finance and Civil Member and Naval Secretary.
In 1912 the ex-clipper ship Sobraon was acquired, renamed Tingira and commissioned as a Boys' Training Ship. On 1st March 1913 a Naval College, providing for the training of Australian naval officers, was opened at Geelong in Victoria. Among the first boys enrolled were two future Admirals, John Augustine Collins and Harold Bruce Famcomb.
In July 1913 all Royal Navy Establishments in Australia were transferred to Australian control.
Creswell was 67 years of age at the time of his retirement to a farming property in Silvan, situated outside Melbourne. The loss of two of Creswell's three sons during the 1914-1918 Great War was a heavy blow to Creswell. Randolph, a Captain in the Camel Corps, was killed whilst serving in Palestine in November 1917. Colin, a Naval Lieutenant, who served in submarines, was lost in August the same year. A third son, Edmund, who served as a Lieutenant in the Australian Pioneers, was severely wounded at Bullecourt in France, in May 1917.
During the remaining years of his life, Creswell continued to take a keen interest in issues of public importance. Among other things, he propounded a scheme, which he advocated most assiduously, for giving the Murray River direct communication with the sea by extending the Coorong Channel to Lacepede Bay.
Creswell died on 20 April 1933, in his 81st year, and is buried in Brighton General Cemetery, Melbourne, Victoria., together with his daughter Margaret, who died on 5 April 1913, aged 20 years, and his wife Adelaide, who died on 14 February 1945.
Captain (later Admiral) Bertram Chambers, Second Naval Member of the first Naval Board appointed on 11 March 1911, said of Creswell:
"His life story is one which should be held in remembrance by coming generations of naval officers, for his career was unique and one which can never be duplicated for the conditions which led to the creation of the present Australian Navy can hardly arise again in any other part of the British Empire."
Creswell's place in history as the professional father of our Navy is secure and publicly acknowledged by the commissioning of the naval I establishment HMAS Creswell, at Jervis Bay, in 1958.
I believe that, were he alive today, Creswell would applaud the motto assigned to the Establishment which bears his name:

'HONOR INTEGRITAS VIRTUS'
(Honour, integrity, virtue)

Appendix C Maria Creek Hydrographic and Beach Survey (PHS January 2020)



LOCATION DIAGRAM:



GENERAL NOTES :

Survey undertaken between 30/01/2020 and 31/01/2020

Survey Vessel: Corsair
- Length: 6.1m
- Beam: 1.8m
- Draft: 0.5m
- Survey installation: Overside Pole

Survey Equipment:
- Valeport Midas Surveyor Echosounder
- Airmar S5510 200kHz transducer
- Trimble SPS585 RTK Rover
- Valeport SWiFT SVP
- Trimble terramodel charting / CAD software

Refer to report PHS-20-003-WLC-R001 for further information
ASCII 1m XYZ file used: PHS-20-003-WLC 200130_GDA94_1m Mean GRID.pts

LEGEND :

Depth Labels are shoal biased on a 10m radius, to the nearest decimetre.

2_5 below datum
-0_7 above datum

Contours and labels are generated from 2m interpolated GRID depth. Contour interval 1m.

Soundings colourised by the following depths:

< −4m	0m−1m
−4m − −3m	1m−2m
−3m − −2m	2m−3m
−2m − −1m	> 3m
−1m − −0m	

Coastline & Features
Contours

HORIZONTAL SCALE : 1 / 1500 (A1)

0 30 60 90 120 150

GEODETIC PARAMETERS :

Coordinate System	:	MGA94 (Zone 54)
Geodetic Datum	:	GDA94
Semi Major Axis	:	6378137.0m
Inverse Flattening (1/f)	:	298.257222101
Projection	:	Transverse Mercator (South)
Central Meridian	:	141° 00' East
Reference Latitude	:	0° North
Scale Factor at CM	:	0.9996
False Easting (X)	:	500,000.0m
False Northing (Y)	:	10,000,000m
Ellipsoid	:	GRS 1980

VERTICAL CONTROL :

Kingston CD lies 3.321 metres below TBM 6824/1160.

Soundings were reduced to Kingston CD by means of AUSGeoid09 and the AHD−Kingston CD offset value of −0.765m.

UNCERTAINTY :

Survey uncertainty is assessed to be no greater than:

Total Horizontal Uncertainty +/− 0.5m

Total Vertical Uncertainty +/− 0.2m

SURVEY STANDARDS:

N/A

APPROVAL :

PHS Representative : Andrew Bembrick CPHS2

Signature: Date: 14/02/2020

Client Representative : Annabel Sandery

Signature: Date:

CHART TITLE :

BATHYMETRIC SURVEY

REV	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
0	14/02/20	Released for client approval	MD	AB	
A	13/02/20	Issued for internal review	MD	MB	AB

Precision Hydrographic Services Pty Ltd operates under a Quality Management System certified ISO 9001:2015 by ECAS (JAS-ANZ registered).

PROJECT :

MARIA CREEK BATHYMETRIC SURVEY

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Contract No. PHS-20-003-WLC Doc. Format A1

PHS Doc. No: PHS-20-003-WLC-C001 Sheet:01/01

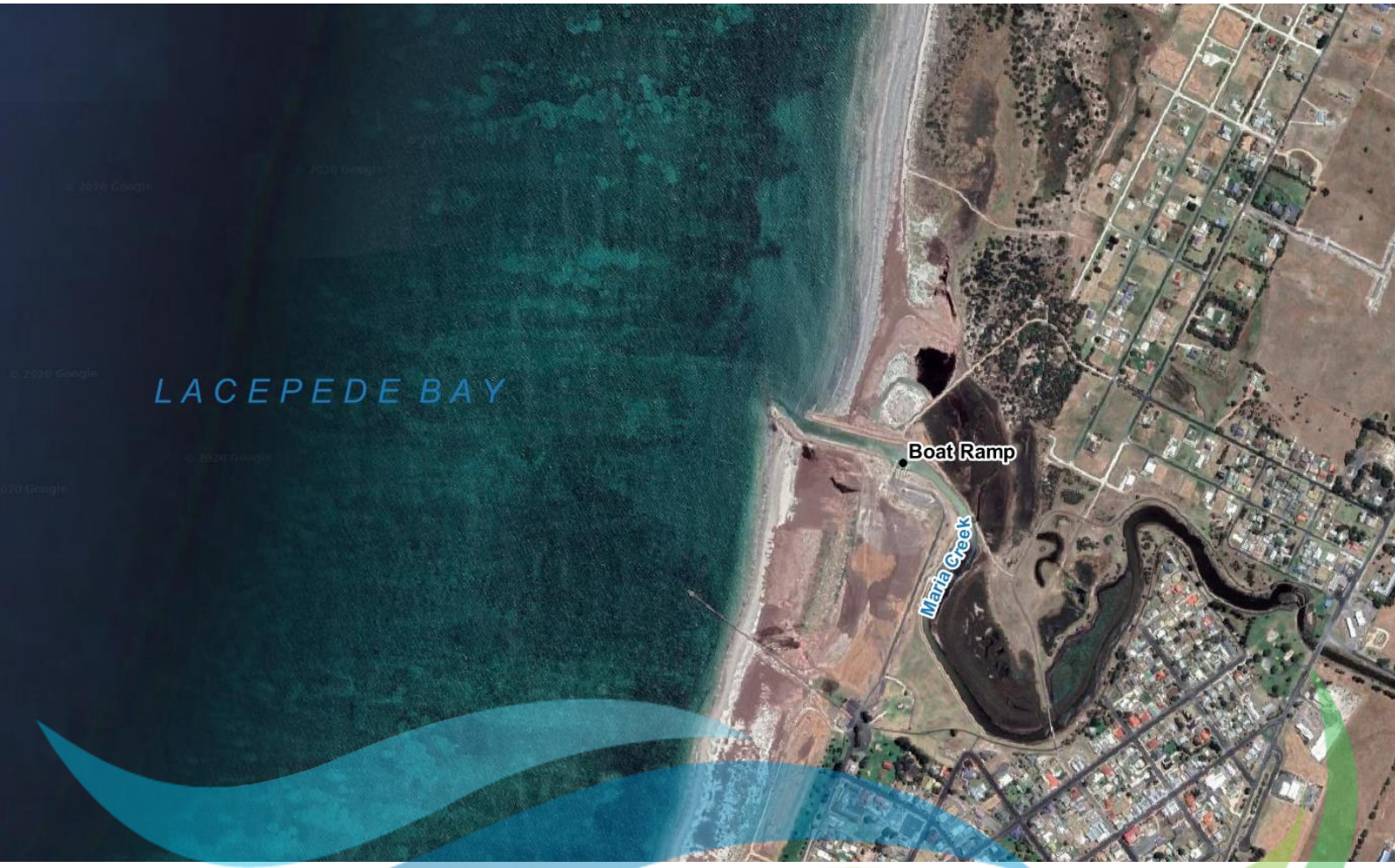
Appendix D Maria Creek Modelling Summary Report (PCS, 2020)

PORT & COASTAL SOLUTIONS

Maria Creek Concept and Design Study

Numerical Modelling Report

Version 0.2



Wavelength

Technical Report
June 2020

PORT & COASTAL
SOLUTIONS

Maria Creek Concept and Design Study

Numerical Modelling Report

Version 0.2

June 2020

P026_R01v02

Version	Details	Authorised By	Date
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0.2	Client Reviewed	Rachel White	15/06/2020

Document Authorisation		Signature	Date
Project Manager	Andy Symonds		15/06/2020
Author(s)	Rachel White Anna Symonds		15/06/2020
Reviewer	Andy Symonds		15/06/2020

Disclaimer

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Executive Summary

Wavelength Consulting, on behalf of Kingston District Council, commissioned Port and Coastal Solutions (PCS) to provide data analysis and numerical modelling services to inform the Maria Creek Concept and Design study. The study relates to the management of a boat ramp facility within Maria Creek, which over recent years has incurred untenable maintenance costs. These costs relate to maintaining the integrity of the existing training wall structures and maintaining access following periods of high seagrass wrack and sediment build up in the Creek. Council is seeking to identify a long-term solution that is financially sustainable and can be delivered through an affordable capital solution.

The aim of this study is the undertake an analysis of the baseline metocean conditions (based on existing data and outputs from numerical modelling tools developed specifically for this study) to provide an understanding of the sediment transport processes and how they vary spatially and temporally in the study area. The numerical modelling tools have also been applied to assess the implications of different concept design options at Maria Creek on the hydrodynamics, waves, sediment transport as well as potential for wrack build-up.

The numerical models developed as part of the study have been subject to an extensive calibration and validation exercise to give confidence that the models are able to accurately represent the hydrodynamic and wave processes at Maria Creek. In the absence of local calibration data, sensitivity testing of the key model parameters has been undertaken and where practical conservative assumptions have been adopted. It is recommended that local measurements of wave conditions in the Maria Creek region (e.g. measurements adjacent to the offshore end of Kingston Jetty) be obtained during winter months (when larger wave events occur) prior to any detailed design stages of the project to help improve the confidence in the modelled wave climate.

The analysis of the metocean conditions indicate that at Maria Creek there are/is:

- low wind and wave driven flows with a dominance in flows to the north due to the circulation patterns in Lacepede Bay and the effect of local wind influences (which can reverse the direction of tidal currents during periods of strong winds);
- a flood dominance within the Creek indicating that it will typically act as a net importer of both sediment and wrack;
- irregular increased freshwater discharge during flood events which could intermittently help to remove sediment build up in the Maria Creek channel, although it is likely that any mobilised sediment would subsequently be redeposited close to the mouth of the Creek in an ebb bar formation;
- relatively low wave heights, but sufficient to drive a longshore transport of sediment in the nearshore region;
- a net northward longshore sediment transport with predicted net transport rates of between 15,000 to 30,000 m^3/yr based on the present day shoreline orientation; and
- potential for periods of significant increases in longshore sediment transport in the form of sand slugs, particularly following storm events when large volumes of sediment have been made available through local shoreline erosion.

For the present (baseline) configuration of the training walls at Maria Creek, the area immediately within the entrance to the Creek and especially on the southern side of the entrance is sheltered from waves. As a result, any sediment or wrack which is transported there by waves and tidal/wind-driven currents during the flood stage of the tide is expected to be deposited and is unlikely to be remobilised.

Without the training walls in place the Creek channel would be expected to be very unstable, with it mainly being closed. The tidal prism of the Creek would need to be approximately an order of magnitude larger for the entrance channel to be more stable.

Four concept design options were proposed by Wavelength Consulting to test how they influence the physical processes around Maria Creek by applying the numerical models. The following concepts were assessed:

- Concept 1: Dredging of sand build-up (south of Southern training wall and within Maria Creek) and ongoing bypassing, with no structural changes;
- Concept 2: Extended and reconfigured training walls and dredging within Maria Creek;
- Concept 3: North training wall extended and widened within Maria Creek and dredging of sand build-up (south of Southern training wall and within Maria Creek); and
- Concept 4: Removal of training walls to return coastline to natural orientation.

The results from the modelling of the four options indicate:

- Concept 1: this concept does not include any structural changes to the breakwaters, so any changes to the wave and current dynamics are due to the capital dredging campaign. It is likely that the shoreline orientation would change quite quickly (likely over months to years depending on occurrence of larger wave events) to reach a dynamic equilibrium in the future. It is possible that the potential for wrack to be imported into the Creek could be reduced compared to the existing (baseline) case due to the reduced tidal current speeds flowing into the Creek. Despite this, the flows within the Creek on the whole remain flood dominant and the entrance remains unprotected from storm winds and waves, with some wrack accumulation still expected within the Creek for Concept 1. Sediment import into the Creek would be reduced relative to the existing (baseline) case since the dredging removed the sand bar present at the entrance to the creek which provides a supply of sediment to be imported into the Creek;
- Concept 2: it is expected that sediment would continue to build-up on the southern side of the extended southern training wall and eventually the build-up would allow the sediment to naturally bypass the structure. The additional sheltering of the shoreline directly to the north of the northern training wall could also result in an increased build-up of sediment in this location. The protection of the entrance from storm winds and waves and the reduction in tidal prism which flows into and out of the Creek indicates a reduced potential for wrack to be imported into the Creek, although the flood dominance means that any wrack which is imported is likely to remain within the Creek. As for Concept 1, the sediment import into the Creek would be reduced relative to the baseline case since the dredging removed the sand bar present at the entrance to the Creek which provides a supply of sediment to be imported into the Creek;
- Concept 3: this design does not result in any significant changes compared to Concept 1; and
- Concept 4: the shoreline would likely evolve over time to become relatively straight over the Maria Creek section. Without the training walls in place the Creek channel would be expected to be very unstable, with it mainly being closed and occasionally opening during large freshwater discharge events.

Overall, the results indicate that ongoing longshore sediment transport would result in a build-up of sediment in the lee of all of the Concept structures and that in order to maintain the dredged depths within the Creek and to avoid the eventual bypassing of any structures, some form of sediment management would be required. While the import of wrack into Maria Creek could be reduced, none of the Concept designs would be likely to completely stop this process and so ongoing management of wrack would also be required to keep the Maria Creek channel clear.

1. Introduction

Wavelength Consulting, on behalf of Kingston District Council (Council), commissioned Port and Coastal Solutions (PCS) to provide data analysis and numerical modelling services to inform the Maria Creek Concept and Design study. The study relates to the management of a boat ramp facility within Maria Creek, which over recent years has incurred untenable maintenance costs. These costs relate to maintaining the integrity of the existing training wall structures and maintaining access following periods of high seagrass wrack and sediment build up in the Creek. Council is seeking to identify a long-term solution that is financially sustainable and can be delivered through an affordable capital solution.

The aim of the data analysis and numerical modelling assessment detailed in this report is to:

- provide an understanding of the local metocean conditions and the sediment transport processes occurring at the study area;
- determine the wave conditions at the site for a range of Average Recurrence Interval (ARI) wave events;
- quantify the longshore transport rates along the shoreline to the north and south of Maria Creek;
- develop an understanding of the tidal prism and flow speeds in the Creek and assess the stability of the Creek entrance; and
- assess the implications of a number of design concepts on sediment transport and wrack accumulation to help estimate the ongoing maintenance requirements.

1.1. Background

Council operates and maintains a recreational boating facility within the downstream reaches of Maria Creek, which is located towards the southern end of Lacepede Bay in South Australia (Figure 1). The existing facility consists of a boat ramp with floating pontoons within the Creek and two rock armoured training wall structures at the Creek Entrance to provide sheltering and to stabilise the Creek entrance. The original facility was constructed between 1988 and 1996 and upgraded (including a widening and deepening of the entrance channel and an extension of the training wall structures) between 1999 and 2000.

Between 1996 and the end of 2015 there was limited build-up of sediment adjacent to the training walls, with an analysis of the beach profiles immediately to the south of the southern wall (from 2005 to 2012) suggesting that cyclical build-up and removal of sediment had occurred as opposed to ongoing sedimentation (Tonkin Consulting, 2018). This was mainly due to sand bypassing undertaken at Maria Creek prior to 2016, with any build-up of sediment on the south side of the southern training wall (henceforth 'the south beach') moved to the north of the northern training wall (henceforth 'the north beach'). Based on the beach profile data and other aerial imagery this approach appears to have been successful in maintaining the general beach position and managing any build-up of sediment and wrack along the southern training wall. Details of this bypassing are limited and it is not possible to ascertain the relative contribution of natural processes and anthropogenic intervention in the removal of sediment build-up. Since 2016 there has been limited management of sand on the south beach and as a result significant sedimentation has occurred (estimated by Tonkins Consulting (2018) to be in the order of 250,000 m^3 of accumulation between 1987 and 2018, expected to mostly have occurred after 2016) as well as sedimentation within the entrance to Maria Creek and on the north beach. Post 2016, only a 'small wedge' of sand was removed from the southern beach at some point between June 2018 and July 2019, but exact volumes and dates of removal are not known (B Smith 2020, pers. comm., 6 May).

Within the Creek entrance, on going sedimentation and accumulation of seagrass wrack have also required periodic maintenance to remove material and ensure ongoing access to the boat ramp facility. Between November 2017 and February 2018, 35,000 m^3 (disturbed

volume) of material (sediment and seagrass wrack in unknown proportions) was removed and between June 2018 and July 2019, an additional 40,000 m^3 (disturbed volume) of material was removed from the Creek entrance and channel. Additional rock armour protection has also periodically been placed on the training wall structures, following storm damage to the structures.

Council has reported that in the last 5 years (since 2015) there has been a noticeable increase in the severity of storm events resulting in damage to the training walls and in sand and seagrass wrack deposition and accumulation on the south beach. As a result, the maintenance costs for the facility are untenable and Council is seeking to identify a long-term solution that is financially sustainable and can be delivered through an affordable capital solution.

Figure 1. Location plot of the Maria Creek region.

1.2. Report Structure

The report herein is set out as follows:

- available metocean data are described to characterise the site conditions in **Section 2**;
- the setup of the numerical modelling tools is detailed in **Section 3**;
- the performance of the numerical modelling tools is assessed in **Section 4**;
- the flows within and around the Creek are described in **Section 5**;

- details on the local wave climate are presented in **Section 6**;
- the setup and results of the longshore drift modelling is described in **Section 7**;
- a conceptual understanding is provided in **Section 8**;
- an overview of the design concepts is provided in **Section 9**;
- the results from the modelling of the different concepts are presented in **Section 10**; and
- a summary of the assessment is provided in **Section 11**.

Unless stated otherwise, levels are reported to Australian Height Datum (AHD). The AHD level at Maria Creek is 0.76 m above the Lowest Astronomical Tide (LAT) level.

Wind and wave directions are reported as the direction coming from in degrees clockwise from True North. Current direction is quoted as direction going to. Sediment volumes are reported as in-situ cubic metres.

2. Site Conditions

This section provides a summary of the metocean and sediment transport conditions at Maria Creek (the study area) based on available data and literature. The location of all available data used for the characterisation of the study area is shown in Figure 2.

2.1. Astronomical Tides

Maria Creek is located at the southern end of Lacepede Bay, where tidal ranges are less than 1 m on spring tides, indicating a micro-tidal system. Of particular importance in this region is the occurrence of dodge tides periodically throughout the year when there is very little change in tidal level for a significant portion of the daily tidal cycle.

A summary of the tidal planes at Maria Creek (based on levels at Kingston) relative to LAT and Australian Height Datum (AHD) is provided in Table 1, with AHD being 0.76 m above LAT. Note that low water levels are not available and are approximated based on the Victor Harbour and Cape Jaffa levels.

Table 1. Tidal Planes at Maria Creek (Kingston).

Tidal Plane	Elevation (m LAT)	Elevation (m AHD)
Highest Astronomical Tide (HAT)	1.6	0.84
Mean High Water Springs (MHWS)	1.2	0.44
Mean High Water Neaps (MHWN)	1.0	0.24
Mean Sea Level (MSL)	0.8	0.04
Mean Low Water Neaps (MLWN)	*0.6*	*-0.16*
Mean Low Water Springs (MLWS)	*0.3*	*-0.46*
Lowest Astronomical Tide (LAT)	*0*	*-0.76*

The closest water level records to Maria Creek are from:

- Cape Jaffa (approximately 18 km southwest of Maria Creek at the southern end of Lacepede Bay) where there is a 5 month record from mid September 2003 to mid February 2004, with a 5 minute sampling frequency; and
- Victor Harbour (approximately 175 km to the northwest of Maria Creek at the northern end of Lacepede Bay) where there are hourly water levels from 1964 to January 2019.

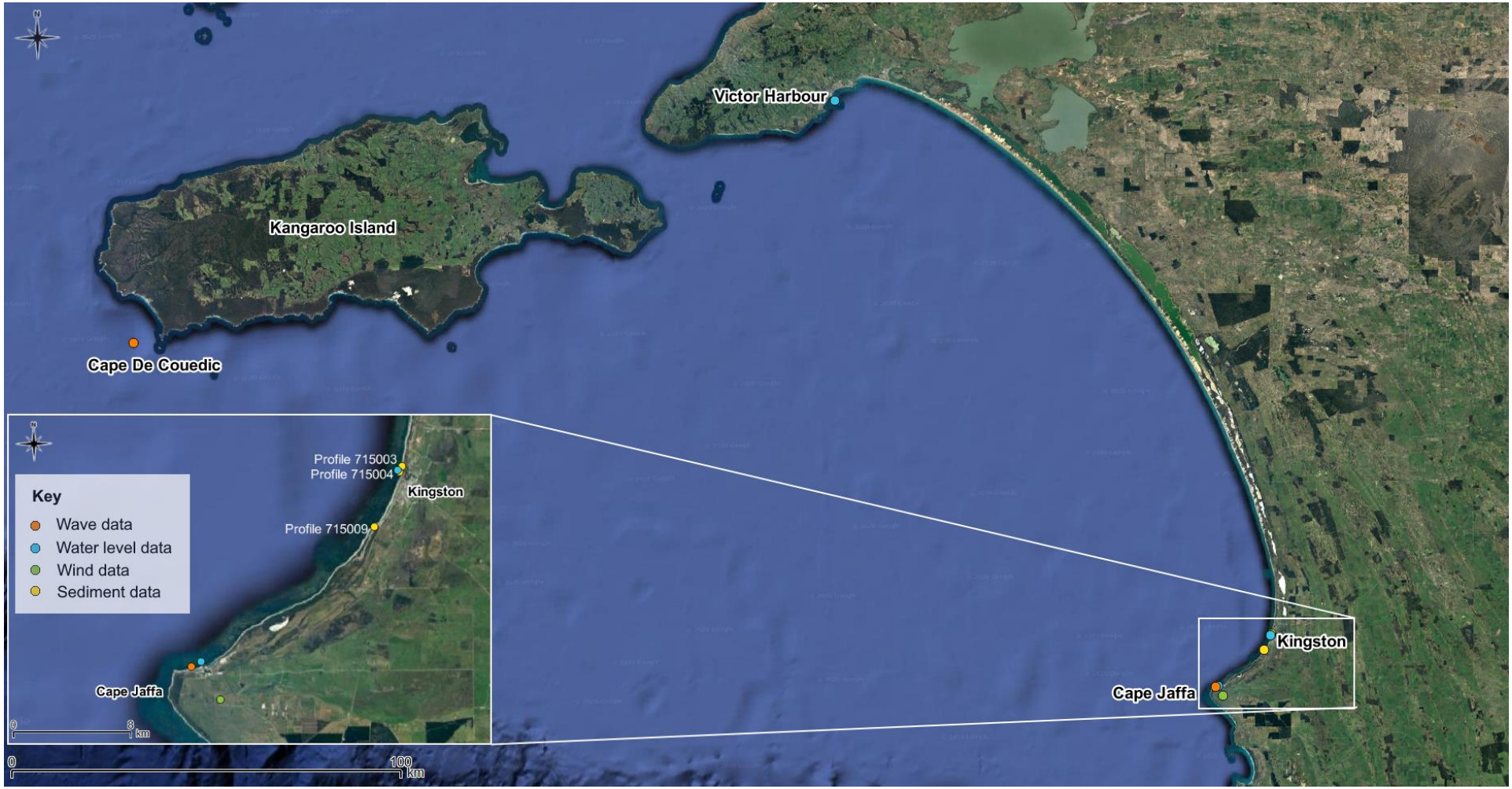


Figure 2. Summary of data locations.

While Victor Harbour is located some distance from Maria Creek, comparisons of predicted water levels at Kingston and Victor Harbour indicate that the timing and amplitude of the tidal signal does not vary significantly across Lacepede Bay. The analysis of measured water levels at Victor Harbour is therefore considered to be able to provide a reasonable indication of historical extreme water level and surge events at Maria Creek.

The Victor Harbour water level data have been analysed to determine the highest measured water levels and the largest surge events which occurred between 2000 and 2018 (Table 2 and Table 3, Figure 3 and Figure 4) The results show that:

- the highest measured water level occurred in May 2016, with a water level of 1.54 m AHD. The second highest water level was 1.40 m AHD in January 2007;
- the highest water levels are typically associated with a surge height in excess of 0.5 m along with a predicted water level of more than 0.4 m AHD;
- the largest surge event occurred in July 2007 with a surge height of 1.17 m and a total measured water level of 1.16 m AHD. Two of the five largest surge events occurred in 2016, with peak surges of 0.97 and 0.92 m occurring in May and September, respectively; and
- there is a negative correlation between the peak in the surge events and the predicted water level, with the majority of the largest surge events occurring when the predicted water levels are low (less than 0.4 m AHD). This suggests that it is unlikely that the peak of a large surge event would coincide with high water.

Table 2. Twenty highest measured water levels at Victor Harbour (2000 to 2018).

Time	Measured Water Level (m AHD)	Predicted Water Level (m AHD)	Surge Height (m)
9/05/2016 14:30	1.54	0.75	0.79
21/01/2007 1:30	1.40	0.54	0.87
5/05/2015 13:30	1.36	0.70	0.66
6/06/2003 15:30	1.34	0.48	0.86
21/06/2000 14:30	1.32	0.59	0.73
25/04/2009 13:30	1.32	0.63	0.69
4/07/2011 14:30	1.30	0.73	0.57
21/11/2018 23:30	1.28	0.42	0.86
29/10/2007 1:30	1.27	0.61	0.66
3/05/2007 13:30	1.27	0.63	0.63
22/05/2011 15:30	1.26	0.62	0.64
26/05/2001 14:30	1.25	0.71	0.54
3/08/2004 13:30	1.25	0.53	0.72
20/07/2008 14:30	1.24	0.67	0.57
6/08/2017 12:30	1.23	0.62	0.61
15/09/2008 12:30	1.22	0.40	0.82
28/06/2014 13:30	1.21	0.75	0.47
19/05/2003 14:30	1.21	0.72	0.49
18/08/2013 12:30	1.21	0.46	0.75
11/07/2016 15:30	1.20	0.41	0.80

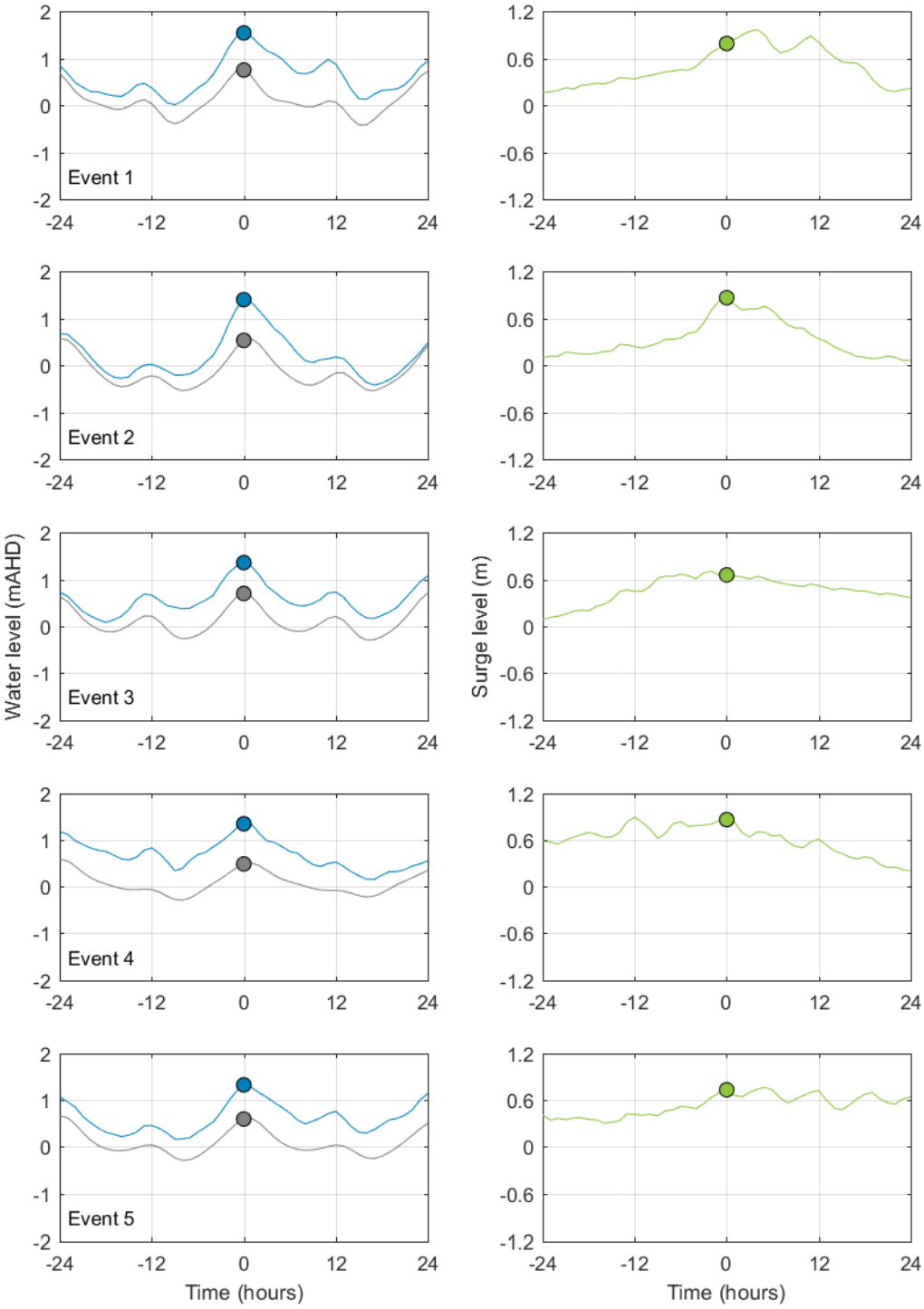


Figure 3. Time series of the five highest measured water levels at Victor Harbour (2000 to 2018).

Table 3. Twenty largest surge events at Victor Harbour (2000 to 2018).

Time	Measured Water Level (m AHD)	Predicted Water Level (m AHD)	Surge Height (m)
4/07/2007 20:30	1.16	-0.01	1.17
14/04/2018 9:30	1.04	0.00	1.04
9/05/2016 18:30	1.09	0.12	0.97
20/06/2011 21:30	0.92	0.00	0.92
29/09/2016 20:30	0.79	-0.12	0.92
6/06/2003 3:30	0.83	-0.06	0.89
12/07/2016 7:30	1.03	0.14	0.88
28/11/2009 18:30	0.77	-0.10	0.87
21/01/2007 1:30	1.40	0.54	0.87
22/05/2011 20:30	0.93	0.07	0.86
21/11/2018 23:30	1.28	0.42	0.86
23/06/2014 8:30	1.20	0.35	0.85
20/07/2000 22:30	0.72	-0.13	0.85
25/08/2009 14:30	1.10	0.25	0.85
15/09/2008 11:30	1.13	0.29	0.83
13/05/2000 8:30	0.74	-0.09	0.83
21/03/2013 12:30	0.73	-0.10	0.83
6/07/2018 23:30	0.85	0.02	0.82
1/07/2009 12:30	1.12	0.30	0.82
2/10/2013 11:30	1.10	0.28	0.82

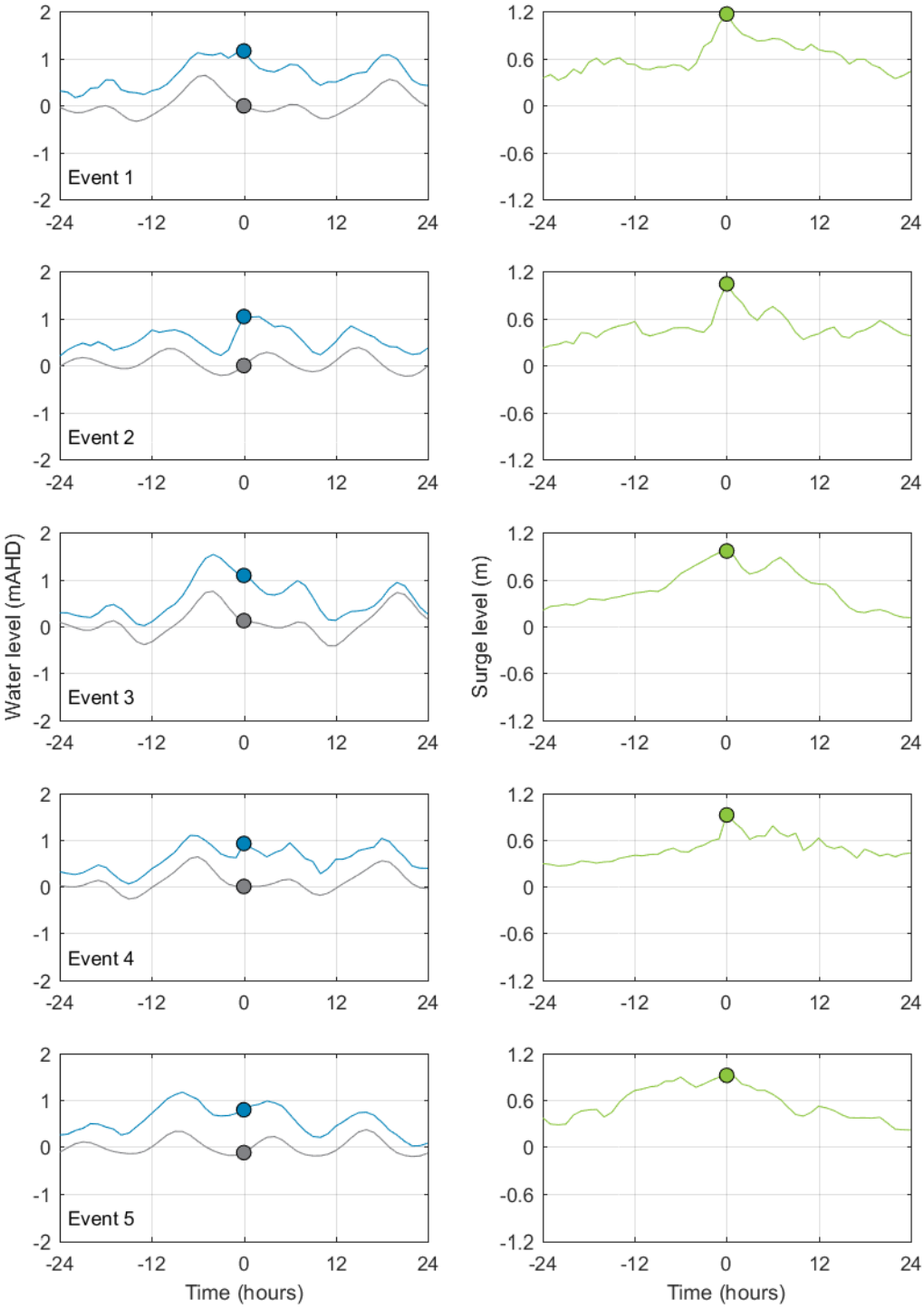


Figure 4. Time series of the five largest surge events at Victor Harbour (2000 to 2018).

No local data are available to provide information on the tidal currents in the Maria Creek region. Therefore, the hydrodynamic model will be used to provide an indication of the magnitude of tidal currents both within Maria Creek and offshore of it. Additional information on local tidal flows is presented in Section 5.

2.2. Wind and Waves

Wind data are available at a number of meteorological stations around the study area. The closest station to Maria Creek is at Cape Jaffa (see Figure 2).

The annual wind conditions in the study region show a high variability in direction, with a dominance of winds from the south to southwest (Figure 5). There is a high degree of seasonal variability with prevailing winds from the south in the summer and from the north in the winter (Figure 6). These seasonal trends are varied by sea breezes through the day, with stronger winds typically developing in the afternoon when winds tend to blow onshore (Figure 7 to Figure 9).

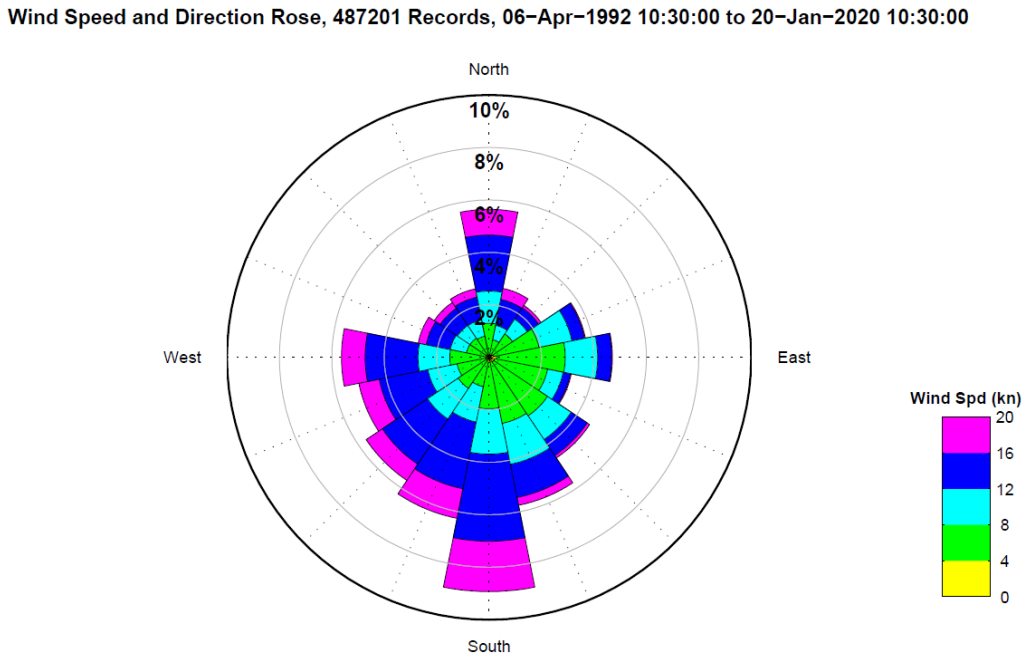


Figure 5. Annual wind rose from Cape Jaffa Station (April 1992 to Jan 2020).

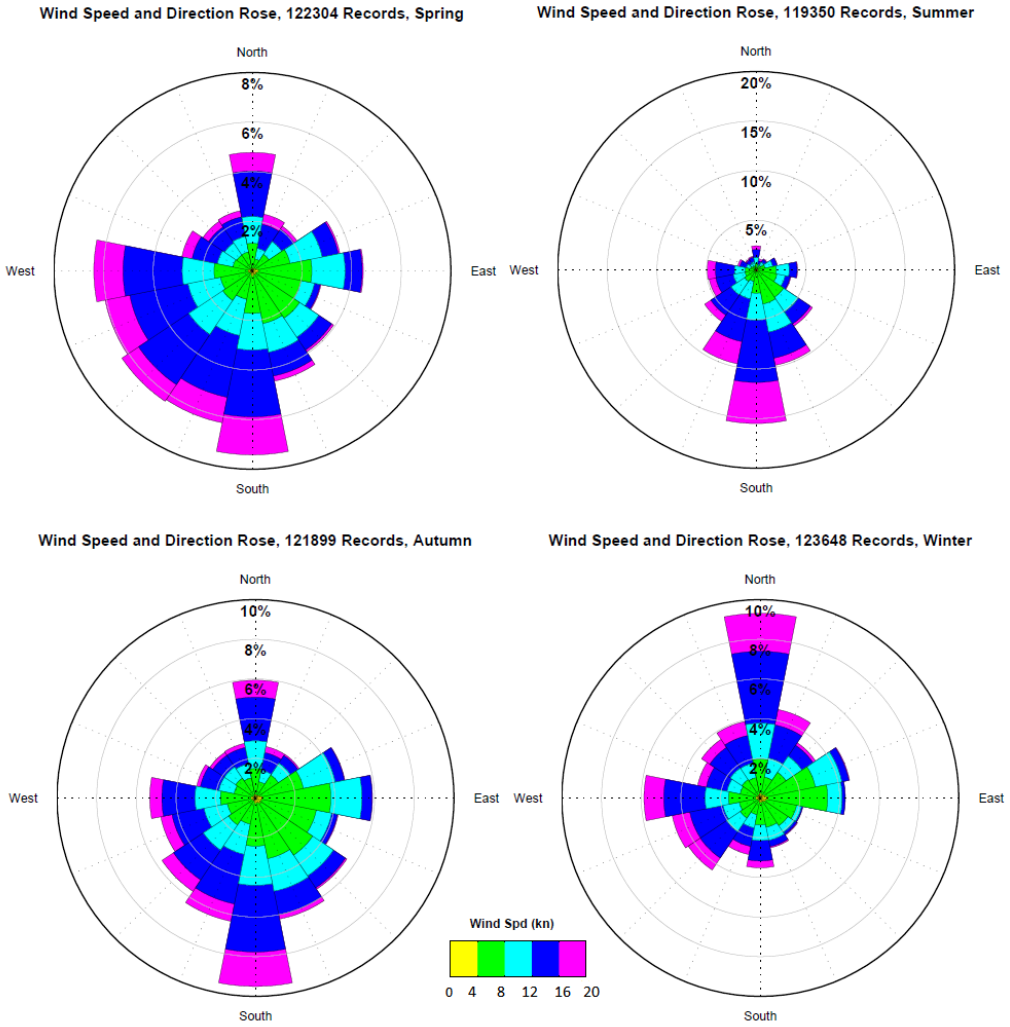


Figure 6. Seasonal wind roses from Cape Jaffa Station (April 1992 to Jan 2020).

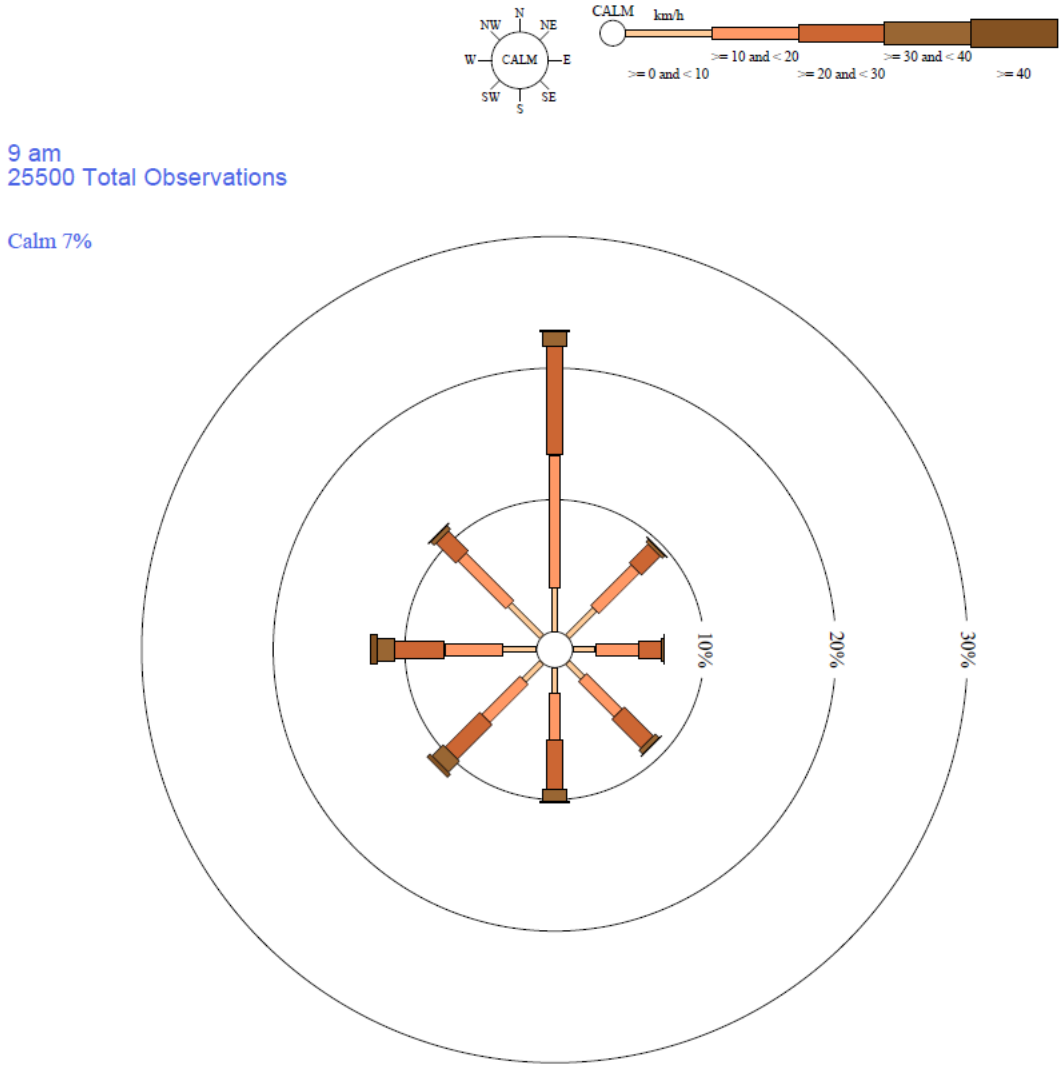


Figure 7. Annual wind rose at 9am from the Cape Jaffa Station (April 1992 to Jan 2020) (www.bom.gov.au, March 2020).

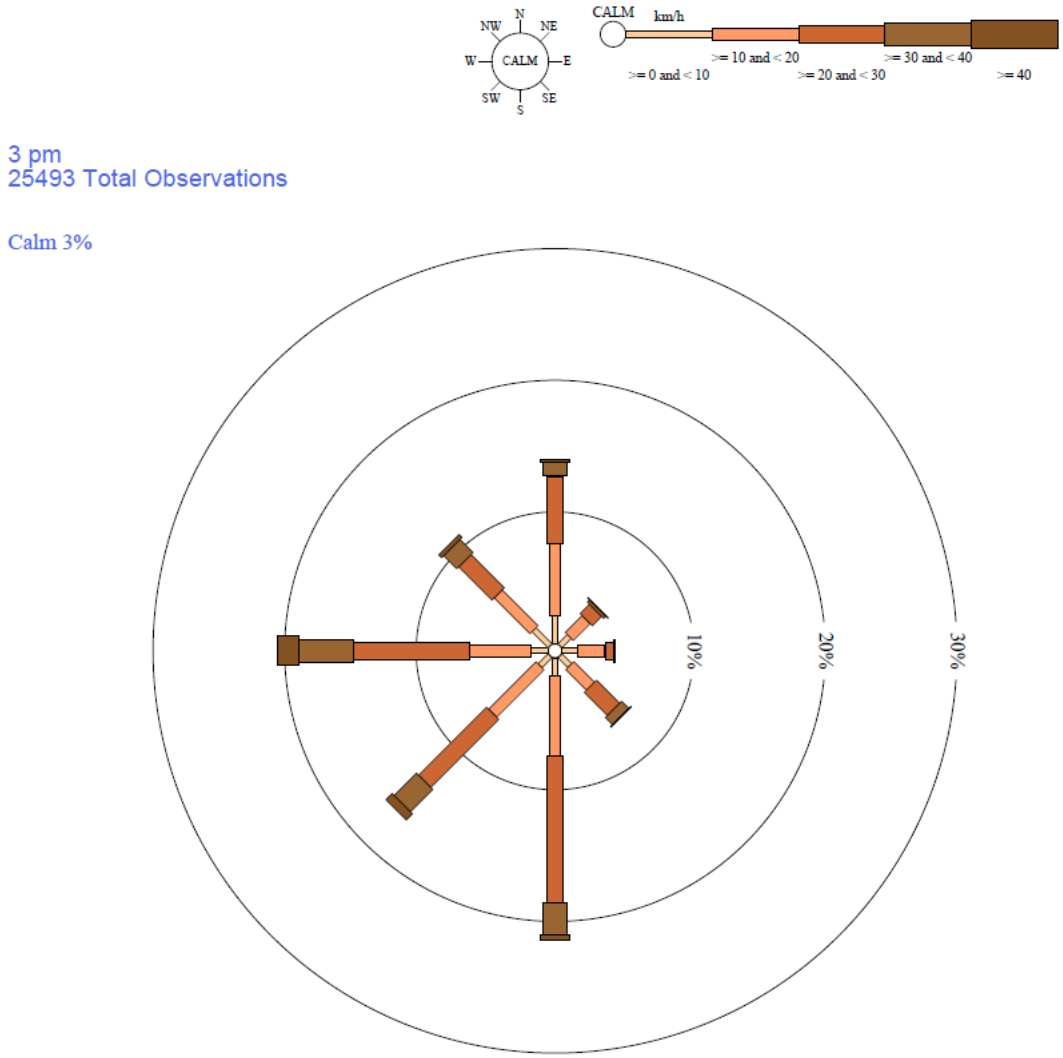


Figure 8. Annual wind rose at 3pm from the Cape Jaffa Station (April 1992 to Jan 2020) (www.bom.gov.au, March 2020).

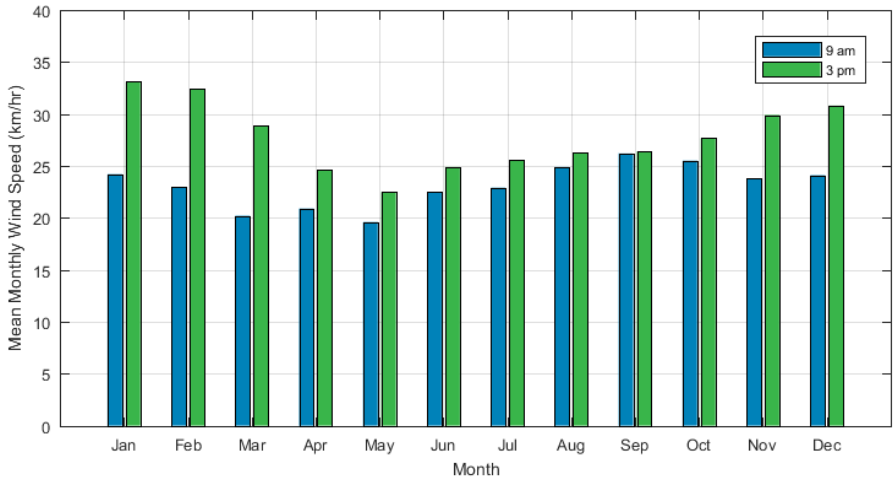


Figure 9. Average monthly wind speed recorded at the Cape Jaffa Station at 9am and 3pm (www.bom.gov.au, March 2020).

The closest long-term observational record of the wave climate is from a Waverider Buoy (WRB) deployed at Cape de Couedic, approximately 300 km to the west northwest of Maria Creek. The WRB was deployed in late November 2000 and is still operational at the time of writing. The original WRB was non-directional, this was replaced with a directional WRB in June 2019. The largest waves at the WRB occur in the winter (when the median wave height is 2.87 m) and smallest waves occur in the summer (with a median wave height of 2.19 m).

The storm waves are significantly larger than these more typical wave conditions. The top ten waves recorded at the Cape de Couedic WRB are detailed in Table 4. Only one of the wave events was recorded after the directional WRB was deployed. Wave directions for all other events have been obtained from the WaveWatch III model (WW3DG, 2016). The following observations can be made about the extreme waves at the WRB:

- the largest wave, with a significant wave height (H_s) of 9.6 m, occurred in July 2019;
- eight of the ten largest waves from the twenty year record occurred in the last ten years;
- the largest waves are from the southwest sector;
- the wave periods are indicative of swell waves; and
- the largest extreme waves occur between late Autumn and mid Spring (i.e. mainly over the winter months).

Table 4. Ten largest waves recorded at the Cape de Couedic WRB.

Date/Time	Peak Hs (m)	Wave period (s)	Wave direction (Deg)
23/07/19 02:00	9.6	18.2	212
31/07/14 21:10	9.26	15.1	232
29/10/17 19:30	8.89	17.3	240
18/08/13 09:10	8.82	17.4	245
15/09/08 08:40	8.71	15.4	247
12/05/15 12:10	8.53	17.4	217
29/09/16 12:40	8.53	13.8	227
03/05/16 10:10	8.52	17.6	225
04/09/02 02:00	8.45	18.0	241
29/06/12 12:50	8.37	17.3	241

The dominant wave direction at the WRB is from the southwest (Figure 10 and Figure 11), even during the winter months when local winds are most frequently from the north (Figure 12). The waves at the Cape de Couedic WRB are dominated by swell waves generated many thousands of kilometres from the site by temperate low pressure systems blowing over the Southern Ocean, rather than by locally generated wind waves. This is further confirmed by the long period of the waves (the largest waves have a peak period of between 10 and 18 seconds) (Figure 13).

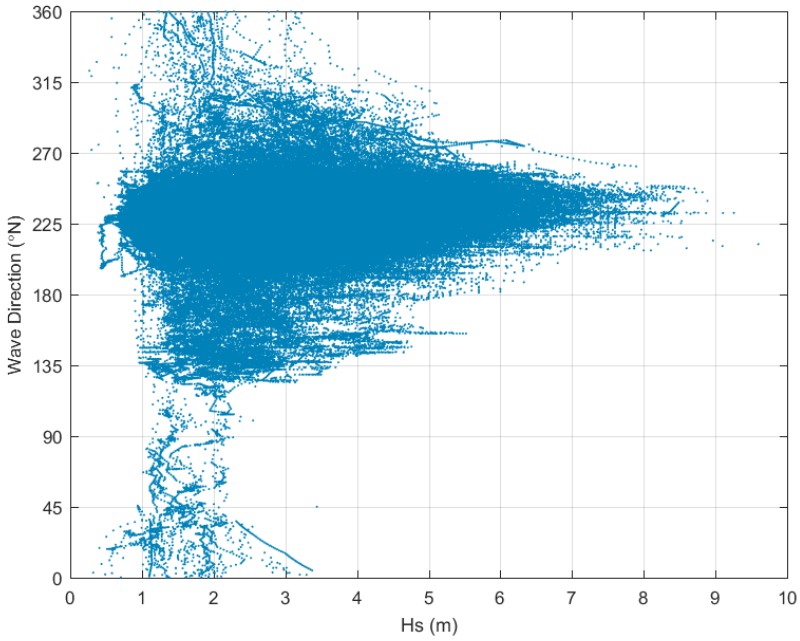


Figure 10. Significant wave height vs wave direction for the Cape de Couedic WRB (June 2019 to Jan 2020).

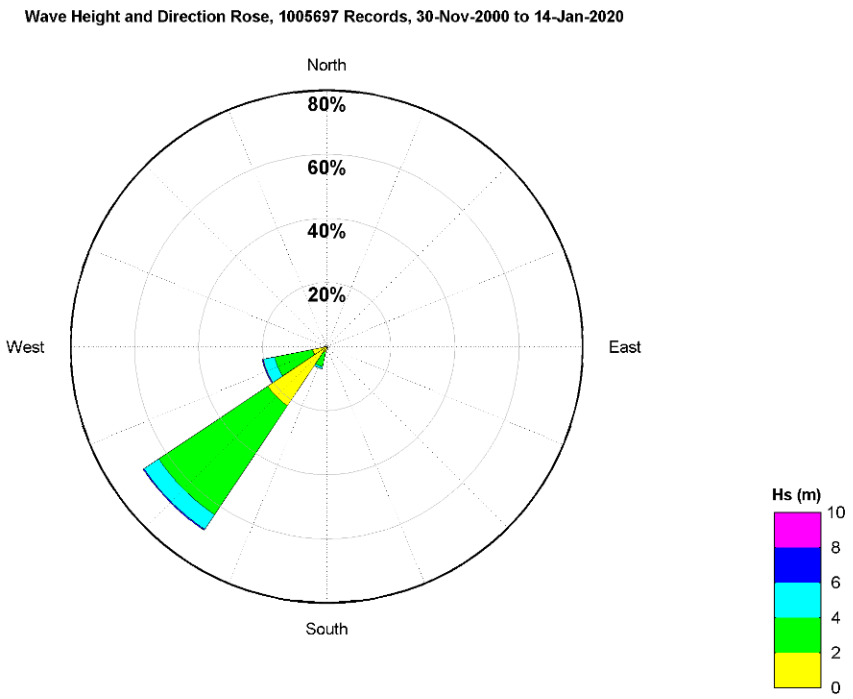


Figure 11. Annual wave rose from the Cape de Couedic WRB for data collected from November 2000 to January 2020. Directions prior to June 2019 are derived from WaveWatchIII.

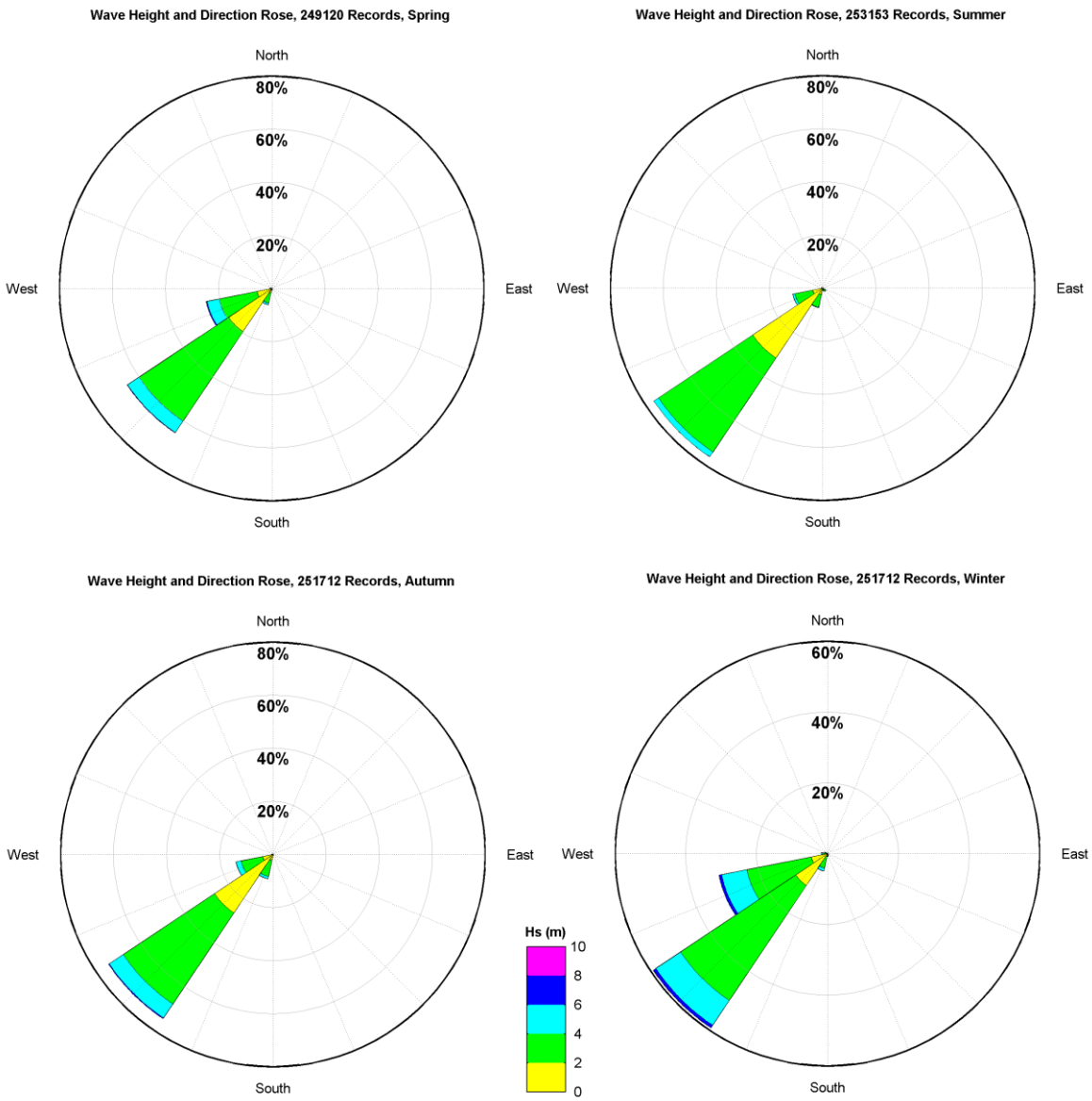


Figure 12. Seasonal wave roses from Cape de Couedic WRB for data from November 2000 to January 2020. Directions prior to June 2019 are derived from WaveWatchIII.

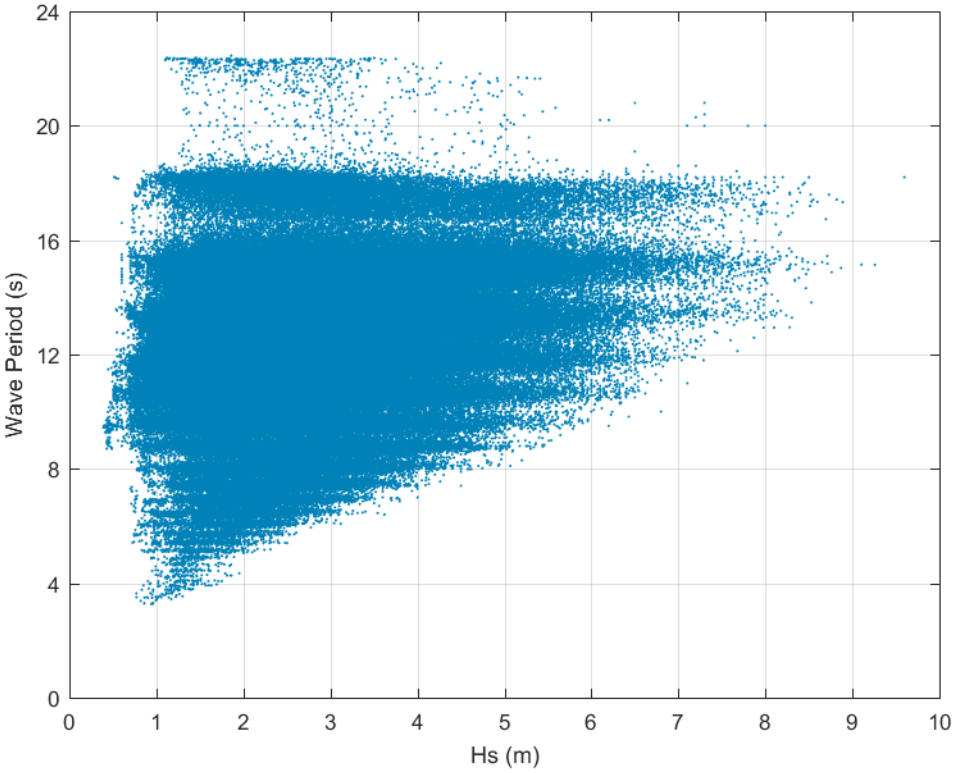


Figure 13. Significant wave height vs peak wave period for the Cape de Couedic WRB (November 2000 to January 2020).

The wave environment in Lacepede Bay and Maria Creek is expected to be significantly less energetic than at Cape de Couedic, with wave energy attenuated as waves propagate across the wide zone of relatively shallow water in Lacepede Bay. Wave data collected at Cape Jaffa in 2013 suggests that wave heights could be a factor of ten smaller than those at Cape de Couedic, with H_s typically less than 1 m. This is consistent with the presence of seagrass beds close to the shoreline local to the study area. The wave climate at Maria Creek is investigated in more detail using the wave model developed as part of this study in Section 6.

2.3. Rainfall and River Discharge

The Maria Creek region has a Mediterranean climate with cool to mild winters with moderate rainfall and warm to hot, generally dry summers. Based on measured data at the BoM Kingston station, the mean annual rainfall is 533 mm. The long term monthly mean rainfall (between 2000 to 2020) and the monthly measured rainfall since 2010 are shown in Figure 14 and Figure 15, respectfully. The plots (and additional summary statistics provided on the BoM website) show that the majority of the rainfall occurs over the winter months between May and September, with monthly averages of between 52 mm and 85 mm.

There are no river discharge records available for Maria Creek, but freshwater discharge rates will follow the same trend as the rainfall records, peaking in July and August. Prior to July and after August, peak flows in the Creek are reported to be insufficient to naturally flush out any deposits brought into the Creek by storm events (GHD, 2013).

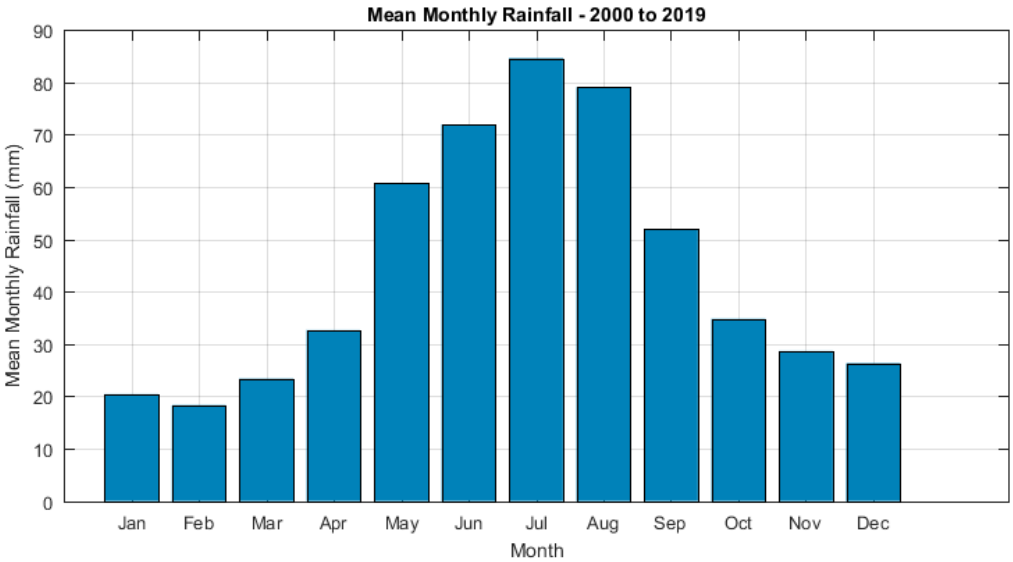


Figure 14. Mean monthly rainfall at Kingston (2000 to 2020).

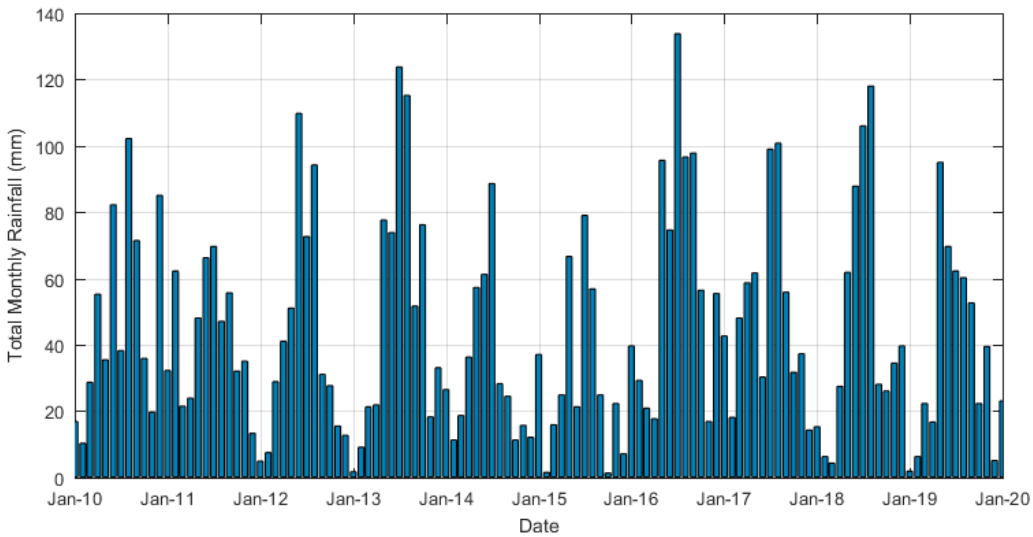


Figure 15. Monthly rainfall at Kingston (2010 to 2020).

2.4. Sediment Characteristics

Sediment samples have been taken at six locations in the southern half of Lacepede Bay (Wavelength, 2020a). The closest sample points to Maria Creek are at 715004 (on the south beach) and 715003 (on the north beach), see Figure 2 for locations. The particle size analysis of these samples indicates the presence of medium sand (with a median (d_{50}) of 0.24 mm) and coarse sand (with a d_{50} of 0.70 mm) on the beaches to the south and north of Maria Creek, respectively. This is consistent with aerial imagery which clearly shows sandy sediments along the adjacent beaches.

2.5. Sediment Transport

Sediment transport at Maria Creek can be identified from the following:

- analysis of satellite and aerial imagery to identify net trends in sediment transport;

- analysis of repeat beach profiles to provide some quantification of accretion/erosion rates; and
- application of empirical formulae to estimate rates of longshore transport.

Data from each of these methods is reviewed and interpreted in the following sections to provide an understanding of the sediment transport drivers and their variability in the study area.

2.5.1. Satellite imagery

The regular repeat imagery which satellite imagery can provide (typically monthly images depending on cloud cover) can be a useful tool to understand under what conditions significant changes to the shoreline have occurred.

Tonkin Consulting (2018) identified the location the visible water line on the south beach using aerial imagery from 1949 to 2018. The results from their analysis (shown in Figure 16) indicate a relatively stable shoreline location, with only slight seaward migration of the shoreline up until 2005. The 2018 shoreline position showed a significant seaward migration and this was attributed to the occurrence of storm surges and dune erosion south of Maria Creek (at Wyomi) (Tonkin Consulting, 2018).

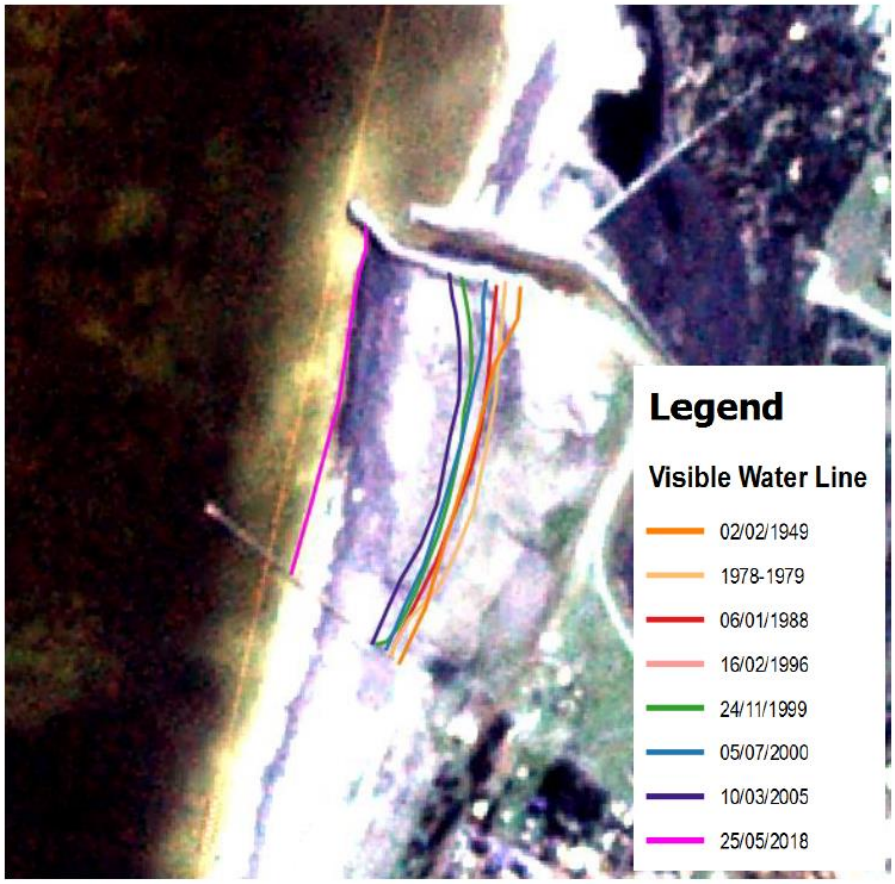


Figure 16. Visible water line from Tonkin (2018), aerial imagery courtesy of Planet Labs, Inc.

To assess the ongoing sedimentation at the south beach and to help determine the processes causing it, true colour satellite imagery from the Sentinel-2 satellite has been used. Satellite images of the Maria Creek region are shown from December 2015 to March 2020 in Figure 17 to Figure 24. The plots show the beach adjacent to Maria Creek and extend just under 1 km to the north and south of Maria Creek. The plots show the following:

- significant sedimentation occurred between August and December 2016 adjacent to the southern training wall; and
- since this time, ongoing sedimentation has occurred around Maria Creek, with the March 2020 image indicating that the sedimentation has extended beyond the offshore end of the southern training wall extension, across the entrance to Maria Creek and adjacent to the northern training wall.

Due to the spatial resolution of the satellite imagery (10 m), it is not possible to determine if any shoreline erosion occurred at the beaches to the south of Maria Creek over the period from August to December 2016, but based on aerial imagery it appears that in the order of 10 to 15 m of erosion occurred between December 2015 and October 2018 at Wyomi, which is approximately 4 km to the south of Maria Creek. It is likely that this sediment was eroded in 2016 and resulted in the release of a large volume of sediment which contributed to the high sedimentation which has occurred at Maria Creek.

Figure 17. Sentinel 2 true colour satellite image on Maria Creek on 24/12/2015.

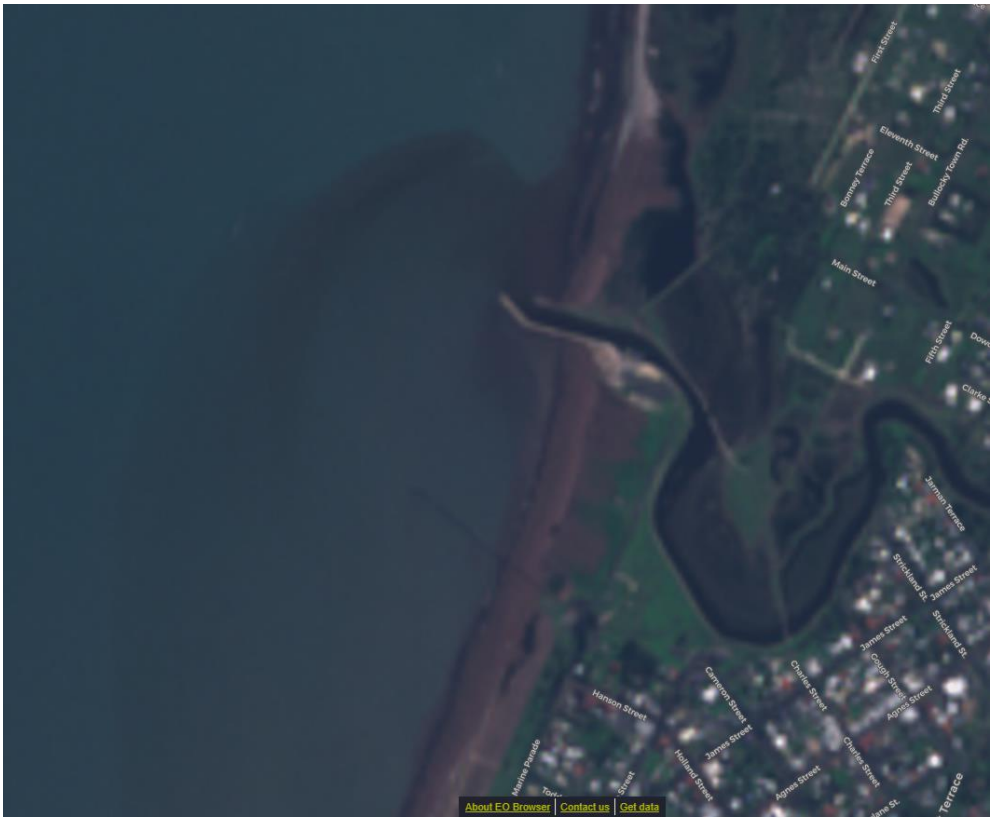


Figure 18. Sentinel 2 true colour satellite image on Maria Creek on 31/07/2016.

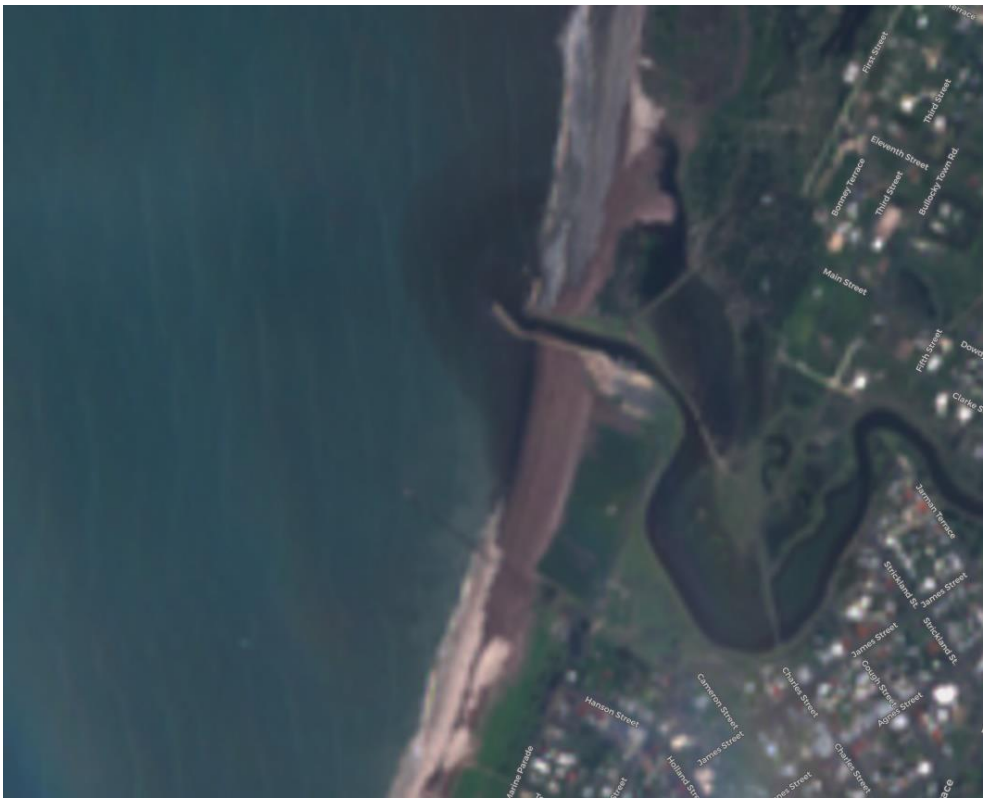


Figure 19. Sentinel 2 true colour satellite image on Maria Creek on 19/10/2016.

Figure 20. Sentinel 2 true colour satellite image on Maria Creek on 29/10/2016.

Figure 21. Sentinel 2 true colour satellite image on Maria Creek on 18/12/2016.

Note: there was no significant change between December 2016 and December 2017 and so the 2017 image has not been presented.

Figure 22. Sentinel 2 true colour satellite image on Maria Creek on 23/12/2018.

Figure 23. Sentinel 2 true colour satellite image on Maria Creek on 23/12/2019.

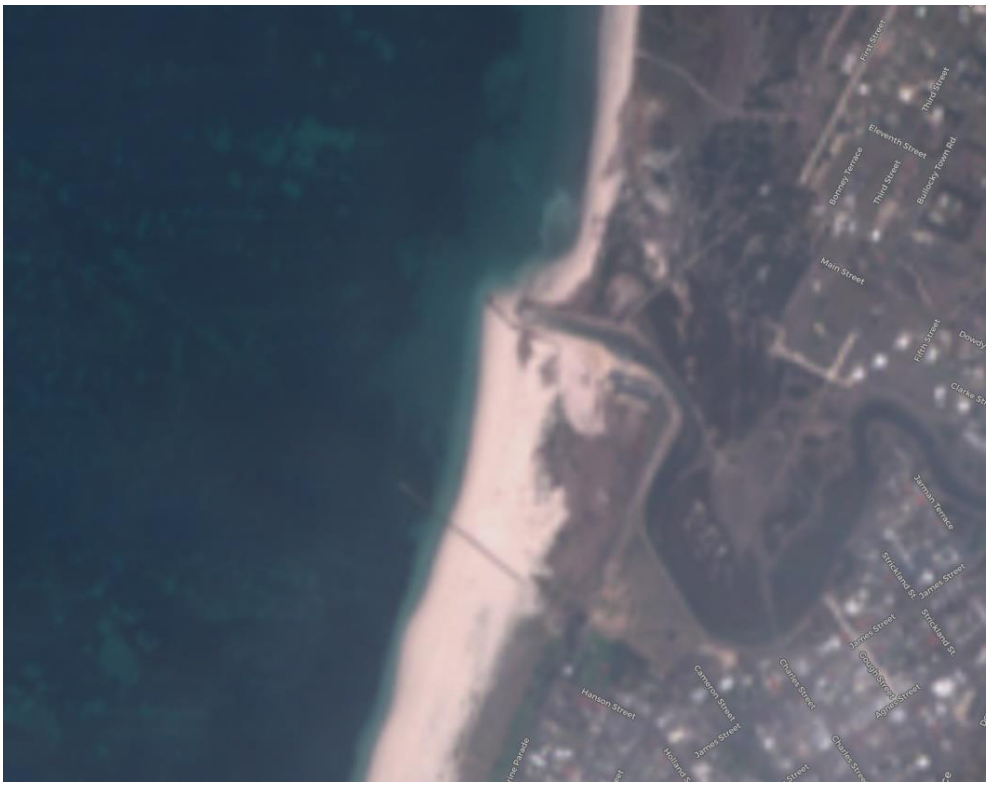


Figure 24. Sentinel 2 true colour satellite image on Maria Creek on 27/03/2020.

2.5.2. Beach profile analysis

A number of repeat beach profiles in the vicinity of Maria Creek were obtained from the Coast Protection Board (CPB, 2020). These repeat beach profiles (715004 and 715003, in particular) have been analysed by Tonkin (2018) and only a brief summary of their findings is included here. The location of the beach profiles in shown in Figure 25.

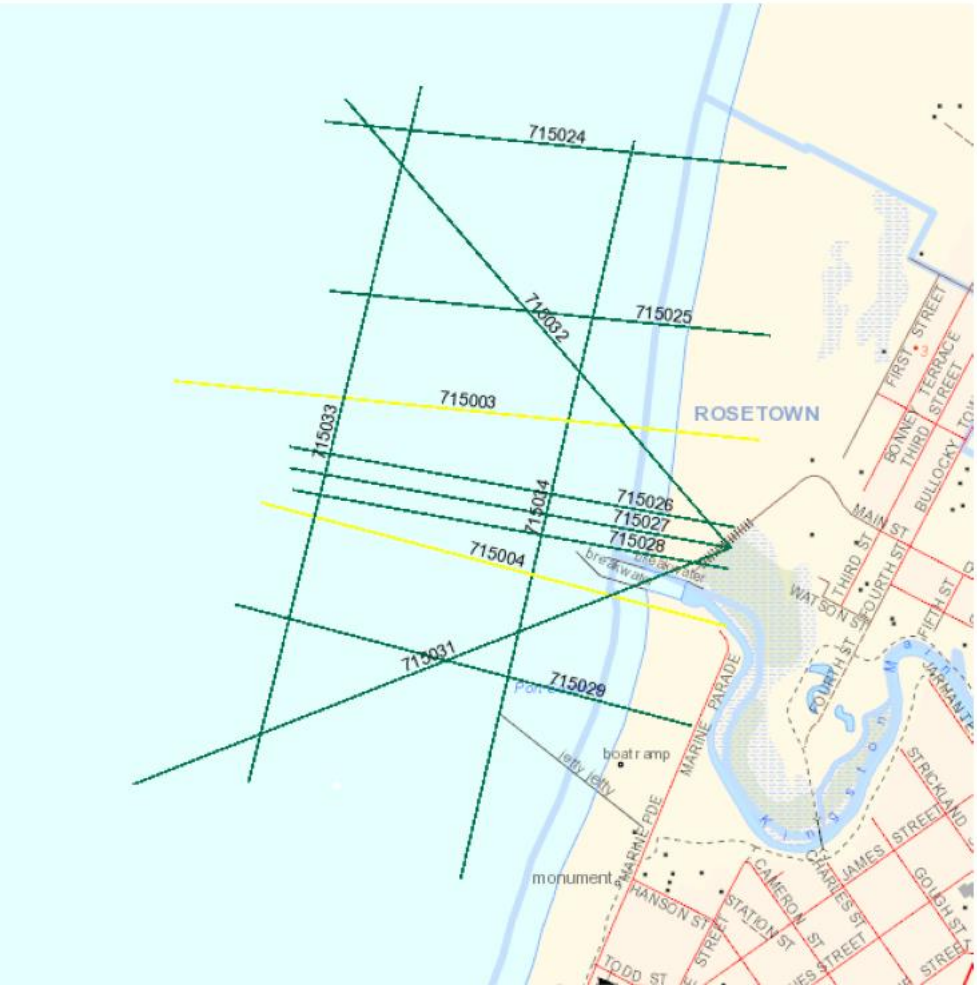


Figure 25. Location of beach profiles at Maria Creek (Source: CPB, 2020).

The key findings from Tonkin Consulting (2018) were as follows:

- the construction of the training wall structure has caused an overall accretion of sand to the south of the training wall;
- cyclical accretion and erosion of the coastline occurred between 2005 and 2012; and
- approximately 246,000 m^3 of material has accumulated in the section of coast between the jetty to the south of Maria Creek and the southern training wall between 1987 and 2018.

Up to 2016, a relatively stable shoreline location was maintained by sand bypassing, with most of the build-up of sediment on the south beach moved to the north beach by a volunteer operated dredge (B Smith 2020, pers. comm., 17 April). Based on this and the shoreline location analysis from the satellite imagery, it is apparent that most of the 246,000 m^3 of sedimentation noted to have occurred between 1987 and 2018 occurred post 2016, since which time there has been only limited management of sediment at Maria Creek.

At the southern end of Lacepede Bay, accretion of sand has also occurred to the leeward (western) side of similar breakwater structures at Cape Jaffa Anchorage, which was constructed in 2007/2008 (Magryn, 2020). The requirement for management of littoral sand drift along the beach was identified in the Cape Jaffa Environmental Impact Statement (PIRSA, 2005). While some sand carting operations have been undertaken in the years since construction (predominantly by trucking sand from west of the western breakwater (west beach) to east of the eastern breakwater (east beach)), sand has continued to build up on the west beach. The total accretion of sand on west beach (based on volumetric changes) and the volumes of sand removed by sand carting indicates a longshore transport rate of approximately 47,000 m^3/yr (Magryn, 2020).

2.5.3. Empirically derived longshore transport

The Cape Jaffa Marina Assessment of Coastal Processes and Impact report produced by WBM Oceanics Australia (WBM, 2005) investigated the rate of longshore sand transport based on the application of an empirical formula. Cape Jaffa is located approximately 20 km to the south-west of Maria Creek at the southern end of Lacepede Bay. The two sites have similar beach gradients and levels of exposure to waves. The longshore transport rates derived at Cape Jaffa are therefore expected to provide a reasonable indication of longshore transport at Maria Creek.

WBM (2005) used the CERC empirical longshore transport formula (CERC, 1984) to derive daily and annual longshore transport rates. The key findings from their report were:

- mean annual transport rates calculated for 2000 to 2002 were 15,000 m^3/year on average;
- upper annual transport rates calculated for 2000 to 2002 were 21,000 m^3/year on average;
- there was a clear seasonal trend to the transport, with highest rates occurring during the winter to spring months (May to October);
- daily transport rates were typically less than 200 m^3/day, but can exceed 500 m^3/day;
- swell waves consistently caused transport up the coast in a net northerly-easterly direction; and
- locally generated ‘sea’ waves led to down coast transport to the south-west from time to time.

3. Modelling Approach

To better understand the processes which have resulted in sedimentation at Maria Creek, in addition to analysing and processing measured data, numerical modelling has also been used. This section provides details of the numerical modelling software and modelling tools developed for this study.

3.1. Software

The MIKE software suite has been adopted for the assessment. The MIKE software has been developed by the Danish Hydraulics Institute (DHI) and is internationally recognised as state-of-the-art and has been adopted elsewhere in Australia and internationally for similar projects. The MIKE suite includes:

- the hydrodynamic (HD) module which has been applied to simulate the local tidal and wind driven flows in and around the study area with the aim of better understanding the local flow regime (i.e. tide and wind driven flows) and its effect on sediment transport; and
- the spectral wave (SW) module which has been applied to transform 20 years of measured wave conditions from the Cape de Couedic WRB to the study area, so that the local wave climate and its effect on longshore transport can be better understood.

The MIKE modules allow a flexible mesh (FM) to be adopted which enables the spatial resolution of the model mesh to be varied throughout the model domain. Adopting this allows suitable model mesh resolutions to be adopted throughout, ensuring the model accuracy and efficiency can be balanced. This means that areas of interest can have a higher mesh resolution while a lower mesh resolution can be adopted in offshore areas and areas away from any areas of interest. This is particularly relevant to hindcast simulations which span several decades.

3.2. Model Configuration

Details of the mesh, bathymetry and boundary conditions adopted for the HD and SW models are provided in the following sections. Both models have also been calibrated against available measured data, the model calibration and validation is detailed in Section 4.

3.2.1. Model Mesh

The HD and SW modules have been applied with two different meshes due to differences in the aims of the models. Details of the meshes are provided in the subsequent sections.

3.2.1.1. Hydrodynamic Model

The HD mesh with the interpolated bathymetry is shown in Figure 26, which also includes a zoom into the Maria Creek region. The unstructured mesh enables the resolution to be varied within the domain, with higher resolution applied around Kingston and Maria Creek. The varying HD mesh resolution is shown in Figure 27, where the resolution is defined as the approximate length of one side of the triangular mesh element in each area.

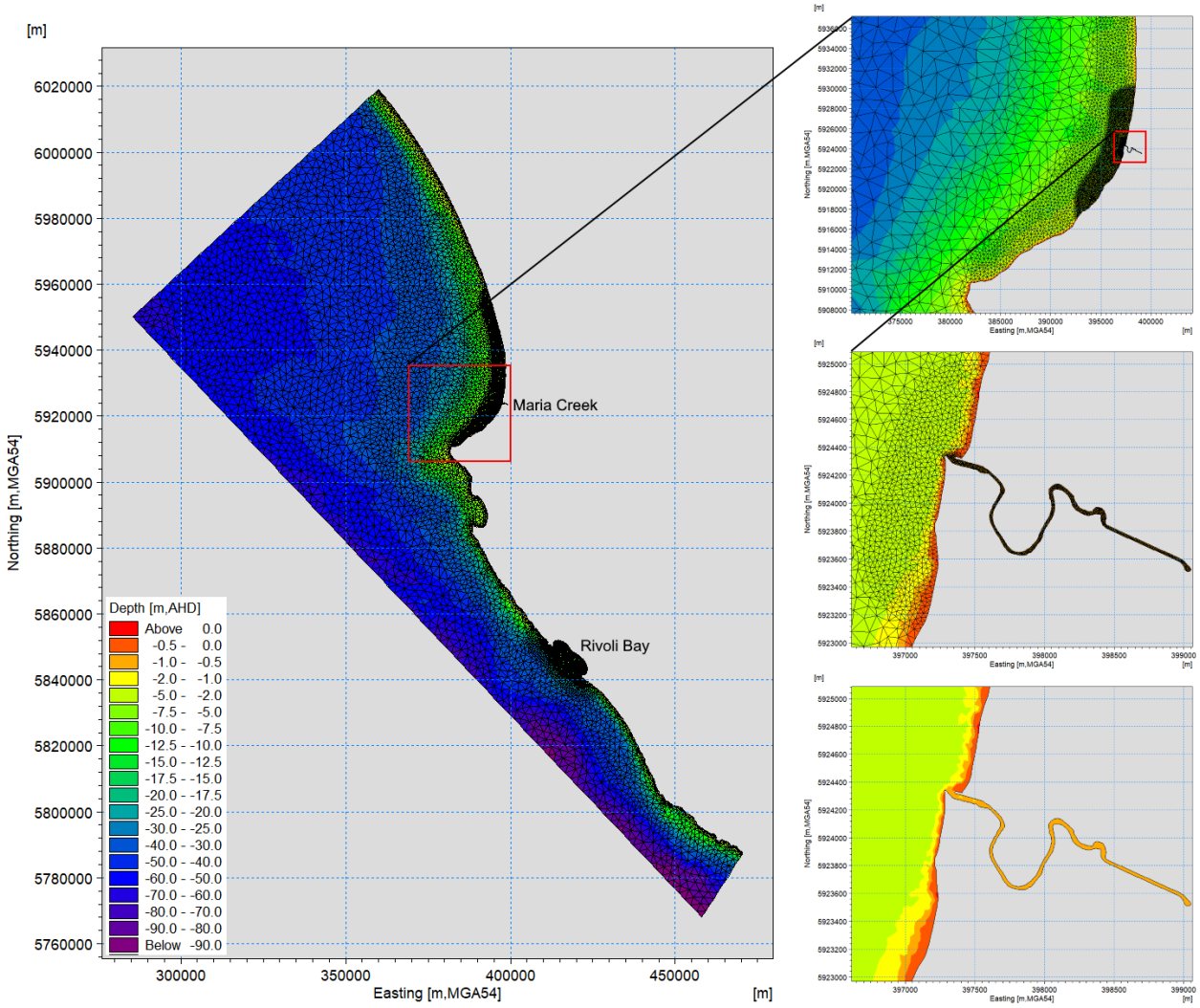


Figure 26. HD model mesh.

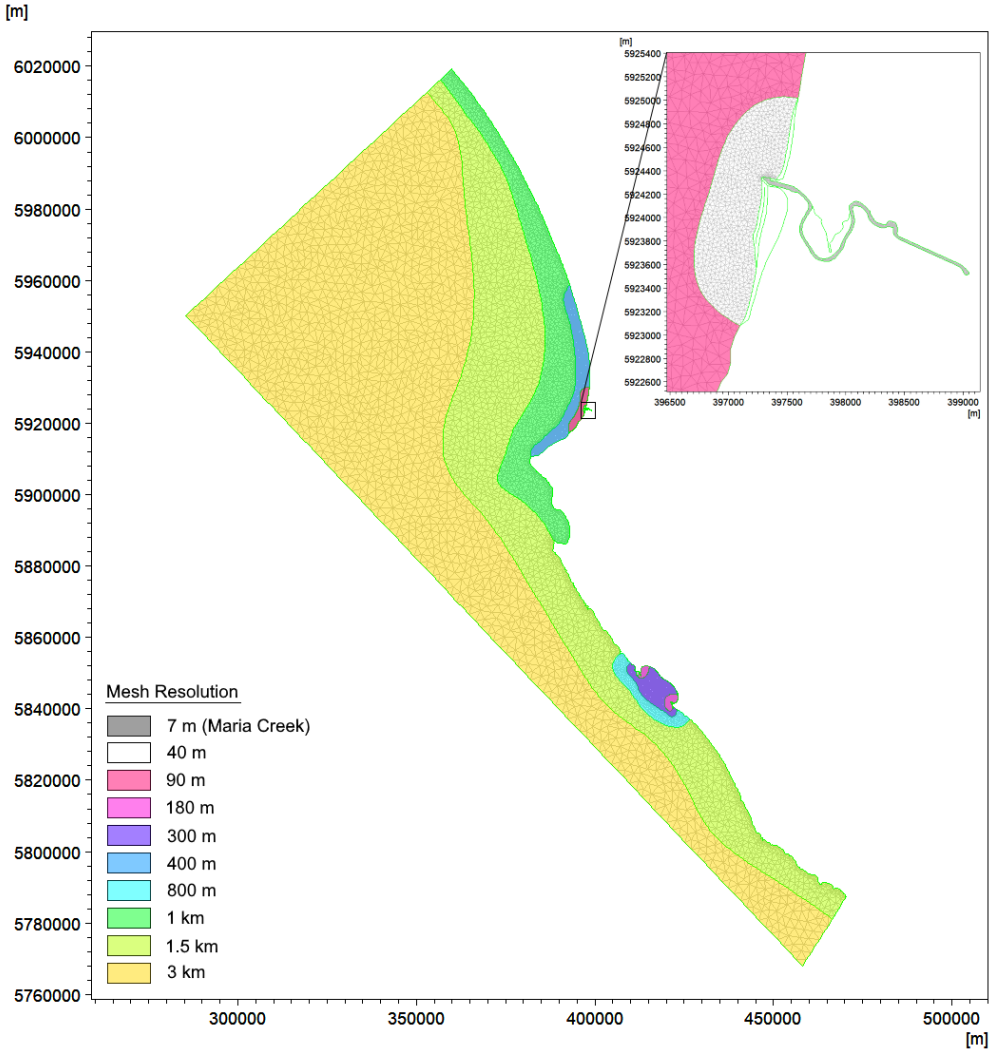


Figure 27. HD model mesh resolution.

3.2.1.2. Spectral Wave Model

The SW mesh is shown in Figure 28. The mesh is shown at both regional scale (showing the full mesh extent) and local scale (zoomed into the Maria Creek region) to show the mesh in more detail. The colour scale shows the interpolated bathymetry. The unstructured mesh enables the resolution to be varied within the domain, with higher resolution (in the order of 50 m or less) applied around Kingston and Maria Creek. The SW mesh does not include the channel of Maria Creek as the wave conditions within the Creek will be very calm and therefore resolving the Creek channel would increase the model run time without providing any additional information.

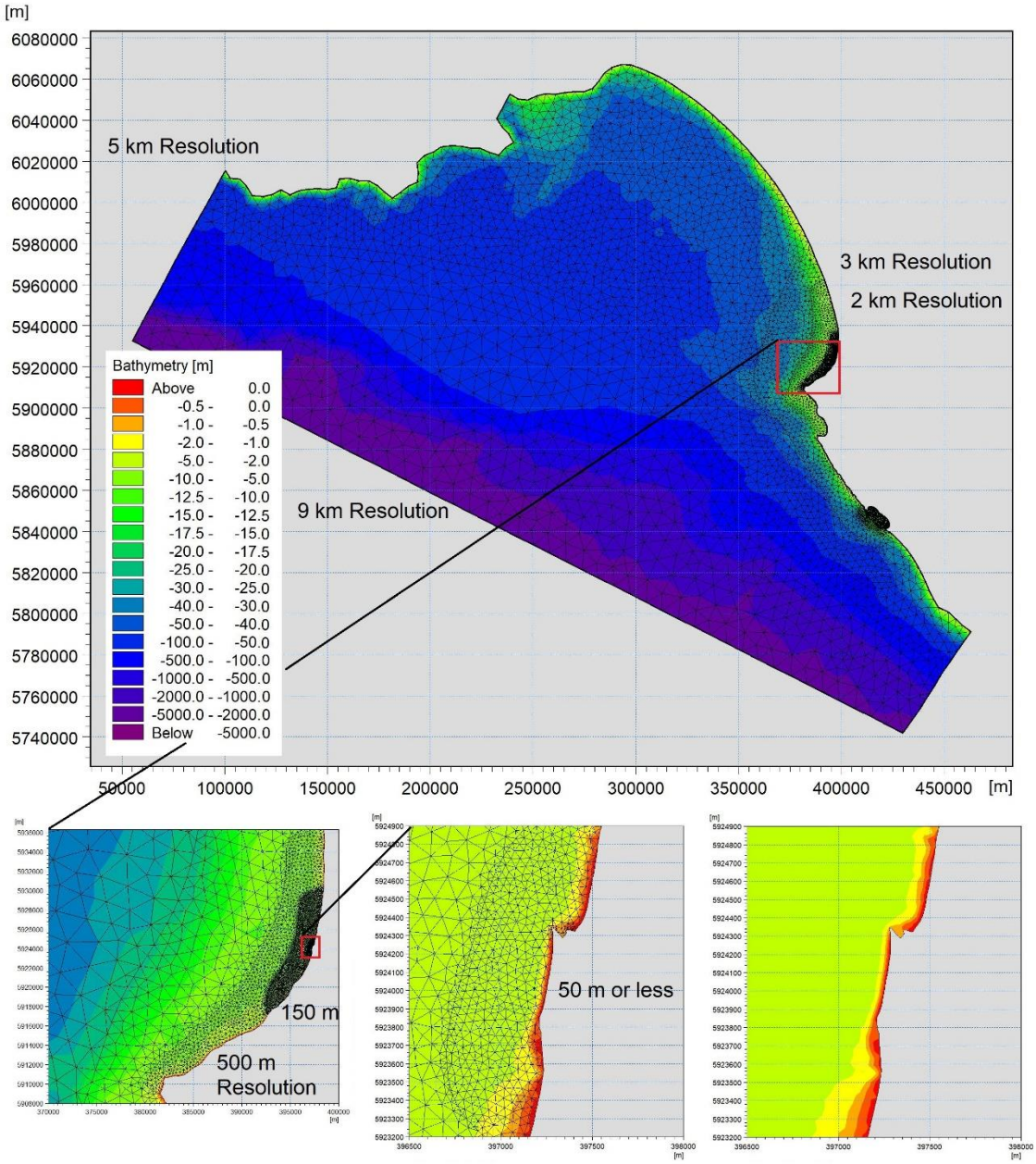


Figure 28. SW model mesh.

3.2.2. Bathymetry

The model bathymetry used in both the HD and SW models is comprised of four key datasets:

1. Bathymetric sounding data (PHS, 2020a)

Processed bathymetric sounding data from the hydrographic survey of Maria Creek undertaken in January 2020 by Precision Hydrographic Services (PHS-20-003-WLC 200130_GDA94_1m_Mean_GRID_Soundings_mAHD.xyz) were used as the primary dataset for the Maria Creek region. This dataset provides the most recent hydrographic survey data of the area at the highest resolution available (Figure 29).

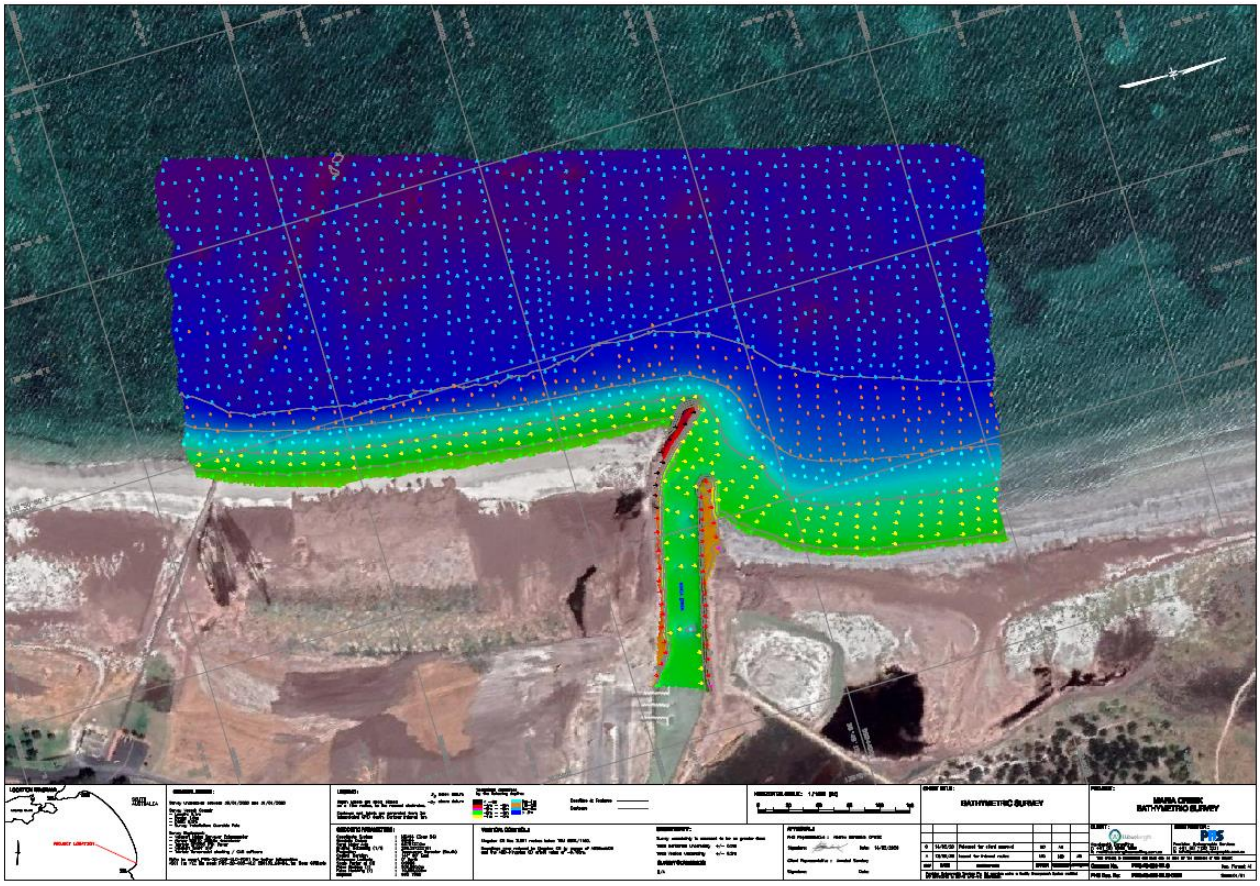


Figure 29. Maria Creek bathymetric survey extent (PHS, 2020b).

2. Beach profile data (CPB, 2020)

Beach profile data along the Kingston frontage were obtained from the Coastal Protection Board, as discussed in Section 2.5.2 and shown in Figure 25. The most recent beach profile data were used to fill data gaps outside of the PHS bathymetric survey extent along the Kingston frontage and extending offshore.

3. Digitised contour data from Australian Hydrographic Charts (AHO, 2001)

Bathymetric contour data from the following Australian Hydrographic Office Charts were manually digitised to supplement the beach profile data:

- AUS 127 – Plans in South East Coast;
- AUS 127, Panel 2 – Rivoli Bay;
- AUS 127, Panel 4 – Beachport; and
- AUS 127, Panel 5 – Kingston SE.

4. 2009 Bathymetric Grid of Australia data (Geoscience Australia, 2009)

Data from the 2009 bathymetric grid of Australia dataset were used to supplement data gaps further offshore within the model domains. The dataset comprises bathymetric data gridded at a resolution of 0.0025 decimal degrees and constitutes the coarsest resolution dataset of the four listed above.

Data from each of these datasets were interpolated onto the HD and SW model meshes using the above list order as the priority preference.

3.2.3. Boundary Conditions

Details of the boundary conditions adopted to drive the SW and HD models are provided in the following sections.

3.2.3.1. Hydrodynamic Model

The HD model has three open tidal boundaries, namely north, offshore and south as shown in Figure 30. The boundary conditions were generated using tidal constituents from the Global Tidal Model developed by DTU Space (DHI, 2007). Tidal constituents were extracted along the three model boundaries from the 0.125° x 0.125° version of the Global Tide Model, which includes the following 10 major constituents: M2, S2, K2, N2 (semidiurnal), S1, K1, O1, P1, Q1 (diurnal) and M4 (shallow water).

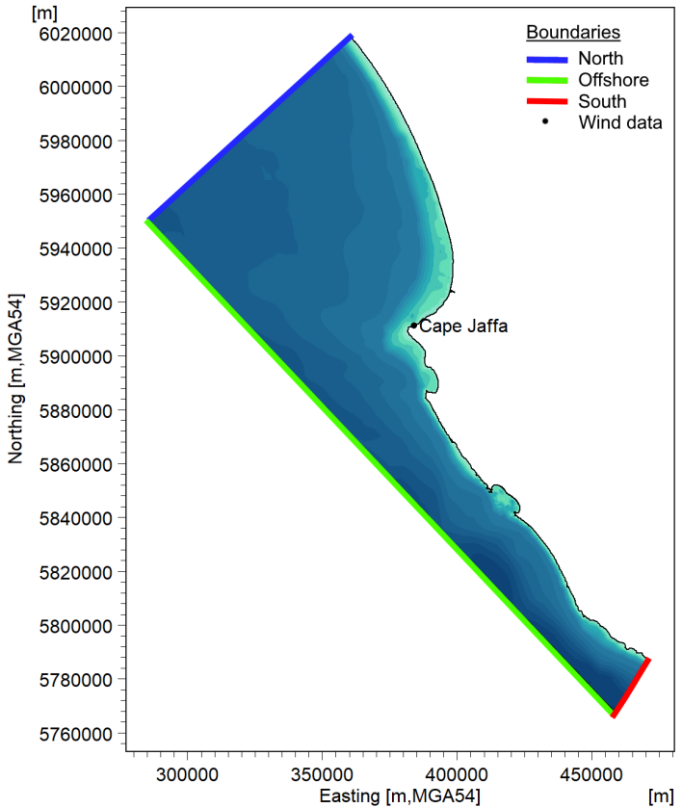


Figure 30. HD model tidal boundaries and wind data source.

A spatially uniform wind was applied in the HD model domain. The wind conditions applied were taken from the measured wind speed and direction at Cape Jaffa (location shown in Figure 30).

3.2.3.2. Spectral Wave Model

The boundary conditions for the SW model were derived from the measured wave data from the Cape de Couedic WRB. As noted in Section 2.2, prior to June 2019 a non-directional WRB was deployed, and so for this period the wave directions were derived from WaveWatchIII. Wave conditions from WaveWatchIII were also used to fill any gaps in the WRB record (approximately 7% of the time).

As with the HD model the SW model applied spatially uniform, time varying winds based on the measured data at Cape Jaffa. In addition, a spatially uniform, time varying water level (based on predicted water levels from Victor Harbour) was also applied in the SW model.

4. Model Calibration and Validation

Model calibration is the process of specifying model parameters so that the model reproduces observed data to a suitable level of accuracy. Model validation is used to confirm that the calibrated model continues to consistently represent the natural processes to the required level of accuracy, in periods other than the calibration period, without any additional adjustment to the model parameters. The calibration and validation processes provide confidence in the model results and are essential to ensure the accurate representation of hydrodynamics and waves.

This section provides details of the calibration and validation undertaken for the hydrodynamic and spectral wave models adopted as part of this assessment.

4.1. Calibration and Validation Standards

For quality control in the hydrodynamic model calibration, performance criteria have been defined to demonstrate that the model is capable of accurately representing the natural processes. Different standards are applicable for coastal and estuarine waters, for this study the more stringent coastal performance criteria have been adopted as follows:

- Modelled water levels (WL) should be within 10 to 15% of the tidal range over a spring neap tidal cycle, or within ± 0.1 m;
- Timing of high water (HW) and low water (LW) should be within 15 minutes;
- Root Mean Square (RMS) difference should be within ± 0.1 to 0.3 m; and
- Phasing should be within 15 minutes.

These standards provide a good basis for assessing model performance, but experience has shown that sometimes they can be too prescriptive and it is also necessary for visual checks to be undertaken. Under certain conditions, models can meet statistical calibration standards but appear to perform poorly. Conversely, seemingly accurate models can fall short of the guidelines. Consequently, a combination of both statistical calibration standards and visual checks has been used to ensure that the model is representative.

For the HD calibration and validation we have undertaken a quantitative comparison between predicted and modelled water level amplitudes and phasing, while a qualitative comparison of current speeds and directions has been made.

For the calibration and validation of the wave model we have undertaken a qualitative comparison between measured and modelled wave height, wave period and wave direction. In addition, a comparison of wave percentiles and an assessment of the correlation between modelled and measured wave heights has also been provided.

4.2. Hydrodynamic Model

To ensure that the hydrodynamic model is able to represent the natural conditions within Maria Creek and the wider model domain, predicted and measured water levels and measured flows (speed and direction) have been compared against modelled conditions.

4.2.1. Data Availability

For the purpose of this study measured Acoustic Doppler Current Profiler (ADCP) data, comprising of water levels, current speeds and current directions, were made available for two sites approximately 80 km south of the Maria Creek study area, namely Beachport and Southend (Figure 31).

Water level data were collected for a duration of five months at Cape Jaffa, these data were subsequently analysed to determine tidal constituents to enable water levels at Cape Jaffa to

be predicted for the HD model calibration and validation periods. The harmonics with longer phases were taken from Victor Harbour (this is discussed further in Section 4.2.2).

No measured water level or current data were available along the Kingston frontage or within Maria Creek and so no local calibration was possible.

To supplement the lack of water level data within the study area, water levels were predicted at Kingston and Robe using SeaFarer Tides (AHO, 2010), which is the digital equivalent of the Australian National Tide Tables publication.

The locations of available data (referred to as calibration data) are shown in Figure 31.

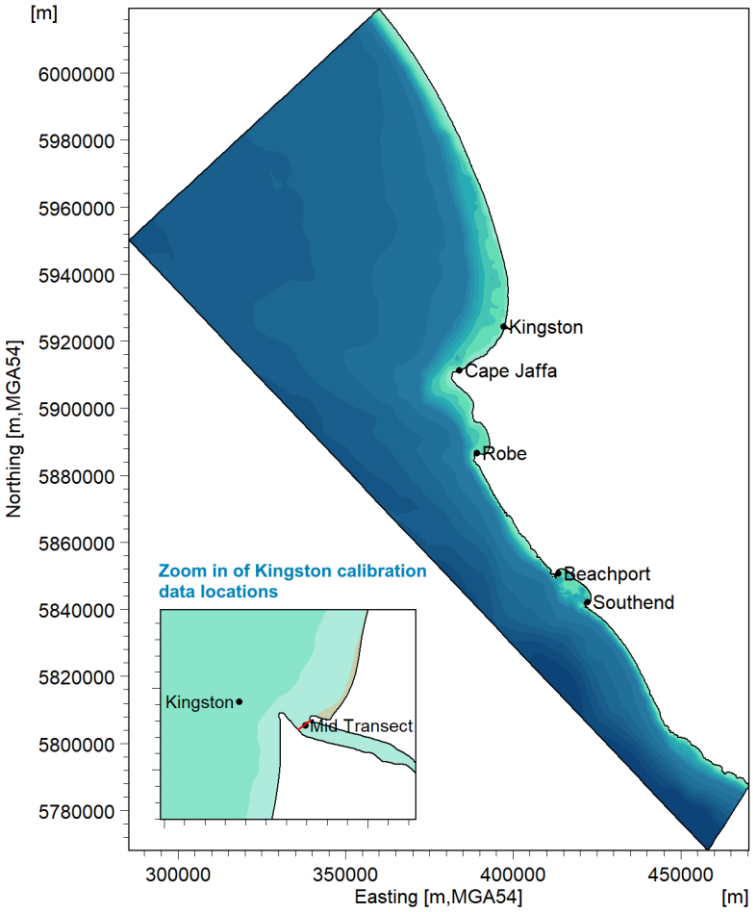


Figure 31. HD model calibration data locations.

4.2.2. Data Limitations

While it is beneficial having data available to assist with calibrating the HD model, there are some limitations associated with the datasets that were available for this study including:

- the proximity of Beachport and Southend to Maria Creek: while it is beneficial having some measured data to calibrate/validate the HD model to, the hydrodynamic regime at these sites differs to the study area. The more enclosed nature of the embayment and more complex bathymetry with numerous rocky outcrops and headlands creates greater complexity from a modelling perspective, especially given that the bathymetric data in the vicinity is not very detailed; and
- predicted water level datasets: there is uncertainty associated with the duration of the measured datasets which have been used to subsequently determine the harmonic constituents and then predict water levels. For example, at Cape Jaffa the measured dataset was only 5 months, requiring harmonics with longer phases to be taken from

Victor Harbour which has a much longer measured dataset. There is also some uncertainty associated with the predicted water levels at Kingston, Robe and Beachport as the duration of the measured data at these stations are unknown. This uncertainty has implications for the calculation of mean sea level (MSL) at each station in addition to characteristics of the tidal signal which are dependent upon longer phase harmonic constituents.

4.2.3. HD Calibration and Validation Periods

Based on the available data, modelled water levels and flows have been compared against the measured calibration data for the following periods:

- Calibration Period: a 14 day spring-neap cycle between 25/07/2019 00:00 and 08/08/2019 00:00, when wind conditions were gentle (0 to 8.7 m/s, average 3.8 m/s); and
- Validation Period: a 14 day spring-neap cycle between 10/07/2019 00:00 and 24/07/2019 00:00, when stronger winds prevailed (0 to 17 m/s, average 8.7 m/s).

Wind data recorded at Cape Jaffa during the model calibration and validation periods are shown in Figure 32 and Figure 33 to assist with interpreting the model results.

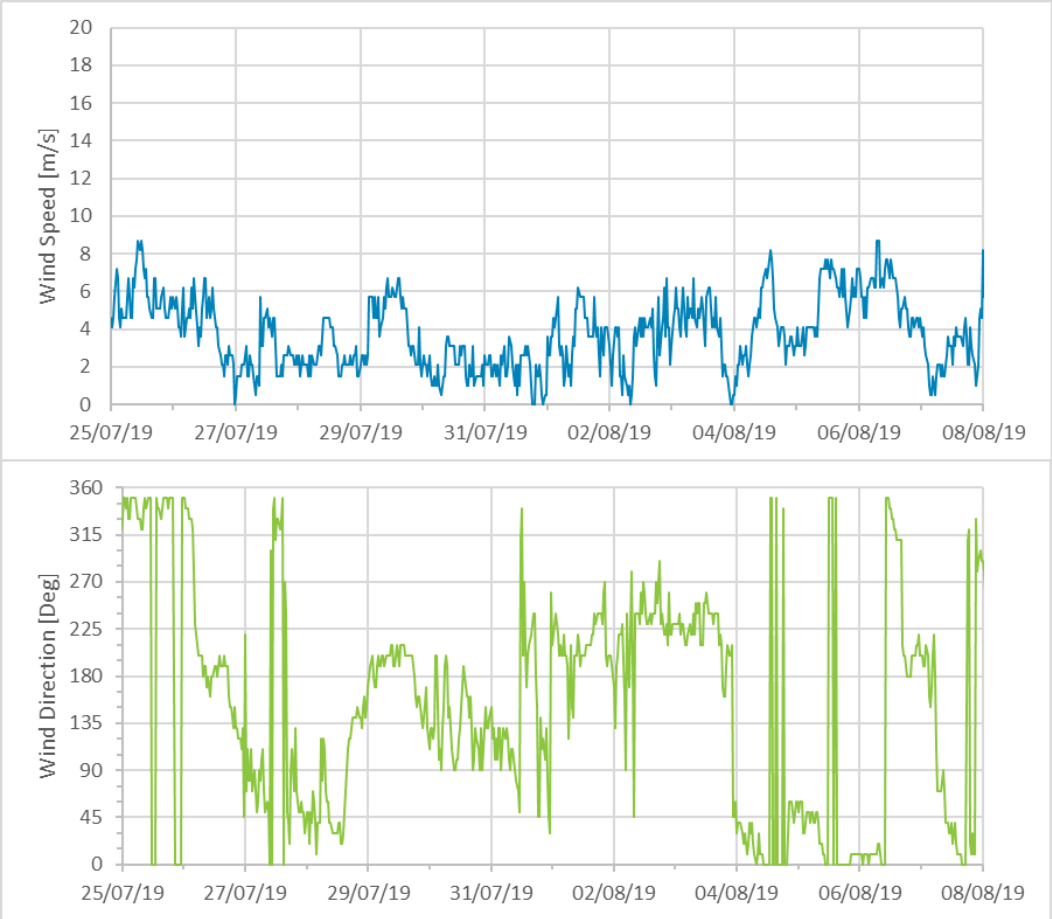


Figure 32. Wind conditions recorded at Cape Jaffa over the calibration period.

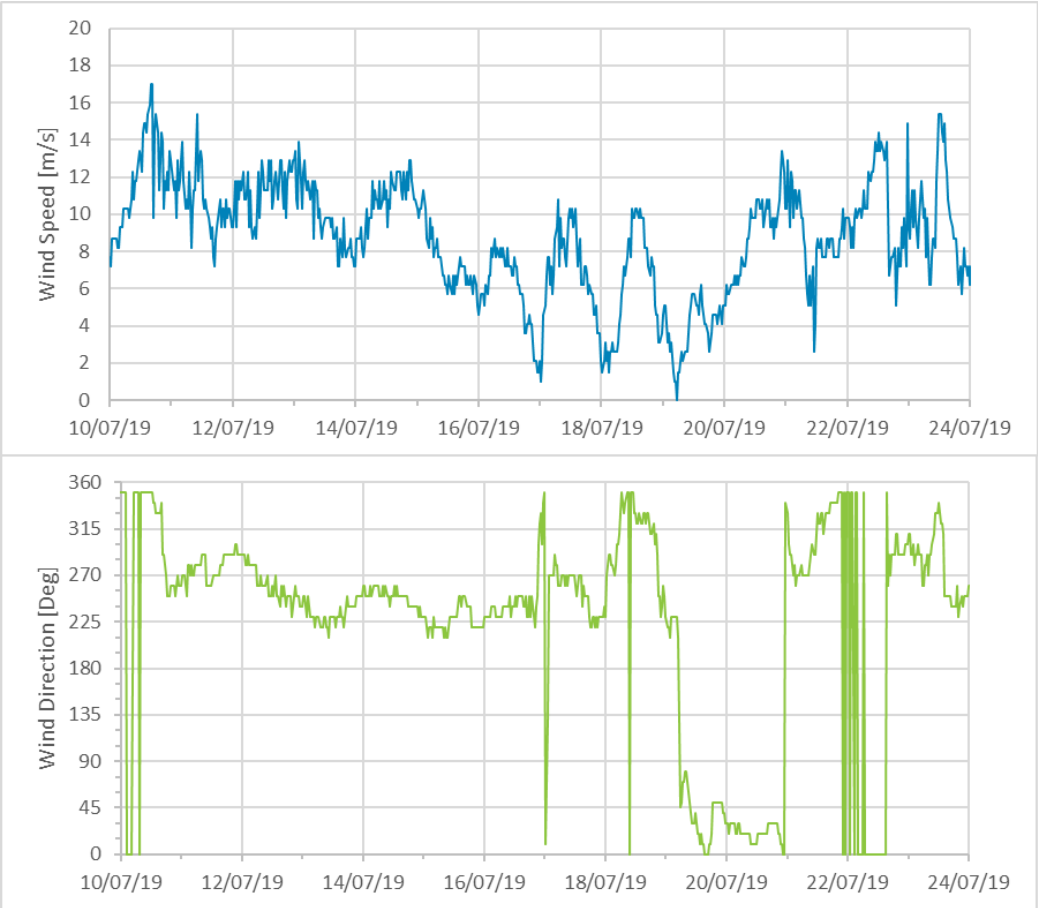


Figure 33. Wind conditions recorded at Cape Jaffa over the validation period.

4.2.4. Results

The HD model has been compared to predicted water levels at Kingston, Robe and Cape Jaffa, and measured water levels, current speeds and current directions at Beachport and Southend for the calibration and validation periods.

All results are presented as timeseries plots, showing modelled and measured/predicted data. For predicted water levels, statistics have been calculated for the criteria defined in Section 4.1. In addition to timeseries plots, currents are also presented as maps to show the spatial variation in speeds and directions through the tidal cycle and with varying wind conditions.

Timeseries plots comparing modelled and predicted water levels are shown in Figure 34 for the calibration and validation periods. The plots demonstrate the complexity of the tidal signature in the region with alternating periods of semi-diurnal (two per day) and diurnal (one per day) tides, with the former typically occurring during neap tides and the latter occurring during spring tides. The time series plots demonstrate that overall, the model replicates the predicted/measured variations in water levels that occur in the region, capturing the magnitude and timing of peak (HW and LW) levels as well as the complex shape of the tidal curve. There are times within the tidal cycle at each station where the modelled water level is consistently above or below the predicted water level. It is likely that this anomaly can partly be attributed to limitations in the predicted water level data mentioned previously in Section 4.2.2. At Beachport the measured water levels show periods with storm surge which the model has not been setup to accurately represent. As a result, the modelled water levels at Beachport agree well with the predicted data but not so well with the measured data due to the storm surges.

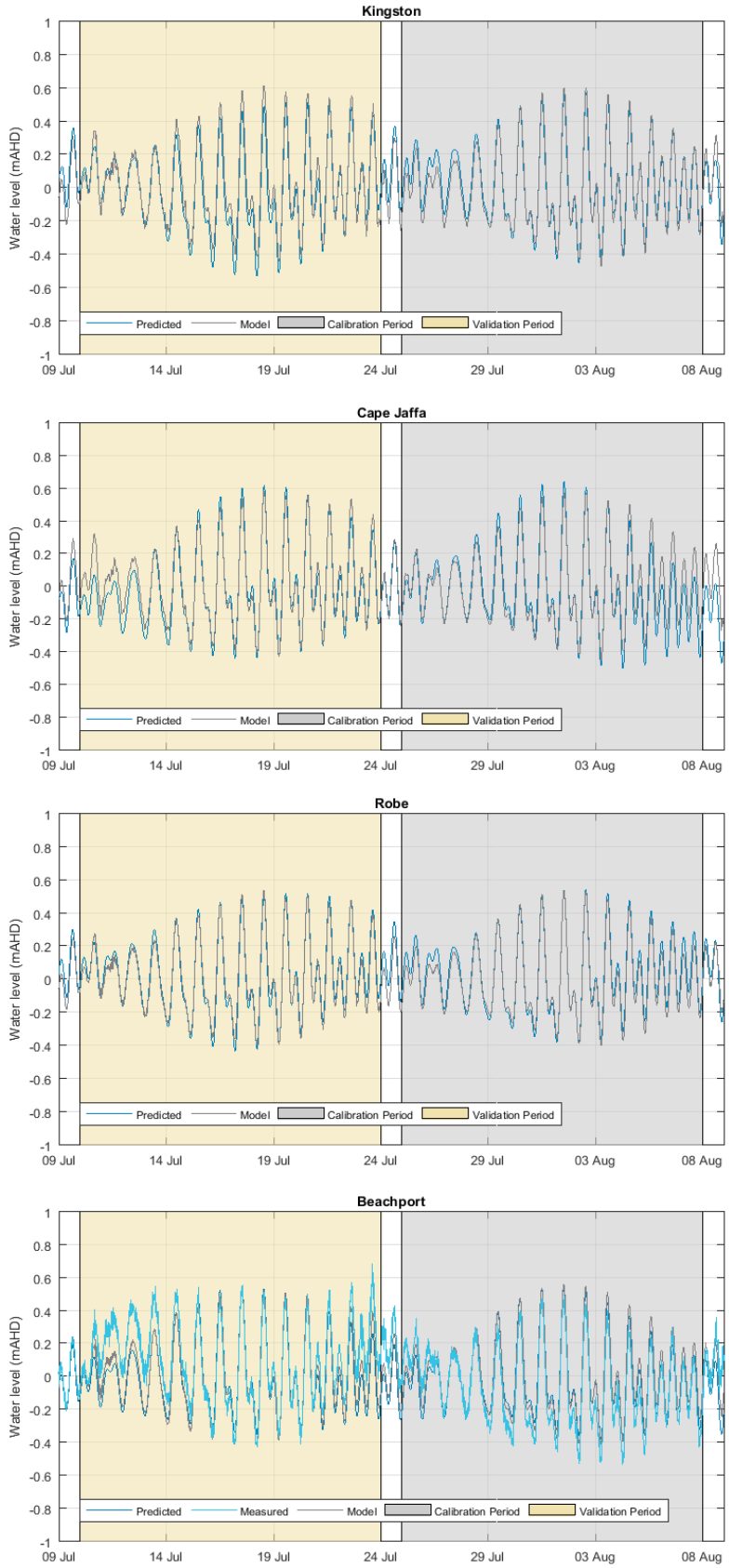


Figure 34. Comparison of modelled and measured/predicted water levels for the calibration and validation periods.

To further assess the level of calibration achieved, a statistical analysis was undertaken to quantify the difference in elevation and timing between the modelled and predicted water levels. The results of the statistical analysis are presented in Table 5.

Table 5. Statistics for comparison of modelled and measured water levels during the calibration and validation periods.

Site	WL difference (m)			WL difference (%)		Phase difference (minutes)		
	HW	LW	RMS	HW	LW	HW	LW	All
Calibration period								
Kingston	-0.01	-0.02	0.05	-2	-4	7	14	11
Cape Jaffa	0.03	0.03	0.08	6	6	-4	8	9
Robe	-0.03	-0.03	0.05	-7	-7	5	9	4
Beachport	0.04	0.01	0.04	9	2	7	-1	4
Validation period								
Kingston	0.06	0.02	0.07	12	4	1	6	6
Cape Jaffa	0.02	0.03	0.07	4	6	-3	1	0
Robe	-0.01	0	0.04	-2	0	-8	-1	-3
Beachport	0.03	0	0.05	7	0	-5	-2	-1

Notes: Differences are modelled minus predicted so that positive values indicate that the model value is high/late relative to predicted.

The guideline standards are achieved for all statistics at all four sites during both the calibration and validation periods.

In addition to water levels, the model calibration and validation also compared current speeds and directions within Rivoli Bay against ADCP data collected at Beachport and Southend. Timeseries plots comparing measured water level, current speed and current direction for the calibration and validation periods are shown for Beachport in Figure 35 and Southend in Figure 36. Current speeds at the two sites during both the calibration and validation periods are consistently low, around 0.05 – 0.1 m/s. The model is able to replicate the low current speeds, but at times finds it challenging to reproduce the variation in current speeds throughout the simulation periods. At times both the current speeds and directions differ to the measured data. It is most likely that these differences can be attributed to the lack of detailed bathymetric data available within Rivoli Bay. The bathymetry within the area, particularly around Beachport and Southend is complex, with numerous shallow rocky outcrops within the deeper areas (Figure 37). Whilst aerial photography was used to guide where these features are and supplement the bathymetric dataset, it is likely that additional data would be required to more accurately replicate the complex bathymetry and eddies in Rivoli Bay.

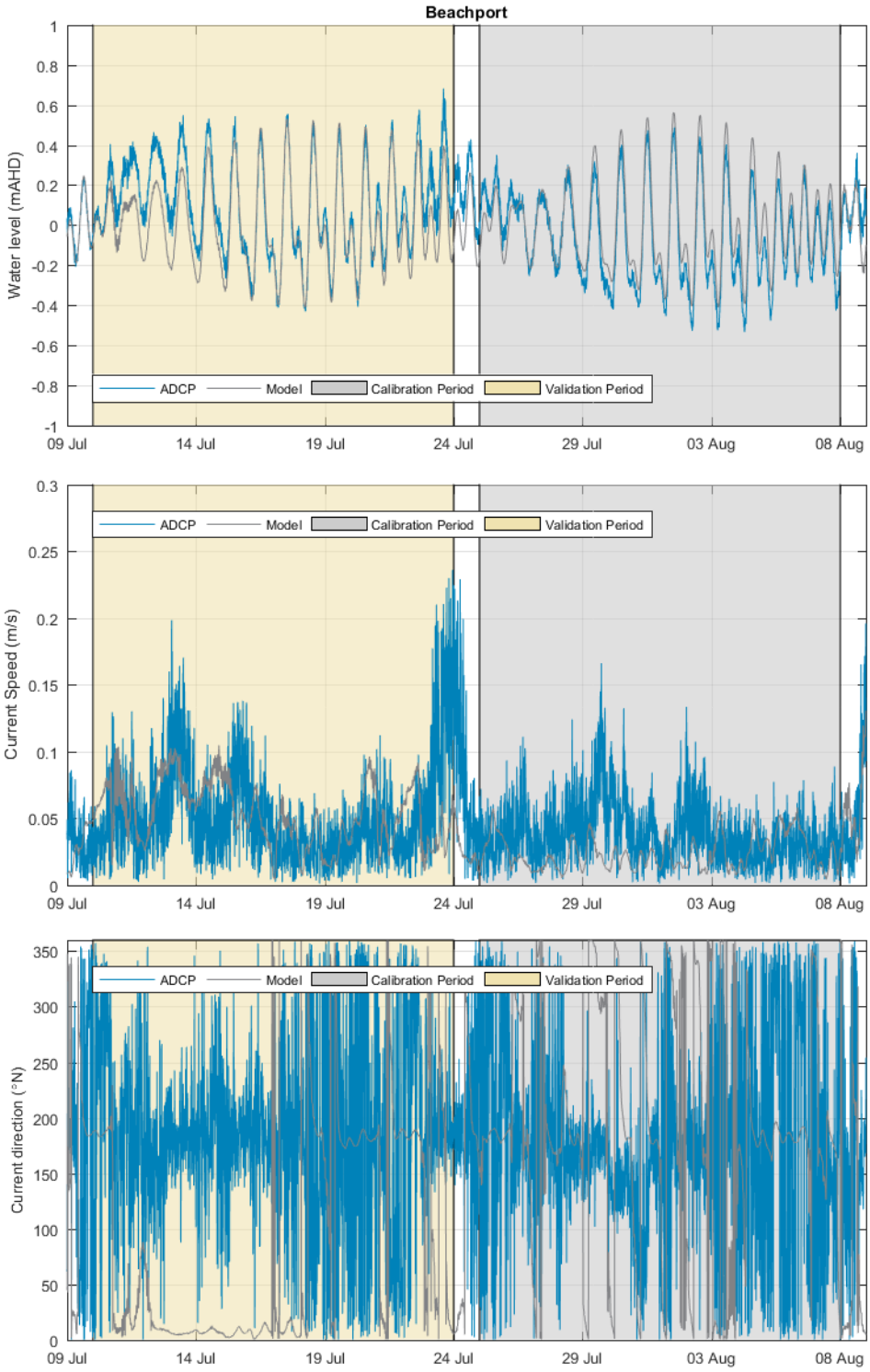


Figure 35. Comparison of measured and modelled water levels and currents at Beachport for the calibration and validation periods.

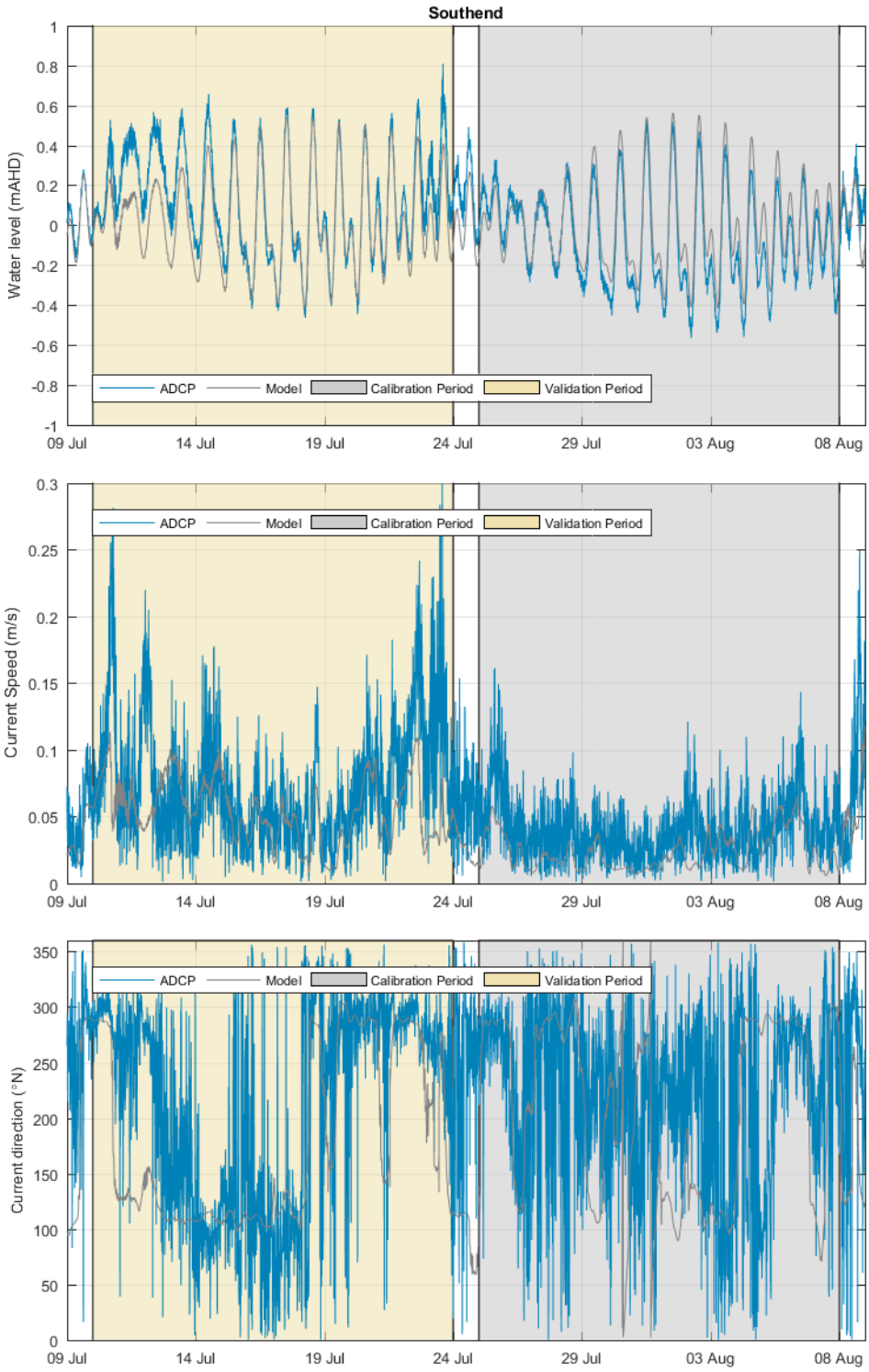


Figure 36. Comparison of measured and modelled water levels and currents at Southend for the calibration and validation periods.

Source: CPB (2020).

Figure 37. Aerial imagery of Rivoli Bay.

To provide additional confidence in the model's ability to replicate the flow patterns in Rivoli Bay, scatter plots comparing modelled and measured current vectors at Beachport and Southend for both the calibration and validation periods are shown in Figure 38. The plots show that the model is able to approximate the distribution of currents (northwards and eastwards flow directions), but as would be expected the degree of scatter within the modelled data is less than the measured data.

To show further detail of the complex flow patterns within Rivoli Bay a selection of spatial plots with vectors are shown at various stages through the tidal cycle in Figure 39 and Figure 40. Eddies are a regular feature around the Beachport and Southend ADCP sites and vary through the tidal cycle.

Variations in wind conditions also influence the behaviour of the eddy structures, as shown in Figure 41, which compares currents at peak flood for a spring tide with low winds, high winds from the west and high winds from the north. High winds from the north can dramatically reduce peak flood current speeds, while high winds from the west can increase them.

It is possible that the agreement between modelled and measured data within Rivoli Bay could be improved by extracting the modelled results from a neighbouring mesh element, however, this was considered unnecessary given the uncertainty associated with the bathymetry in the area. It is considered that the level of calibration achieved within Rivoli Bay is sufficient to give confidence that the model is able to replicate tidal propagation and current directions and magnitudes in the region, especially given that the model will subsequently be used for the Kingston/Maria Creek area.

The statistics combined with the timeseries and spatial plots show the model provides a good representation of the tide at Cape Jaffa, Robe and Beachport, providing confidence that the modelled tides in the Kingston and Maria Creek region are suitably replicated.

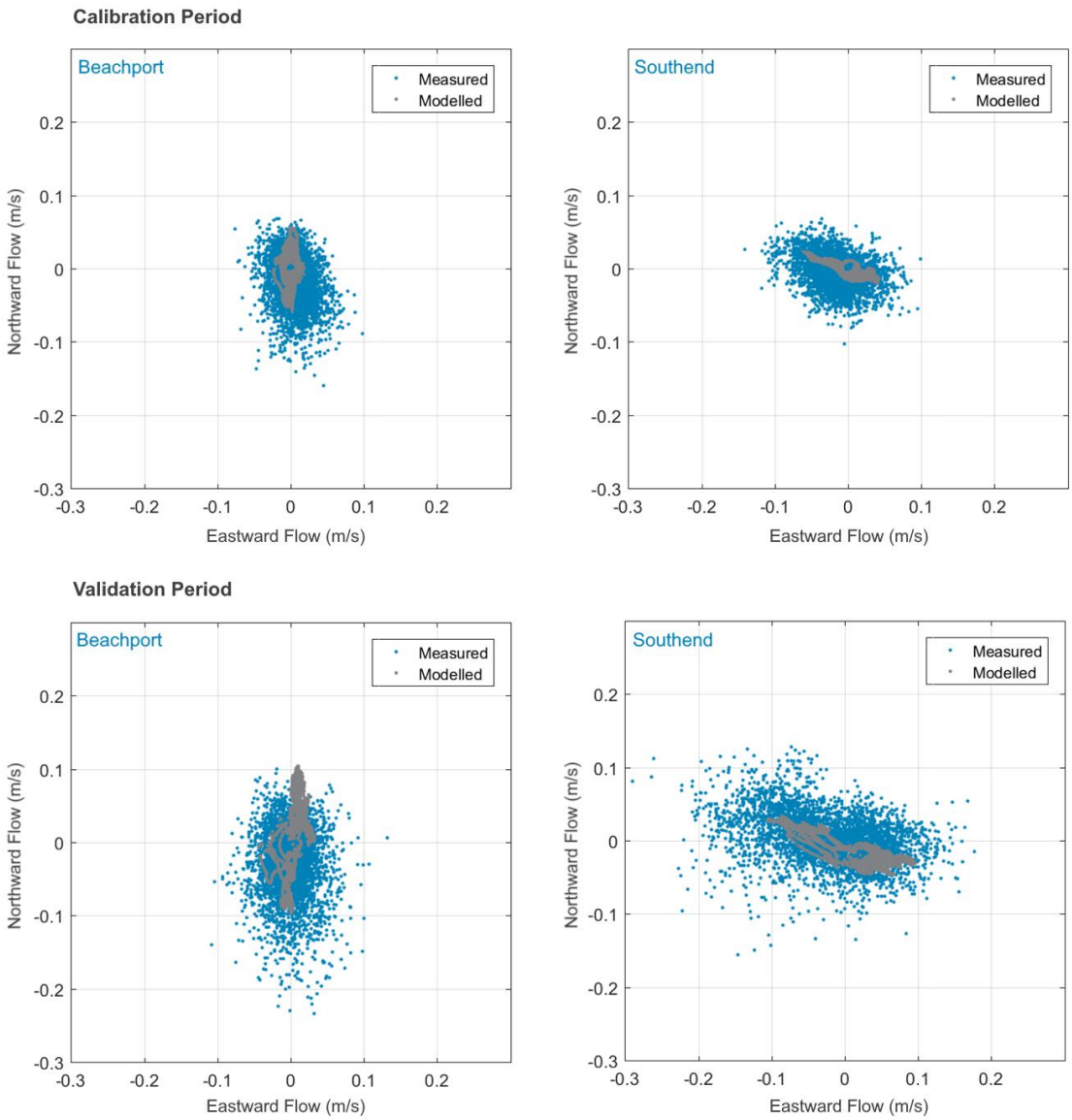


Figure 38. Comparison of measured (ADCP) and modelled current at Beachport and Southend for the calibration (top plots) and validation (bottom plots) period.

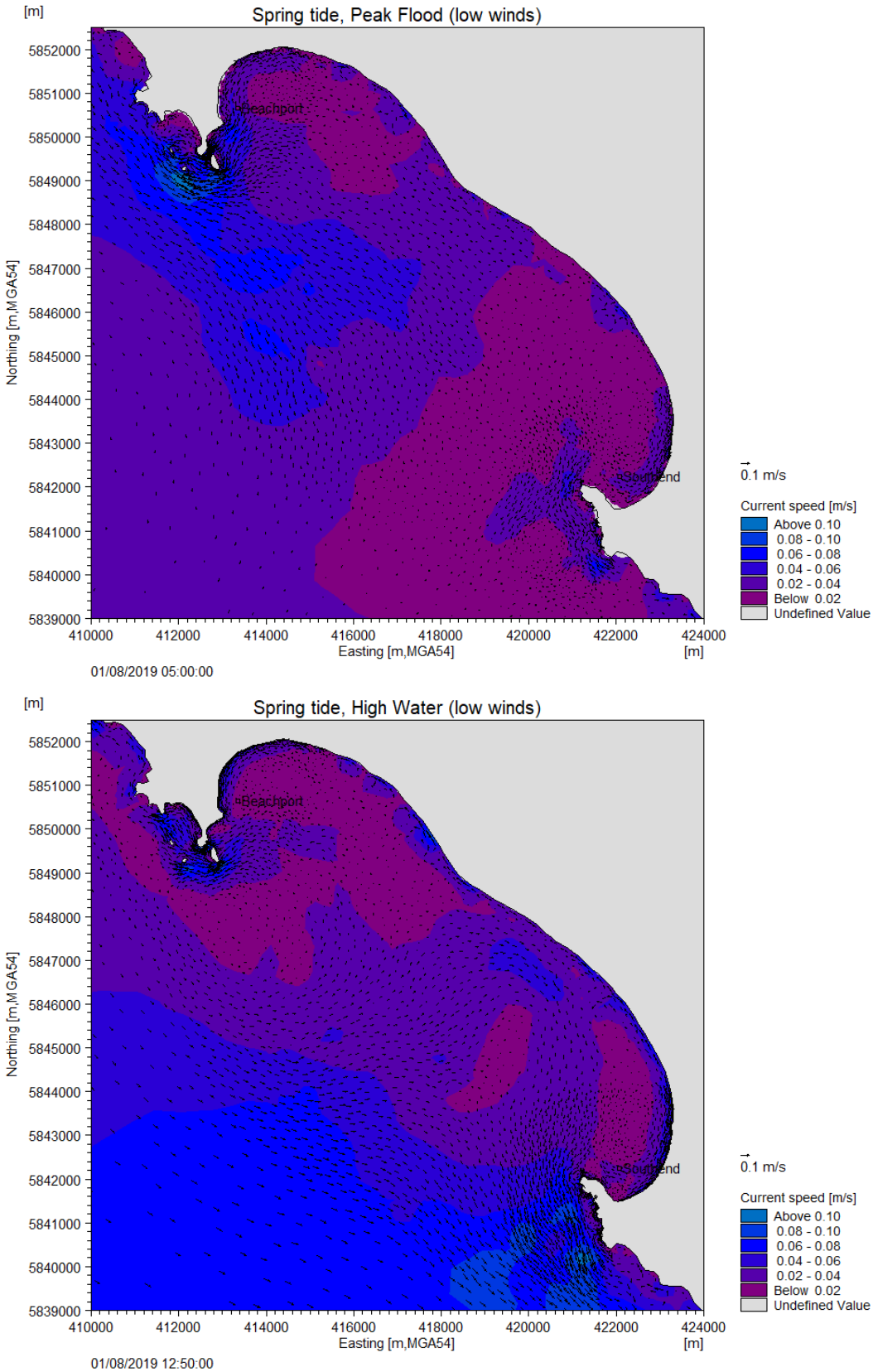


Figure 39. Modelled tidal current speeds within Rivoli Bay at peak flood (top) and high water (bottom) for a spring tide with low winds.

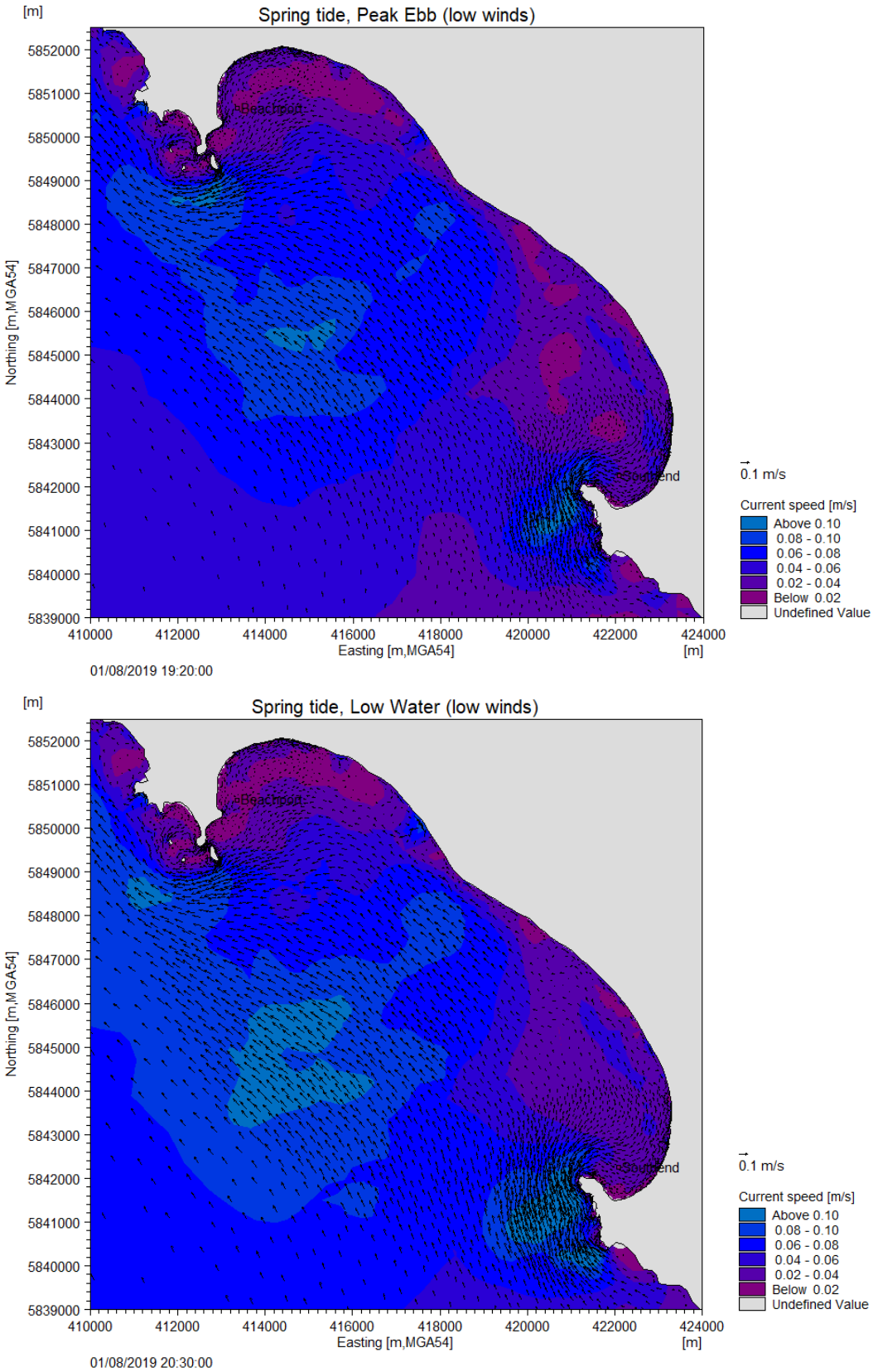


Figure 40. Modelled tidal current speeds within Rivoli Bay at peak ebb (top) and low water (bottom) for a spring tide with low winds.

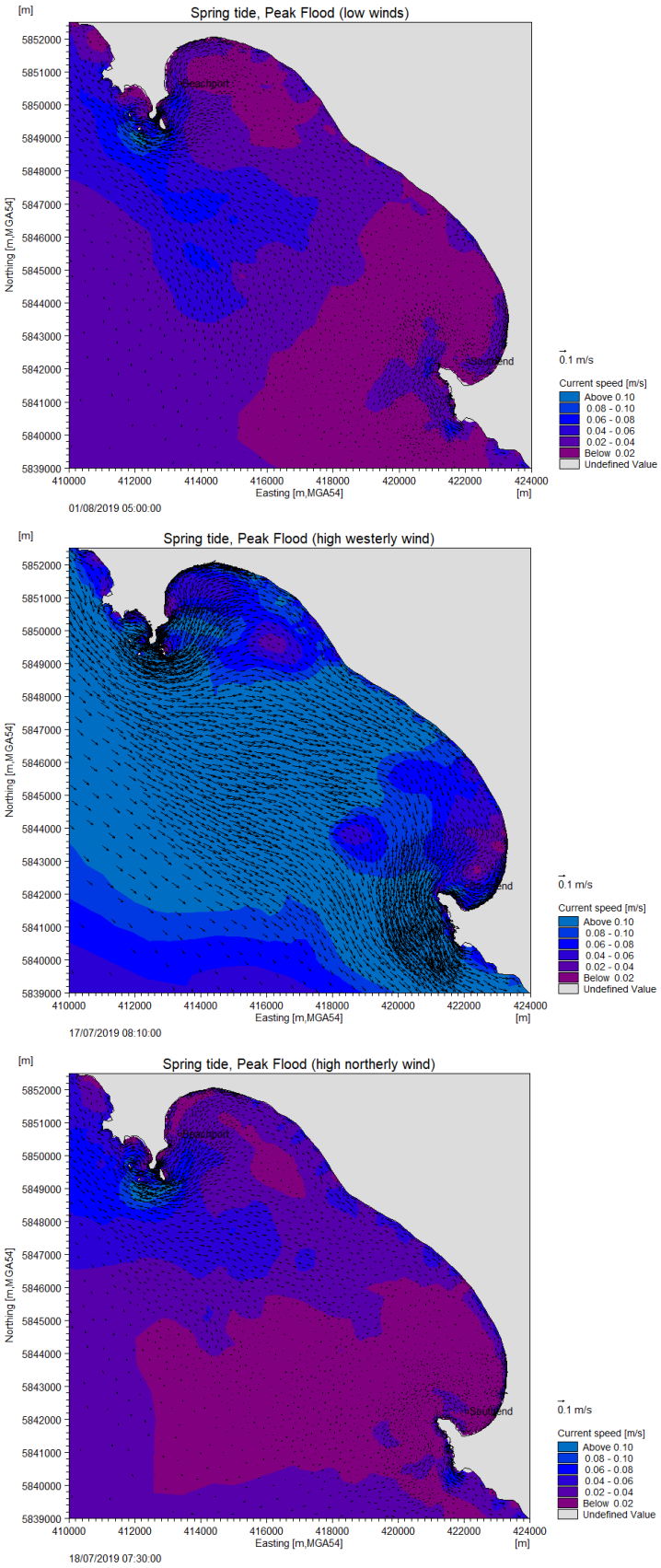


Figure 41. Modelled tidal current speeds within Rivoli Bay during peak flood on a spring tide with low winds (top), a high westerly wind (middle) and a high northerly wind (bottom).

4.3. Wave Model

The wave model was set up to replicate the wave conditions at the Cape de Couedic Waverider Buoy (WRB) and to transform these wave conditions to the shoreline at Maria Creek. The model was run for the 20 year period between November 2000 and January 2020 for which the WRB has been recording wave data.

4.3.1. Data Availability

For the purpose of this study, in addition to wave data from the Cape de Couedic WRB, measured wave data from Acoustic Wave and Current (AWAC) devices deployed at the Rivoli Bay calibration sites (Beachport and Southend) were used to assess the ability of the model in transforming wave conditions across the model domain. As noted in Section 4.2.1, the Rivoli calibration data are located approximately 80 km from Maria Creek, within a relatively sheltered and shallow embayment. More local to Maria Creek, a two month record of wave heights is available at Cape Jaffa (approximately 20 km south of Maria Creek) and the model performance at this site has also been considered.

The locations of the available wave data are shown in Figure 42.

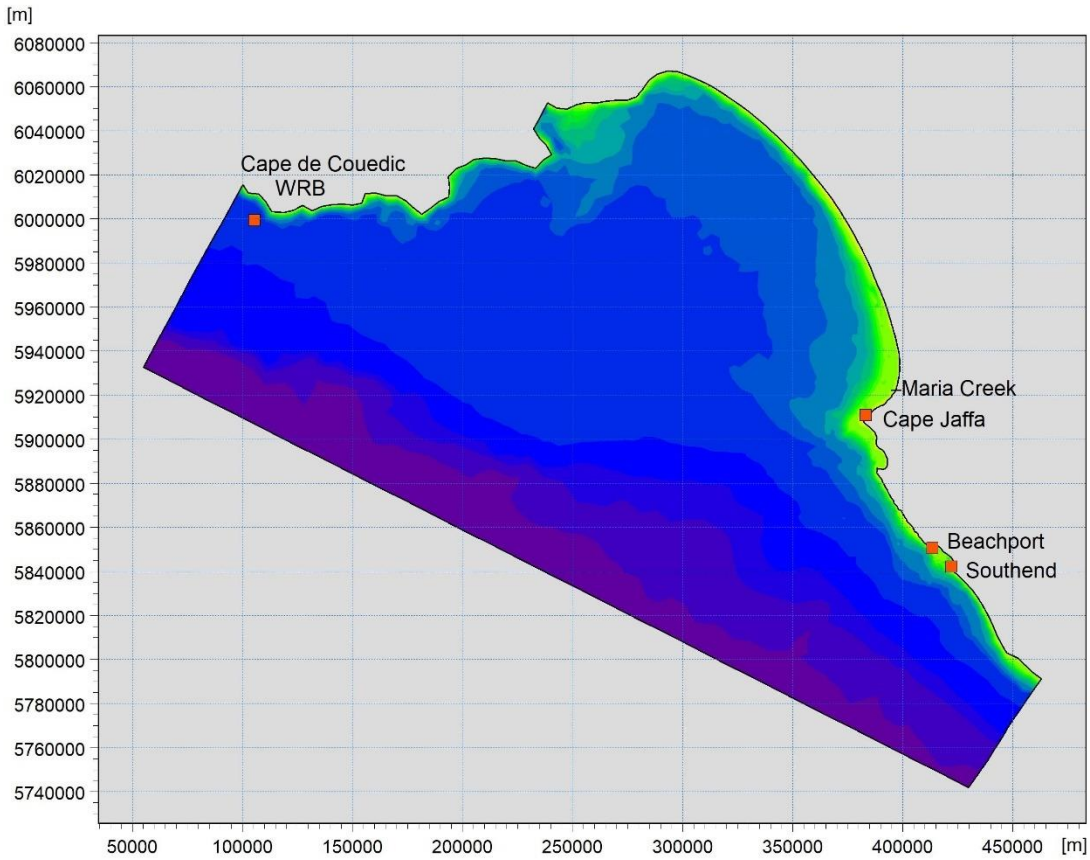


Figure 42. SW calibration data locations.

4.3.2. SW Calibration and Validation Periods

The model was setup to replicate wave conditions over a two month calibration period (July and August 2019), selected as a period with relatively large waves and coinciding with measured wave data in Rivoli Bay. The model performance was then validated against a separate two months (mid September to mid November 2003) to coincide with the period of measured data at Cape Jaffa (WBM, 2005).

4.3.3. Results

Time series plots of H_s, peak wave period (T_p) and mean wave direction are shown at the Cape de Couedic WRB over the full 20 year model simulation period in Figure 43 and for the calibration and validation periods in Figure 44 and Figure 45, respectively. The plots show that the model provides a good representation of the measured wave height, period and direction, during both calmer periods and stormier, more extreme wave conditions.

A quantitative assessment of the model calibration at the WRB site is provided in Table 6, with percentile statistics presented for both measured and modelled H_s over the full model run period and also for the calibration and validation periods. The statistics confirm that the modelled waves agree well with the measured data (typically to within 0.1 to 0.2 m).

The correlation between the modelled and measured waves is also shown as a scatter plot in Figure 46. Figure 46 shows a higher degree of scatter between the measured and modelled wave heights during the calmer periods. This is because the model effectively accounts for the effect of winds on waves (which will be more significant when swell waves are small) at the WRB twice since the effects of winds is included in the boundary condition and is applied again over the fetch between the boundary and the WRB. Although the exclusion of winds improves the model performance at the WRB, if winds are not applied in the model, the wave field at Maria Creek would only consider the effect of swell waves. While swell waves will be the dominant source of wave energy at Maria Creek, wind waves may play an important role in driving southward sediment transport along the coast and it was therefore considered necessary to include wind forcing in the model. To improve the model performance at the WRB, the wind wave component could be removed from the boundary condition. However, the separation of wind and swell wave components is complex and given the short distance between the boundary and the WRB in relation to the distance between the WRB and Maria Creek, this was not considered necessary for ensuring the accuracy of the modelled wave climate at Maria Creek. Further, despite the poorer agreement between modelled and measured waves at the WRB when winds are applied in the model, the correlation between modelled and measured waves remains high, indicating that the model accurately represents the wave conditions at the WRB.

Table 6. Percentiles of modelled and measured H_s during the model simulation period at the WRB.

Percentile	November 2000 to Jan 2020		Calibration Period		Validation Period	
	Modelled H_s (m)	Measured H_s (m)	Modelled H_s (m)	Measured H_s (m)	Modelled H_s (m)	Measured H_s (m)
99th	5.88	5.83	7.13	7.30	5.82	5.75
95th	4.74	4.68	5.92	5.90	5.22	5.11
90th	4.15	4.09	5.22	5.20	4.60	4.51
80th	3.51	3.45	4.28	4.30	3.90	3.80
50th	2.58	2.49	3.23	3.20	2.81	2.68
20th	1.88	1.79	2.18	2.20	2.10	1.96
10th	1.59	1.51	1.85	1.80	1.74	1.64
5th	1.40	1.31	1.63	1.60	1.50	1.40

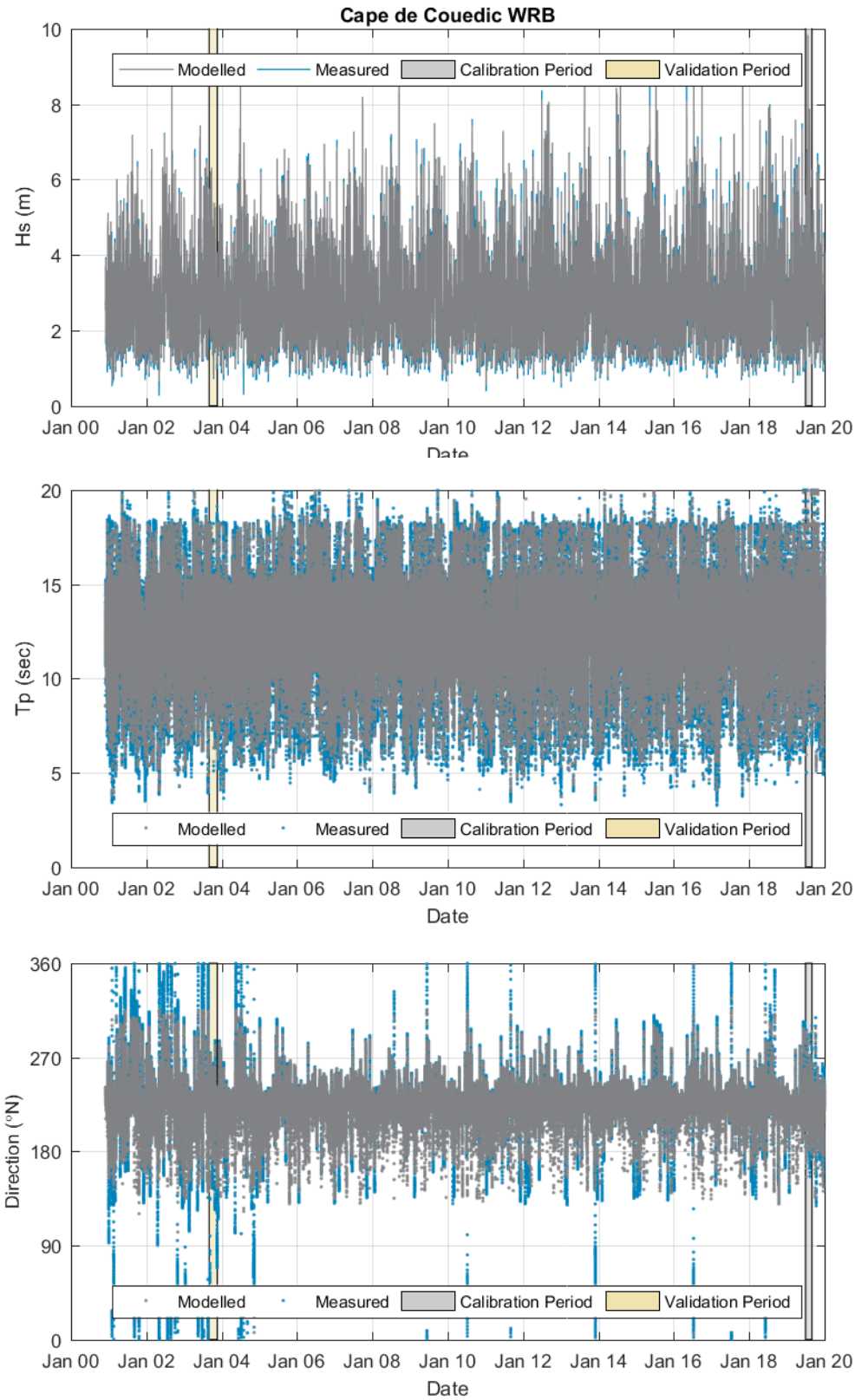


Figure 43. Comparison of modelled and measured waves at the Cape de Couedic WRB for the 20 year model run period, showing H_s (upper), peak wave period (middle) and mean wave direction (lower).

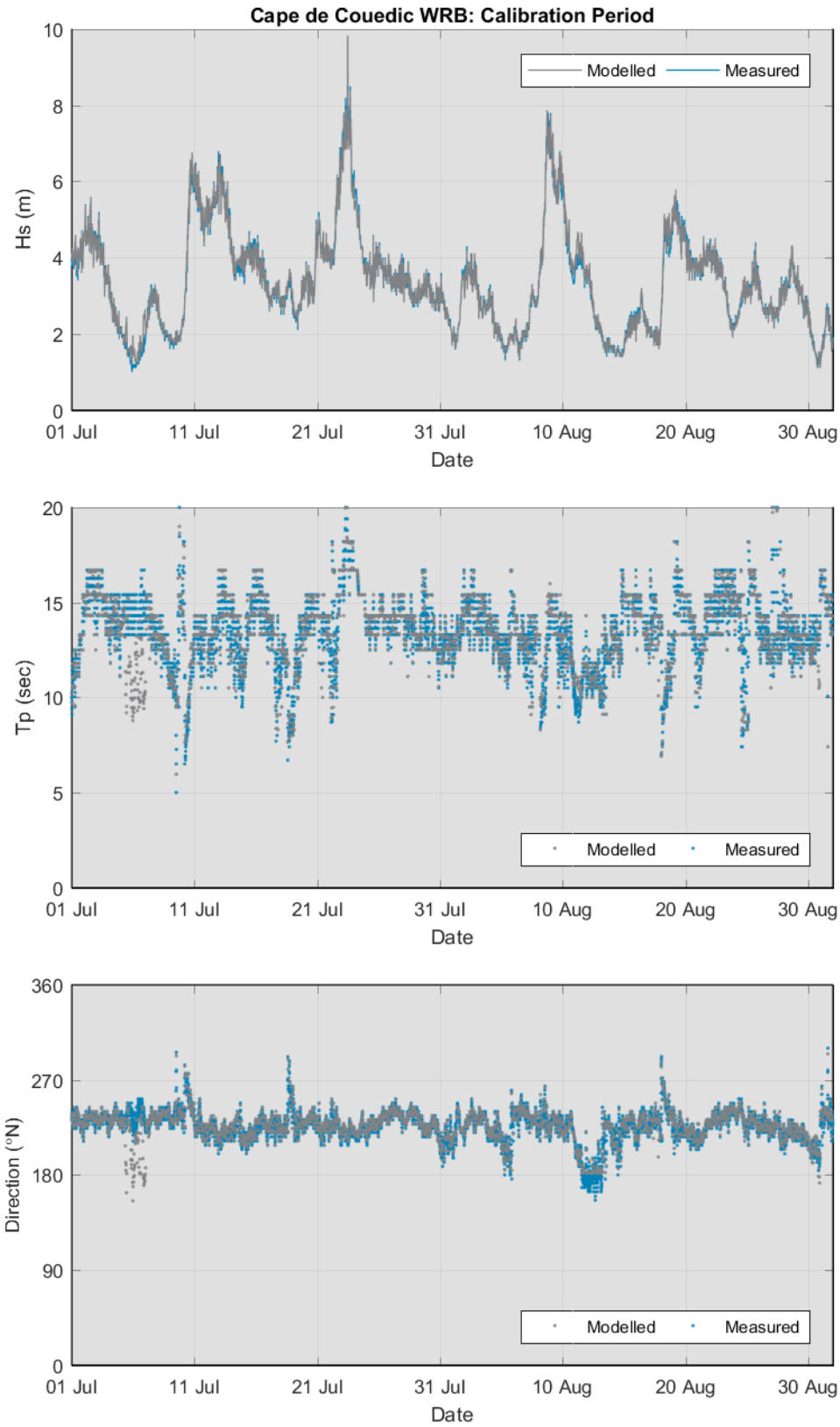


Figure 44. Comparison of modelled and measured waves at the Cape de Couedic WRB for the model calibration period, showing H_s (upper), peak wave period (middle) and mean wave direction (lower).

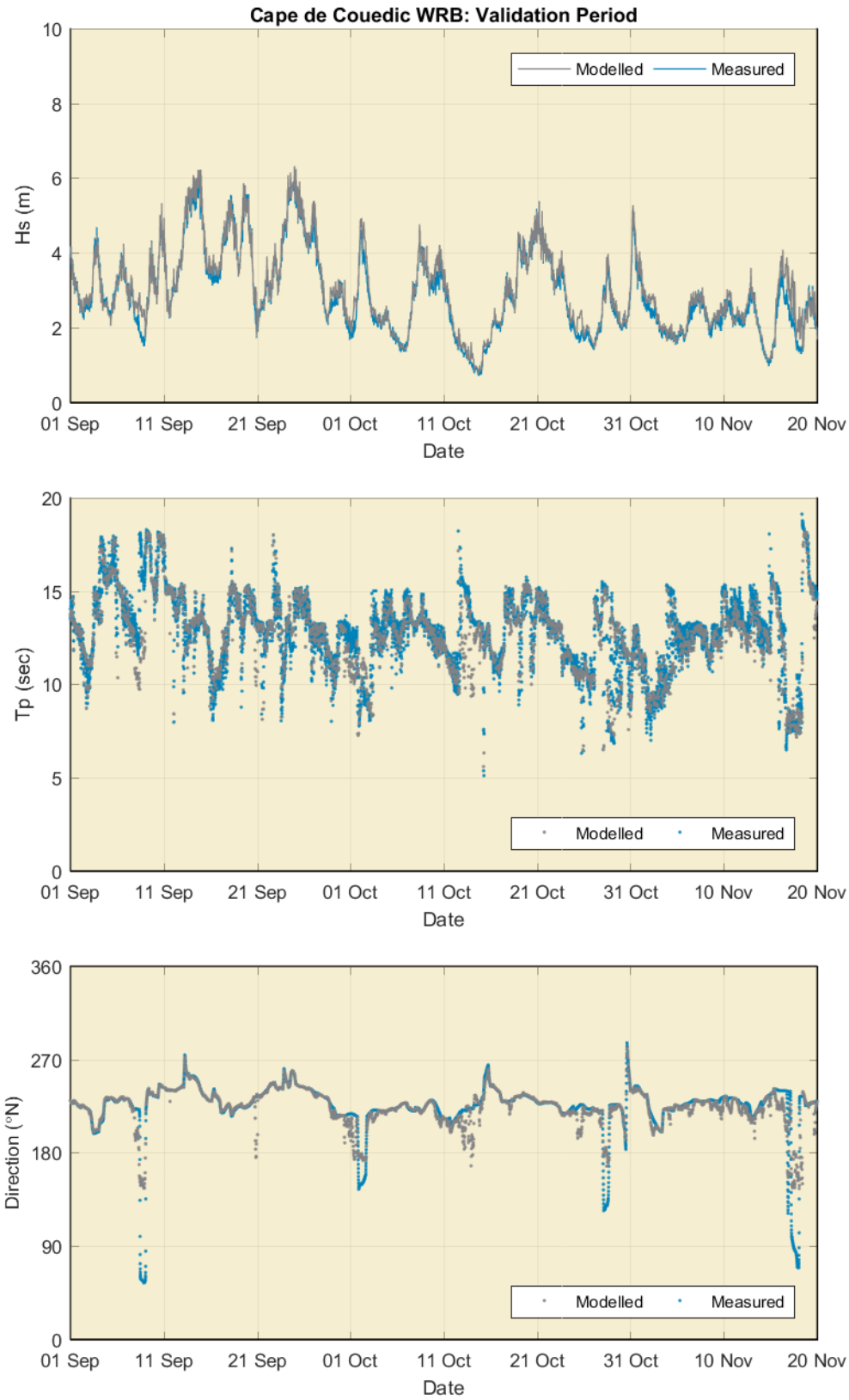


Figure 45. Comparison of modelled and measured waves at the Cape de Couedic WRB for the model validation period, showing H_s (upper), peak wave period (middle) and mean wave direction (lower).

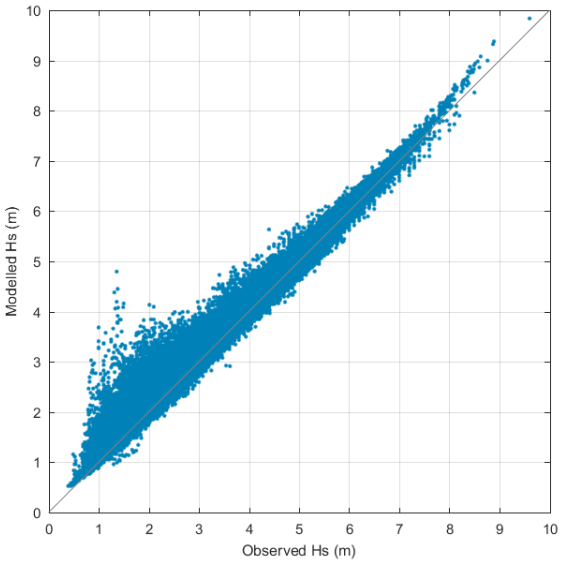


Figure 46. Correlation between modelled and measured H_s at the Cape de Couedic WRB.

Time series plots of the wave conditions at Beachport and Southend during the calibration period are shown in Figure 47 and Figure 48, respectively. The plots show that the model provides a good representation of the wave conditions with respect to wave height, wave period and wave directions. The measured wave periods from the AWAC are only reported in incremental blocks with steps of up to 4 seconds for the higher wave periods. The actual wave periods would be expected to vary in a more gradual way and based on this the variations in wave periods shown by the model are expected to provide a good representation of the wave period. The measured data at Beachport show a larger degree of temporal variability in the wave direction compared to the model, but as described in Section 4.2.2, the AWAC was deployed in an area of complex bathymetry with numerous rocky outcrops and headlands, which are not fully resolved in the numerical model.

A quantitative assessment of the model calibration at Beachport and Southend is provided in Table 7, with percentile statistics presented for both measured and modelled H_s over the calibration period. The statistics show that the model has a tendency to slightly under predict (by up to 0.05 m or 5%) the peak wave heights, and slightly over predicts the waves during calmer conditions. Despite this, overall the model is considered to provide a good representation of the measured wave conditions in Rivoli Bay, particularly in view of the complex local bathymetry.

Table 7. Percentiles of modelled and measured H_s during the calibration period at Rivoli Bay.

Percentile	Beachport		Southend	
	Modelled H_s (m)	Measured H_s (m)	Modelled H_s (m)	Measured H_s (m)
99th	0.95	1.00	1.41	1.44
95th	0.87	0.86	1.28	1.28
90th	0.82	0.79	1.22	1.18
80th	0.75	0.69	1.13	1.04
50th	0.60	0.52	0.90	0.71
20th	0.43	0.34	0.66	0.53
10th	0.36	0.28	0.54	0.44
5th	0.32	0.24	0.46	0.35

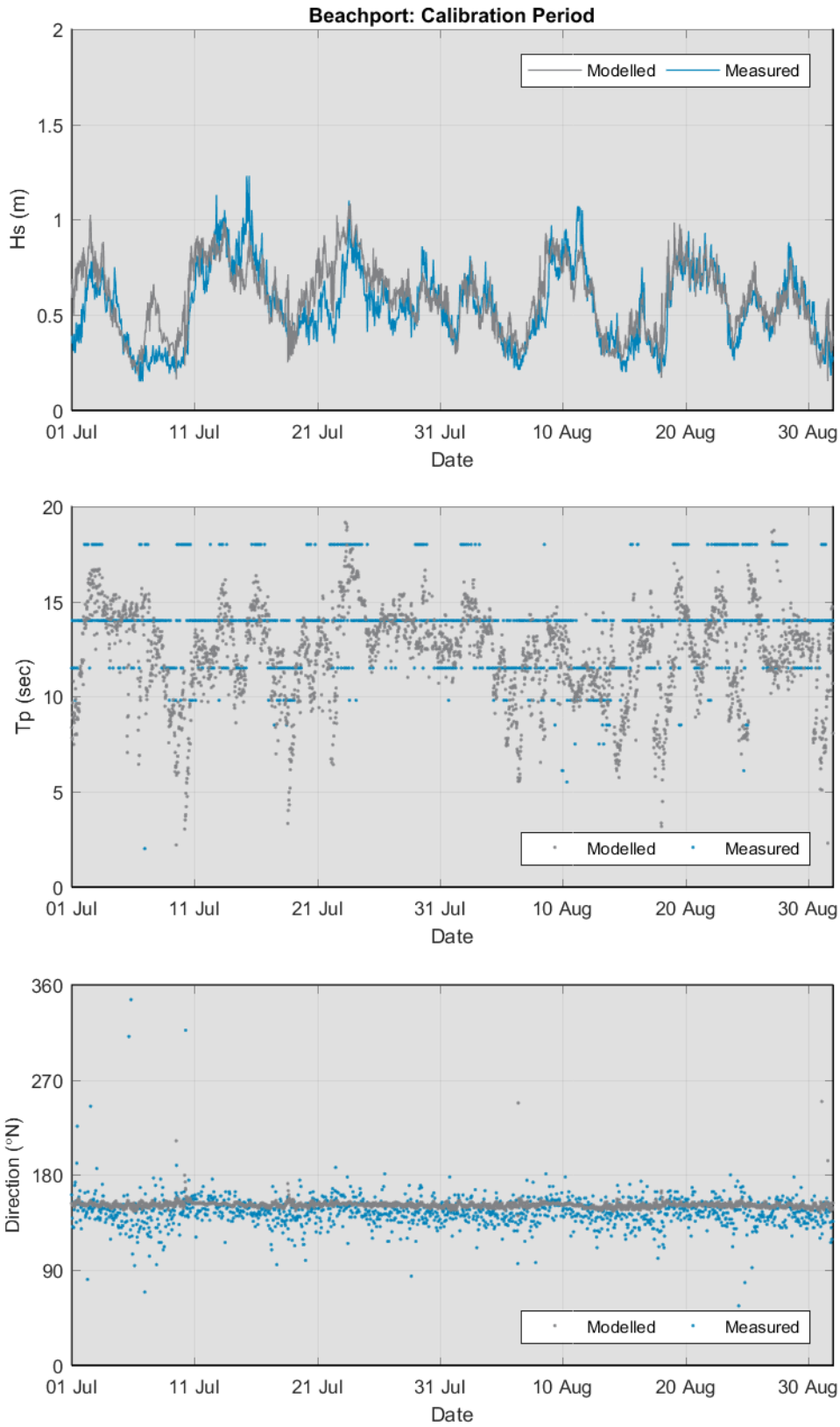


Figure 47. Comparison of modelled and measured waves at the Beachport ADCP for the model calibration period, showing H_s (upper), peak wave period (middle) and mean wave direction (lower).

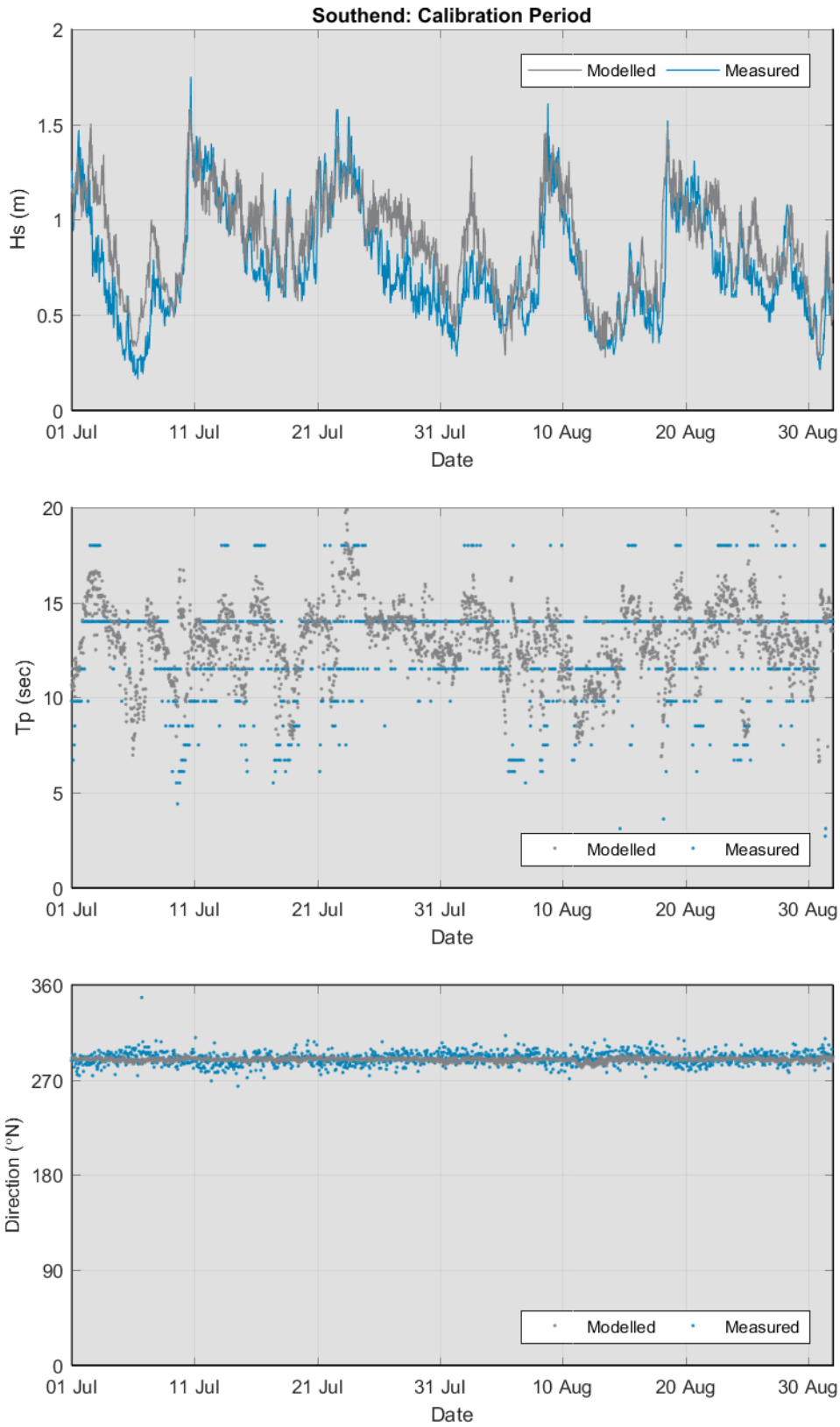


Figure 48. Comparison of modelled and measured waves at the Southend ADCP for the model calibration period, showing H_s (upper), peak wave period (middle) and mean wave direction (lower).

To assess the performance of the wave model closer to the study area, time series plots of the wave heights at Cape Jaffa during the validation period are shown in Figure 49, the measured wave heights for comparison are shown in Figure 50. The plots show that the model captures the timing of the peaks in wave activity and provides a good representation of wave heights during both the calmer and stormier conditions.

While there are no data at Maria Creek for calibration, the wave climate at Maria Creek is expected to be broadly similar to that at Cape Jaffa with comparable levels of exposure at both sites. The modelled wave heights, wave period and wave directions at Maria Creek are shown for the validation period in Figure 51. The wave heights are slightly smaller than those at Cape Jaffa, with peaks generally less than 0.8 m during the validation period, indicating a low energy wave climate. This is confirmed by the presence of sea grass beds close to the shoreline at Maria Creek and the modelled waves are therefore believed to provide a reasonable representation of the wave climate in the study area. The local wave climate at Maria Creek is discussed in more detail in Section 6.

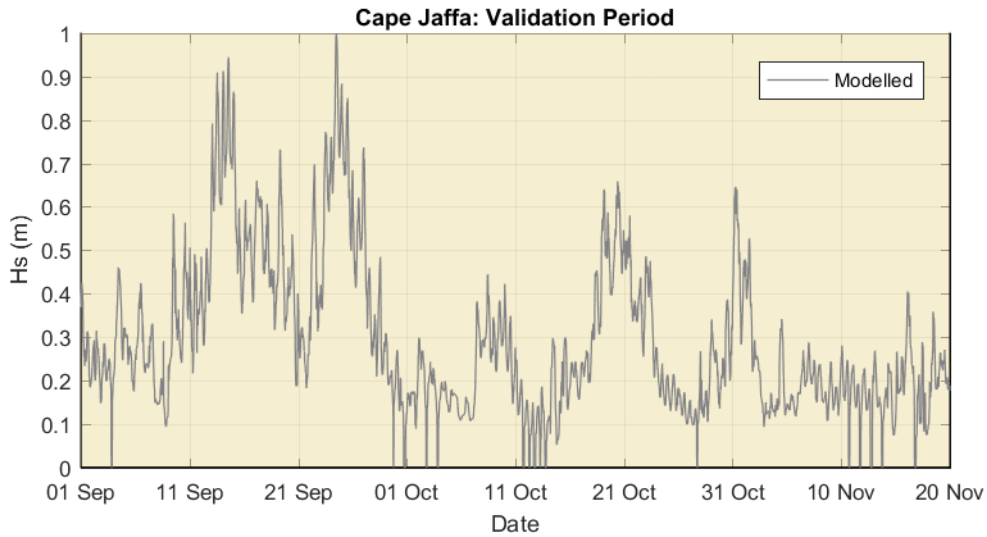


Figure 49. Modelled waves at Cape Jaffa during the model validation period (2003), showing H_s.

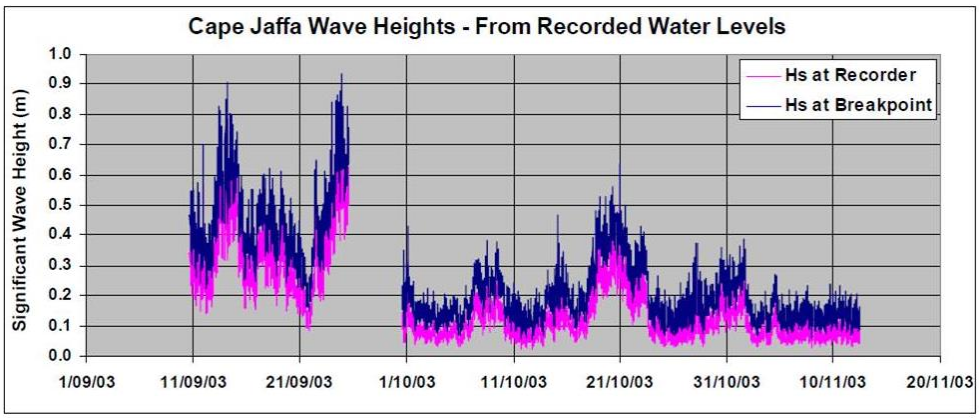


Figure 50. Measured waves at Cape Jaffa during the model validation period, showing H_s.

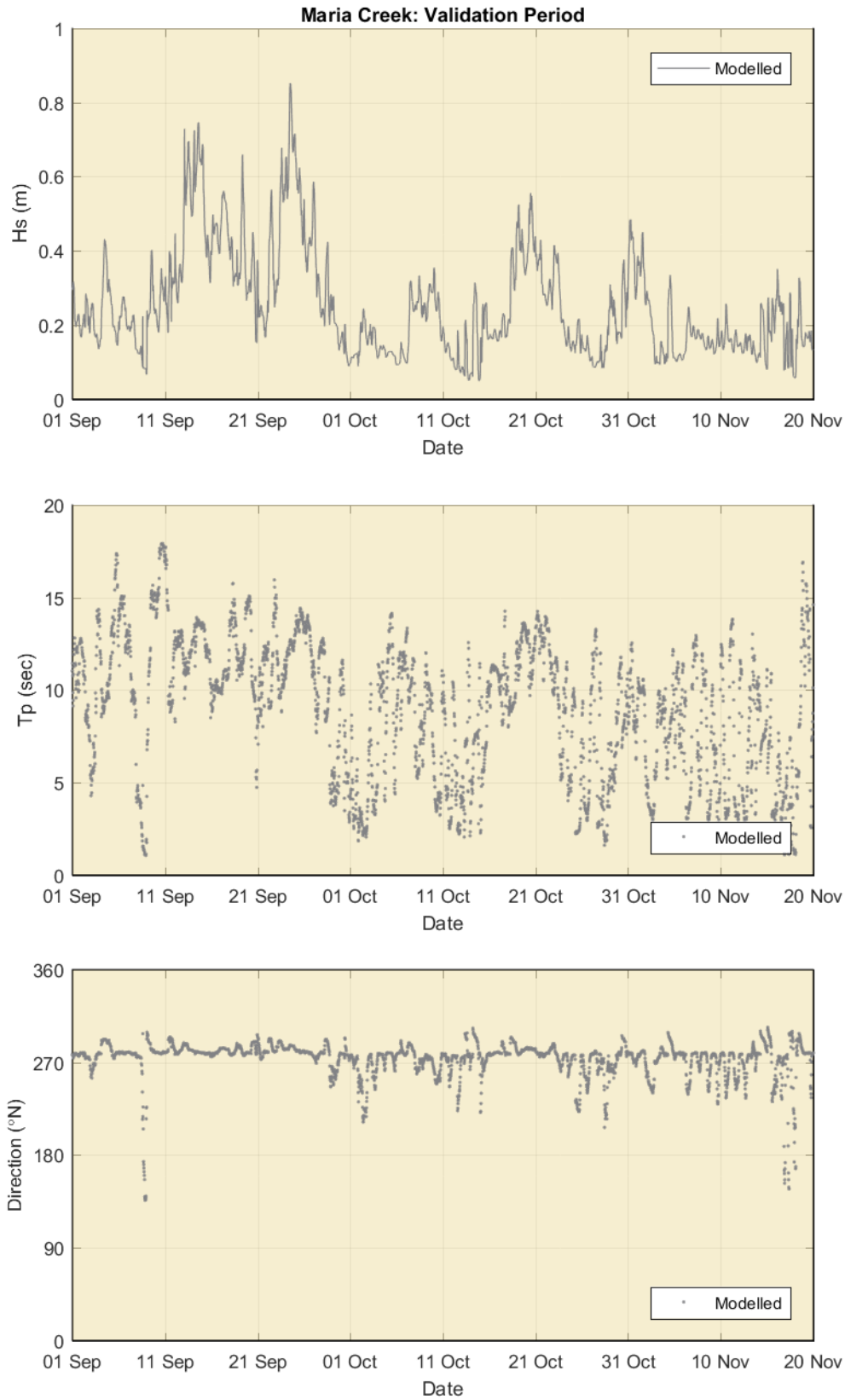


Figure 51. **Modelled waves at Maria Creek during the model validation period, showing H_s (upper), peak wave period (middle) and mean wave direction (lower).**

5. Hydrodynamic Modelling

The hydrodynamic model has been applied to help quantify the flow velocities in the Creek and the tidal prism on both spring and neap tides for the existing (baseline) case. The baseline case applies the present day bathymetry with a sand bar extending along the southern training wall, a shallow creek entrance and a saturated southern beach. This baseline case does not provide a navigable entrance channel to the Creek and the results presented here therefore represent the currents without sand and wrack management.

The hydrodynamic model was run to simulate conditions over a five week winter period between the 2nd July 2019 and the 9th August 2019. A selection of model outputs, including timeseries and spatial maps are presented within this section, to help explain the physical processes within Maria Creek and in the adjacent offshore region. The locations where timeseries data have been extracted from the model are shown in Figure 52.

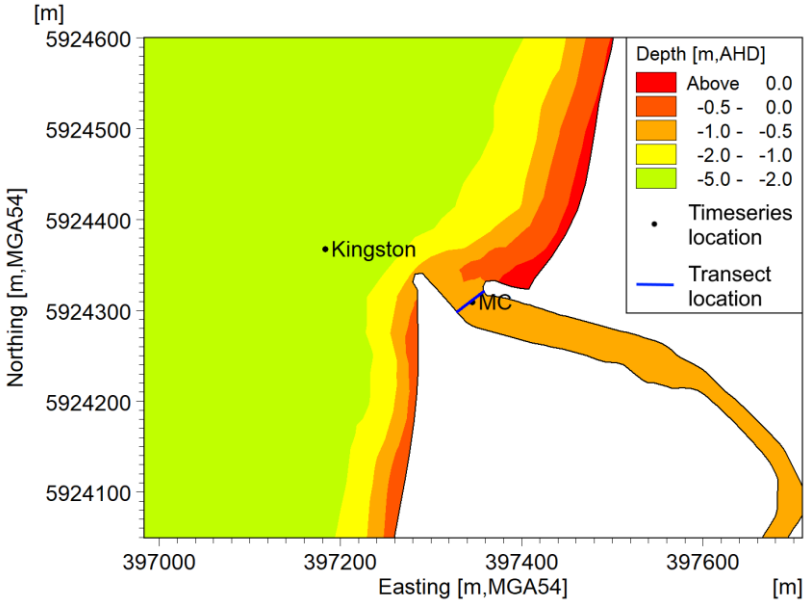


Figure 52. Location of timeseries and transect extraction locations from the hydrodynamic model.

Key observations from the hydrodynamic modelling which are relevant to the conceptual understanding include the following:

- within Lacepede Bay the tide generally floods to the north and ebbs to the south. However, the modelling shows that a large-scale eddy can form within the bay, resulting in a reversal of the ebb currents in the nearshore region extending from Cape Jaffa to approximately 50 km north of Cape Jaffa. The reversal commonly occurs during larger spring tides when slightly faster currents occur and even though the currents are to the south during the ebb stage of tide throughout most of the Bay, in the nearshore area noted the currents continue to flow to the north throughout the tide. In addition, local wind conditions can also influence the currents, with strong winds having the potential to enhance or even reverse the currents within the bay, depending on the direction of the wind and the state of the tide;
- spatial plots of modelled tidal current speeds around Maria Creek are presented at low water, peak flood (Figure 55), high water and peak ebb (Figure 56) for a spring tide with low winds. The plots show that the nearshore currents are to the north throughout the tide during spring tides. This is a result of the large-scale eddy which forms in Lacepede Bay during the ebb tide (as detailed in the previous point), meaning that flows remain to the north throughout the tide in the Maria Creek nearshore region;

- in the vicinity of Maria Creek the tidal currents are generally shore parallel, typically with lower magnitude current speeds observed offshore with the highest current speeds in Maria Creek due to the channel constriction. Variations in current speed and direction over the model simulation period are shown in Figure 53 at the Kingston extraction location. The wind speed and direction at Cape Jaffa have also been plotted to show how the currents can vary depending on the wind conditions;
- the wind conditions can influence the currents in the Maria Creek area. During the model simulation period there are periods of strong northerly (wind coming from the north) and south westerly winds. Strong south westerly winds can increase current speeds to the north on the flood tide, while strong northerly winds can reverse the current direction causing a southerly current as shown in Figure 57;
- tidal current dominance to the north during calm conditions and during periods with strong winds from the south through to west could potentially enhance longshore transport in a northwards direction, while the potential for increased flows to the south during periods of strong northerly winds could act to enhance longshore transport in a southwards direction. Overall, it is likely that tidal currents will act to increase the overall gross longshore transport rates, while also increasing the net longshore transport in a northwards direction (this is further discussed in Section 7);
- the hydrodynamic model results indicate that there is a flood dominance into Maria Creek on larger spring tides, with flood flows around 50% higher than ebb flows. The highest current speeds occur during the flood stage of the tide during the largest spring tides, for smaller range tides the current speeds are more equal between the flood and ebb stages (Figure 54). This indicates a tendency for the Creek entrance to be importing sediment and wrack; and
- the flood dominance in Maria Creek is further shown by the model discharge results, with higher peak flood discharges observed (peaks of 7m^3/s) compared to the peak ebb discharges (4.5 m^3/s) (Figure 58). Consequently, a longer ebb discharge duration occurs to balance the total volume of water imported and exported over a tidal cycle. The discharge of water out of the Creek would be higher during flood events, however, it is hard to estimate what this could be as there is currently insufficient data available to quantify it.

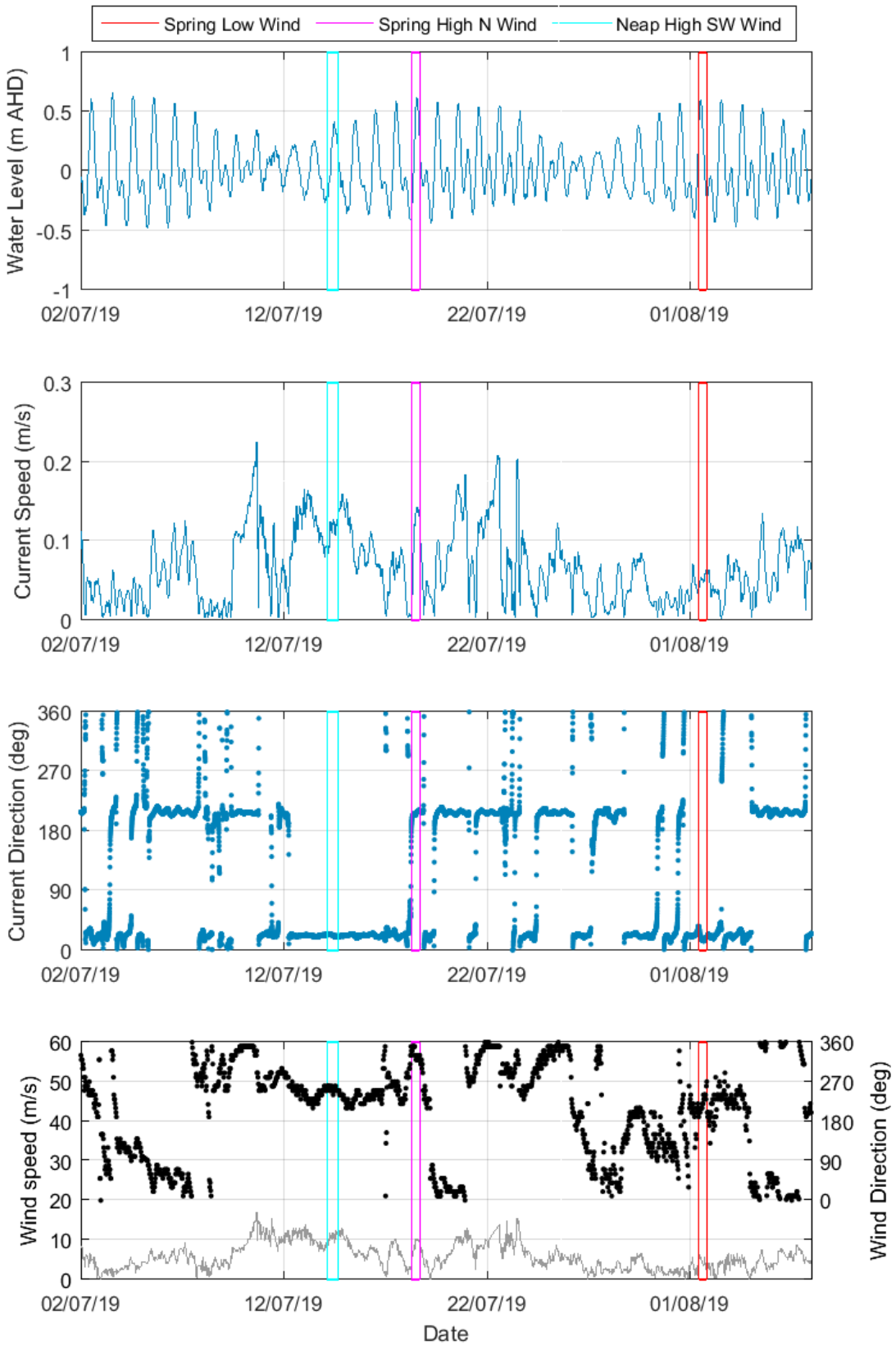


Figure 53. Modelled water levels and currents at Kington and measured winds at Cape Jaffa.

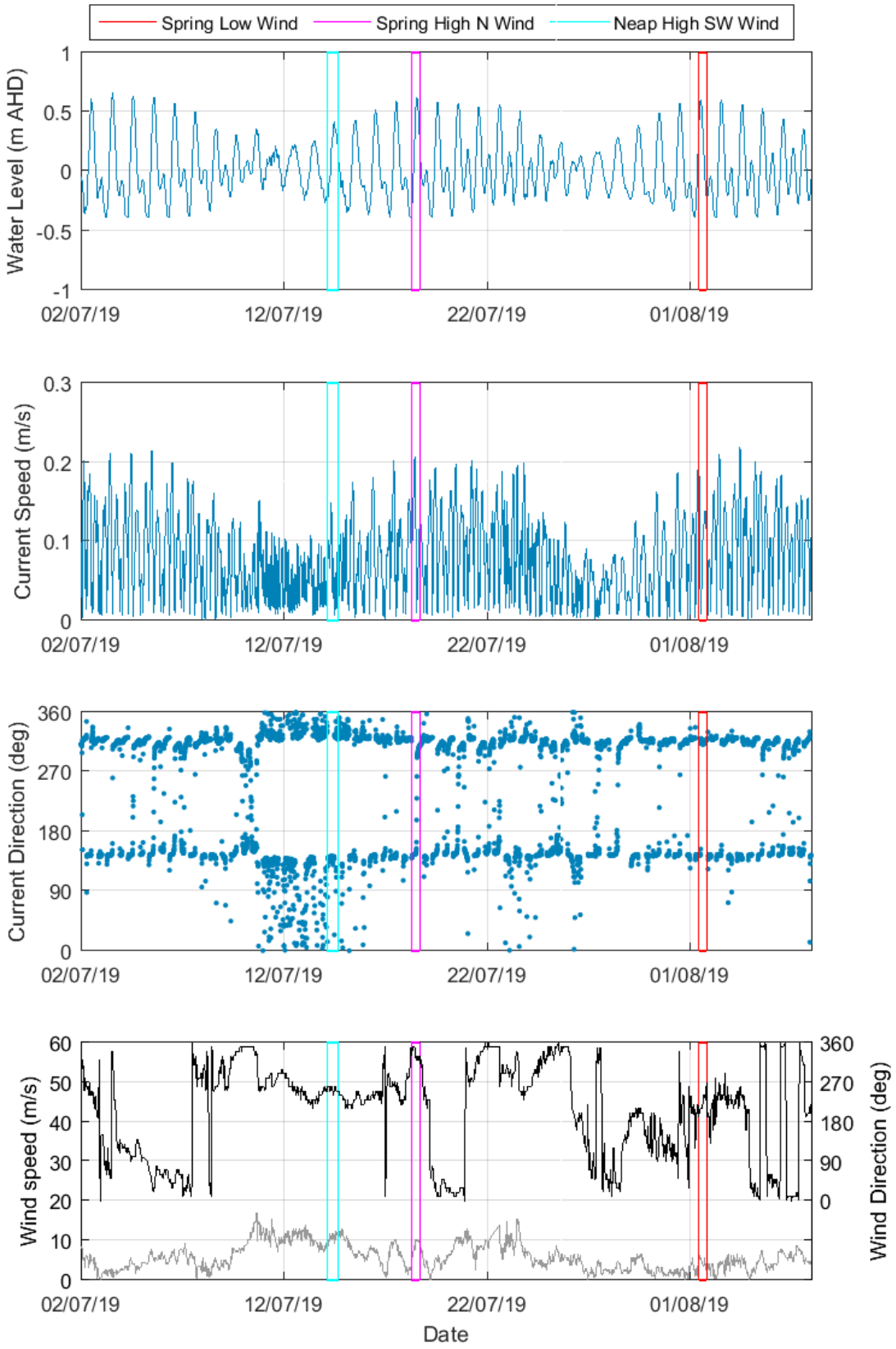


Figure 54. Modelled water level and currents inside Maria Creek and measured winds at Cape Jaffa.

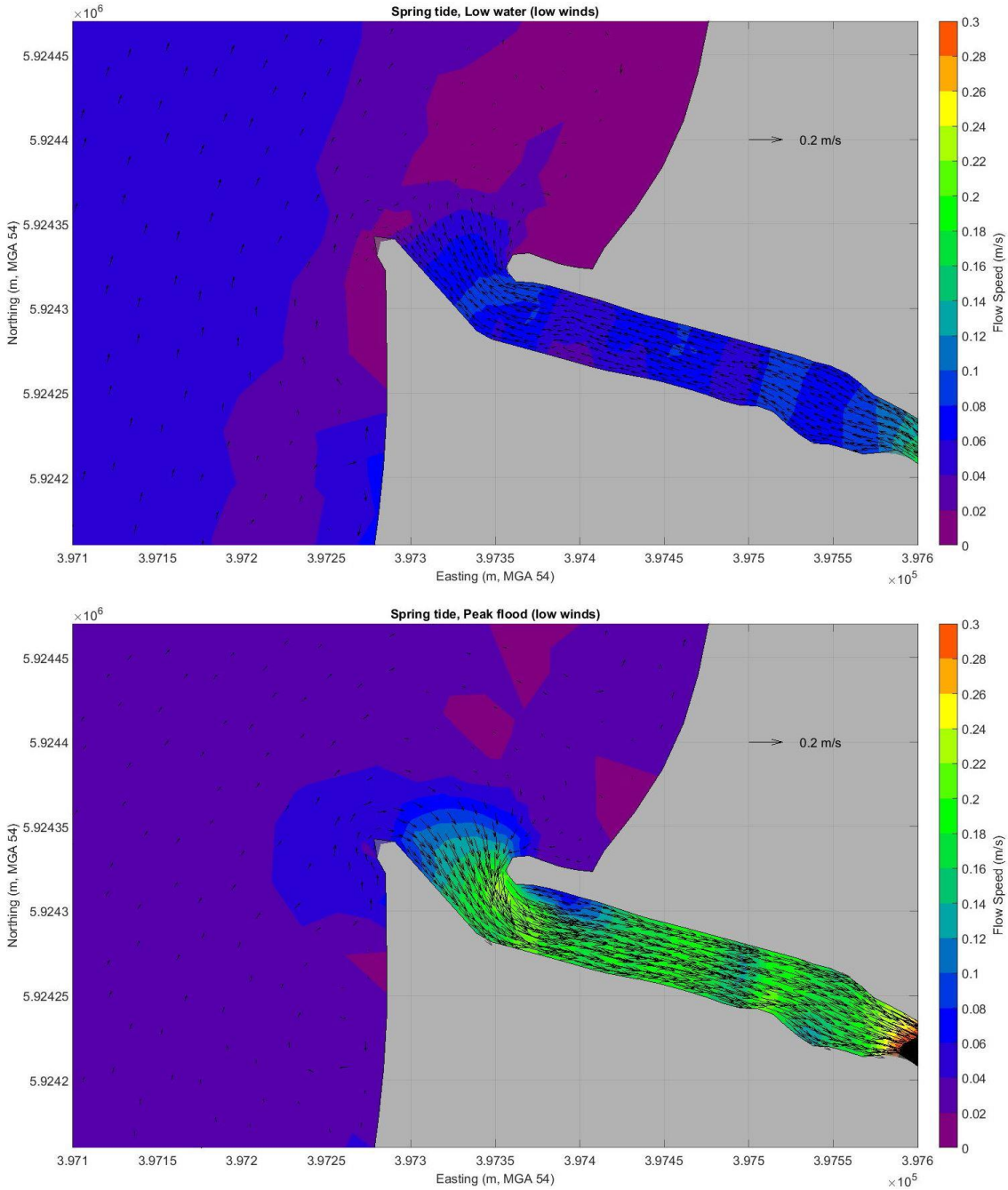


Figure 55. Modelled tidal current speeds around Maria Creek at low water (top) and peak flood (bottom) for a spring tide with low winds.

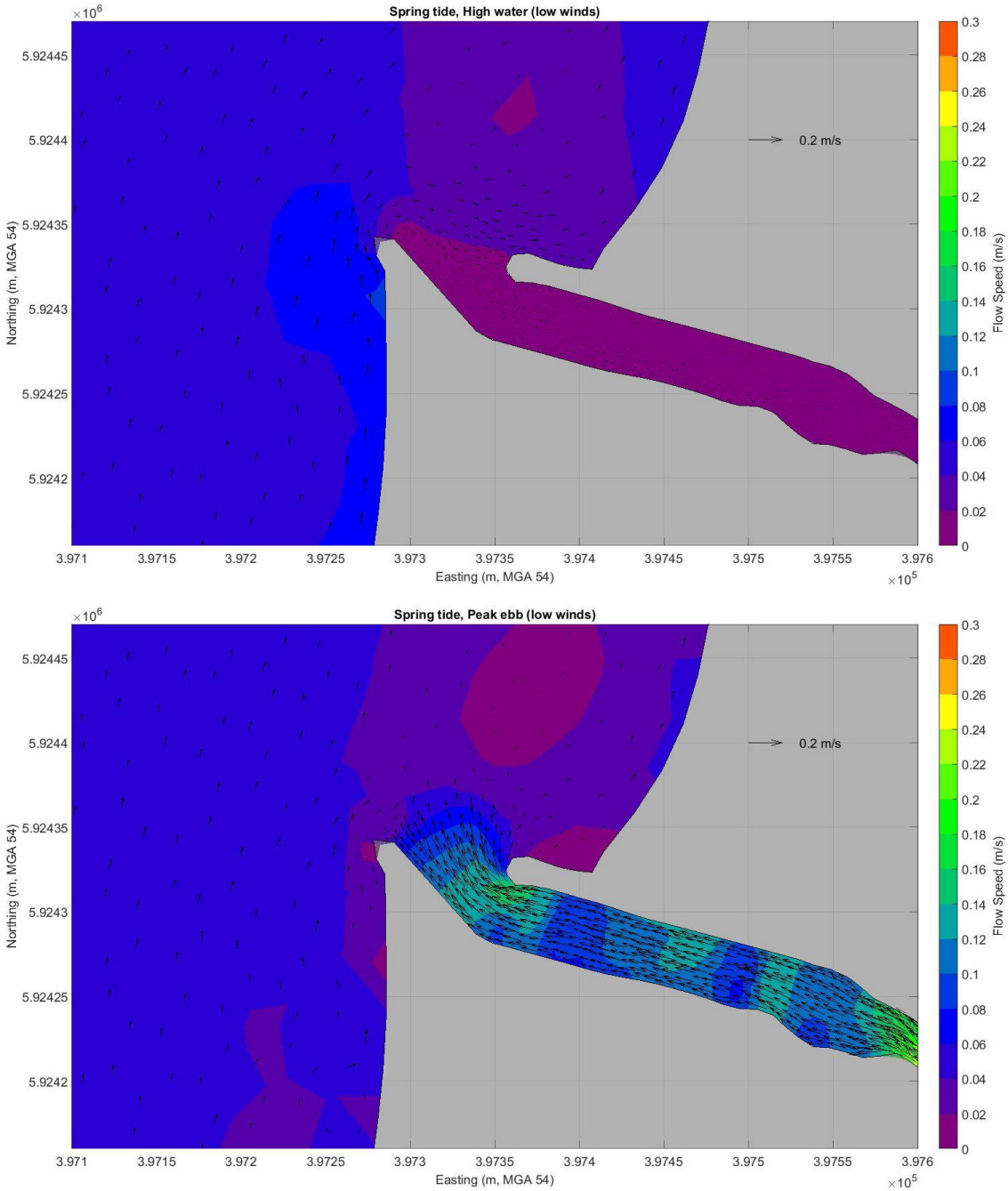


Figure 56. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with low winds.

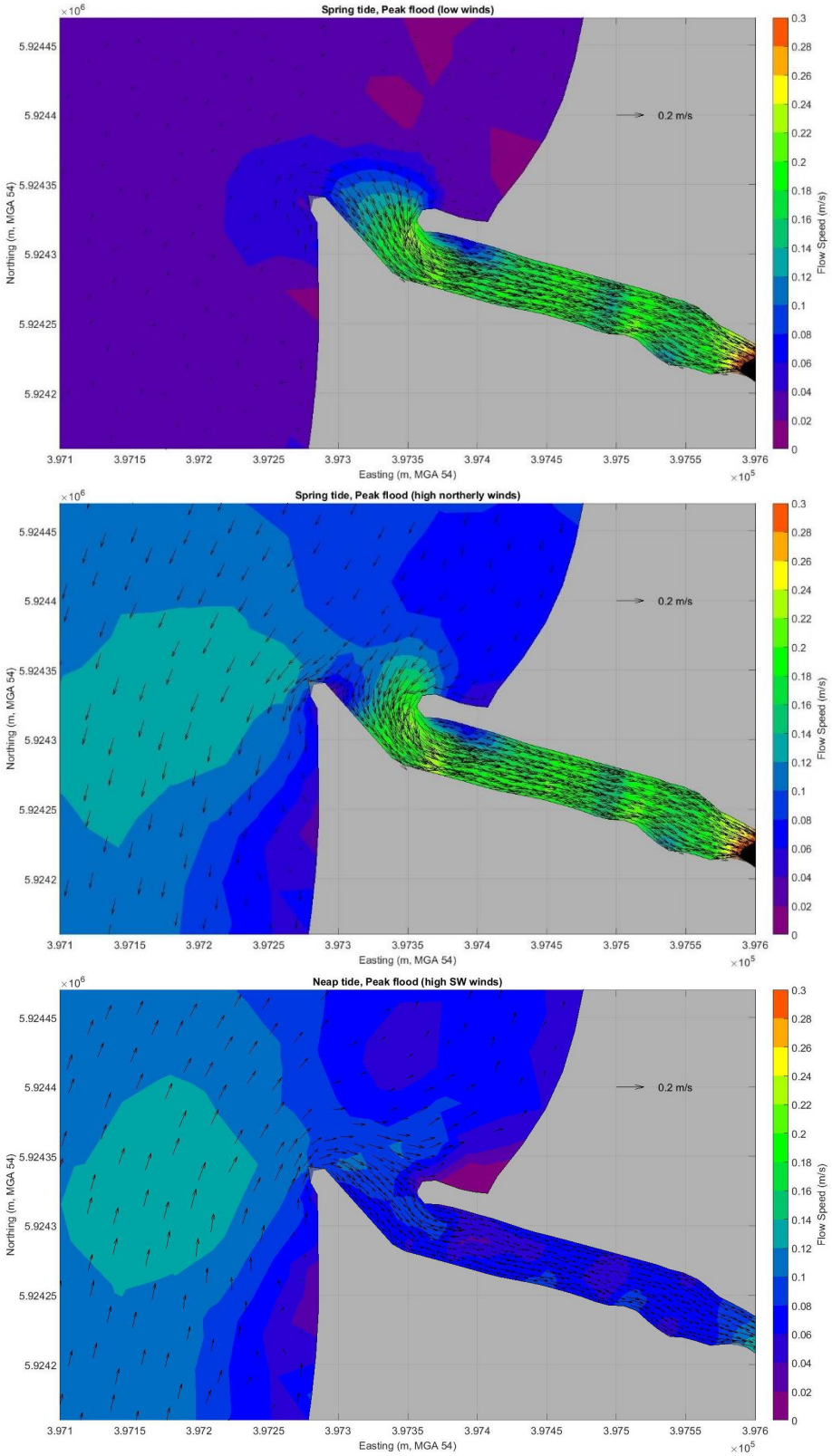
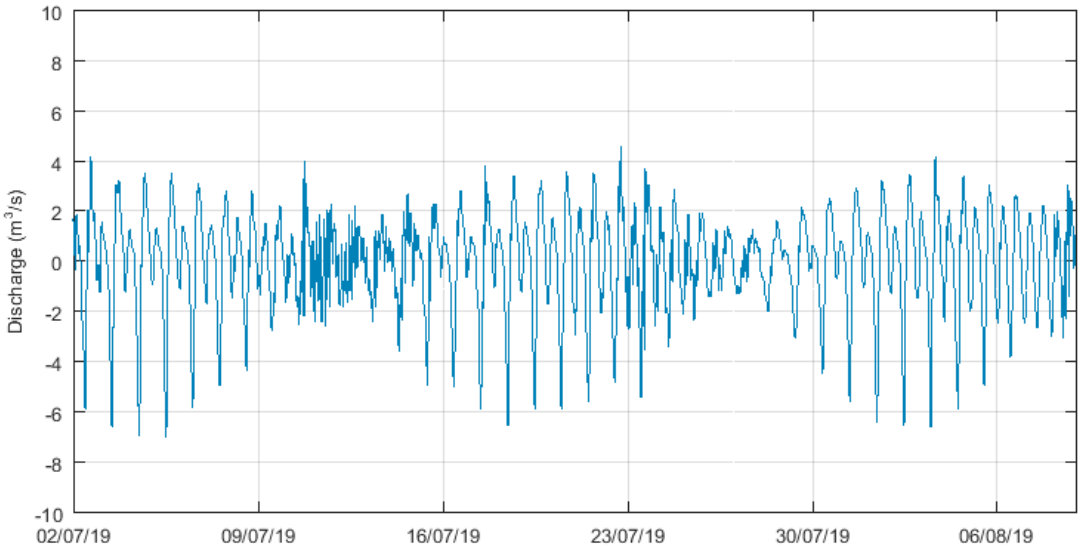


Figure 57. Modelled tidal flows around Maria Creek at peak flood for a spring tide with low winds (top), a spring tide with high northerly winds (middle) and a neap tide with high south westerly winds (bottom).



Note: Positive values denote discharge out of Marina Creek and negative values discharge into the Creek.

Figure 58. Modelled discharge through Maria Creek.

6. Wave Climate

Measured wave conditions from the Cape de Couedic WRB for the twenty year period between 2000 and 2020 have been transformed to the study area using the SW model. The modelled waves at Maria Creek have then been used to:

- define the local wave climate (Section 6.1);
- inform the specific wave design criteria to help inform the requirements of any structures required as part of concept designs (Section 6.2); and
- to drive the longshore transport model (see Section 7).

As noted in Section 5, the model setup considers the existing (baseline) case and the modelling results represent the wave conditions without sand and wrack management.

6.1. Local Wave Climate

Wave conditions have been extracted from the SW model at a location approximately 150 m northwest of the southern training wall in a water depth of 3.25 m below AHD (the 'Maria Creek wave extraction' point - see Figure 59 for location). The wave conditions at the Maria Creek wave extraction point (Maria Creek) are shown as a wave rose in Figure 60, as time series in Figure 61 and as a scatter plot of H_s against wave direction in Figure 62. These plots show the following:

- the wave climate at Maria Creek is characterised by a low wave energy environment, with H_s less than 0.2 m for approximately 50% of the time;
- there is a seasonality in the wave climate with larger waves (H_s exceeding 0.6 m), with longer periods (T_p exceeding 15 seconds) occurring in the winter months;
- wave periods are typically in the range of 5 to 20 seconds with a mean of 13 seconds; and
- waves at Maria Creek approach the coast from a relatively narrow offshore band, with the largest waves approaching from 280°N. The shoreline orientation along this stretch of coast is approximately 15 to 20 degrees, indicating that these waves are approaching slightly south of shore normal (by around 5 to 10 degrees) and therefore have the potential to drive northwards longshore transport.

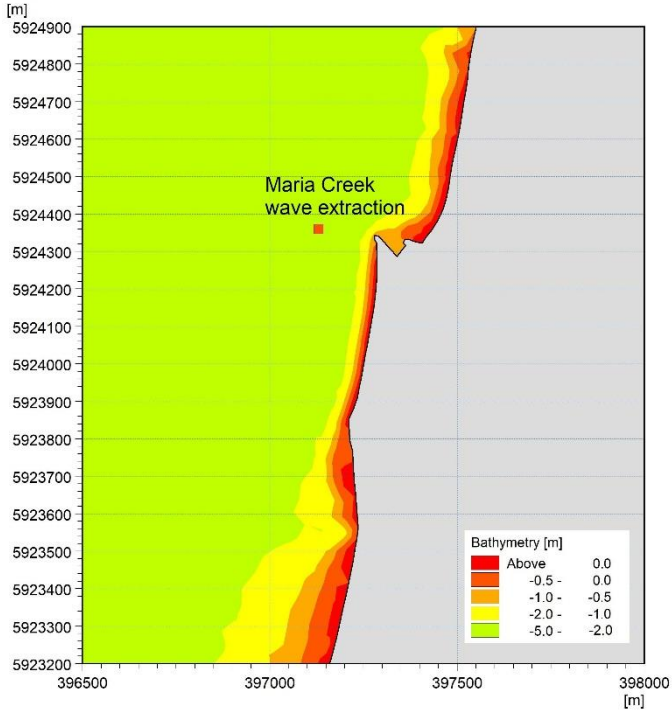
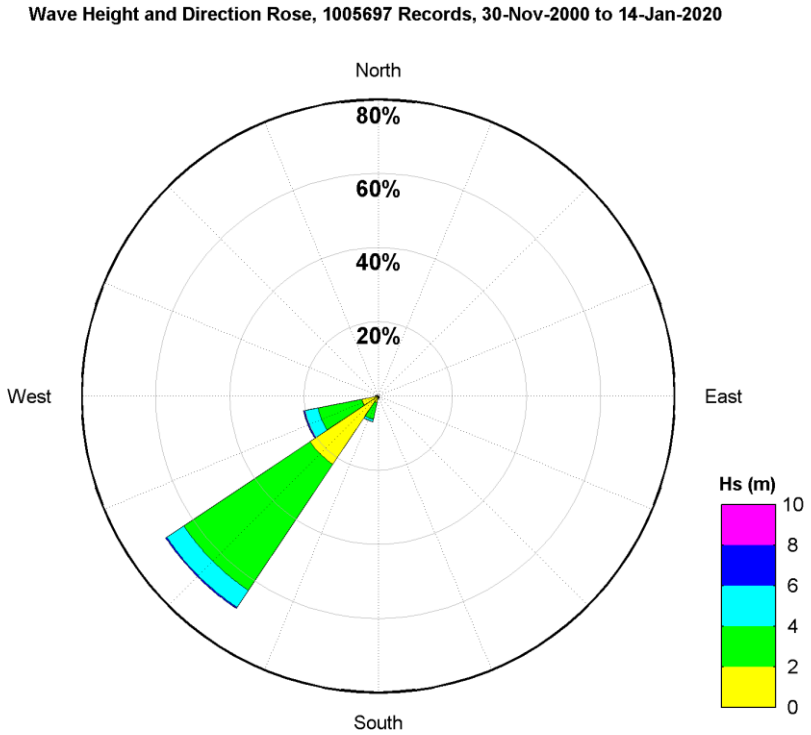
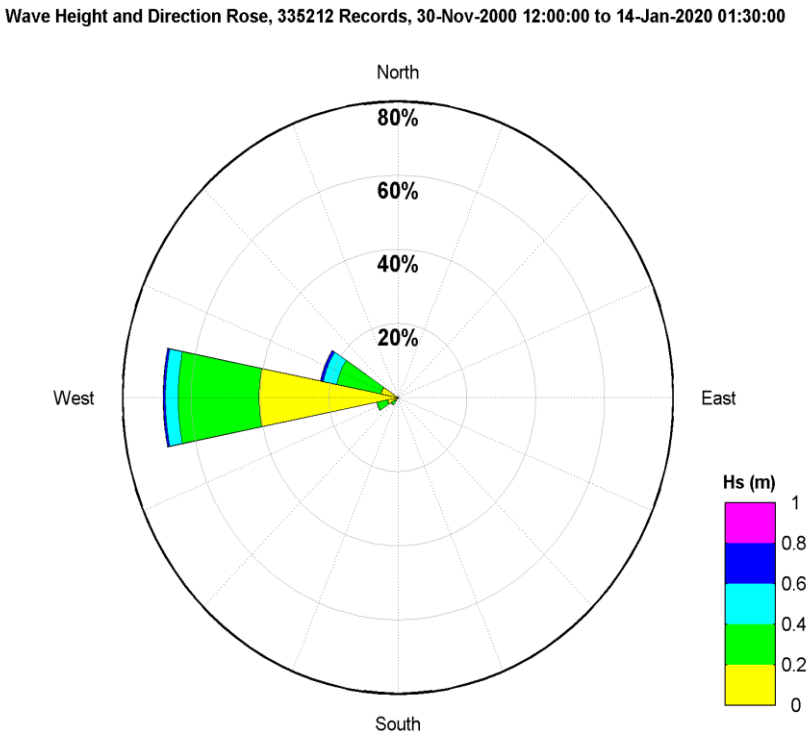


Figure 59. Location of Maria Creek wave extraction point from the SW model.



Cape de Couedic WRB



Maria Creek

Figure 60. Wave Rose at Cape de Couedic WRB and at Maria Creek (note different scales).

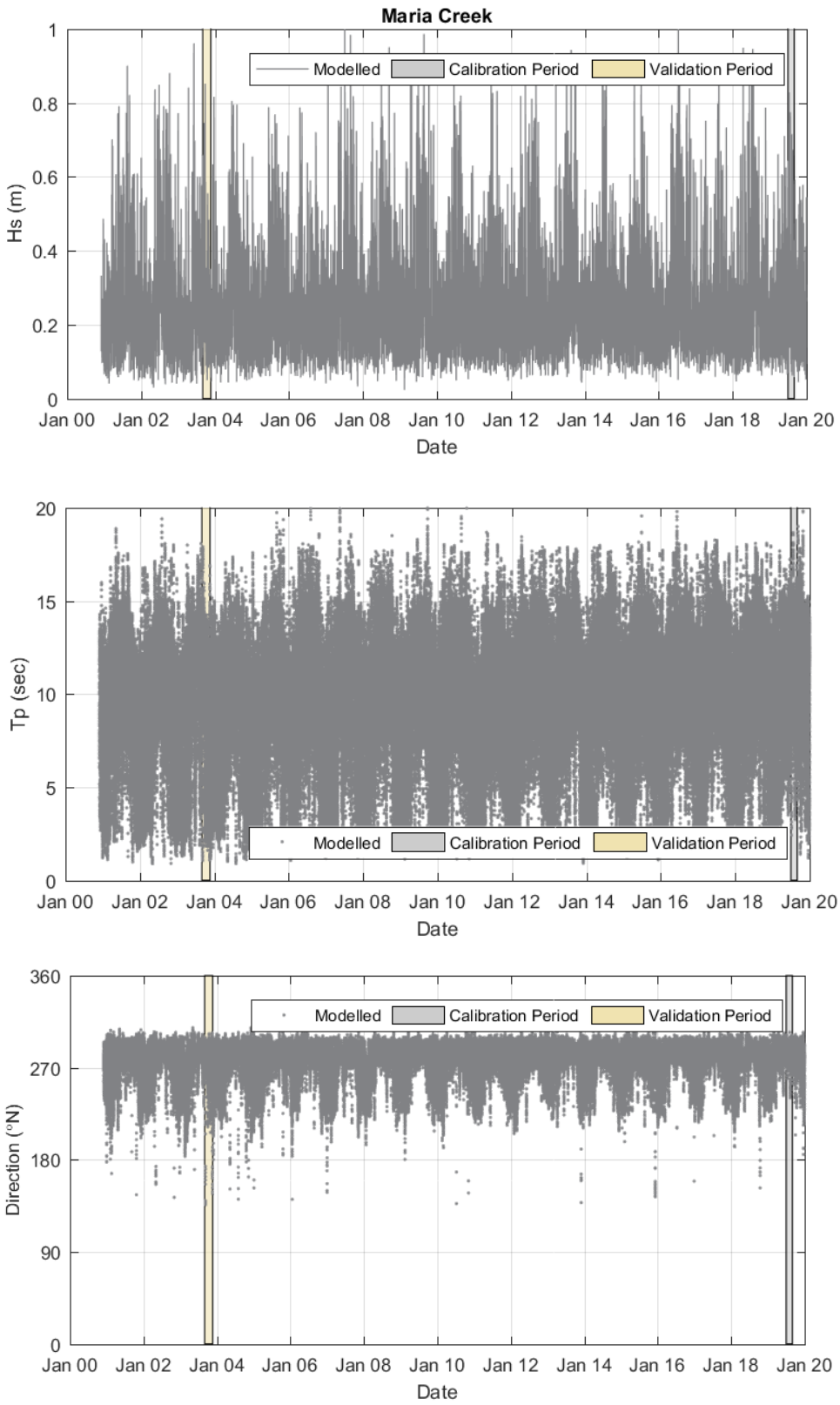


Figure 61. Timeseries of H_s (upper), peak wave period (Tp) (middle) and mean wave direction (lower) at Maria Creek.

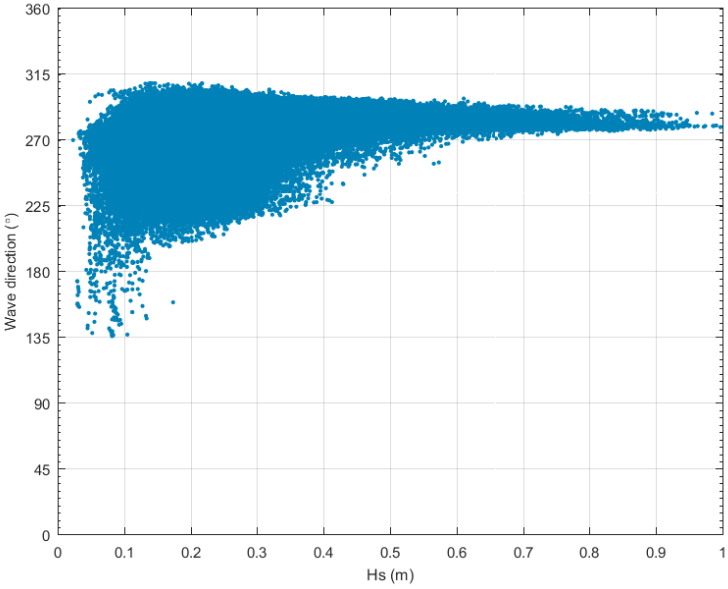


Figure 62. H_s versus wave direction at Maria Creek.

6.2. Wave Design Criteria

The annual recurrence interval (ARI) wave conditions (H_s) have been calculated for waves extracted at the Maria Creek extraction point (see Figure 59 for location), following Goda (2005).

In the absence of calibration data directly at Maria Creek, the sensitivity of the modelled waves to the key calibration parameters was assessed. The modelled waves in Lacepede Bay were found to be particularly sensitive to the bed friction (c_{fw}) applied in the model.

The application of a higher (rougher) c_{fw} (0.016 m) provided the best overall model performance at the Rivoli Bay calibration sites for ambient wave conditions, as well as the best agreement between modelled and measured wave heights at Cape Jaffa. However, it is possible that some of the peak wave heights during storm events could be underestimated when this higher c_{fw} is applied (see percentile statistics in Table 7). The application of a lower (smoother) c_{fw} (0.0115 m) was found to provide a better representation of the peak wave heights during storm events in Rivoli Bay (but overestimated ambient wave conditions) and in lieu of local calibration data it was considered preferable to adopt a more conservative approach using the lower c_{fw} when defining the local wave climate and ARI wave conditions. The effect of the applied c_{fw} on H_s at Beachport, Southend, Cape Jaffa and Maria Creek is shown in Figure 63 to Figure 66. These plots highlight the significant effect of the applied bed friction on wave heights at the study area. It is recommended that local measurements of wave conditions at Maria Creek (e.g. measurements adjacent to the Kingston Jetty) be obtained during winter months (when larger wave events occur) prior to any detailed design stages of the project to help reduce uncertainty in the modelled wave climate.

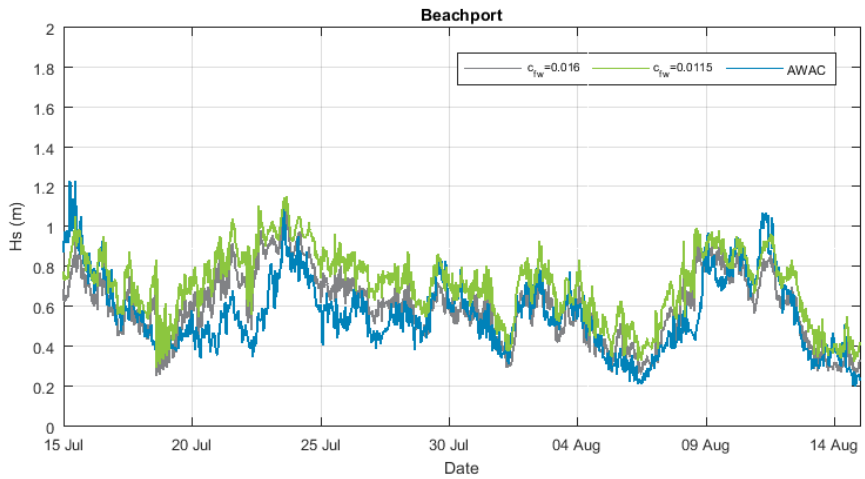


Figure 63. Effect of c_{fw} on H_s at Beachport.

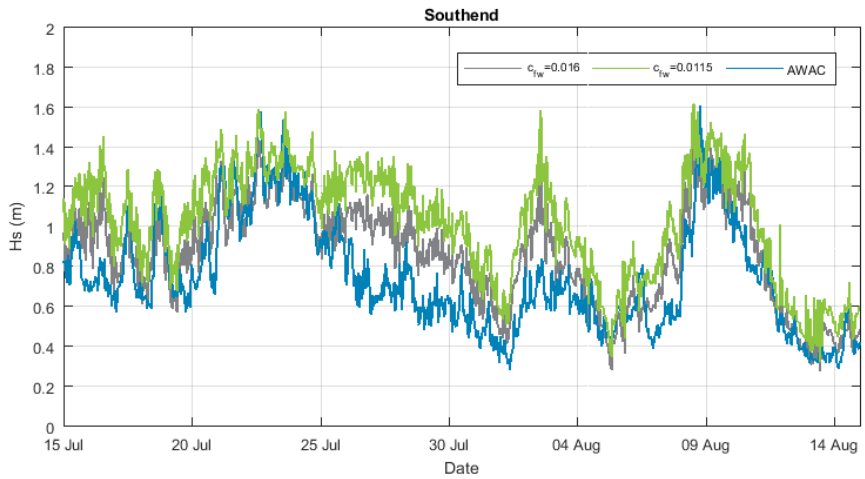


Figure 64. Effect of c_{fw} on H_s at Southend.

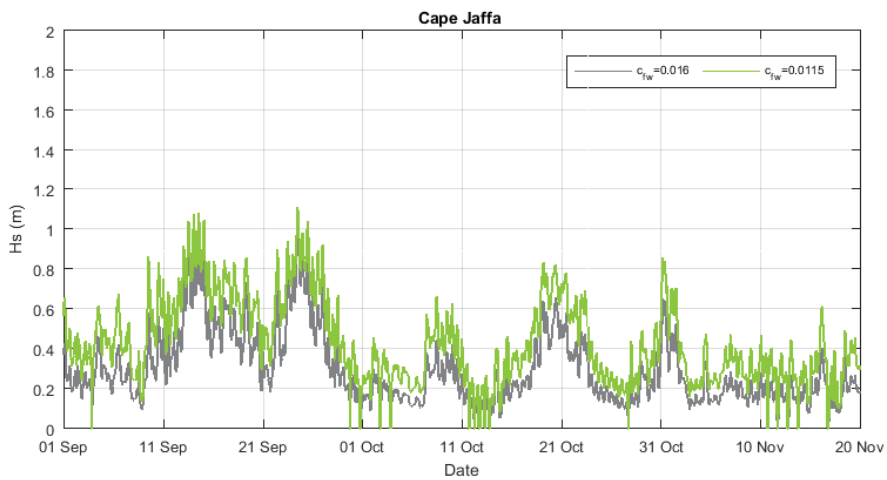


Figure 65. Effect of c_{fw} on H_s at Cape Jaffa.

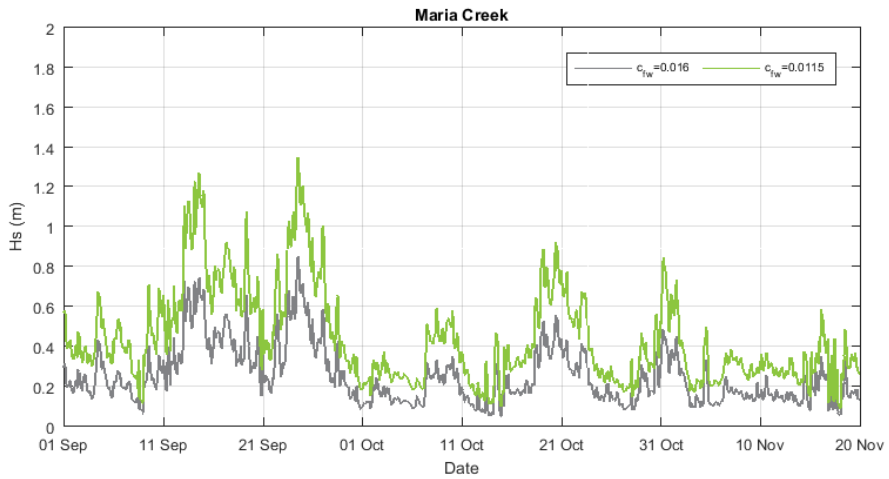


Figure 66. Effect of c_{fw} on H_s at Maria Creek.

Results from the ARI analysis for the lower (more conservative) c_{fw} model run are presented in Table 8. For completeness and to facilitate comparison with the other wave results presented, results from the ARI analysis for the higher c_{fw} model run are also presented (see Table 9).

The uncertainty in the ARI analysis has also been quantified and provided in Table 8 and Table 9. It should be noted that the confidence intervals only reflect uncertainty in the fit in the regression of the data when deriving the ARI conditions and do not include any indication of uncertainty in the modelled H_s. There is a high level of uncertainty in the ARI analysis for the longer recurrence intervals since the analysis is based on slightly less than 20 years of data. The following key observations can be drawn from ARI analysis:

- the conservative ARI analysis (based on the lower c_{fw}) indicates that a 1 in 1 year wave has an H_s of 1.42 m, while a 1 in 100 year wave has an H_s of 1.65 m (Table 8);
- the ARI analysis based on the higher c_{fw})indicates that a 1 in 1 year wave has an H_s of 0.89 m, while a 1 in 100 year wave has an H_s of 1.13 m (Table 9);
- there were 15 wave events larger than a 1 in 1 year ARI wave in the simulated wave data, three of these (including the largest event) occurred over the winter period of 2016; and
- since 2016, there has only been one wave event larger than a 1 in 1 year ARI wave.

The ARI analysis indicates that there does not appear to be an increase in storm activity over recent years, however 2016 was a notably stormy year. To examine the wave conditions in 2016 in more detail, percentiles of H_s have been calculated for each winter/spring period for the model simulation period (Figure 67), clearly demonstrating the anomalous extreme conditions compared to other years.

Table 8. ARI wave conditions at Maria Creek based on modelled waves from November 2000 to January 2020, using the lower c_{fw}.

Return Period (yrs)	Peak H_s (m)	98% Confidence Interval H_s (m)
1	1.42	1.40-1.44
5	1.53	1.49-1.56
10	1.56	1.51-1.61
20	1.59	1.53-1.65
50	1.63	1.55-1.70
100	1.65	1.56-1.74

Table 9. ARI wave conditions at Maria Creek based on modelled waves from November 2000 to January 2020, using the higher c_{fw}.

Return Period (yrs)	Peak H_s (m)	98% Confidence Interval H_s (m)
1	0.89	0.87-0.91
5	0.99	0.95-1.02
10	1.02	0.97-1.07
20	1.06	1.00-1.12
50	1.10	1.03-1.17
100	1.13	1.05-1.22

Table 10. Wave events greater than 1 in 1 year (based on the lower c_{fw}).

Date	Hs (m)	Tp (sec)	Peak Dir (deg)
12/07/2016 14:30	1.63	14.4	281
04/07/2007 17:00	1.55	14.7	280
15/09/2008 12:30	1.52	14.2	280
24/07/2016 17:00	1.52	15.3	278
23/06/2014 11:00	1.52	13.8	279
21/01/2007 02:30	1.51	14.3	281
01/07/2009 08:30	1.51	14.4	280
07/08/2001 16:00	1.51	11.7	284
28/06/2014 12:30	1.50	13.8	281
17/07/2018 15:30	1.48	15.2	279
15/08/2010 03:00	1.48	13.4	282
18/08/2013 12:30	1.47	15.0	279
29/09/2016 20:00	1.46	12.5	279
25/04/2009 14:30	1.46	12.7	279
01/08/2014 07:00	1.45	13.7	279

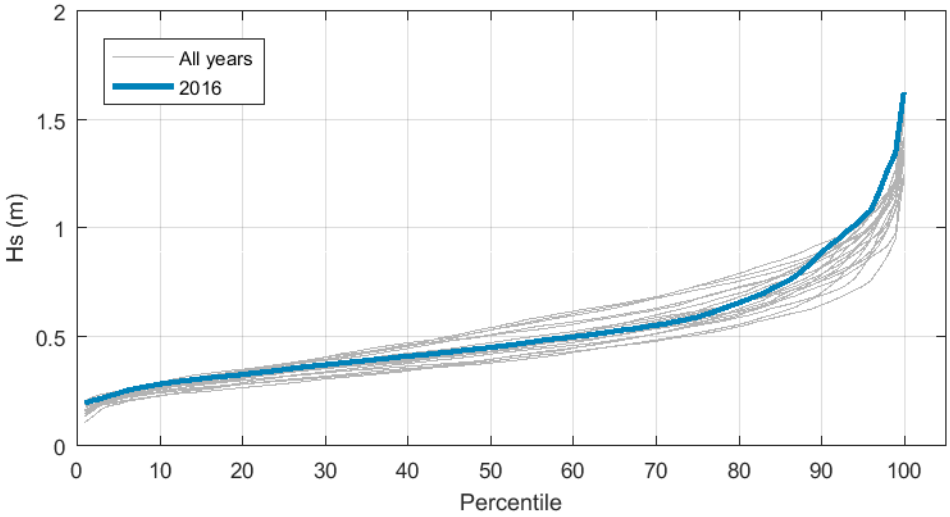


Figure 67. Percentile wave heights for July to September for each year. *Results are shown for the lower c_{fw}.*

Time series of H_s, T_p and wave direction during the three month period between July and September 2016 inclusive are shown in Figure 68. During this period there were three storm events when H_s exceeded the 1 in 1 year ARI wave height. An example map plot of the wave heights and directions during peak wave conditions on the 24th July 2016 (approximately equivalent to a 1 in 5 year ARI wave condition) are shown in Figure 69. The plots shows how the waves lose a significant amount of energy as they travel from offshore across the relatively shallow area of Lacepede Bay (where depths reduce to less than 100 m and the long period waves start to be influenced by the bed) and how the waves refract into the southern end of Lacepede Bay.

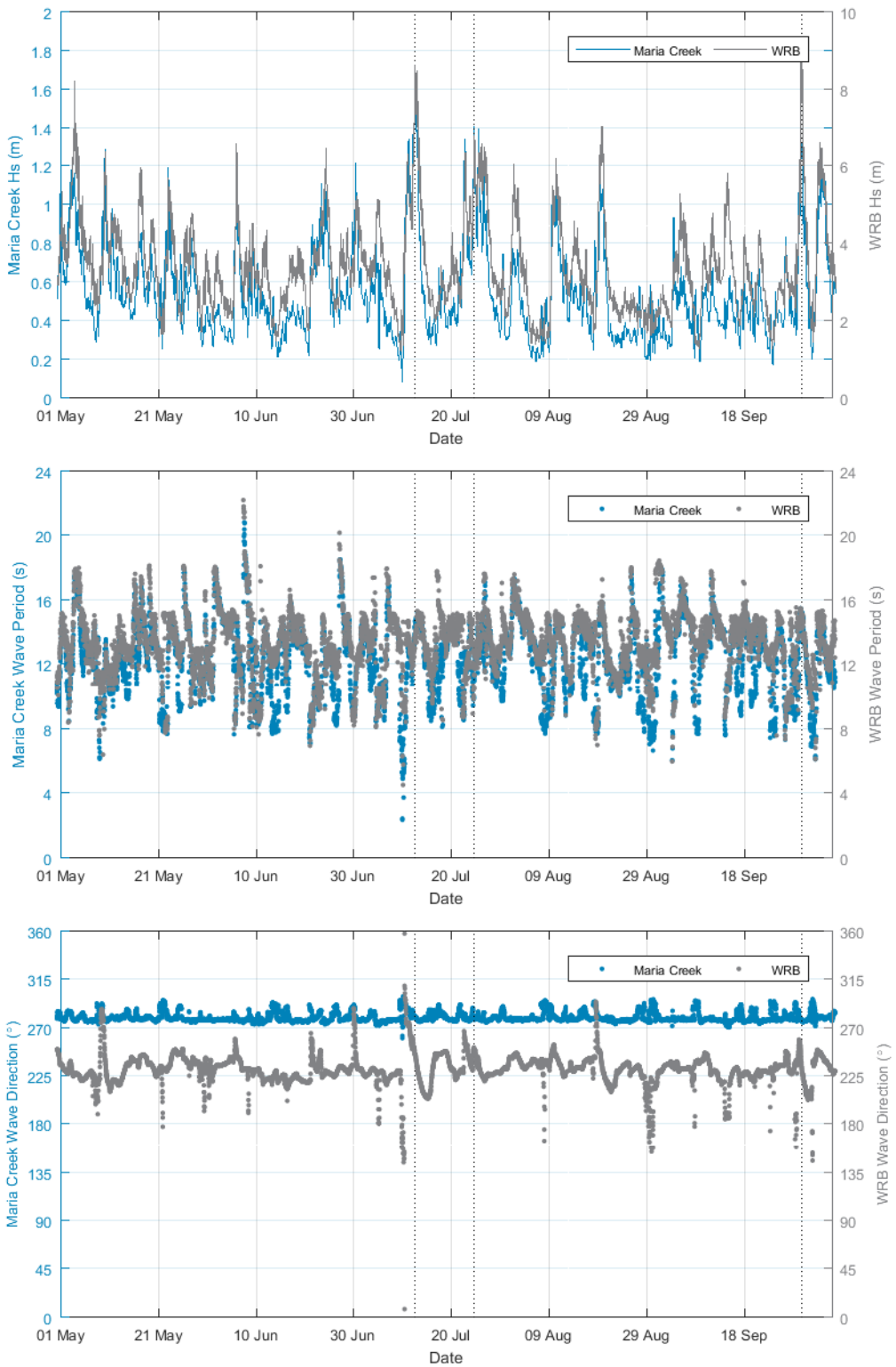


Figure 68. Extreme wave events in 2016 (greater than 1 in 1 year ARI) showing Hs (upper), wave period (middle) and direction (lower).

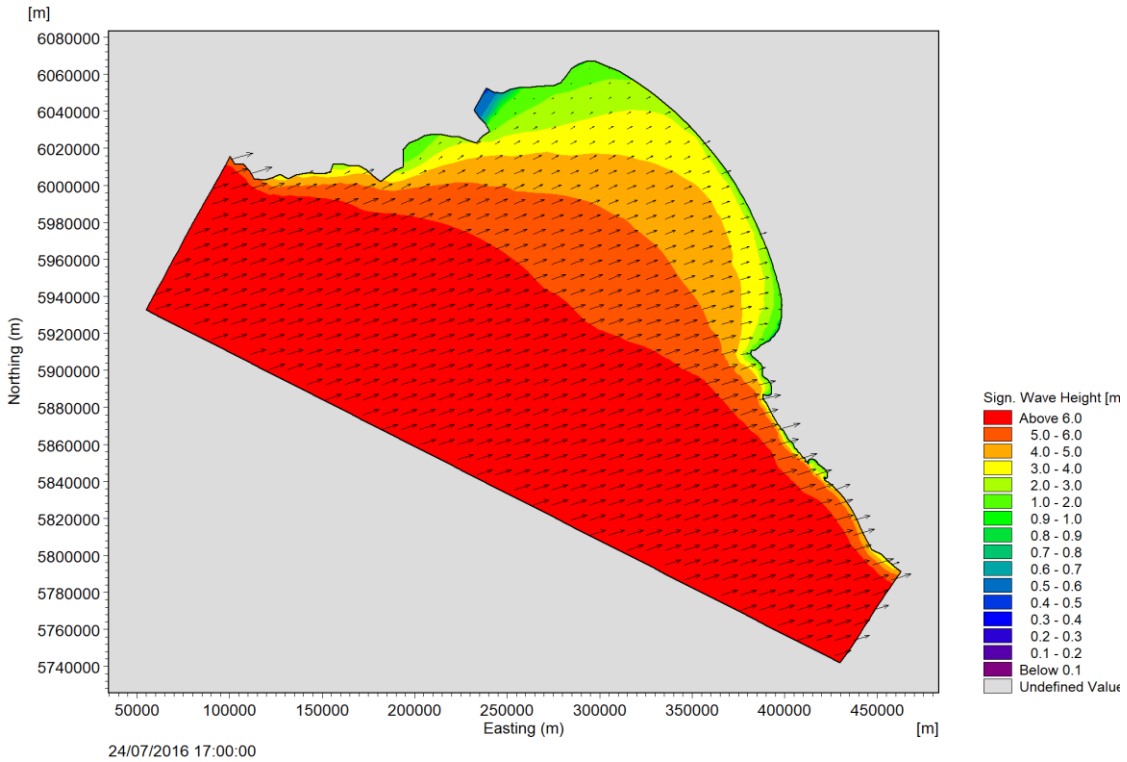


Figure 69. Map plot of modelled waves for 12 July 2016, regional scale.

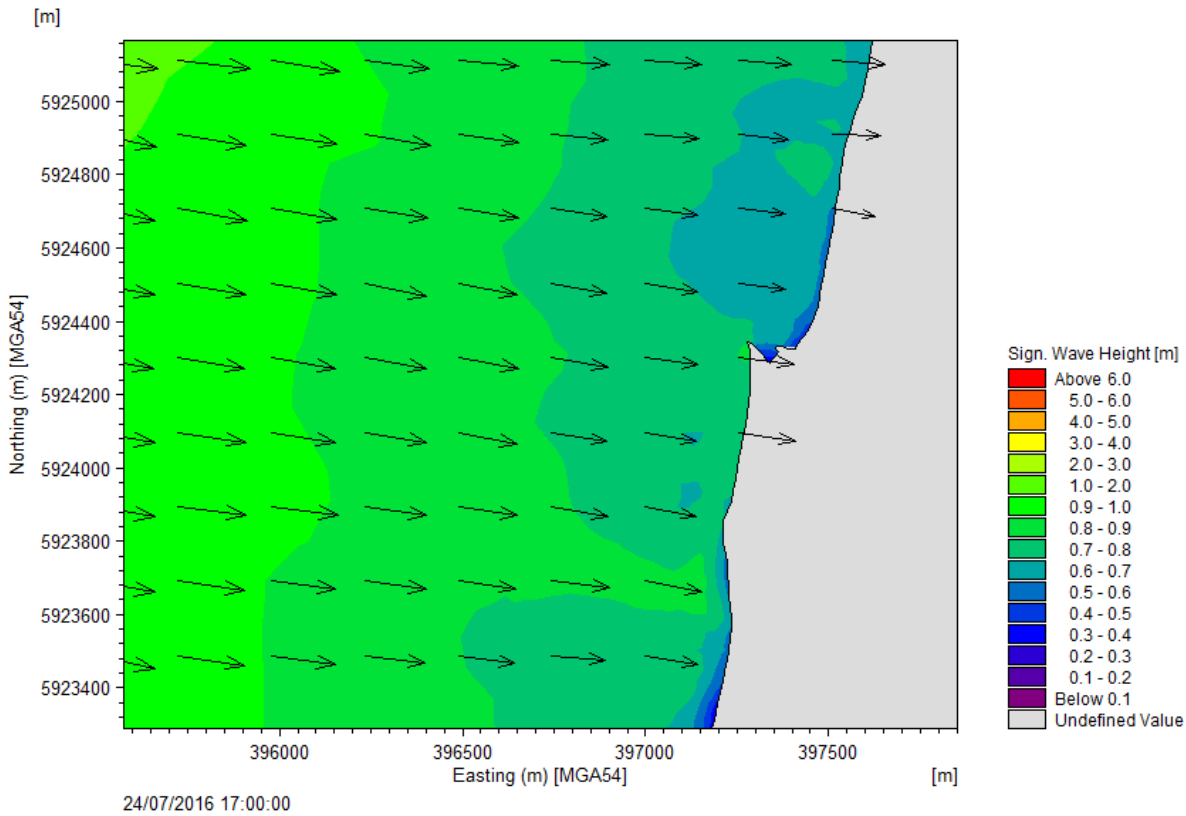


Figure 70. Map plot of modelled waves for 12 July 2016, local scale.

7. Longshore Sediment Transport Modelling

There is the potential for sediment to be transported along the shoreline by the action of waves approaching the coast at an angle (i.e. not perpendicular to the shore) and/or by the action of tidal or wind driven flows.

7.1. Method

There are a number of different formulae available to calculate longshore transport. In this study, the Kamphuis (1991) method was adopted to calculate the wave driven longshore sediment transport as this method has been found to provide the most realistic longshore drift predictions (e.g. Wang et al. (2002) and Shanas and Kumar (2014)). The method uses the twenty years of modelled wave conditions (H_s, T_p and direction) and the grain size, seabed slope and coastline orientation to determine the longshore sediment transport rates.

The contribution of tidal and wind driven flows to the sediment transport has been assessed by applying the formulae (Equations 10 and 11) from Van Rijn (2013).

The longshore transport has been calculated at four locations along the coast at Maria Creek (see Figure 71 for locations):

- P1: immediately to the south of the Kingston Jetty (approximately 400 m south of the southern training wall);
- P2: 50 m south of the southern training wall;
- P3: 50 m north of the northern training wall; and
- P4: 200 m north of the northern training wall.

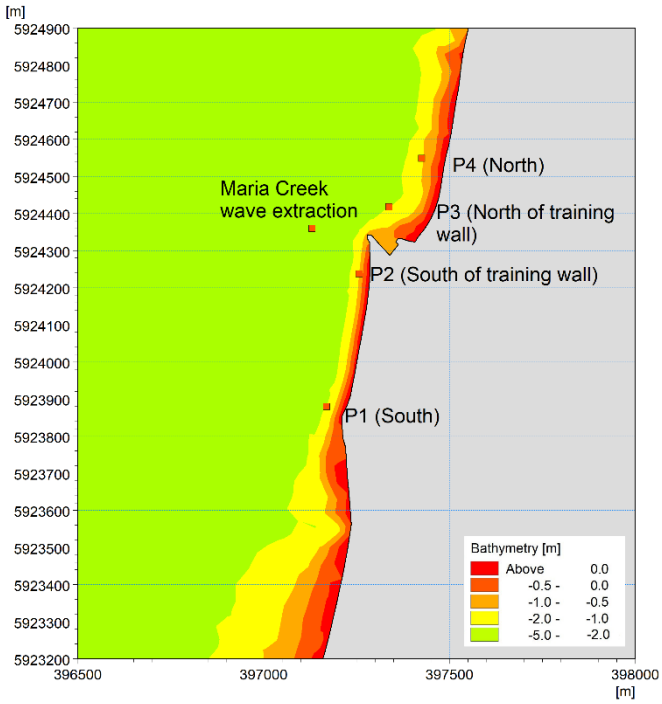


Figure 71. Extraction points for longshore sediment transport calculations.

Before considering the results, it should be noted that the calculated transport rates are potential transport rates, since they assume that there is an unlimited supply of sediment. If there is not sufficient sediment supply, then the actual transport rates which occur will be lower. Further, the model does not account for any changes to shoreline orientation or

sediment grain size which could have occurred over the twenty year period of analysis. The present day shoreline position and bathymetry have been applied throughout the period (representing the longshore sediment transport without sand and wrack management). Any naturally occurring changes to the shoreline orientation are unlikely to have a significant effect on the longshore transport rates as the offshore area is also likely to change allowing the waves to refract so that their relative approach to the coastline remains unchanged. However, following periods when sediment has been anthropogenically removed from the beach, but not from further offshore, the changes to longshore transport could be more significant (due to an increase in the incident angle that waves approach the beach).

7.2. Results

7.2.1. Wave Driven Sediment Transport

The annual mean wave driven longshore sediment transport calculations for the 20 year period between 2000 and 2020 are shown in Figure 72 and tabulated in Table 11. The results show the following:

- there is a clear northerly dominance in the longshore transport of sand at all four points, with very little southerly transport predicted;
- the highest transport rates are typically predicted to be at the locations furthest from the training walls (P1 and P4), while closer to the training walls (P2 and P3), the transport rates are lower. The lower transport rates at P2 are a result of the accumulation of sediment along the south beach, which has resulted in a reorientation of the shoreline so that waves are approaching slightly closer to shore normal. Similarly at P3, the effect of the training walls on sedimentation at the north beach and on wave refraction results in a small reduction of the longshore sediment transport; and
- the transport rates vary between years, with the highest transport rates predicted in 2016 and 2018. The rates are predicted to vary from around 9,000 to 15,000 m^3/yr at P2 and from around 14,000 to 22,000 m^3/yr at P4. As noted, the model does not account for any changes to the bathymetry or shoreline orientation and as such the variability in transport rates could be underestimated, with higher longshore transport expected for (and following) years when dredging has removed beach sediment but not changed the offshore area where waves are refracting and breaking.

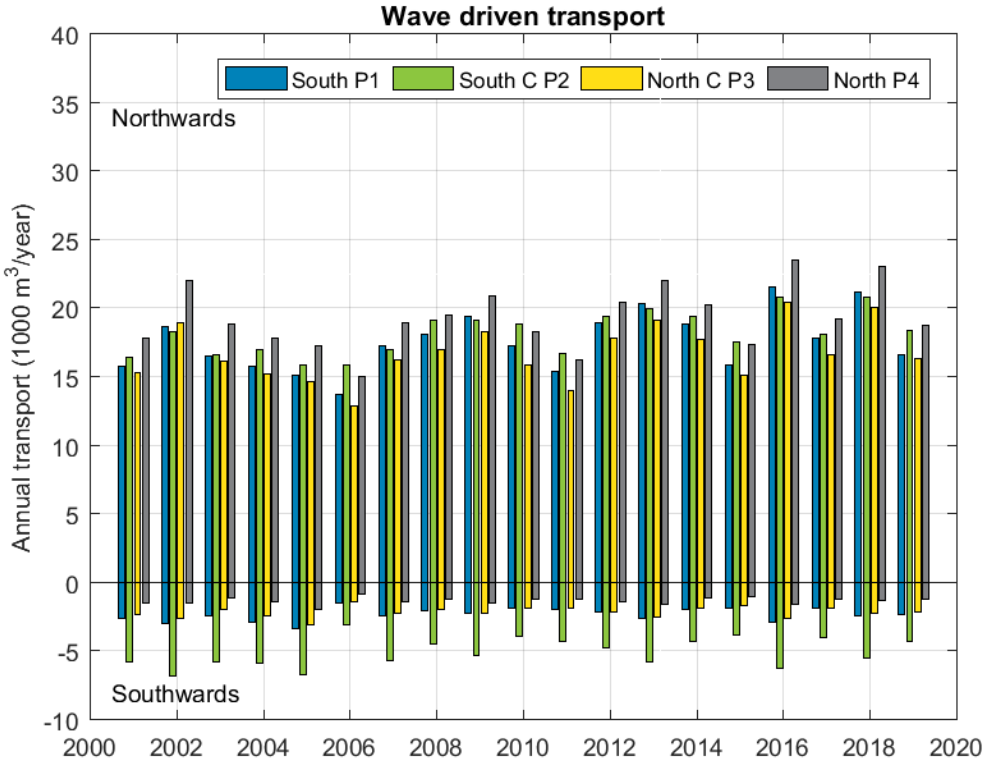


Figure 72. **Predicted wave driven annual longshore sediment transport (based on Kamphuis (1991)).**

Table 11. **Predicted wave driven annual net longshore sediment transport rates.**

Year	Wave Driven Net Transport (1,000 m³/year), Kamphuis (1991)			
	P1	P2	P3	P4
2001	13.1	10.6	12.9	16.3
2002	15.6	11.5	16.2	20.5
2003	14	10.8	14.1	17.6
2004	12.8	11	12.8	16.4
2005	11.7	9.1	11.5	15.3
2006	12.2	12.7	11.4	14.2
2007	14.8	11.3	13.9	17.5
2008	16	14.7	14.9	18.3
2009	17.1	13.8	16	19.4
2010	15.4	14.9	14	17.1
2011	13.4	12.3	12.1	15
2012	16.7	14.6	15.7	19
2013	17.7	14.2	16.6	20.4
2014	16.9	15	15.8	19.1
2015	13.9	13.7	13.3	16.3
2016	18.6	14.4	17.8	21.9
2017	15.9	14	14.7	18
2018	18.7	15.2	17.8	21.7
2019	14.2	14.1	14.1	17.5

The predicted daily wave driven longshore transport rates over the entire 20 year period and during the 2016 winter period (when the highest annual longshore transport occurred) is shown in Figure 73. The plot shows seasonality, with highest daily transport occurring in the winter to spring months (between May and October) and lower daily transport in the summer to autumn months (between November and April). During the winter period of 2016, peak daily wave driven sediment transport was up to 700 m^3/day at the time of the largest wave event in July 2016. Over the three month period between July and September in 2016 there were six separate events which resulted in peak daily transport rates of more than 300 m^3/day, with two of these exceeding 500 m^3/day. There was also a high transport rate of more than 300 m^3/day at the start of May 2016 which coincides with the highest water level event to occur in the 20 year model simulation period (see Section 2.1). This highlights how the wave conditions during the 2016 period were more severe than during other years, and it is possible that the occurrence of multiple events over a short period of time (potentially combined with elevated water levels for some of these events) could have resulted in erosion of the Wyomi dunes releasing a large volume of sand close to Maria Creek.

The longshore sediment transport rates derived in this study are broadly similar to those presented in The Cape Jaffa Marina Assessment of Coastal Processes and Impact report produced by WBM Oceanics Australia (WBM, 2005) (Section 2.5.3).

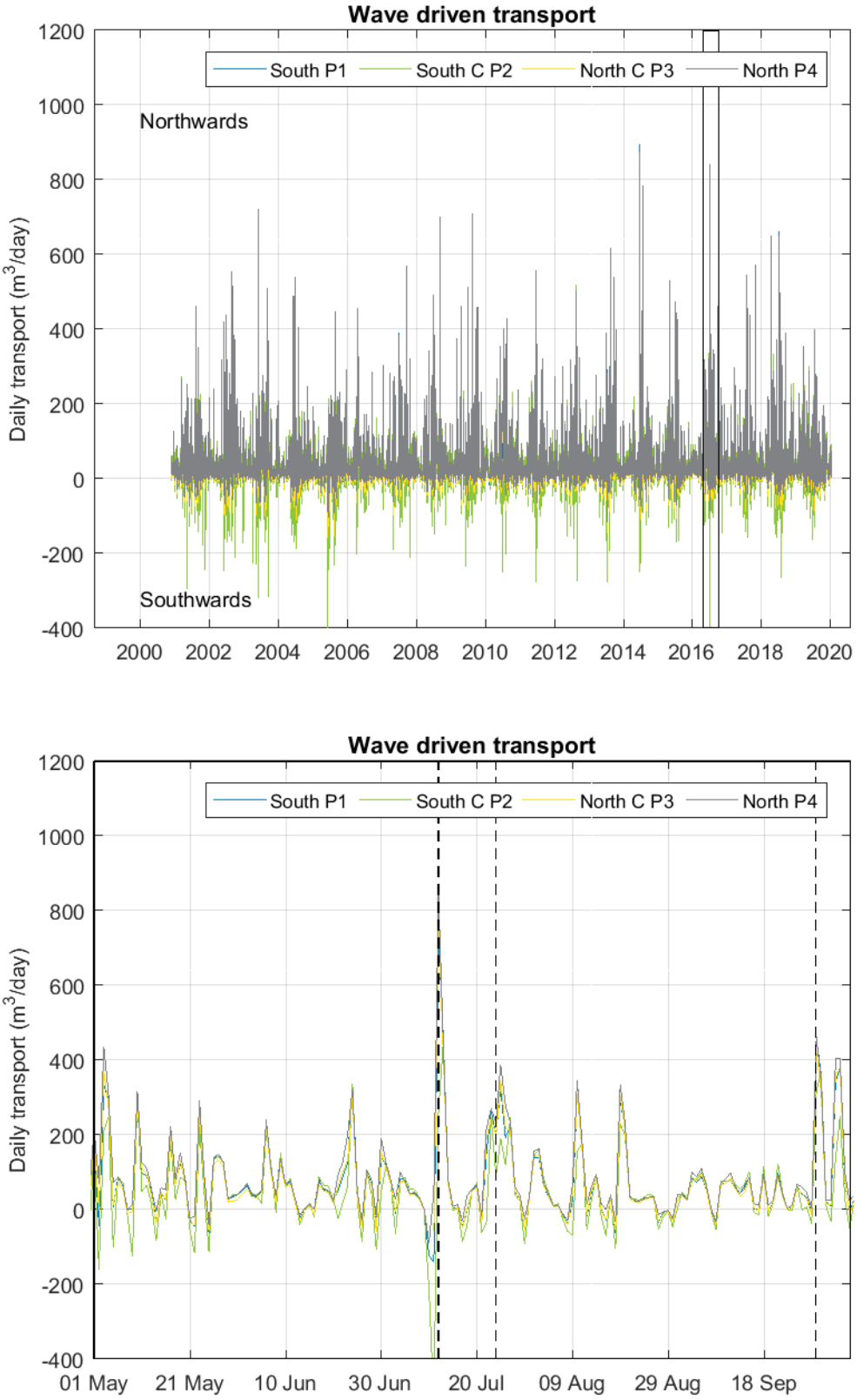


Figure 73. Daily wave driven longshore sediment transport for full simulation period (upper) and 2016 period (lower), based on Kamphuis (1991) formula.

7.2.2. Flow Driven Sediment Transport

Based on results from the hydrodynamic modelling, there is potential for the tide and wind driven flows to contribute to the longshore sediment transport due to the dominance of currents in a northwards direction. The longshore transport formulae proposed in Van Rijn (2013) for combined wave and tide/wind driven flows have been applied as part of sensitivity testing to the longshore transport rates to determine the likely contribution of the tide/wind driven flows to the longshore sediment transport. The results show the following:

- calm wave conditions (H_s < 0.2 m): during periods with calm wave conditions, which occur for approximately 50% of the time at Maria Creek, the nearshore wind/tidal currents at Maria Creek could result in additional longshore transport rates of up to 10 m^3/day in either a northward or southward direction;
- moderate wave conditions (H_s = 0.5 m): during periods with moderate wave conditions, which are only exceeded for approximately 5% of the time, the nearshore wind/tidal currents at Maria Creek could result in additional longshore transport rates of up to 150 m^3/day in either a northward or southward direction; and
- extreme wave conditions (Hs > 0.9 m): during periods with extreme wave conditions, which only occur roughly once a year, the nearshore wind/tidal currents at Maria Creek could result in additional longshore transport rates of up to 500 m^3/day in either a northward or southward direction. However, these events are typically associated with strong west to south-westerly winds and so the transport would be expected to be increased in a northwards direction.

The sensitivity testing detailed above shows that the nearshore currents at Maria Creek have the potential to influence the longshore transport rates in the region. The influence of the tide and wind driven flows is small during periods of calm wave conditions, but during periods with moderate or extreme wave conditions the potential influence of the flows (tide and wind driven) could increase the longshore transport rates by more than 50%.

7.2.3. Combined Net Sediment Transport

To quantify the annual net combined wave and flow driven sediment transport, estimates of flow driven transport have been derived by applying the following assumptions:

- the tidal flows do not contribute to the longshore transport. This is a reasonable assumption in view of the low tidal flows and the negligible effect of flow driven transport during calm conditions; and
- when wind speeds exceed 6 m/s they drive an alongshore flow, this is to the south for winds from the north to north west sector and to the north when winds are from the west to south sector.

Annual rates of the combined flow and wave driven transport are shown in Figure 74 and tabulated in Table 12. Comparison of these against the wave driven sediment transport rates (Figure 72 and Table 11), confirms the significance of the contribution of flows in driving longshore sediment transport at Maria Creek. The magnitude of this contribution varies between years and locations, but is typically around 20% (but can be up to 60%) of the wave driven longshore transport.

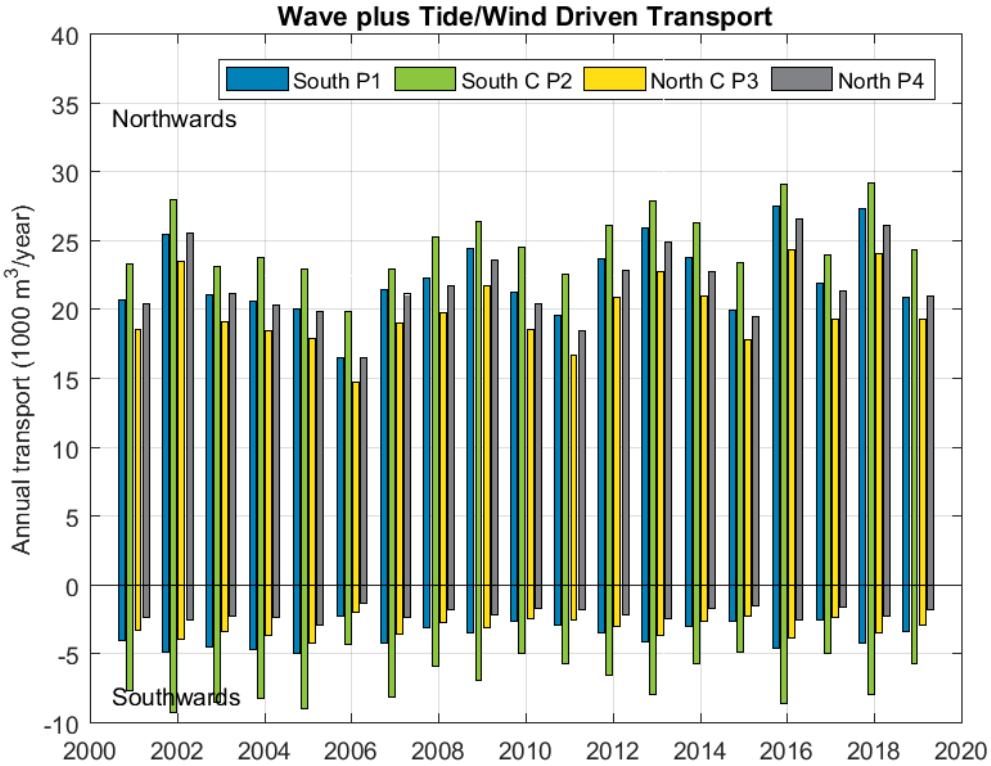


Figure 74. Predicted combined wave and flow driven annual longshore sediment transport (based on Van Rijn (2013)).

Table 12. Predicted combined wave and flow driven annual net longshore sediment transport rates.

Year	Wave and Flow Driven Net Transport (1,000 m³/year), Van Rijn (2013))			
	P1	P2	P3	P4
2001	16.7	15.6	15.3	18.1
2002	20.6	18.6	19.5	23.0
2003	16.6	14.5	15.8	18.9
2004	15.9	15.5	14.7	17.9
2005	15.1	13.9	13.7	16.9
2006	14.2	15.5	12.7	15.1
2007	17.2	14.8	15.4	18.7
2008	19.2	19.3	17.0	19.9
2009	21.0	19.4	18.6	21.4
2010	18.6	19.5	16.1	18.7
2011	16.6	16.9	14.1	16.6
2012	20.2	19.5	17.9	20.7
2013	21.8	19.9	19.1	22.4
2014	20.8	20.5	18.3	21.0
2015	17.3	18.5	15.5	18.0
2016	22.9	20.5	20.5	24.0
2017	19.4	18.9	17.0	19.7
2018	23.1	21.3	20.5	23.8
2019	17.4	18.7	16.3	19.2

Wavelength (2020b) derived an indicative sediment budget for the period March 2016 to October 2018 using beach profile data, a Digital Elevation Model (DEM) based on LiDAR data and Google Earth aerial photography. Their analysis indicated a net northerly longshore transport of sand in the order of 70,000 m^3 from March 2016 to October 2018, with an additional 100,000 m^3 also transported to Maria Creek as a large sand slug from storm erosion at Wyomi beach. The annual mean transport based on the modelled waves and the Kamphuis formula for this period is 15,000 to 21,000 m^3/year, with a net transport rate of 40,000 m^3 calculated between March 2016 and October 2018 adjacent to the southern side of the Maria Creek training walls. When the effects of flows are also included, the net transport for this period was calculated to be 55,000 m^3 adjacent to the southern side of the training walls. While this predicted rate is low relative to the results derived from the indicative sediment budget, there is uncertainty in the sediment budget values due to the limited available data and large number of assumptions required to estimate volumes and subsequent rates. In addition, the predicted transport rates in the model are based on the present day bathymetry and coastline orientation. Following periods of removal of beach sediment to the south of the Maria Creek training walls, the bathymetry and shoreline orientations would have been different and higher longshore transport than modelled based on the present day shoreline position could have occurred.

Further, it is important to note that previous research has found that longshore transport formulae are not able to replicate the higher longshore transport rates associated with the transport of sand slugs (BMT WBM, 2011) and in view of this, and the differences noted above, the sediment transport rates derived using the Kamphuis and Van Rijn formulae are considered to correspond well with the indicative sediment budget results and the method provides a useful tool for relative comparisons of transport under various options.

7.2.4. Entrance Channel Stability

Prior to the completion of the training walls at Maria Creek in 1996, the stability of the entrance channel to Maria Creek would have been controlled by the balance between the rate of sediment supply to the entrance and the flows through the channel. Wave action in Maria Creek is the dominant process which would have driven the longshore transport of sediment to the entrance, acting to close the entrance through deposition within the channel, while the flows through the channel will act to erode sediment and maintain the inlet channel cross-sectional area. An empirical relationship derived by Bruun and Gerittsen (1960) and subsequently refined by Gao and Collins (1994) was applied to determine the stability of the Maria Creek entrance channel. Results from the hydrodynamic modelling show that the tidal prism of the Creek during a large spring tide is in the order of 70,000 m^3 and as noted above the longshore transport rate is predicted to range from 15,000 to 25,000 m^3/yr. Based on this the entrance has a stability criterion value of between 4 and 6 which relates to a very unstable entrance acting mainly as an overflow channel (i.e/ predominantly closed). This appears to correspond to how the channel behaved prior to construction of the training walls. In order for a channel to be expected to be present for the majority of the time without training walls, the tidal prism of the Creek would need to be an order of magnitude larger.

8. Conceptual Understanding

In this section, results from the preceding sections are brought together to present a conceptual understanding on the key physical coastal processes currently acting in and around the study area. The conceptual understanding is discussed below and a schematic representation of the key processes driving sediment transport in Maria Creek region is shown in Figure 75.

The foreshore along the northern section of Lacepede Bay is sandy and backed by sandhills. Further south, towards Kingston (on the south side of the entrance to Maria Creek), the land is more low lying and the vegetation more swampy. Lacepede Bay is a relatively shallow bay which predominantly exhibits low wave energy due to the shoaling and refraction of waves which occurs. The area is dominated by long period waves which typically approach the coast slightly south of shore normal. Based on the numerical wave modelling undertaken as part of this study the nearshore extreme significant wave heights at Maria Creek vary from 1.42 m for a 1 year ARI and up to 1.65 m for a 100 year ARI. However, the wave model results at Maria Creek were found to be very sensitive to the bed friction value adopted in the model (the ARI wave conditions represent what is thought to be the upper range of possible wave heights at Maria Creek) and it is therefore suggested that local wave measurements are collected at Maria Creek during winter months prior to any detailed design being undertaken.

In terms of tidal regime, the study area lies within a micro-tidal environment with low tidal flows (in the order 0.2 m/s or less) orientated parallel to the coast. There is a dominance in tidal currents to the north during calm conditions due to a large-scale eddy which forms in Lacepede Bay and results in northward currents throughout the tidal cycle during spring tides. Wind induced currents can occur during periods of strong winds and under certain conditions the resultant currents can dominate the nearshore currents in the Maria Creek region. When strong winds occur from a southerly through to westerly direction the winds can significantly increase the nearshore currents in a northward direction (strong winds (>8 m/s) occur from these directions for 12% of the time). When strong winds occur from a northerly to north-westerly direction (strong winds (>8 m/s) occur from these directions for 1% of the time) the dominant northward currents can be reversed resulting in southward currents.

At Maria Creek the currents are diverted around the training walls to flow into and out of the Creek. The currents in the Creek are higher than the currents offshore (still generally less than 0.3 m/s) due to the constriction of the Creek banks inland of the boat ramp resulting in a flow acceleration. In the mouth of Maria Creek there is a slight flood dominance in the currents (both speed and discharge), which indicates that the Creek will typically act as a net importer of both sediment and wrack. Without the training walls in place the Creek channel would be expected to be very unstable, with it mainly being closed and acting as a small overflow channel. The tidal prism of the Creek would need to be approximately an order of magnitude larger for the entrance channel to be more stable.

The dominant sediment transport process in the Maria Creek region is wave action. Although the wave heights are relatively low in the area they still have the potential to drive the longshore transport of sediment in the nearshore region, with annual potential wave driven transport rates predicted to be in the region of 10,000 to 20,000 m^3/yr. Analysis has also shown that the nearshore tidal and wind-induced currents which can occur in the Maria Creek region could also influence the longshore transport rates during periods with larger wave conditions (less than 5% of the time), having the potential to result in more than a 50% increase in the potential longshore transport rates with the majority of this increased transport being in a northward direction, although some increased transport to the south could also occur during periods of strong northerly winds.

The area immediately within the entrance to Maria Creek, and especially within the southern side of the entrance, is sheltered from waves, meaning that any sediment or wrack which is

transported there by waves and tidal/wind-driven currents during the flood stage of the tide is expected to be deposited and will be unlikely to be remobilised. It is possible that during occasional high freshwater discharge events, the increased flow at the entrance to the Creek could result in some scour of deposited sediment, although it is likely that any mobilised sediment would subsequently be redeposited close to the mouth of the Creek in an ebb bar formation.

The training walls either side of the entrance to Maria Creek act as groynes, interrupting the longshore transport of sediment and wrack along the coastline. From when the training walls were completed in 1996 to the end of 2015 a limited amount of sediment accumulated on the south beach, as regular sand bypassing (which was carried out until the end of 2015) was used to manage any build-up of sediment adjacent to the training walls. During 2016 significant sedimentation occurred on the south beach and since this time ongoing sedimentation has continued. This highlights how the sand bypassing successfully managed the sedimentation adjacent to the southern Maria Creek training wall on the south beach pre-2016 and that without any sediment management there will be an ongoing build-up of sediment in this area.

In 2016 the combination of multiple extreme wave events combined with extreme water level events (the May 2016 event resulted in a significant wave height of approximately a 1 year ARI as well as the largest measured water level from 2000 to 2018) appear to have resulted in extensive erosion of beaches and dune systems to the south of Maria Creek (e.g. the Wyomi region), which released a large volume of highly mobile sand into the intertidal region. Due to the proximity of Maria Creek relative to where some of the sediment was eroded from (Wyomi), a sand slug was created which meant that the actual longshore transport rates were significantly higher than the predicted potential longshore transport rates, resulting in a significant build-up of sediment on the south beach over approximately a 6 month period following the initial erosion event (May 2016). Since 2016, there has not been additional significant erosion of the shoreline to the south of Maria Creek and so no further sand slugs have occurred. Therefore, the longshore transport rates have returned to rates similar to those predicted by the longshore transport calculations, meaning that there has been a continuing supply of sediment to Maria Creek from the south. At Maria Creek this has meant that the sedimentation on the south beach has continued over time and now the beach has grown beyond the offshore end of the structure and a sandbar has formed across the mouth of Maria Creek connecting to the northern training wall. The area of north beach adjacent to the northern training wall is sheltered from larger waves and therefore acts as a sediment sink.

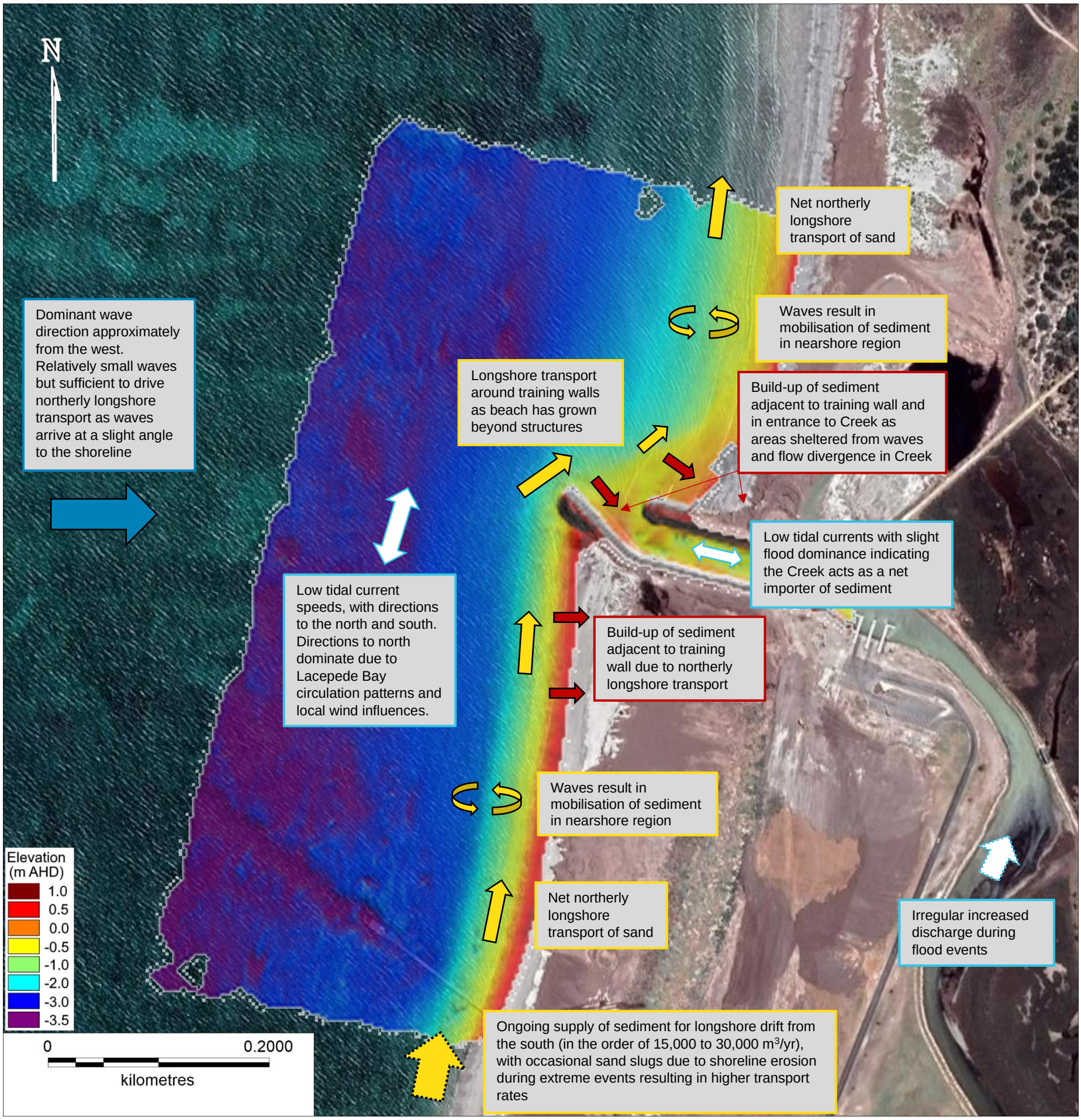


Figure 75. Schematic representation of the conceptual sediment transport understanding for Maria Creek.

9. Concept Design

The hydrodynamic, wave and longshore sediment transport modelling tools have been applied to assess a number of concept designs which have been put forward by Wavelength as potential options to help ease the currently untenable costs associated with maintaining access to the Maria Creek boat ramp. A total of four concepts were assessed, these are discussed in the following sections.

9.1. Concept 1: Ongoing Management

This Concept assumes no change to the Maria Creek training walls and aims to test the impacts of dredging the Creek and bypassing a large amount of the sediment which has built up on the south beach. The key elements of Concept 1 are detailed below and are shown on the HD model mesh[1] in Figure 76:

- a large dredging campaign to -2.7 m AHD from within the Creek and adjacent to the entrance channel;
- a large excavation campaign on south beach extending to the jetty;
- a small dredging campaign to the north of the Creek entrance to -2.7 m AHD; and
- placement of the dredged/excavated material on the northern side of the northern training wall.

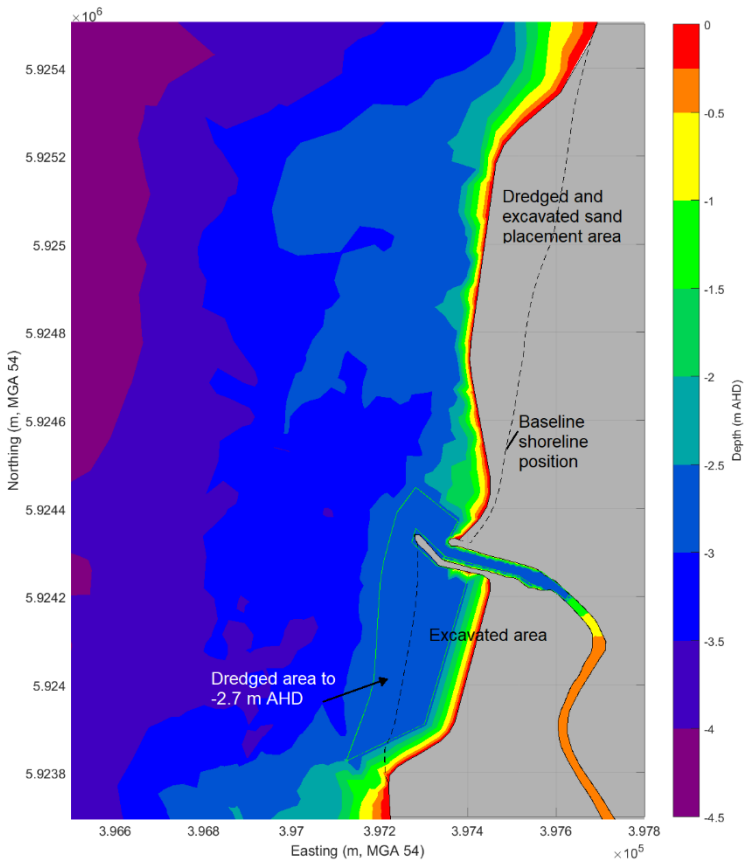


Figure 76. Concept 1: Ongoing management approach model configuration.

[1] the SW model mesh was also updated to include the key elements of the Concept 1 design but does not include the Creek (since this area is sheltered from waves).

9.2. Concept 2: Training Wall Extension

This Concept aims to reduce the exposure of the entrance channel to waves and wind induced currents from the west and north-west. The key elements of Concept 2 are shown on the HD model mesh[2] in Figure 77 and include the following:

- removal of approximately 80 m of the southern training wall;
- an approximate 250 m extension of the southern training wall to the west and north-west;
- an approximate 60 m extension of the northern training wall to the west;
- a dredging campaign to -2.7 m AHD within the Creek and adjacent entrance channel (width of 20 m);
- a small dredging campaign to the north of the Creek entrance to -2.7 m AHD; and
- placement of the dredged/excavated material on the northern side of the northern training wall.

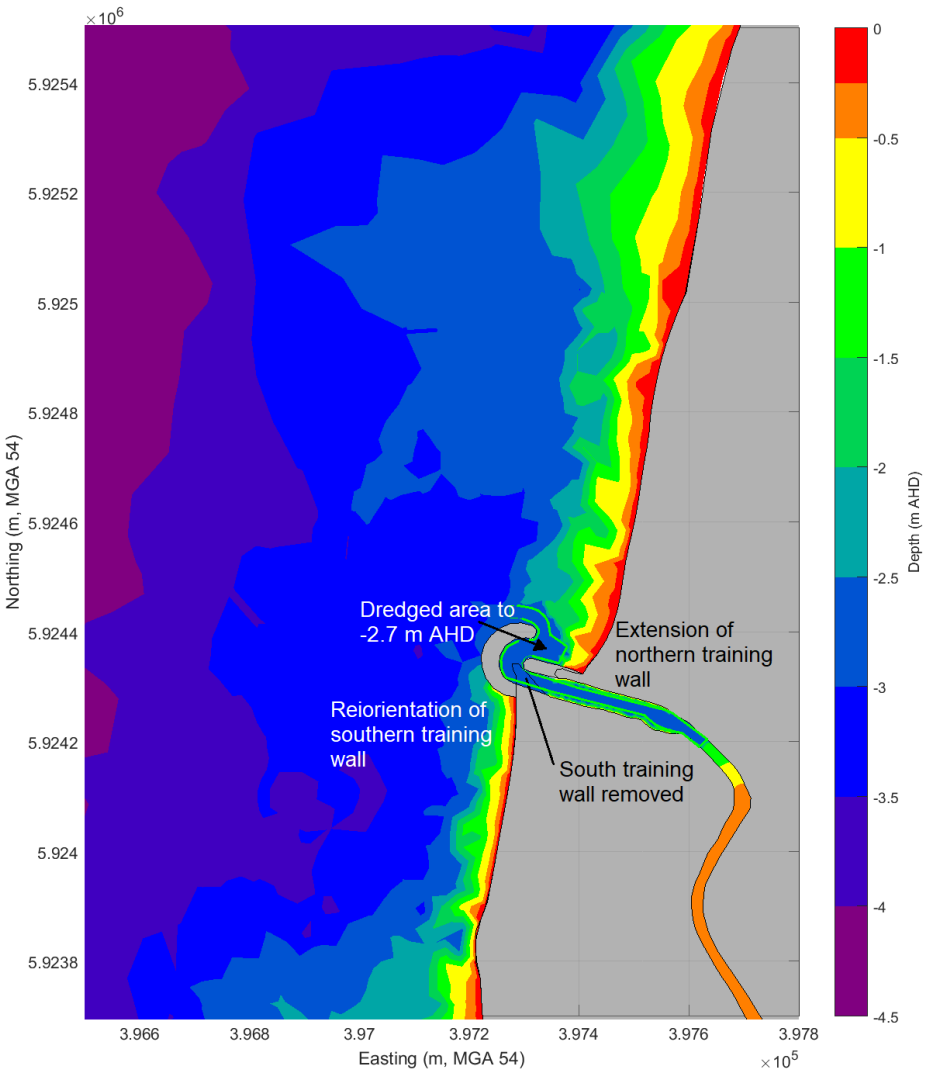


Figure 77. Concept 2: Training wall extension configuration.

[2] the SW model mesh was also updated to include the key elements of the Concept 2 design but does not include the Creek (since this area is sheltered from waves).

9.3. Concept 3: Narrowed Entrance

This Concept aims to narrow the entrance channel thus increasing the flushing potential of the Creek and potentially eliminating the flood tide dominance. This may also reduce wave penetration from the west and north directions. The key elements of Concept 3 are shown on the HD model mesh[3] in Figure 78 and include the following:

- a widening of the northern training wall by 7 m to the south, reducing the entrance channel to 15 m width (at navigable depth of -2.7 m AHD);
- a dredging campaign to -2.7 m AHD within the Creek and adjacent entrance channel;
- a large excavation campaign extending from the southern training wall to the jetty to a depth of -2.7 m AHD;
- a small dredging campaign to the north of the Creek entrance to -2.7 m AHD; and
- placement of the dredged/excavated material on the northern side of the northern training wall.

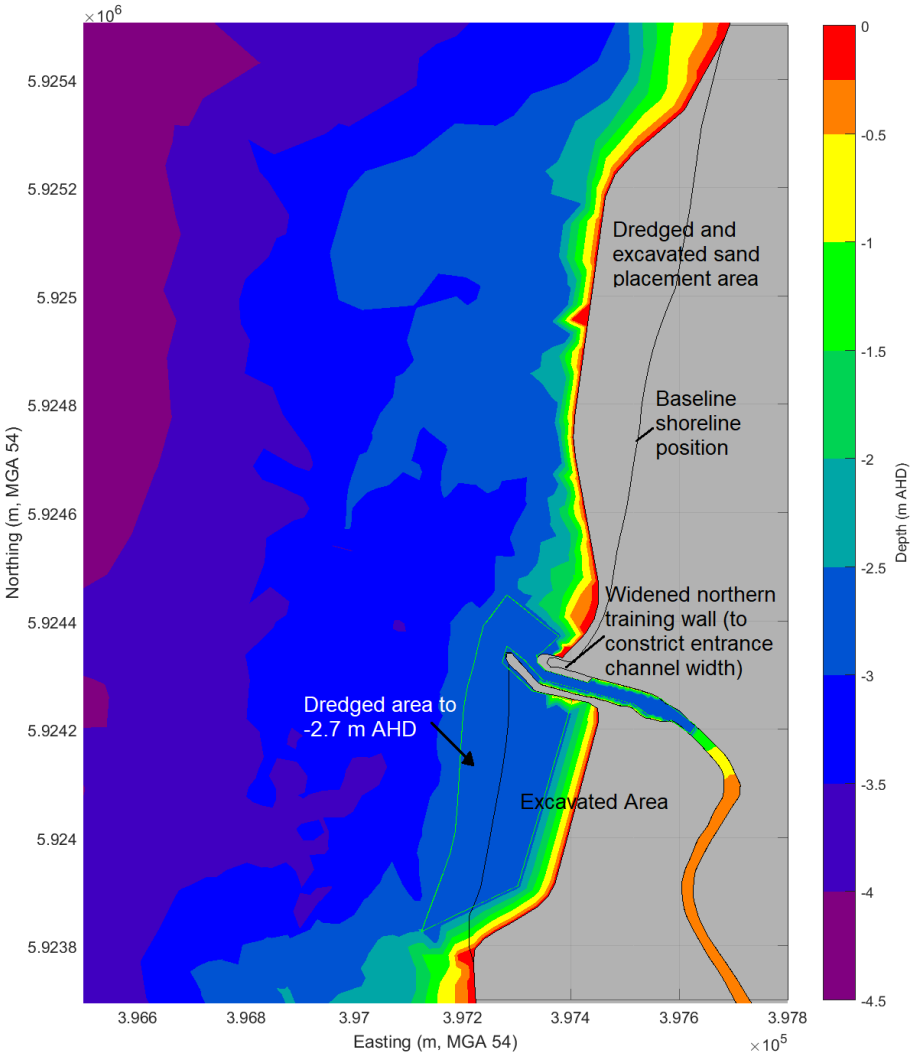


Figure 78. Concept 3: Narrowed entrance model configuration.

[3] Concept 3 was not modelled on the SW mesh, since the only difference between Concept 1 and Concept 3 is a narrowing of the Creek entrance which is not represented in the SW model.

9.4. Concept 4: Removal of Training Walls

This Concept removes the training walls with the aim of returning the coastline to the pre-construction (1988) alignment. This assumes an alternative boat ramp location outside of Maria Creek would be sought. The key elements of Concept 3 are shown on the SW mesh in Figure 79 and include the following:

- removal of approximately 240 m of the southern training wall to beach level;
- removal of approximately 150 m of the northern training wall to beach level;
- natural re-alignment of the shoreline expected to occur over the short-term during and immediately after removal of the training walls. This reflects the expected northwards transport of sand from the south of the southern training wall past the Creek and likely accretion of some of this sediment to north of the Creek; and
- natural re-alignment of the Creek entrance depths.

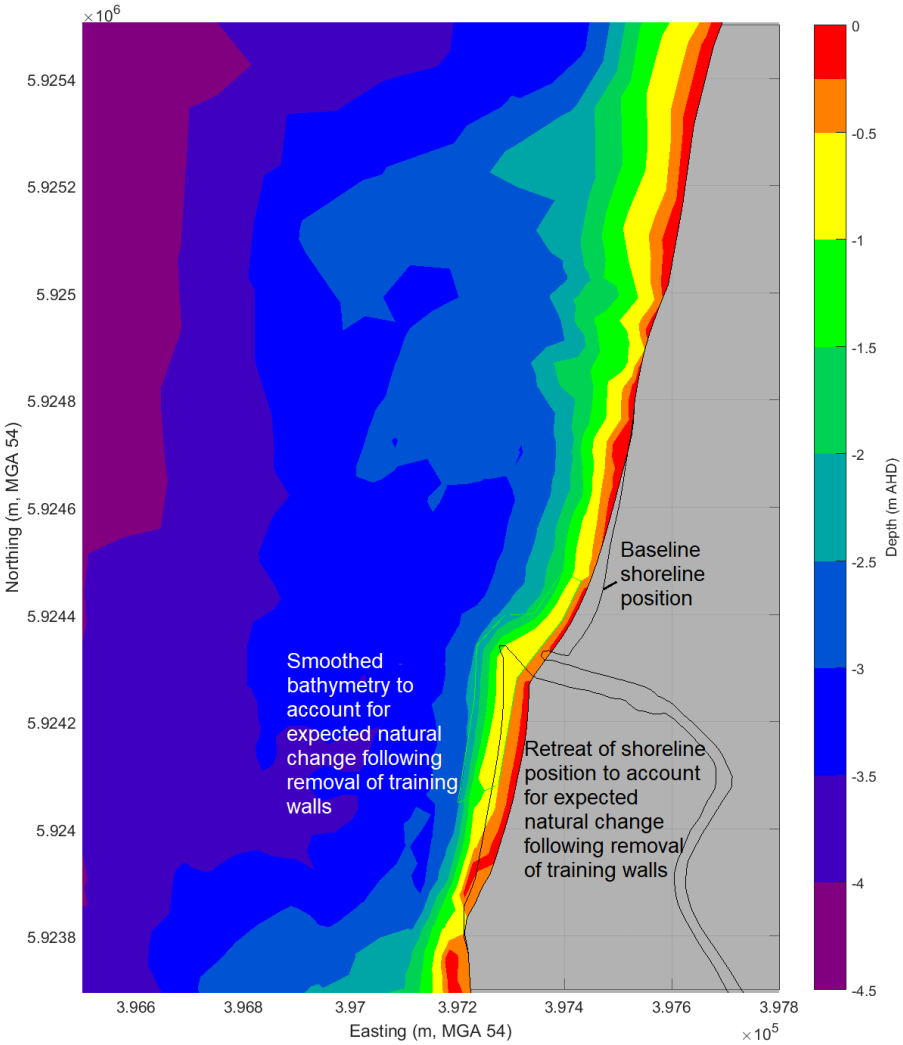


Figure 79. Concept 4: removal of training walls model setup.

Concepts 1 to 3 were modelled in the hydrodynamic model and Concepts 1, 2 and 4 were modelled in the SW model, with the aim of understanding how each design would change the present day (baseline) physical processes.

10. Results

The results from the numerical modelling for each of the concept designs are presented in the following sections.

10.1. Concept 1: Ongoing Management

This Concept was considered to provide an understanding of the baseline Creek dynamics at a design depth of -2.7 m AHD and to provide insight into the sustainability of any dredging work undertaken. The results presented below show the effects of the Concept 1 design with a focus on these aspects.

10.1.1. Effects on Flows

Timeseries plots of flows outside (at Kingston) and inside Maria Creek (at MC – see Figure 52 for location) are shown in Figure 80 and Figure 81, respectively. Results show that relative to the baseline, flows outside of the Creek are largely unchanged, with only a small reduction in flows as a result of the slight deepening in this area. However, within Maria Creek, the tidal currents are significantly reduced during all stages of the tide due to the deepening, with peak speeds reducing from 0.2 m/s to less than 0.1 m/s.

To show the flows in more detail, map plots of the tidal flows for the Concept 1 design are plotted at incremental stages throughout a spring tide during a period of low winds in Figure 82 and Figure 83. Map plots at the time of peak flood are also shown for a spring tide during a period of high northerly winds and a neap tide during a period of high south westerly winds in Figure 84. Additional map plots showing the flows for all tidal states for each wind condition are included in Appendix A (Figure A5 to Figure A8).

The map plots confirm that flows into the Creek at the time of peak flood are reduced relative to the baseline (existing) case due to the dredging deepening the Creek. These slower flows reduce the potential for the transport of sediment and seagrass wrack into the Creek compared to the existing case. This reduction in peak flood flow into the Creek is particularly notable during periods of high south westerly winds when seagrass wrack accumulation within the Creek typically occurs for the existing case. Despite this, the flows within the Creek on the whole remain flood dominant and some wrack accumulation could still be expected. The import of sediment into the Creek is also reduced by the dredging at the entrance, as the dredging removes the sand bar at the entrance to the Creek which currently helps to supply sand which is transported into the Creek.

While flows through the Creek entrance have reduced for Concept 1, as a result of the deepening and increase in cross sectional area across the Creek entrance, there has been a small increase in the Creek discharge with an additional 1,000 m^3 or so of water flowing into and out of the entrance during a large spring tide (72,500 m^3 for Concept 1 compared to 71,500 m^3 for Base Case, see Figure 85). This small change will not influence the stability of the entrance relative to the longshore transport.

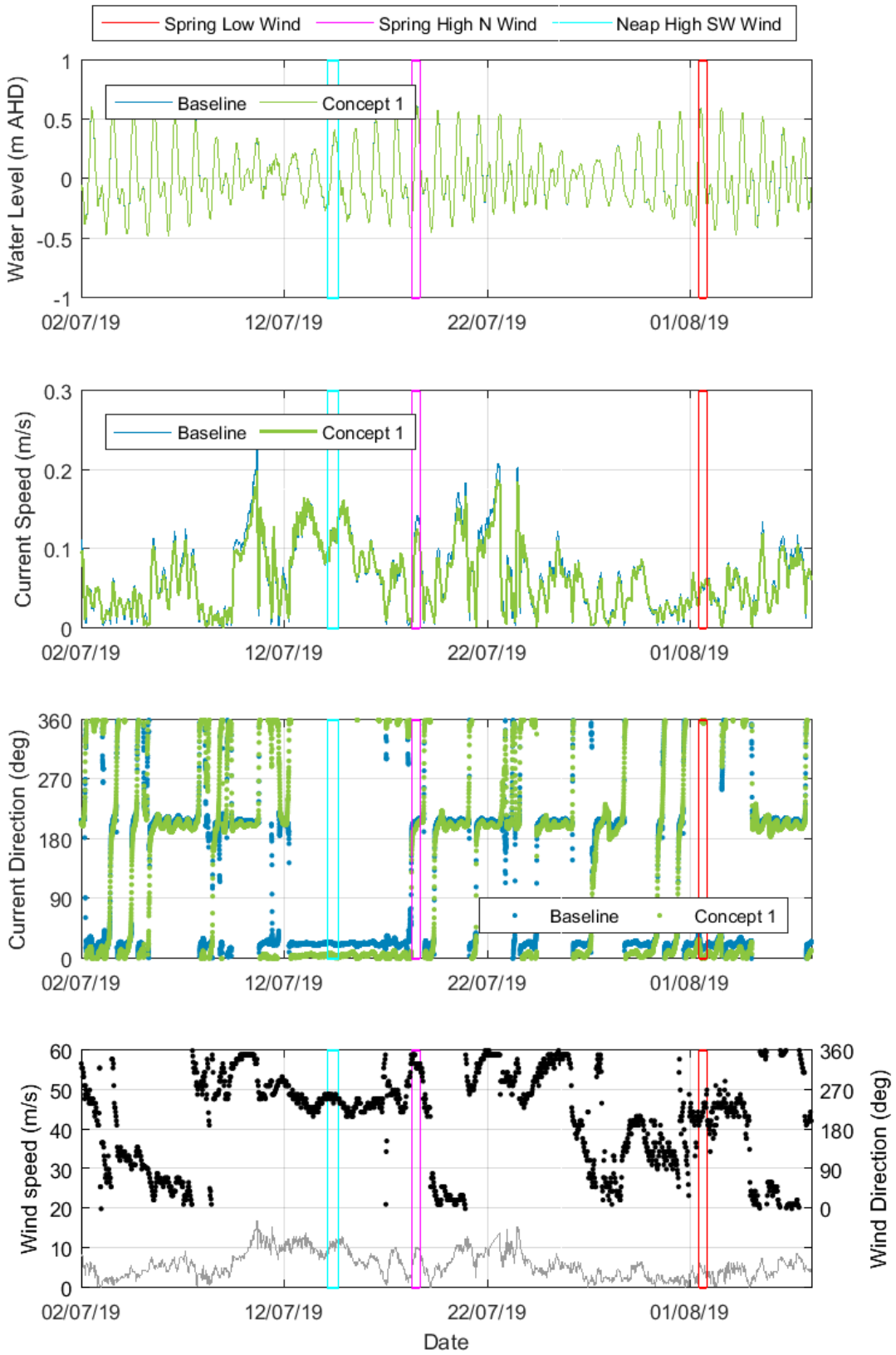


Figure 80. Timeseries of modelled tidal levels and flows at Kingston for the baseline and Concept 1 design.

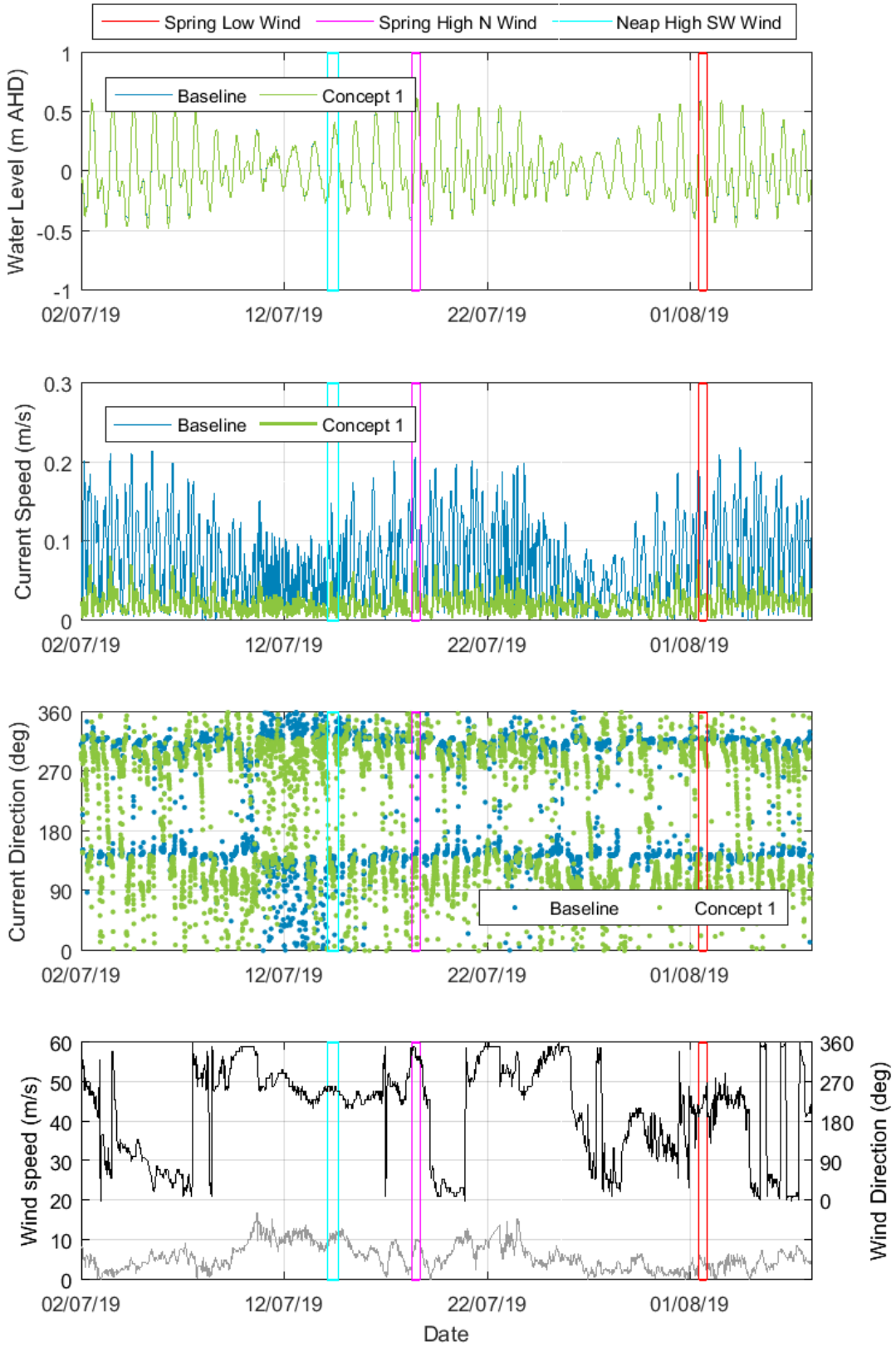


Figure 81. Timeseries of modelled tidal levels and flows in Maria Creek (at MC) for the baseline and Concept 1 design.

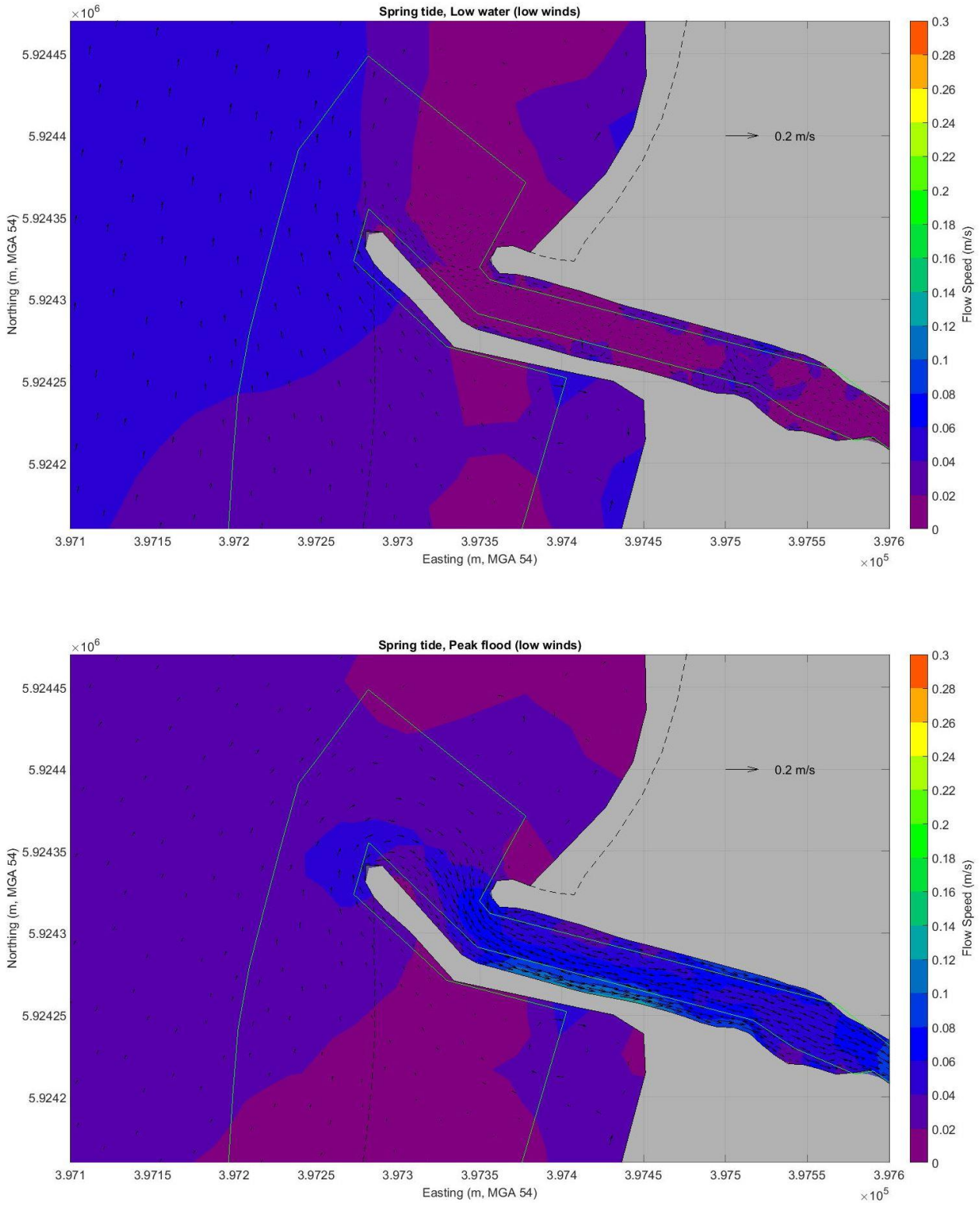


Figure 82. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a spring tide with low winds for Concept 1.

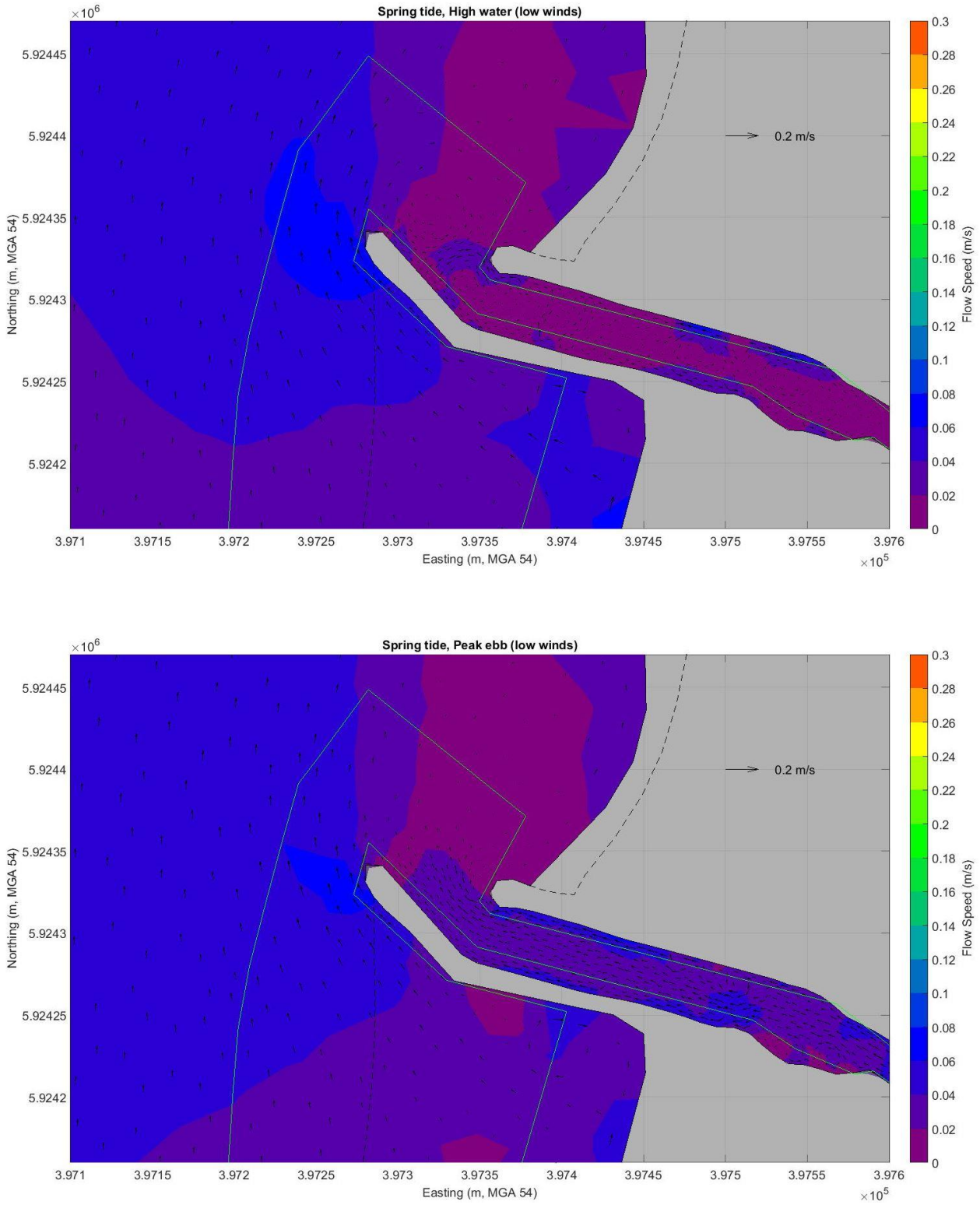


Figure 83. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with low winds for Concept 1.

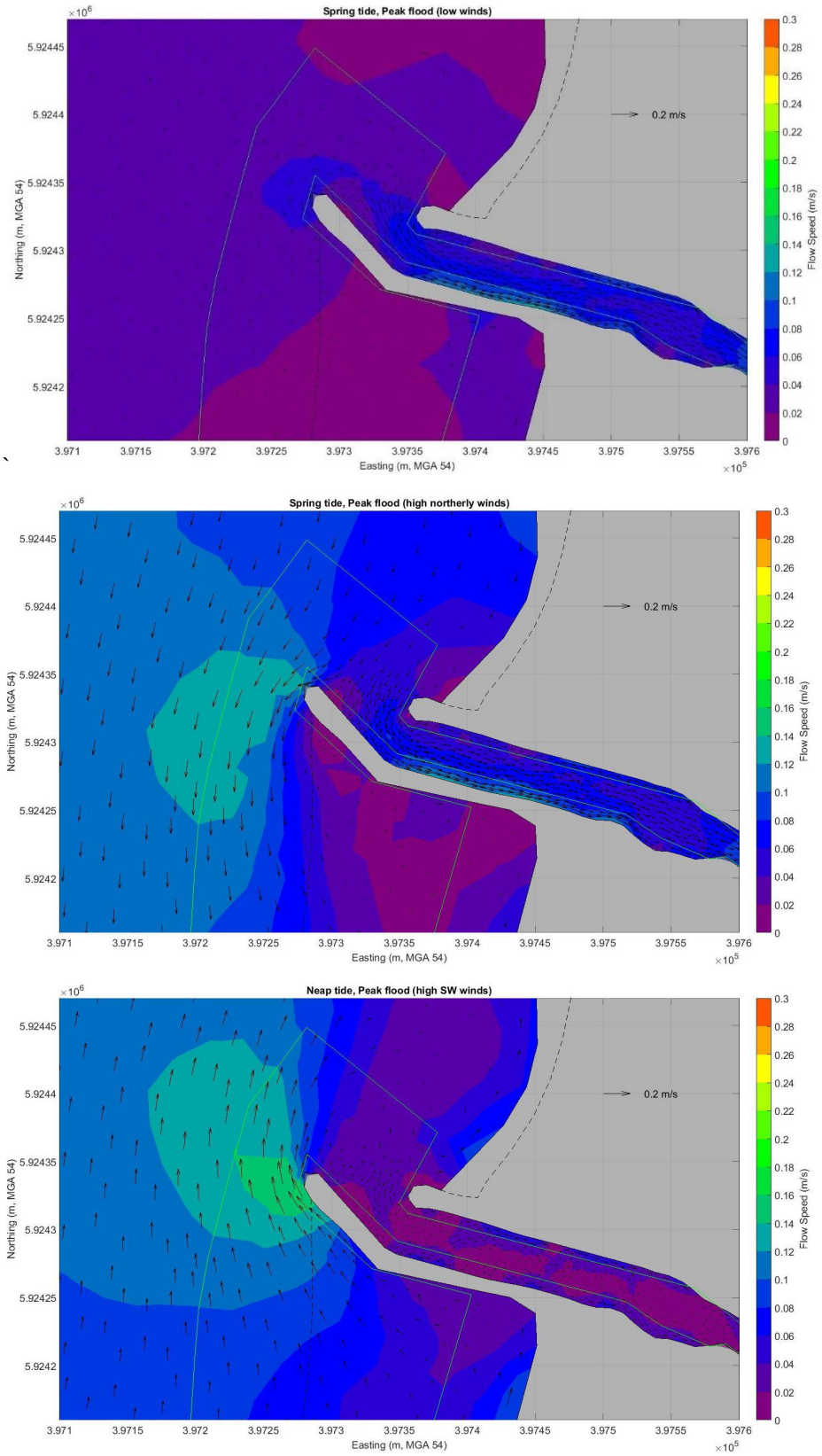
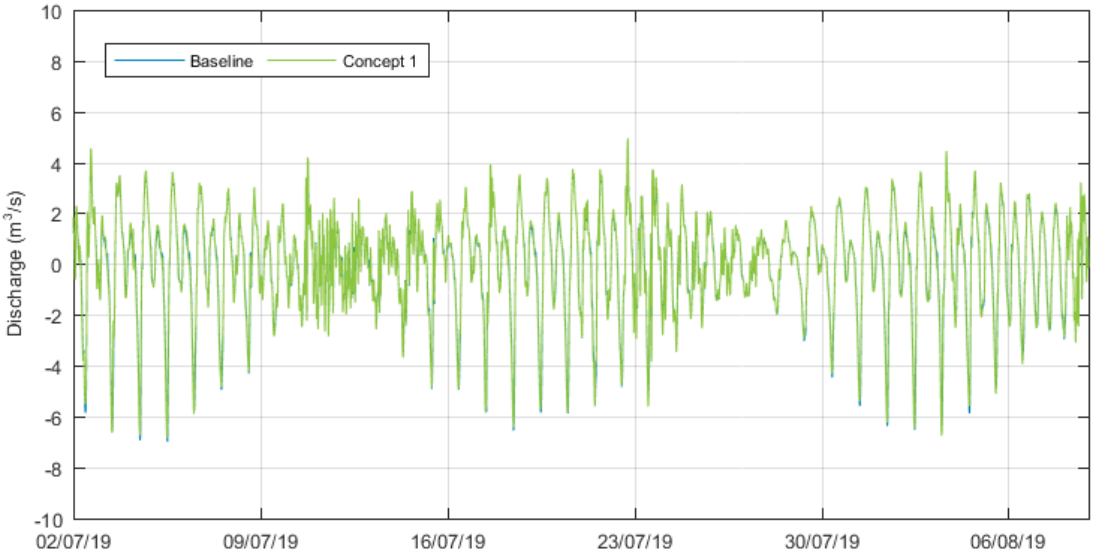


Figure 84. **Modelled tidal flows around Maria Creek at peak flood for a spring tide with low winds (top), a spring tide with high northerly winds (middle) and a neap tide with high south westerly winds (bottom) for Concept 1.**



Note: Positive values denote discharge out of Marina Creek and negative values discharge into the Creek.

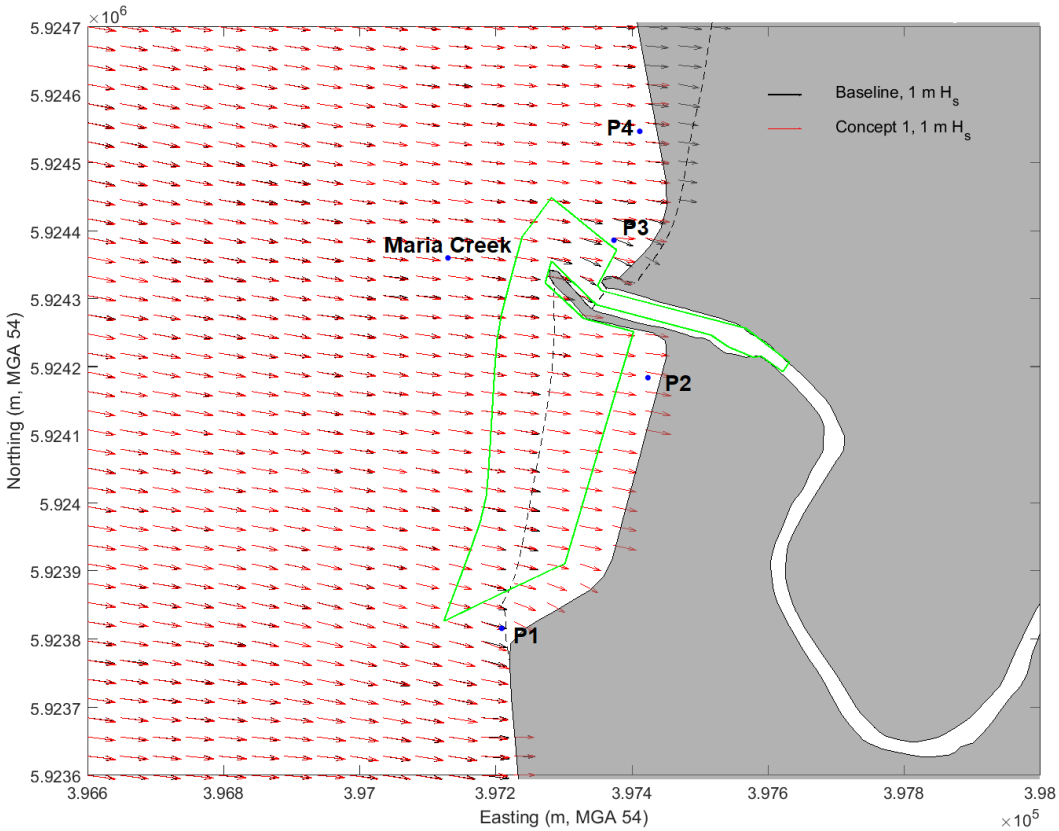
Figure 85. Modelled discharge through Maria Creek for Concept 1.

10.1.2. Effect on Waves

To show how Concept 1 affects waves, a vector plot for an example wave condition is provided in Figure 86. The wave condition shown is for a large wave event (H_s equivalent to approximately a 1 in 5 year ARI), with a long wave period (T_p = 15 seconds). Given how the waves approach the coast from a relatively narrow band irrespective of the local wind direction, this wave vector plot provides a good overview of expected changes to wave conditions, although for smaller, shorter wave periods the effect would be reduced somewhat.

Concept 1 results in a number of small changes to the wave conditions relative to the baseline:

- an increased propagation of waves inshore to the south of the Maria Creek southern training wall in the dredged area, but with little change in wave directions. While wave directions are largely unchanged, as a result of the change in shoreline orientation for Concept 1, the relative direction at which the waves approach the beach is increased, increasing the potential for wave driven longshore sediment transport (see Section 10.1.3);
- a shift in wave direction at the southern edge of the dredged area due to the significant change in the alignment of the bathymetric contours in this area resulting in a change to the wave refraction so that waves approach the shore at a more oblique angle; and
- slight increase in wave heights and change to wave directions (again so that waves approach more oblique to the shoreline for Concept 1) to the north of the training walls as a result of the dredging.



Note: Dashed line shows the coastline position in the baseline case, the grey filled area is the Concept 1 coastline position and the green outline shows the dredged area.

Figure 86. Modelled wave vectors for an example wave event for the baseline case and Concept 1 design.

10.1.3. Effect on Longshore Transport

As expected, the changes to the shoreline orientation as well as some changes to the wave conditions mean that Concept 1 results in a significant change in longshore transport at some of the profiles. The annual wave driven longshore transport rates for Concept 1 are plotted in Figure 87 and tabulated in Table 13.

In particular, the results show:

- a significant increase (about 4x) in the net longshore transport at the southern profile (P1). This is due to P1 being at the southern end of the dredged area where the shoreline orientation has changed by around 40 degrees and so an increase in longshore transport would be expected initially as the shoreline re-orientates;
- an increase (about 1.5x) in net longshore transport at the profile just south of the southern breakwater (P2), this is due to the change in shoreline orientation increasing the transport rates and the sheltering of the breakwater due to the shoreline orientation reducing transport in a southerly direction;
- an increase (about 2x) in net longshore transport at the profile adjacent to the northern breakwater (P3). The increase in longshore transport is due to a small change in the shoreline orientation resulting from the option and a reduction in wave refraction and shoaling due to the deepening directly offshore; and
- a reduction (about 0.6 x) in net longshore sediment transport at the most northerly profile (P4). This is due to the change in shoreline orientation which means that waves approaching from offshore are now closer to shore normal.

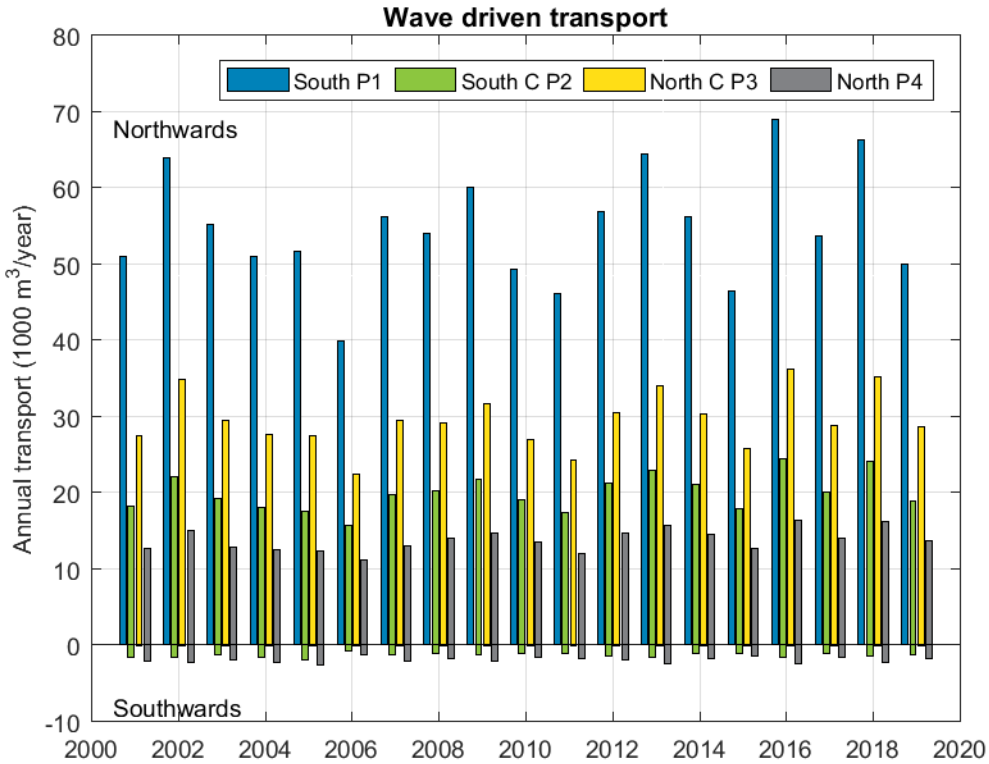


Figure 87. Predicted wave driven annual longshore sediment transport for Concept 1.

Table 13. Predicted wave driven annual net longshore sediment transport rates for Concept 1.

Year	Wave Driven Net Transport (1,000 m³/year), Kamphuis (1991)			
	P1	P2	P3	P4
2001	51	16.6	27.2	10.5
2002	63.9	20.5	34.7	12.6
2003	55.2	18	29.4	10.8
2004	51	16.4	27.5	10.3
2005	51.7	15.5	27.3	9.6
2006	39.8	14.8	22.4	10
2007	56.2	18.3	29.4	10.9
2008	54.1	19.1	29.1	12.2
2009	60	20.4	31.5	12.5
2010	49.2	18	26.7	11.9
2011	46	16.1	24.1	10.1
2012	56.8	19.9	30.3	12.7
2013	64.4	21.4	33.8	13.3
2014	56.2	20	30.3	12.8
2015	46.5	16.8	25.7	11.2
2016	68.9	22.7	36.2	13.9
2017	53.6	18.8	28.8	12.4
2018	66.3	22.6	35.1	13.9
2019	49.9	17.6	28.6	11.9

10.1.4. Summary

The modelling results for Concept 1 indicate that potential for wrack to be imported into the Creek could be reduced due to the reduction in tidal current speeds flowing into the Creek, particularly during periods of high south westerly winds when wrack accumulation in the Creek can be significant for the baseline case. In addition, the dredging removes the sand bar at the entrance to the Creek which supplies sand to be transported into the Creek and so the import of sediment into the Creek will be reduced. Despite this, the flows within the Creek on the whole remain flood dominant and some wrack accumulation could still be expected. These changes in currents relative to the baseline case are due to the dredging (there are no structural changes in this Concept).

Following the implementation of Concept 1, the shoreline orientation would likely change quite quickly (in the order of months to years depending on occurrence of larger wave events) to reach a new equilibrium in the future. The equilibrium would likely be similar orientations to what has been present historically, although the orientation would also be dependent on whether any ongoing bypassing was implemented or not. It could be similar to pre 2016 if ongoing bypassing is adopted or gradually change to present day orientation if no bypassing is undertaken.

In view of the changes to longshore transport rates and Creek discharge, the stability of the entrance to the Creek is unlikely to be significantly changed by the Concept and will depend largely on any ongoing sediment management that is undertaken.

10.2. Concept 2

This Concept was considered to assess the effect of an extended and re-orientated training wall on wave sheltering and subsequent sand accumulation north of the Creek entrance. The results presented in the following sections show the effects of the Concept 2 design with a focus on this aspect.

10.2.1. Effects on Flows

Timeseries plots of flows outside (at Kingston) and inside Maria Creek (at MC, see Figure 52 for locations) are shown in Figure 88 and Figure 89, respectively. Results show that relative to the baseline, flows at Kingston are largely unchanged. However, within the Creek, the tidal currents are significantly reduced during all stages of the tide, with peak speeds reducing from 0.2 m/s to less than 0.08 m/s.

To show the flows in more detail, map plots of the tidal flows for the Concept 2 design are plotted at incremental stages throughout a spring tide during a period of low winds in Figure 90 and Figure 91. Map plots at the time of peak flood are also shown for a spring tide during a period of high northerly winds and a neap tide during a period of high south westerly winds in Figure 92. Additional map plots showing the flows for all tidal states for each wind condition are included in Appendix A (Figure A9 to Figure A12).

The map plots confirm that flows into the Creek at the time of peak flood are reduced relative to the baseline. These slower flows reduce the potential for the transportation of sediment and seagrass wrack into the Creek. As for Concept 1, the import of sediment into the Creek is also reduced by the dredging at the entrance, meaning that there is no sand bar available for feed. Despite the reduction in flows on the flood tide, the tidal currents in the Creek remain flood dominant (i.e. ebb flows are also reduced), with peak flood tidal currents more than double those on the ebb tide during large spring tides.

As for Concept 1, the reduction in flows inside the Creek is largely due to the deepening within the entrance to the Creek due to the dredging. In comparison to Concept 1, the flows for Concept 2 are broadly similar, but are slightly faster on both the flood and ebb tide due to an increased constriction resulting from the changes to the training wall structures.

The maps plots also show that while the tidal currents offshore of Maria Creek are broadly similar to the baseline case, there are some localised increases in current speeds due to an acceleration of currents directly offshore of the extended southern training wall due to its extension and re-orientation.

There has been a 30% decrease in the discharge of Maria Creek, with a reduction of water flowing into and out of the entrance during a large spring tide of more than 25,000 m^3 (45,200 m^3 for Concept 2 compared to 71,500m^3 for the baseline and 72,500 m^3 for Concept 1, see Figure 93). This will reduce the potential of the Creek entrance to remain stable relative to the longshore transport compared to the baseline (and Concept 1). However, it also means that there is a reduced potential of wrack being transported into the Creek during the flooding tide relative to the baseline and Concept 1.

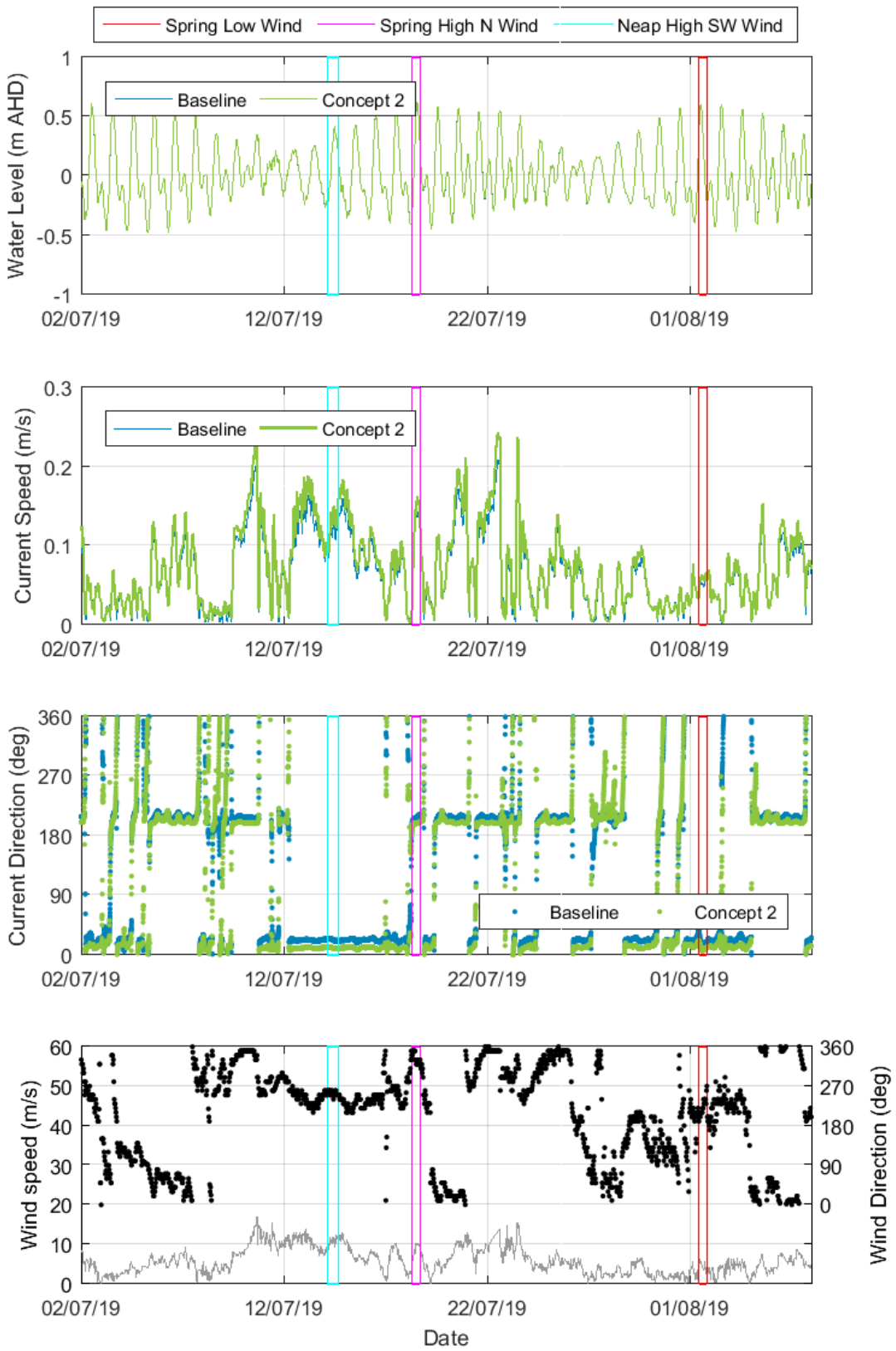


Figure 88. Timeseries of modelled tidal levels and flows at Kingston for the baseline and Concept 2 design.

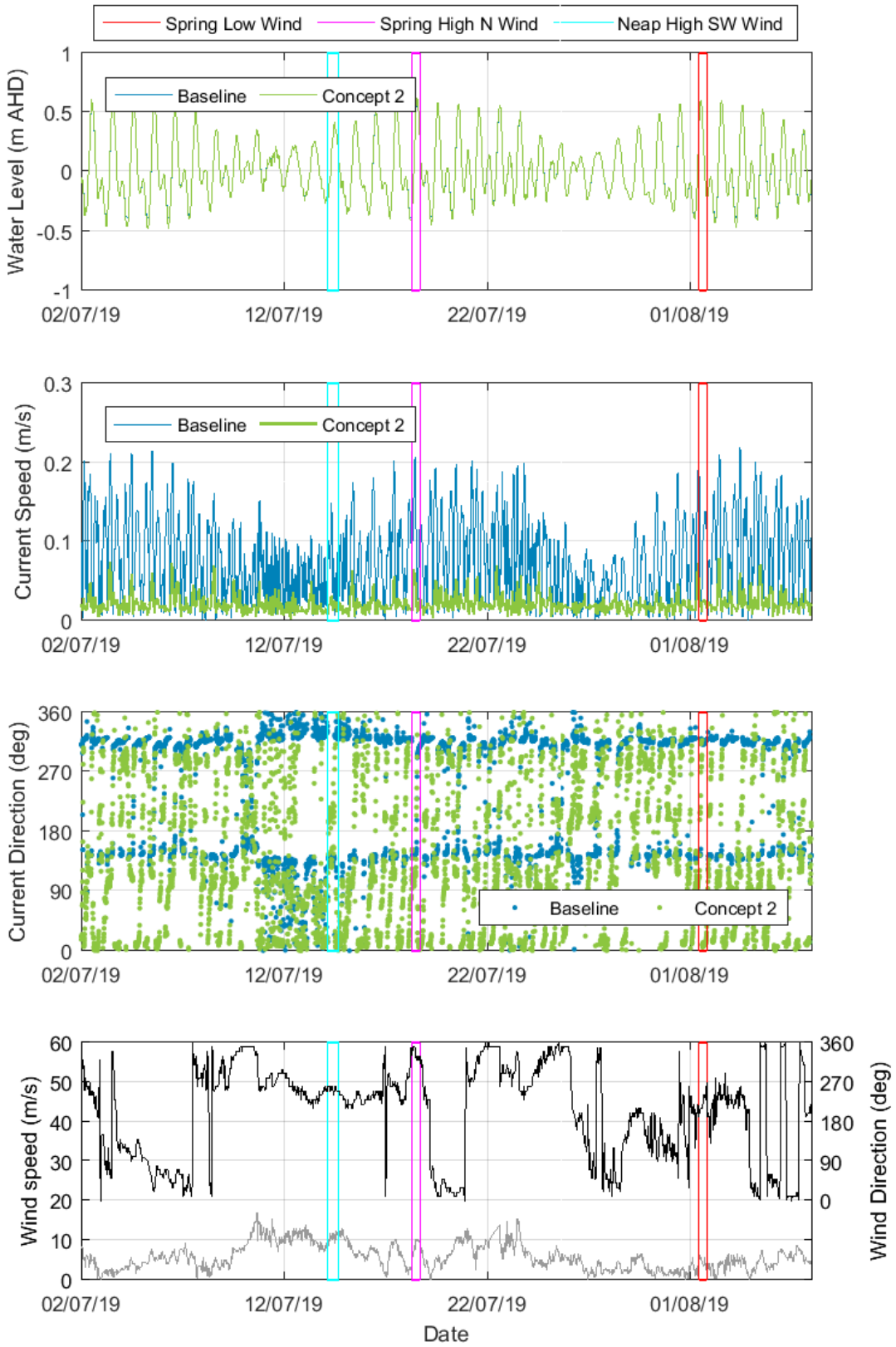


Figure 89. Timeseries of modelled tidal levels and flows in Maria Creek (at MC) for the baseline and Concept 2 design.

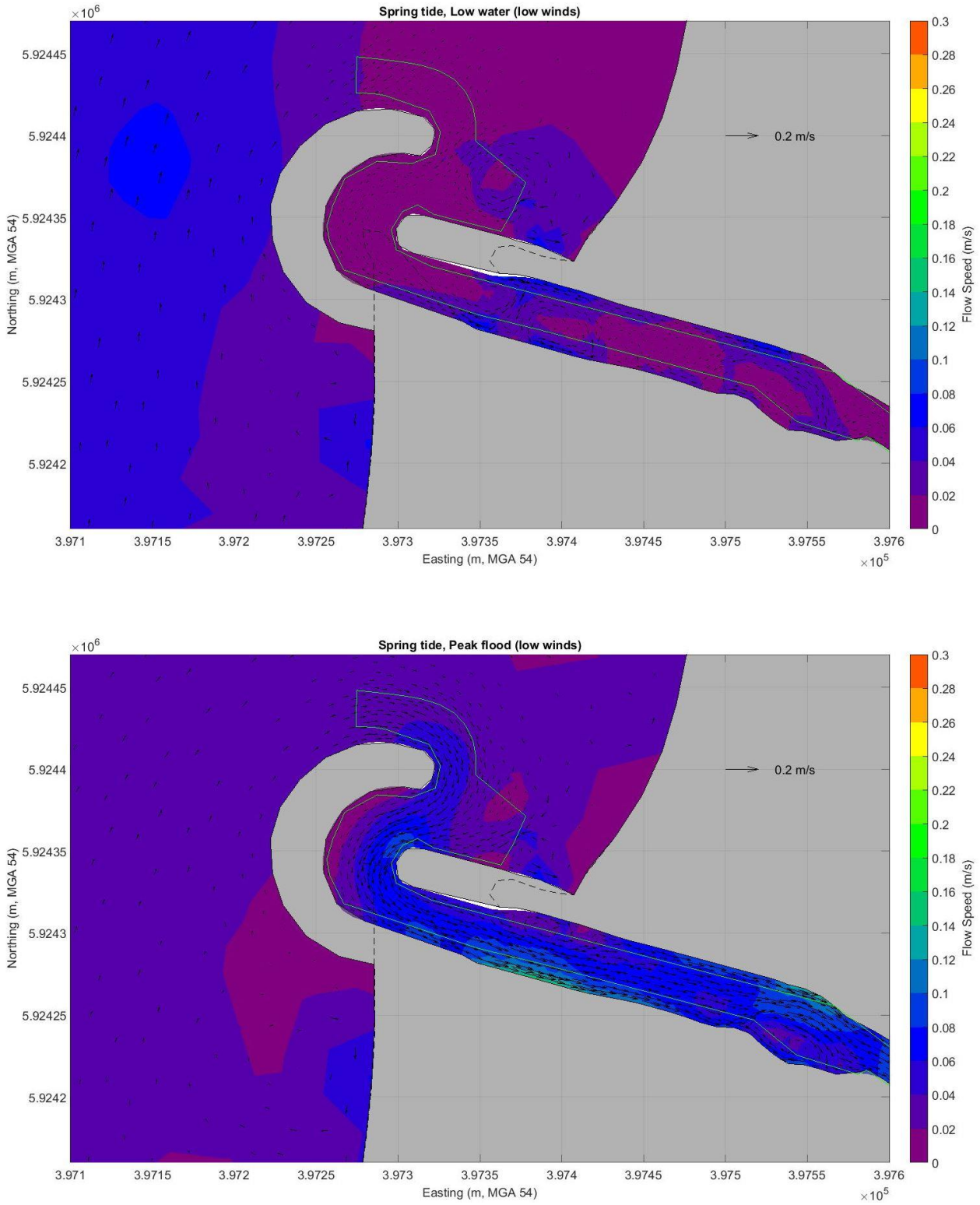


Figure 90. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a spring tide with low winds for Concept 2.

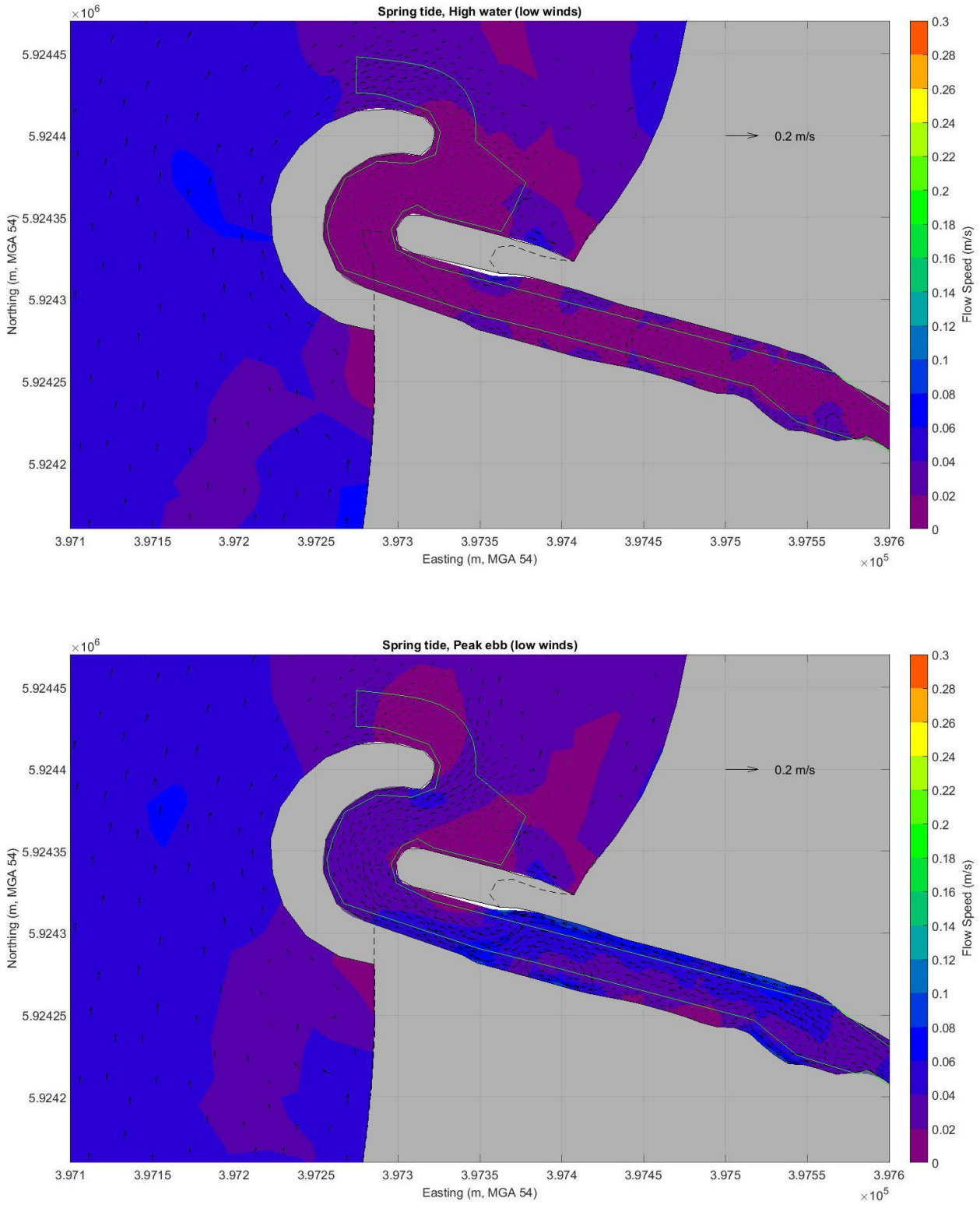


Figure 91. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with low winds for Concept 2.

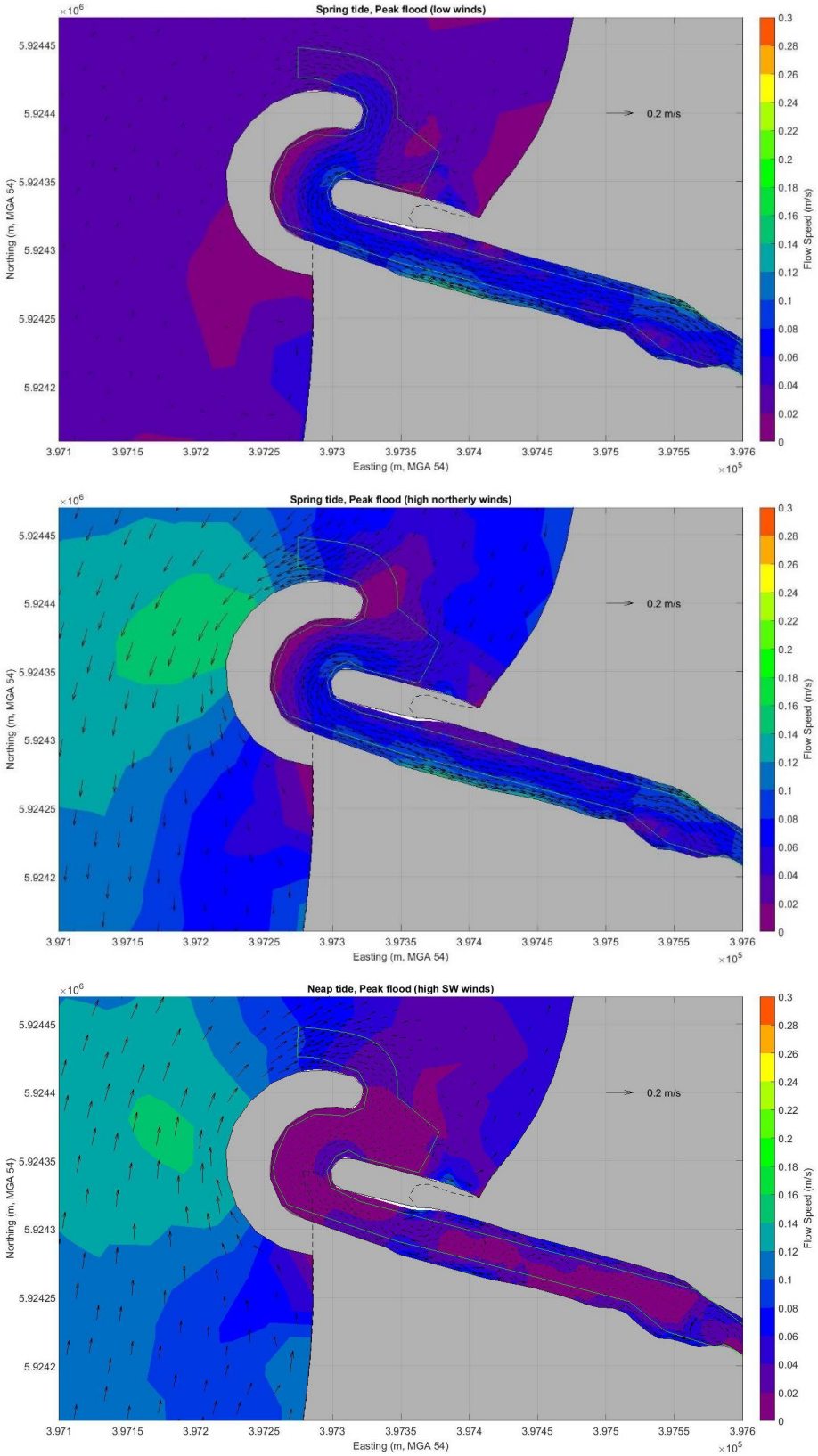
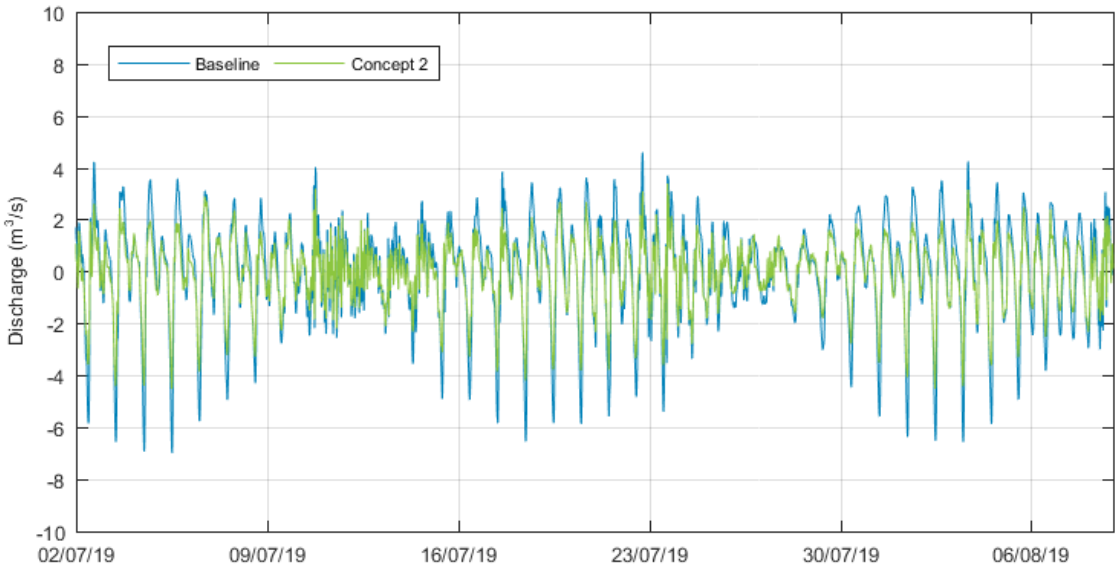


Figure 92. **Modelled tidal flows around Maria Creek at peak flood for a spring tide with low winds (top), a spring tide with high northerly winds (middle) and a neap tide with high south westerly winds (bottom) for Concept 2.**

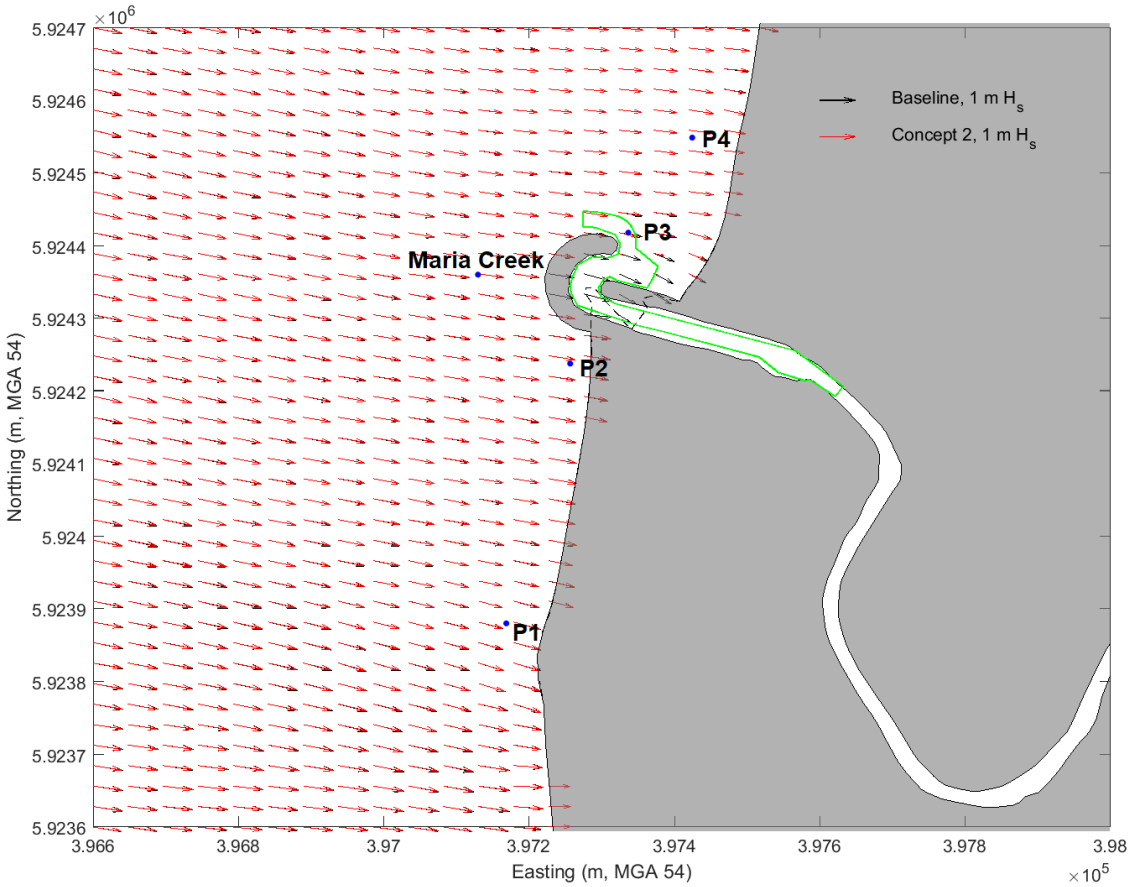


Note: Positive values denote discharge out of Marina Creek and negative values discharge into the Creek.

Figure 93. Modelled discharge through Maria Creek for Concept 2.

10.2.2. Effect on Waves

The effect of the Concept 2 design on wave conditions is shown for the example wave condition in Figure 94. The changes to the waves for Concept 2 relative to the baseline are localised, with reduced wave heights off the shoreline directly adjacent to the northern training wall, due to the increased sheltering provided by the extended and hooked southern training wall. The hooked southern training wall also acts to prevent any wave activity from reaching or propagating into the entrance of Maria Creek. Therefore, the effect of wave driven wrack transport into the Creek is expected to be reduced for Concept 2 relative to the baseline and Concept 1.



Note: Dashed line shows the coastline position in the baseline case, the grey filled area is the Concept 2 coastline position and the green outline shows the dredged area.

Figure 94. Modelled wave vectors for an example wave event for the baseline case and Concept 2 design.

10.2.3. Effect on Longshore Transport

The additional wave sheltering from the extended and re-orientated training walls to the north of Maria Creek results in a localised reduction in longshore transport rates in this area. As a result, a larger volume of sand is likely to become trapped on the north beach adjacent to the northern training wall. Elsewhere (including adjacent to the southern training wall), longshore transport rates are largely unchanged relative to the baseline case.

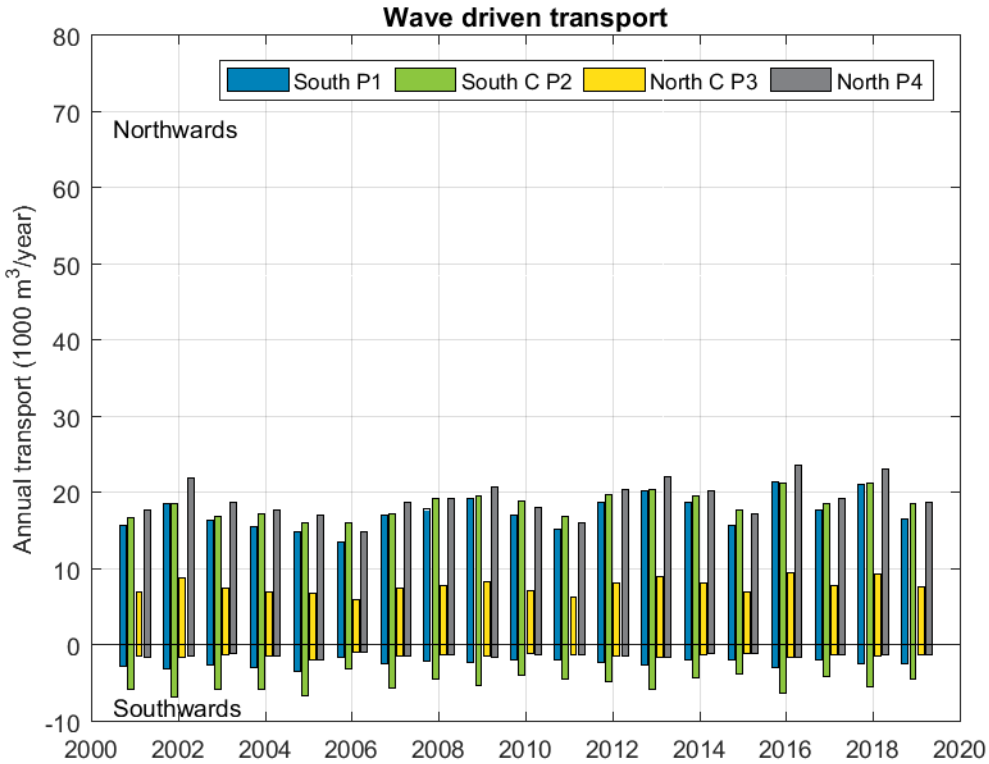


Figure 95. Predicted wave driven annual longshore sediment transport for Concept 2.

Table 14. Predicted wave driven annual net longshore sediment transport rates for Concept 2.

Year	Wave Driven Net Transport (1,000 m³/year), Kamphuis (1991)			
	P1	P2	P3	P4
2001	12.9	10.9	5.5	16.1
2002	15.4	11.8	7.3	20.4
2003	13.8	11.1	6.2	17.6
2004	12.5	11.2	5.4	16.2
2005	11.4	9.4	4.9	15.1
2006	11.9	12.8	5.1	14
2007	14.5	11.5	6	17.3
2008	15.6	14.6	6.4	18
2009	16.9	14.3	6.9	19.2
2010	15	14.9	5.9	16.8
2011	13.2	12.5	5	14.8
2012	16.5	15	6.7	18.8
2013	17.5	14.6	7.2	20.4
2014	16.7	15.3	6.9	19
2015	13.7	13.8	5.8	16.1
2016	18.4	14.9	7.8	21.9
2017	15.7	14.5	6.6	18
2018	18.6	15.7	7.8	21.7
2019	14	14.2	6.4	17.3

10.2.4. Summary

The modelling results for Concept 2 indicate that the additional sheltering provided by the extended and re-orientated training walls would reduce the potential for wrack to be imported into the Creek (relative to both the baseline and Concept 1 cases). In addition, the extended training walls and dredging in the entrance mean that there is no sand bar available at the entrance to the Creek meaning that the import of sediment into the Creek will be reduced (as for Concept 1). However, the flood dominance of the flows through the Creek entrance means that any wrack (and sediment) which is imported is likely to remain within the Creek. In view of the changes to Creek discharge, there is a reduced potential of the Creek entrance to remain stable relative to the longshore transport compared to the baseline (and Concept 1).

The modelled effects of Concept 2 on waves and longshore sediment transport indicate that it is likely that sediment would continue to build up on the southern side of the southern training wall and eventually the build-up would allow the sediment to naturally bypass the structure (and also form a sand bar which could supply sediment to be imported into the Creek unless ongoing maintenance occurs). The additional sheltering of the shoreline directly to the north of the northern training wall could also result in an increased build-up of sediment in this location, with sediment which is transported there during periods of southwards longshore transport remaining there. Overall, this would result in sedimentation occurring adjacent to the entrance to Maria Creek and as the current speeds are very low it is unlikely that the sediment would be naturally removed. It is therefore likely that the Creek entrance would become silted up over time and ongoing sediment management would be required to keep it open.

10.3. Concept 3

This Concept was considered to see how a narrowing of the Creek entrance channel could affect the flushing potential of the Creek in an attempt to limit the flood dominance in flood tidal flows. Other than the narrowing of the Creek entrance, this Concept design is the same as the Concept 1 design. The results from the wave modelling (Section 10.1.2) and longshore sediment transport (Section 10.1.3) for Concept 1 are applicable to this Concept design, since the propagation of waves into the Creek is not included in the wave model.

10.3.1. Effect on Flows

Timeseries plots of flows outside (at Kingston) and inside Maria Creek (at MC, see Figure 52 for locations) are shown in Figure 96 and Figure 97, respectively. The plots show that (as for Concept 1), tidal currents offshore of Maria Creek are similar to the baseline, with only small reductions in current speeds caused by the dredging. However, tidal currents within Maria Creek are significantly lower during all stages of the tide due to the deepening, with peak speeds reducing from 0.2 m/s to less than 0.1 m/s. Despite the narrowing of the entrance channel, the tidal currents in the Creek remain flood dominant (as for Concept 1 and the baseline), with peak flood tidal currents up to 40% higher than those during the ebb tide during large spring tides.

To show the flows in more detail, map plots of the tidal flows for the Concept 3 design are plotted at incremental stages throughout a spring tide during a period of low winds in Figure 98 and Figure 99. Map plots at the time of peak flood are also shown for a spring tide during a period of high northerly winds and a neap tide during a period of high south westerly winds in Figure 100. Additional map plots showing the flows for all tidal states for each wind condition are included in Appendix A (Figure A13 to Figure A16).

The map plots confirm that flows into the Creek at the time of peak flood are reduced relative to the baseline. However, differences in flows relative to the Concept 1 design are insignificant indicating that the narrowing of the channel is unlikely to offer any significant benefit with respect to effects on accumulation of sand and seagrass wrack.

There is no change to the discharge of Maria Creek, with approximately 71,500 m³ flowing into and out of the Creek entrance during a large spring tide for both the baseline and Concept 3 (see Figure 101).

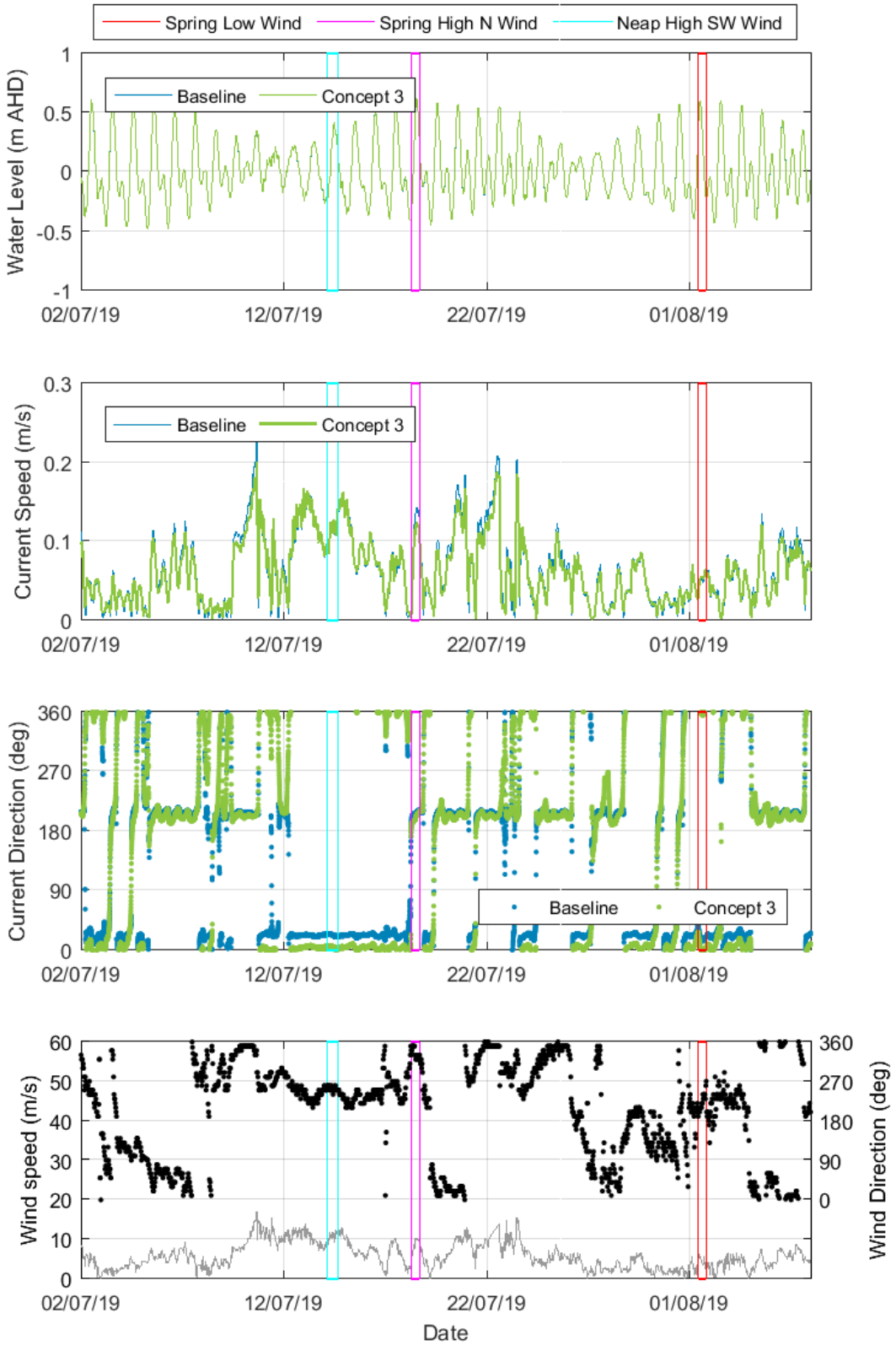


Figure 96. Timeseries of modelled tidal levels and flows at Kingston for the baseline and Concept 3 design.

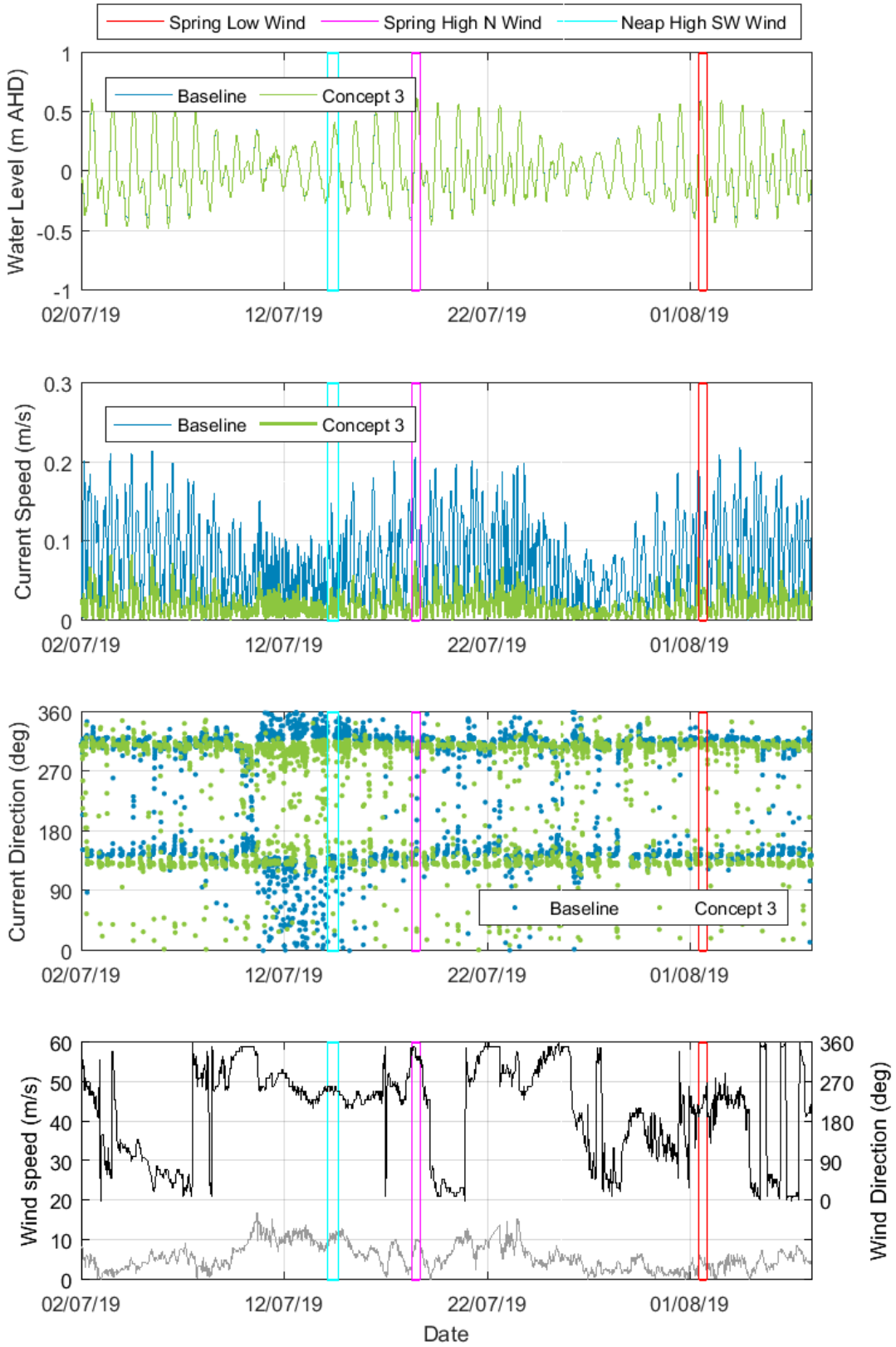


Figure 97. Timeseries of modelled tidal levels and flows in Maria Creek (at MC) for the baseline and Concept 3 design.

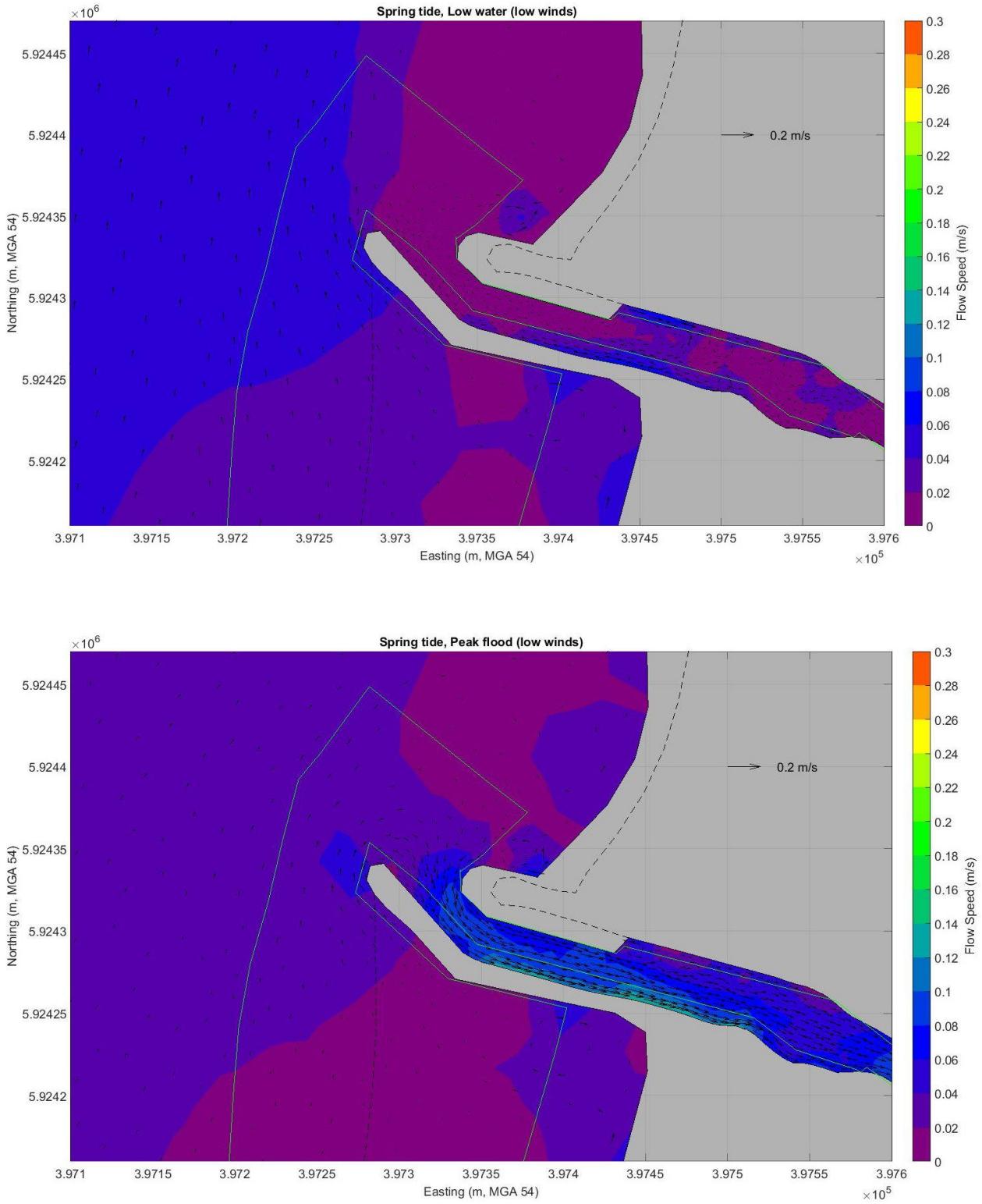


Figure 98. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a spring tide with low winds for Concept 3.

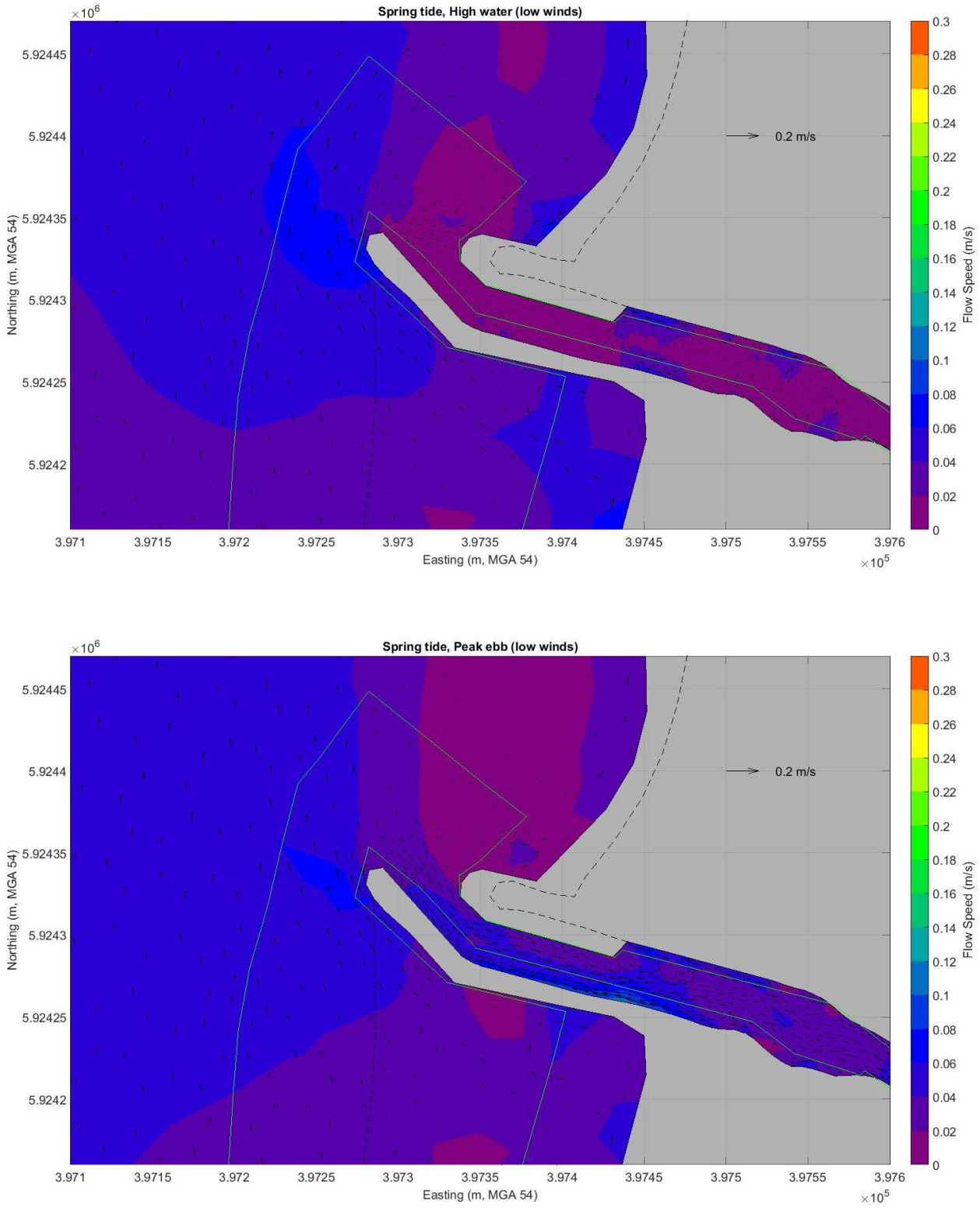


Figure 99. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with low winds for Concept 3.

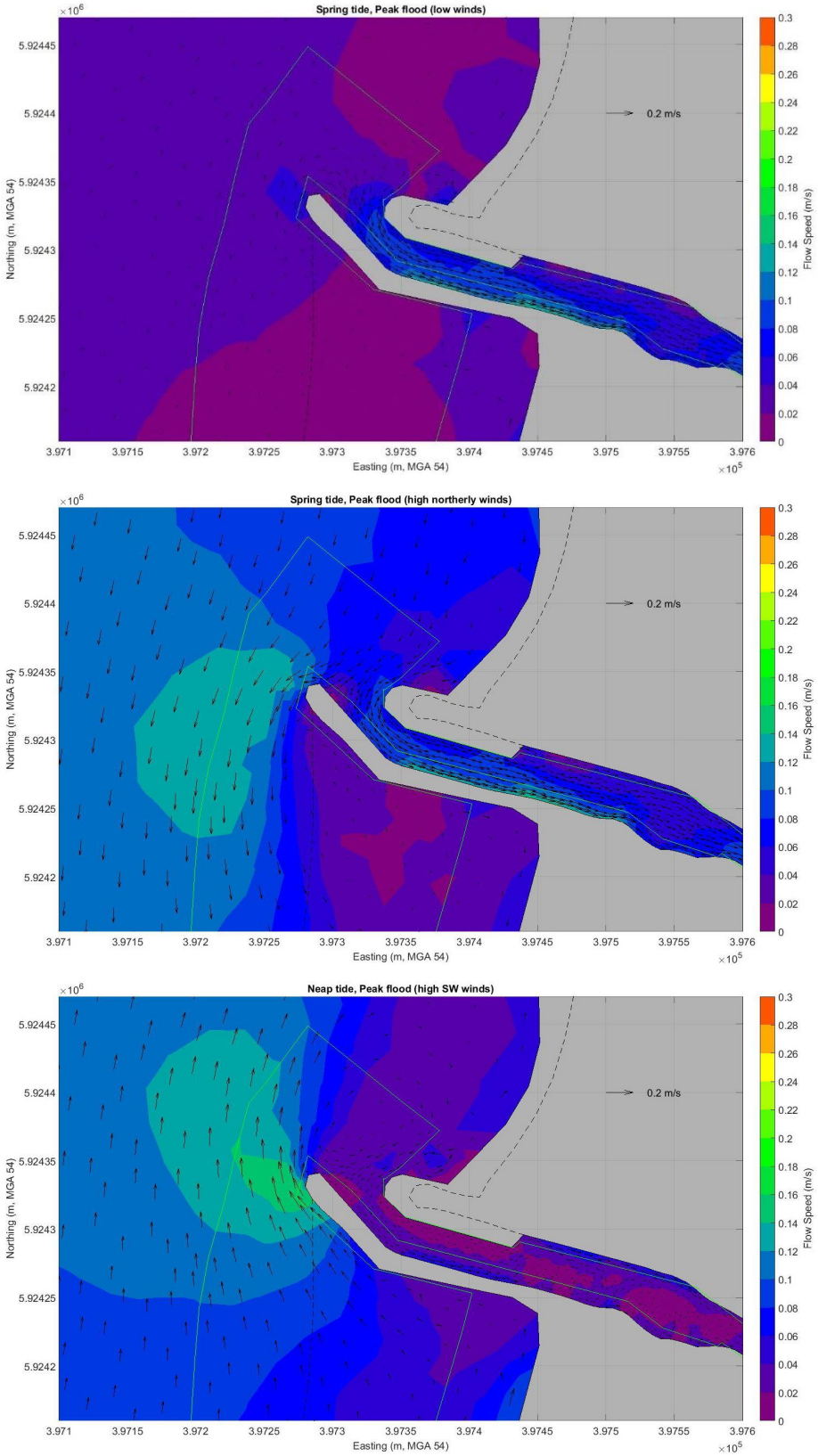
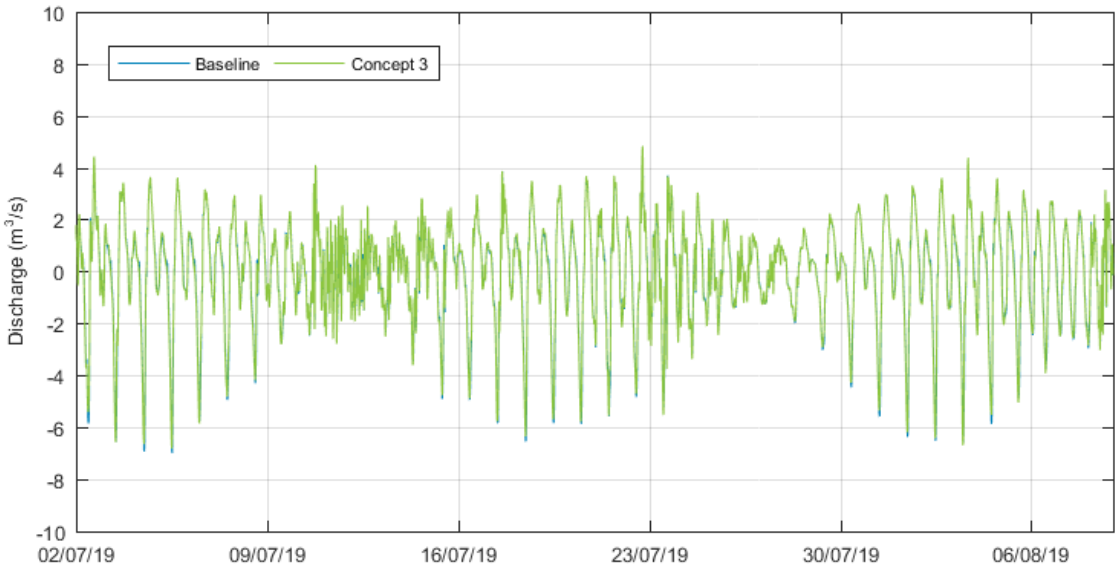


Figure 100. Modelled tidal flows around Maria Creek at peak flood for a spring tide with low winds (top), a spring tide with high northerly winds (middle) and a neap tide with high south westerly winds (bottom) for Concept 3.



Note: Positive values denote discharge out of Marina Creek and negative values discharge into the Creek.

Figure 101. Modelled discharge through Maria Creek for Concept 3.

10.3.2. Effect on Waves

Concept 3 results in the same changes to the wave conditions relative to the baseline as Concept 1 (see results in Section 10.1.2). This includes:

- an increased propagation of waves inshore to the south of the southern training wall in the dredged area, but with little change in wave directions. While wave directions are largely unchanged, as a result of the change in shoreline orientation for Concept 3, the relative direction at which the waves approach the beach is increased, increasing the potential for wave driven longshore sediment transport;
- a shift in wave direction at the southern edge of the dredged area due to the significant change in the alignment of the bathymetric contours in this area resulting in a change to the wave refraction so that waves approach the shore at a more oblique angle; and
- slight increase in wave heights and change to wave directions (again so that waves approach more oblique to the shoreline for Concept 3) to the north of the training walls as a result of the dredging.

10.3.3. Effect on Longshore Transport

Concept 3 results in the same changes to the longshore transport relative to the baseline as Concept 1 (see results in Section 10.1.2). This includes:

- a significant increase (about 4x) in the net longshore transport at the southern profile (P1). This is due to P1 being at the southern end of the dredged area where the shoreline orientation has changed by around 40 degrees and so an increase in longshore transport would be expected initially as the shoreline re-orientates;
- an increase (about 1.5x) in net longshore transport at the profile just south of the southern breakwater (P2), this is due to the change in shoreline orientation increasing the transport rates and the sheltering of the breakwater due to the shoreline orientation reducing transport in a southerly direction;
- an increase (about 2x) in net longshore transport at the profile adjacent to the northern breakwater (P3). The increase in longshore transport is due to a small change in the shoreline orientation resulting from the option and a reduction in wave refraction and shoaling due to the deepening directly offshore; and

- a reduction (about 0.6 x) in net longshore sediment transport at the most northerly profile (P4). This is due to the change in shoreline orientation which means that waves approaching from offshore are now closer to shore normal.

10.3.4. Summary

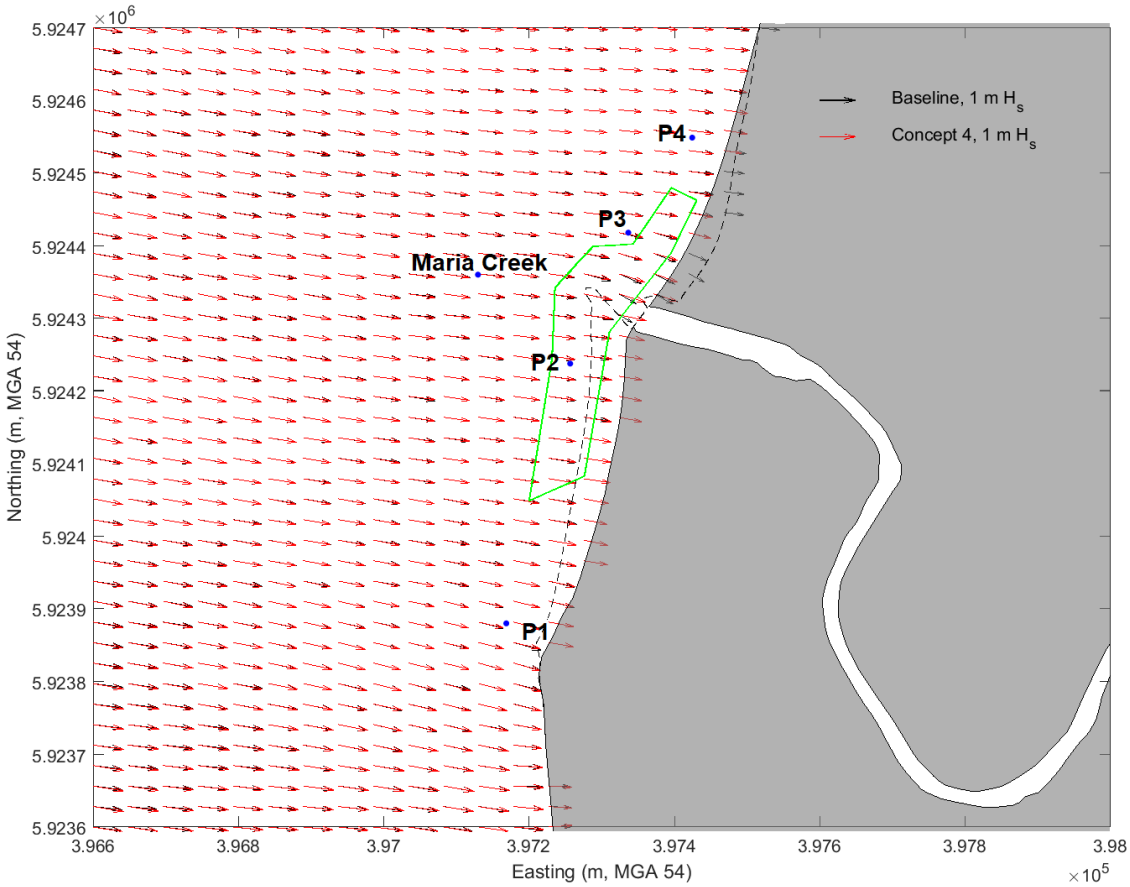
The results for Concept 3 are very similar to Concept 1, this Concept results in slightly higher tidal current speeds in the Creek due to the reduced channel width but overall that change is not expected to result in any significant changes compared to Concept 1. As for Concept 1, it is likely that the shoreline orientation would change quite quickly where the dredging was undertaken (likely over months to years depending on occurrence of larger wave events) to reach some kind of equilibrium in the future. The equilibrium would likely be similar orientations to what has been present historically, although the orientation would also be dependent on whether any ongoing bypassing was implemented or not. The orientation could be similar to pre 2016 if ongoing bypassing is adopted or gradually change to present day orientation if there is no bypassing. In terms of the stability of the entrance to the Creek, this is unlikely to be significantly changed by the Concept and will largely be dependent on any ongoing sediment management, although it is possible that the potential for wrack to be imported into the Creek might be reduced due to the reduced tidal current speeds flowing into the Creek (although the flows are still flood dominant in the Creek).

10.4. Concept 4

This Concept was considered to determine whether following a removal of the training walls on-going natural realignment of the shoreline would occur so that it's position would return to the pre-construction location or whether the current shoreline positions would be maintained. This Concept has therefore only been modelled in the SW model.

10.4.1. Effect on Waves

The effect of the Concept 4 design on wave conditions is shown for the example wave condition in Figure 102. Concept 4 results in minor changes to the wave conditions relative to the baseline, with small changes in wave direction and wave height around where the training walls previously were and in areas where the shoreline has been realigned to the south and north of the Maria Creek. To the south of the training wall there is a small increase in wave refraction due to the retreated coastline in this area, while to the north there is a small reduction in wave refraction due to the shoreline progression.



Note: Dashed line shows the coastline position in the baseline case, the grey filled area is the Concept 4 coastline position and the green outline shows the area where the bathymetry has been smoothed.

Figure 102. Modelled wave vectors for an example wave event for the baseline case and Concept 4 design.

10.4.2. Effect on Longshore Transport

The changes to the shoreline orientation as well as some changes in wave conditions result in the following changes to the longshore transport:

- there is an increase in northerly transport (almost 2x) at the most southerly profile (P1), this is due to the reorientation of the shoreline in this location;
- there was also a slight (approx. 5%) increase in the longshore transport rate at P2 due to the change in shoreline at this location; and
- limited change in longshore transport predicted at P3 and P4 as the shoreline orientation and nearshore bathymetry has not significantly changed.

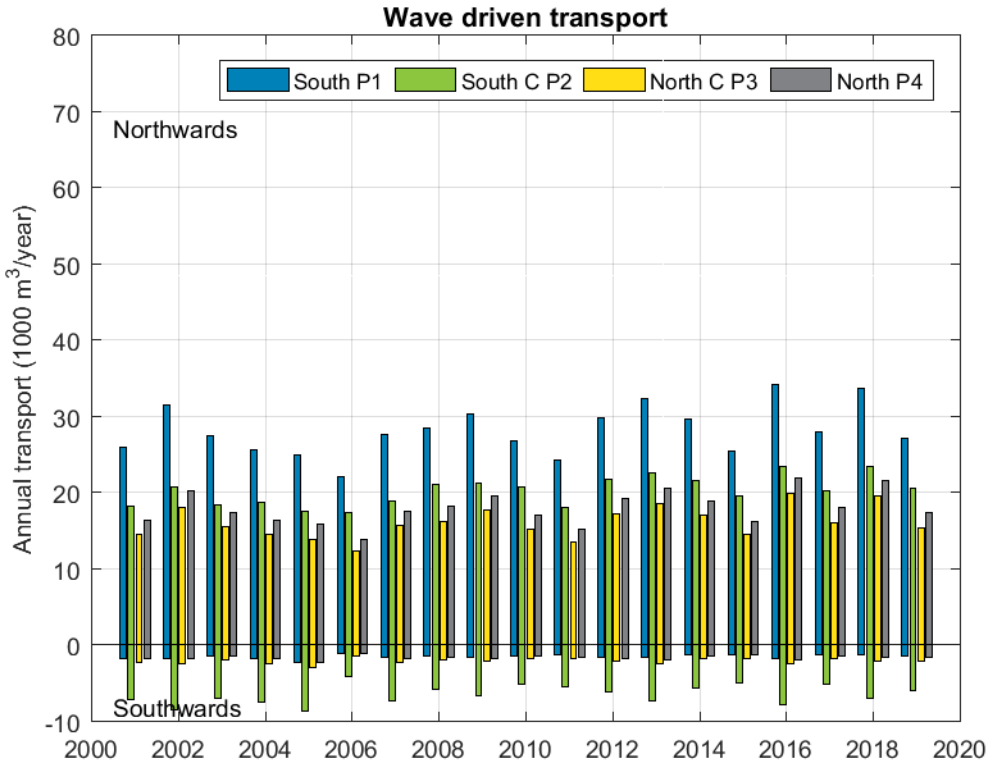


Figure 103. Predicted wave driven annual longshore sediment transport for Concept 4.

Table 15. Predicted wave driven annual net longshore sediment transport rates for Concept 4.

Year	Wave Driven Net Transport (1,000 m³/year), Kamphuis (1991)			
	P1	P2	P3	P4
2001	24.1	11.1	12.3	14.6
2002	29.7	12.2	15.4	18.3
2003	26	11.3	13.6	15.8
2004	23.8	11.2	12.1	14.5
2005	22.7	8.9	10.9	13.4
2006	21	13.2	10.8	12.8
2007	26.1	11.6	13.4	15.7
2008	27	15.2	14.3	16.6
2009	28.8	14.6	15.5	17.6
2010	25.4	15.5	13.4	15.6
2011	23	12.6	11.7	13.6
2012	28.3	15.5	15.1	17.4
2013	30.7	15.2	16.1	18.6
2014	28.5	16	15.3	17.4
2015	24.1	14.5	12.8	14.9
2016	32.5	15.4	17.4	20
2017	26.7	15.2	14.3	16.5
2018	32.3	16.4	17.4	19.9
2019	25.6	14.5	13.2	15.7

10.4.3. Summary

The modelling results for Concept 4 suggest that over time the shoreline would likely become relatively straight over this section, with the higher longshore transport rates at P1 relative to P2, P3 and P4 indicating that there would initially be erosion of the southern end of the area. The longshore transport rates would change over time as the shoreline orientation changes due to ongoing transport. The duration of time that it takes for the shoreline to reach some form of new dynamic equilibrium could be approximately estimated by undertaking a volumetric analysis and comparing this with the predicted annual longshore transport rates. However, it is possible that following the removal of the training walls the sand which has built-up to the south of the training walls could be transported as a ‘slug’, with the longshore transport rates being much higher than predicted by the models.

11. Summary

This report has presented results from an analysis of existing metocean data and outputs from numerical modelling tools developed specifically for this study (which provide further insight into local metocean conditions) to help develop an understanding of the sediment transport processes and how they vary spatially and temporally in the study area. The numerical modelling tools have also been applied to assess the implications of different concept design options at Maria Creek on the hydrodynamics, waves, sediment transport as well as the potential for wrack build-up.

The numerical models developed as part of the study have been subject to an extensive calibration and validation exercise to give confidence that the models are able to accurately represent the hydrodynamic and wave processes at Maria Creek. In the absence of local calibration data, sensitivity testing of the key model parameters has been undertaken and where practical conservative assumptions have been adopted. It is recommended that local measurements of wave conditions in the Maria Creek region (e.g. measurements adjacent to the offshore end of Kingston Jetty) be obtained during winter months (when larger wave events occur) prior to any detailed design stages of the project to help improve the confidence in the modelled wave climate.

The analysis of the metocean conditions indicate that at Maria Creek there are/is:

- low wind and wave driven flows with a dominance in flows to the north due to the circulation patterns in Lacepede Bay and the effect of local wind influences (which can reverse the direction of tidal currents during periods of strong winds);
- a flood dominance within the Creek, indicating that it will typically act as a net importer of both sediment and wrack;
- irregular increased freshwater discharge during flood events which could intermittently help to remove sediment build up in the Maria Creek channel, although it is likely that any mobilised sediment would subsequently be redeposited close to the mouth of the Creek in an ebb bar formation;
- relatively low wave heights, but sufficient to drive a longshore transport of sediment in the nearshore region;
- a net northward longshore sediment transport with predicted net transport rates of between 15,000 to 30,000 m^3/yr based on the present day shoreline orientation; and
- potential for periods of significant increases in longshore sediment transport in the form of sand slugs, particularly following storm events when large volumes of sediment have been made available through local shoreline erosion.

For the present (baseline) configuration of the training walls at Maria Creek, the area immediately within the entrance to the Creek and especially on the southern side of the entrance is sheltered from waves. As a result, any sediment or wrack which is transported there by waves and tidal/wind-driven currents during the flood stage of the tide is expected to be deposited and is unlikely to be remobilised.

Without the training walls in place the Creek channel would be expected to be very unstable, with it mainly being closed. The tidal prism of the Creek would need to be approximately an order of magnitude larger for the entrance channel to be more stable.

Four concept design options were proposed by Wavelength Consulting to test how they influence the physical processes around Maria Creek by applying the numerical models. The following concepts were assessed:

- Concept 1: Dredging of sand build-up (south of Southern training wall and within Maria Creek) and ongoing bypassing, with no structural changes;
- Concept 2: Extended and reconfigured training walls and dredging within Maria Creek;

- Concept 3: North training wall extended and widened within Maria Creek and dredging of sand build-up (south of Southern training wall and within Maria Creek); and
- Concept 4: Removal of training walls to return coastline to natural orientation.

The results from the modelling of the four options indicate:

- Concept 1: this concept does not include any structural changes to the breakwaters, so any changes to the wave and current dynamics are due to the capital dredging campaign. It is likely that the shoreline orientation would change quite quickly (likely over months to years depending on occurrence of larger wave events) to reach a dynamic equilibrium in the future. It is possible that the potential for wrack to be imported into the Creek could be reduced compared to the existing (baseline) case due to the reduced tidal current speeds flowing into the Creek. Despite this, the flows within the Creek on the whole remain flood dominant and the entrance remains unprotected from storm winds and waves, with some wrack accumulation still expected within the Creek for Concept 1. Sediment import into the Creek would be reduced relative to the existing (baseline) case since the dredging removed the sand bar present at the entrance to the creek which provides a supply of sediment to be imported into the Creek;
- Concept 2: it is expected that sediment would continue to build up on the southern side of the extended southern training wall and eventually the build-up would allow the sediment to naturally bypass the structure. The additional sheltering of the shoreline directly to the north of the northern training wall could also result in an increased build-up of sediment in this location. The protection of the entrance from storm winds and waves and the reduction in tidal prism which flows into and out of the Creek indicates a reduced potential for wrack to be imported into the Creek, although the flood dominance means that any wrack which is imported is likely to remain within the Creek. As for Concept 1, the sediment import into the Creek would be reduced relative to the baseline case since the dredging removed the sand bar present at the entrance to the Creek which provides a supply of sediment to be imported into the Creek;
- Concept 3: this design does not result in any significant changes compared to Concept 1; and
- Concept 4: the shoreline would likely evolve over time to become relatively straight over the Maria Creek section. Without the training walls in place the Creek channel would be expected to be very unstable, with it mainly being closed and occasionally opening during large freshwater discharge events.

Overall, the results indicate that ongoing longshore sediment transport would result in a build-up of sediment in the lee of all of the Concept structures and that in order to maintain the dredged depths within the Creek and to avoid the eventual bypassing of any structures, some form of sediment management would be required. While the import of wrack into Maria Creek could be reduced, none of the Concept designs would be likely to completely stop this process and so ongoing management of wrack would also be required to keep the Maria Creek channel clear.

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Appendices

Appendix A – Additional Plots

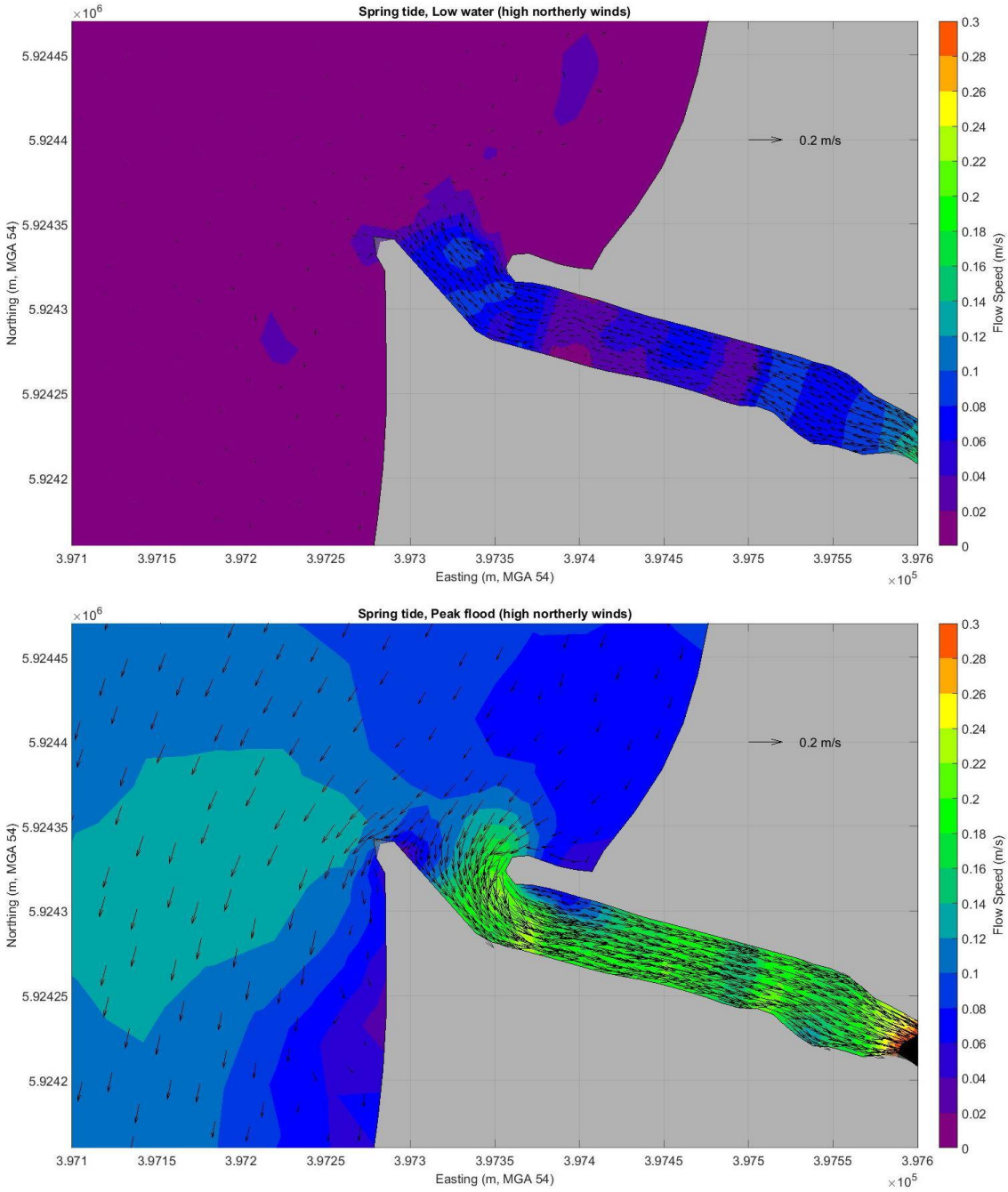

Figure A1. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a spring tide with high northerly winds for Baseline.

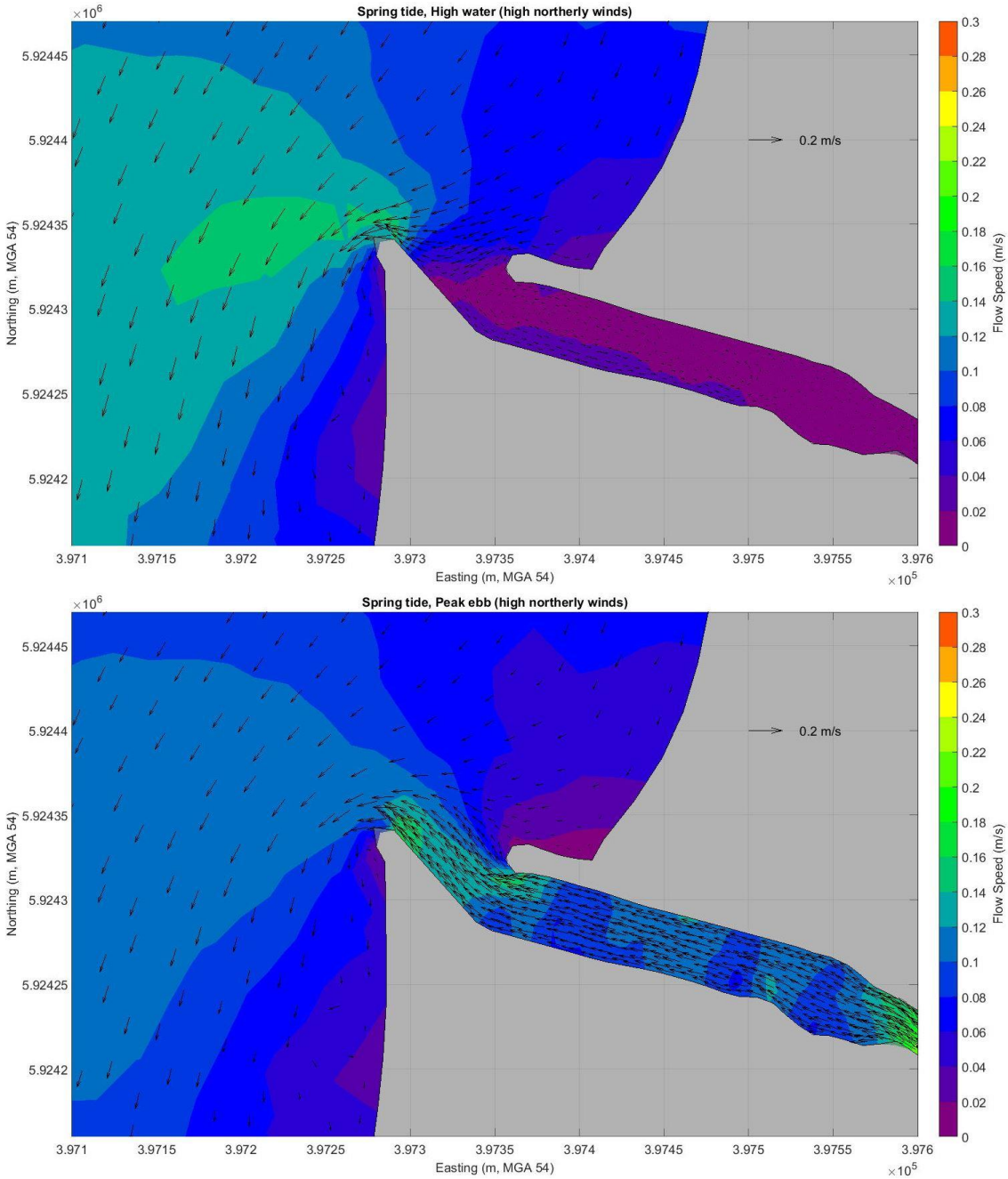

Figure A2. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with high northerly winds for Baseline.

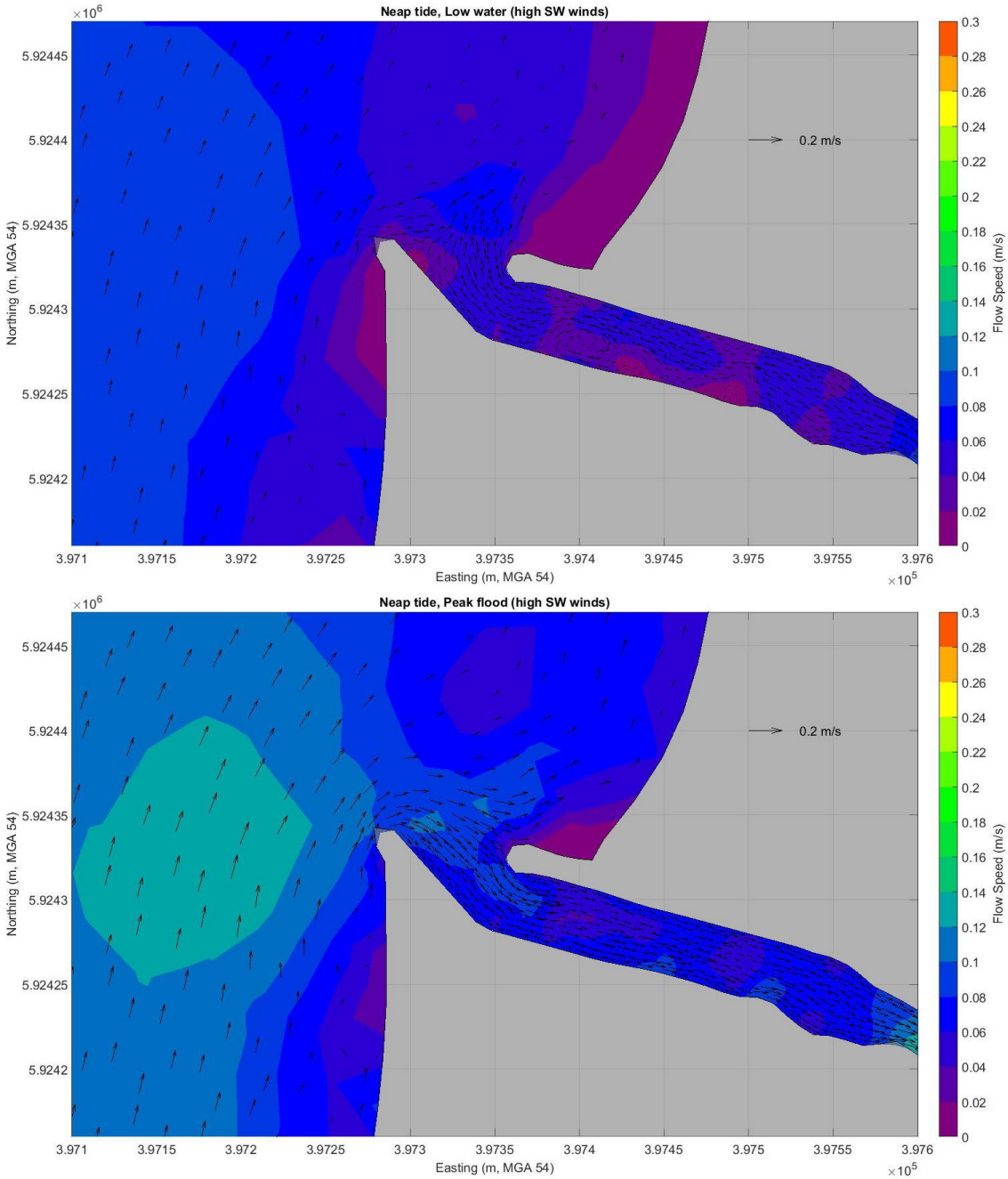

Figure A3. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a neap tide with high south westerly winds for Baseline.

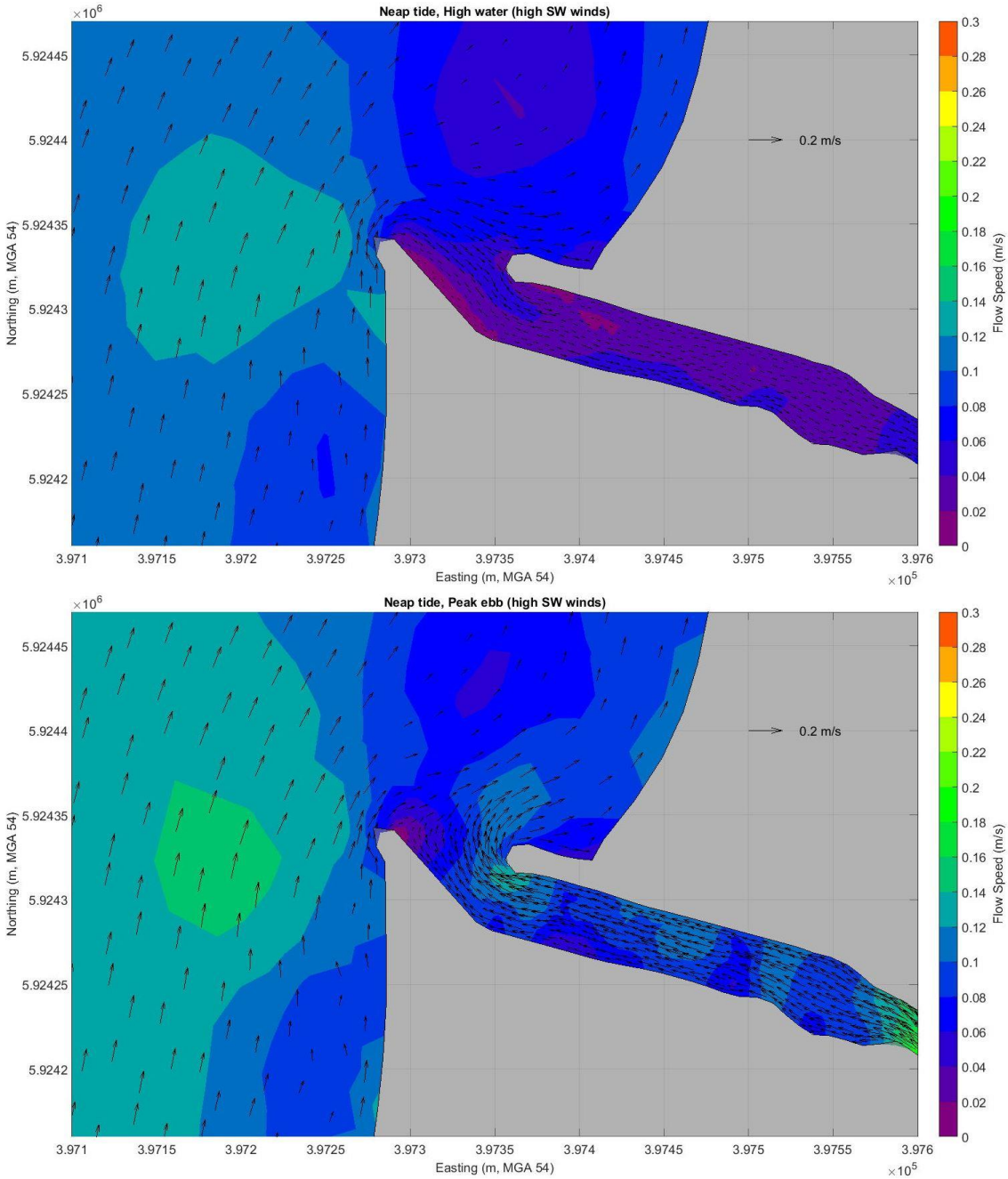

Figure A4. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a neap tide with high south westerly winds for Baseline.

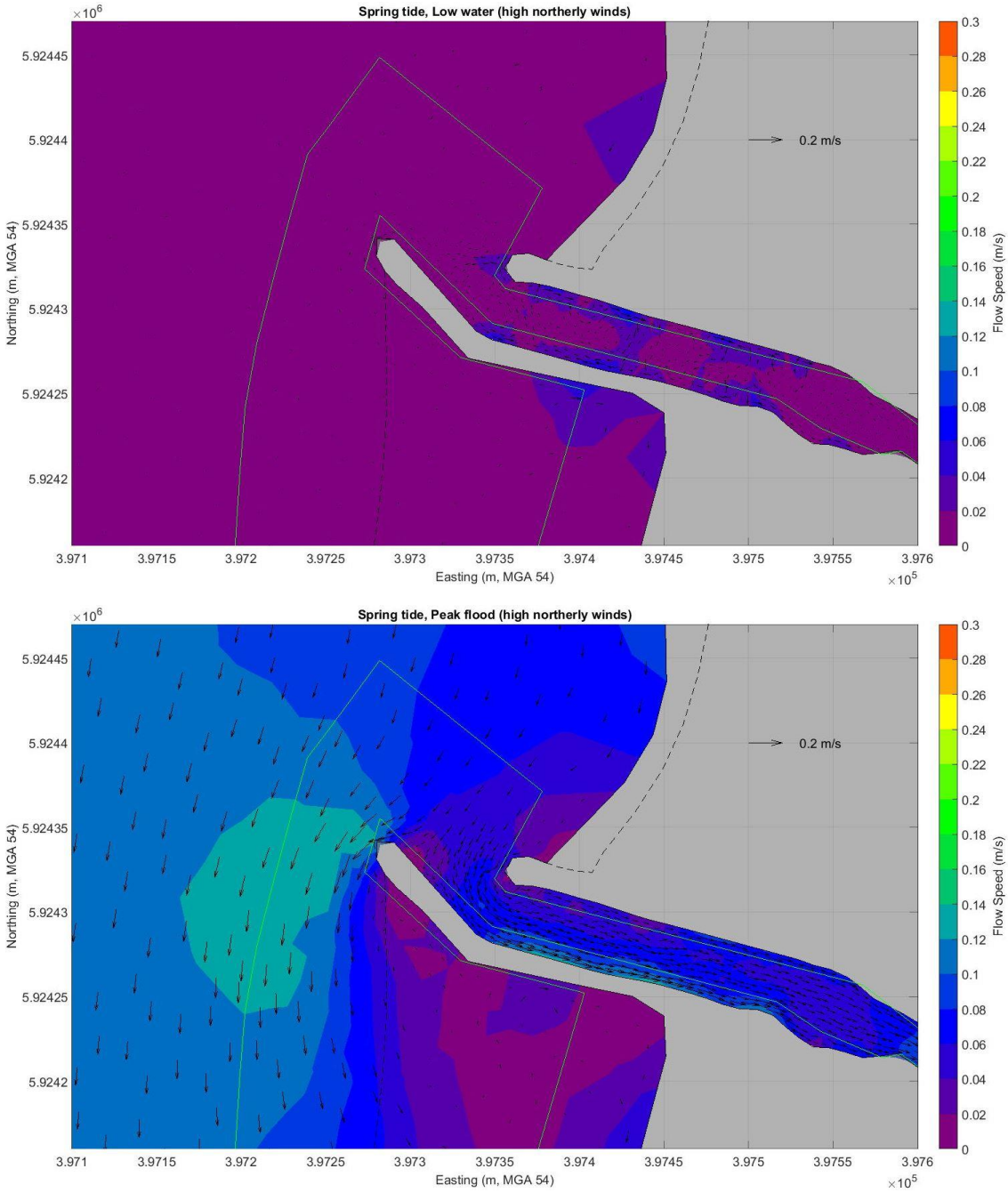

Figure A5. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a spring tide with high northerly winds for Concept 1.

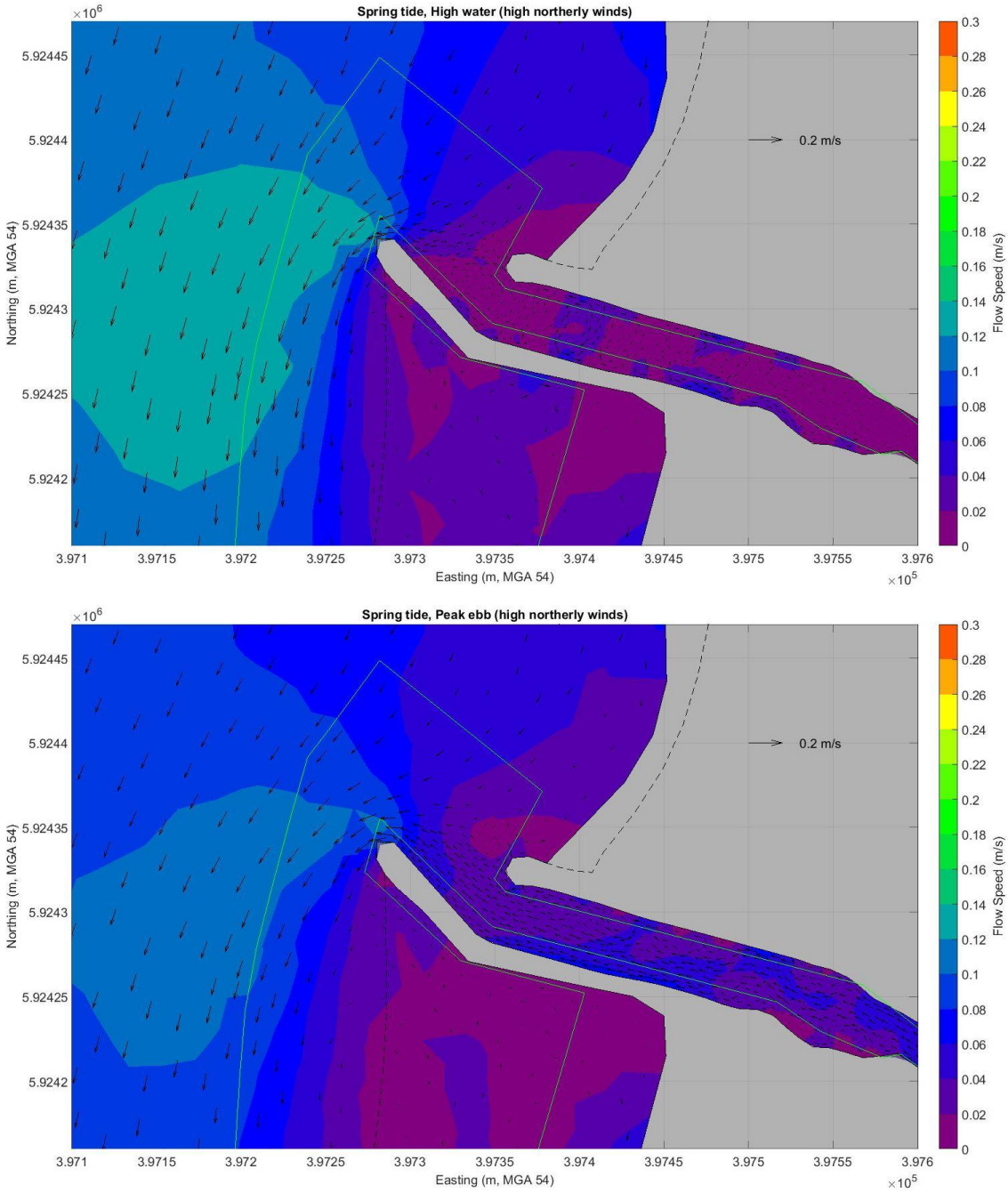

Figure A6. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with high northerly winds for Concept 1.

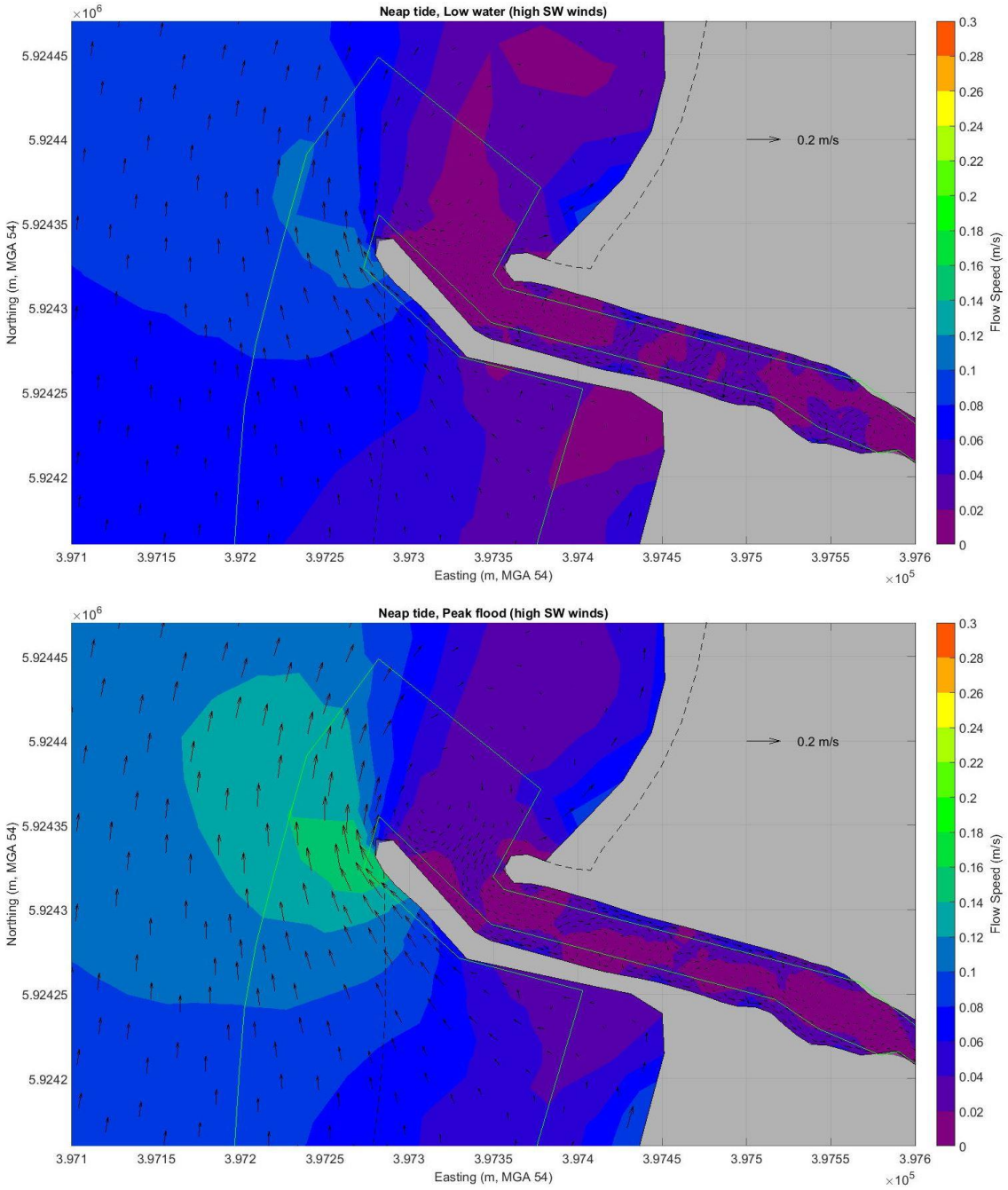

Figure A7. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a neap tide with high south westerly winds for Concept 1.

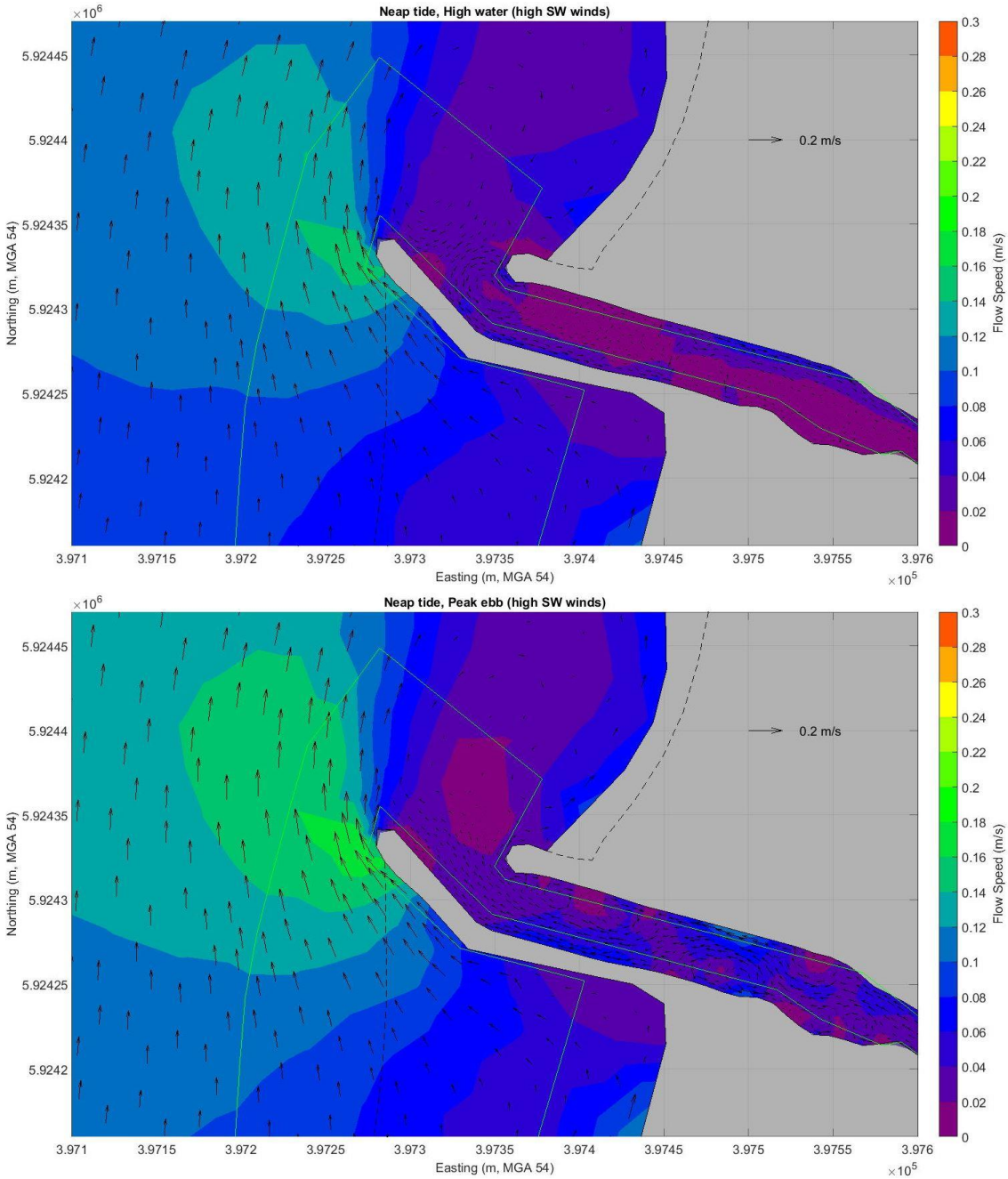

Figure A8. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a neap tide with high south westerly winds for Concept 1.

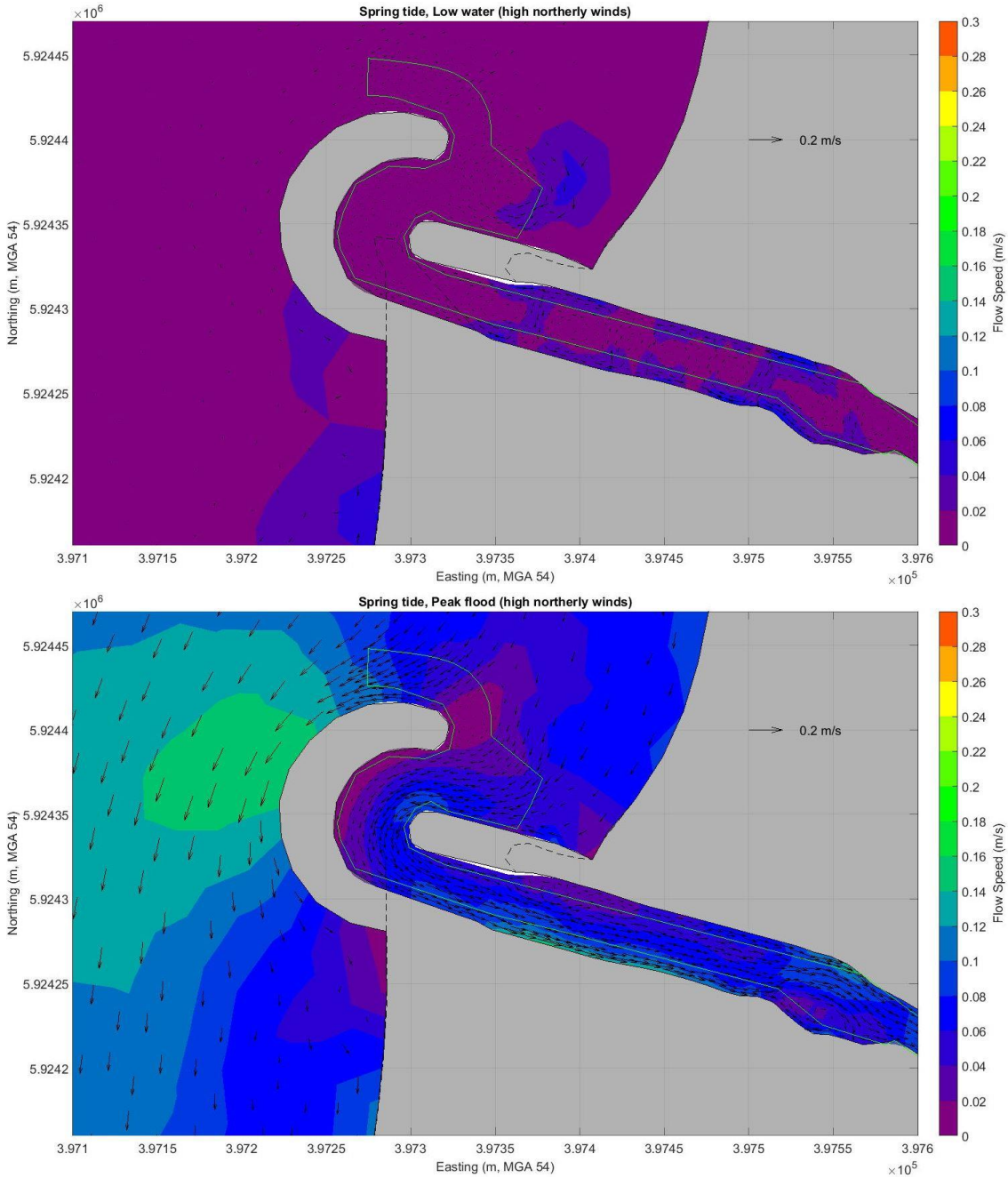

Figure A9. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a spring tide with high northerly winds for Concept 2.

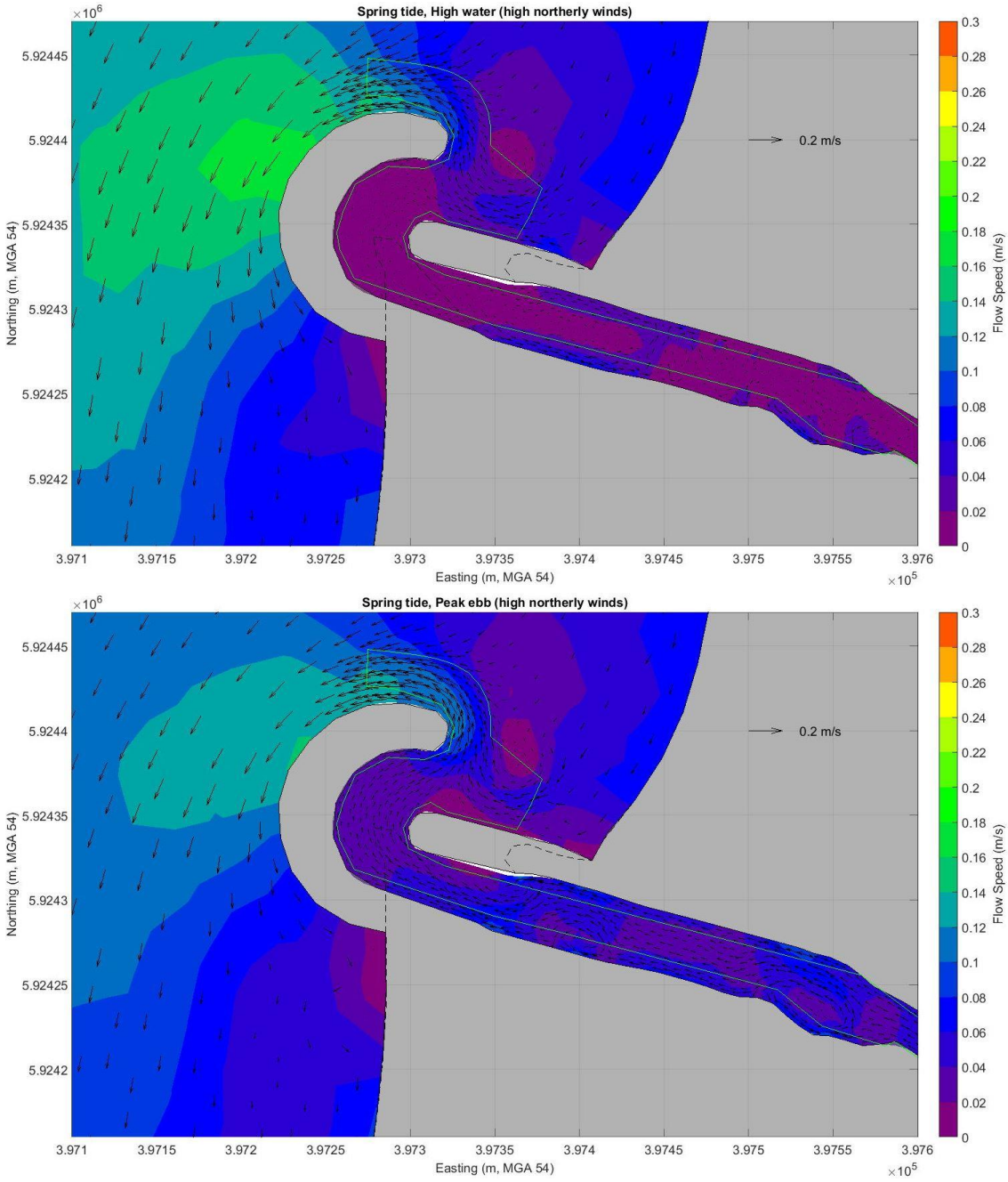

Figure A10. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with high northerly winds for Concept 2.

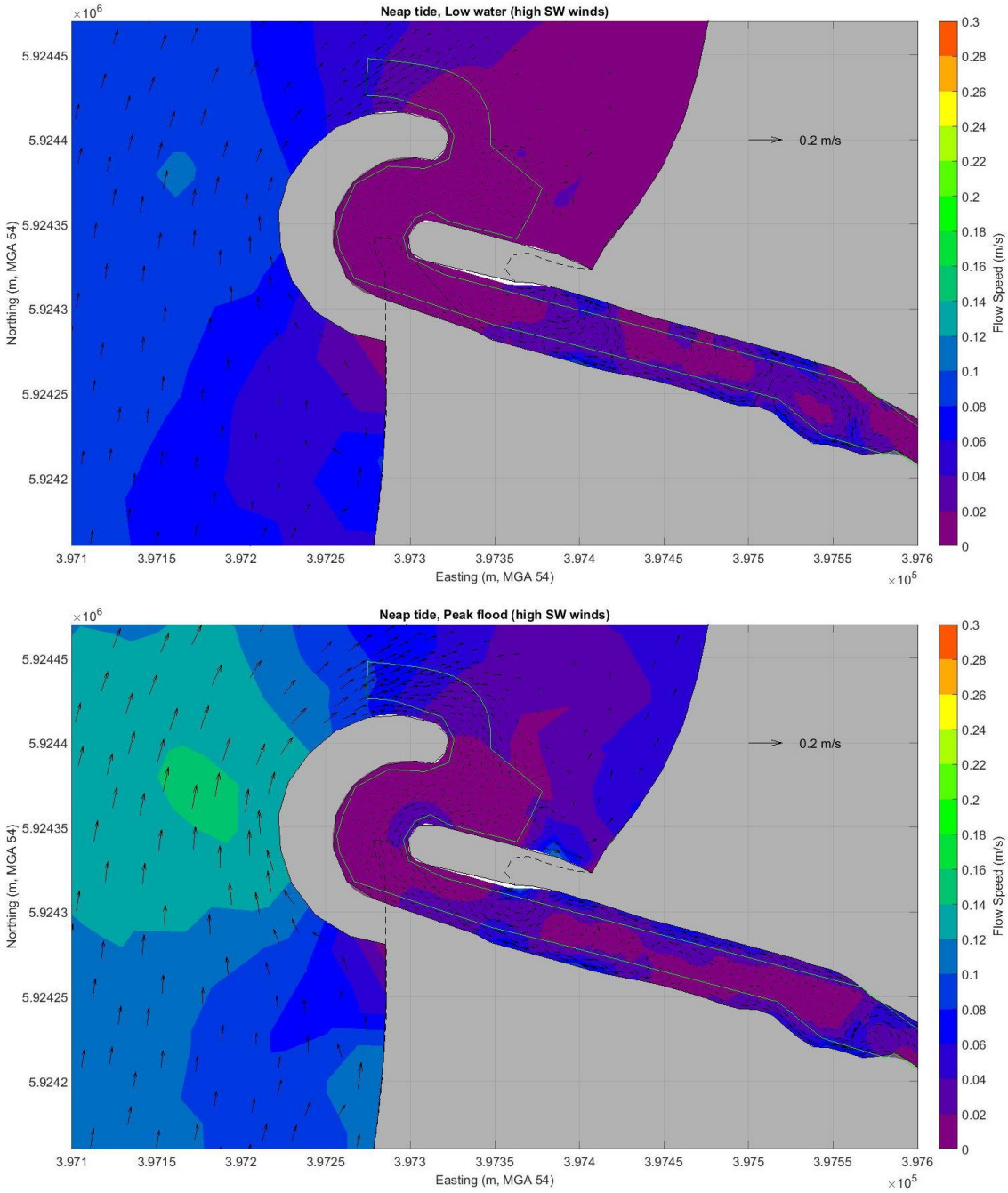

Figure A11. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a neap tide with high south westerly winds for Concept 2.

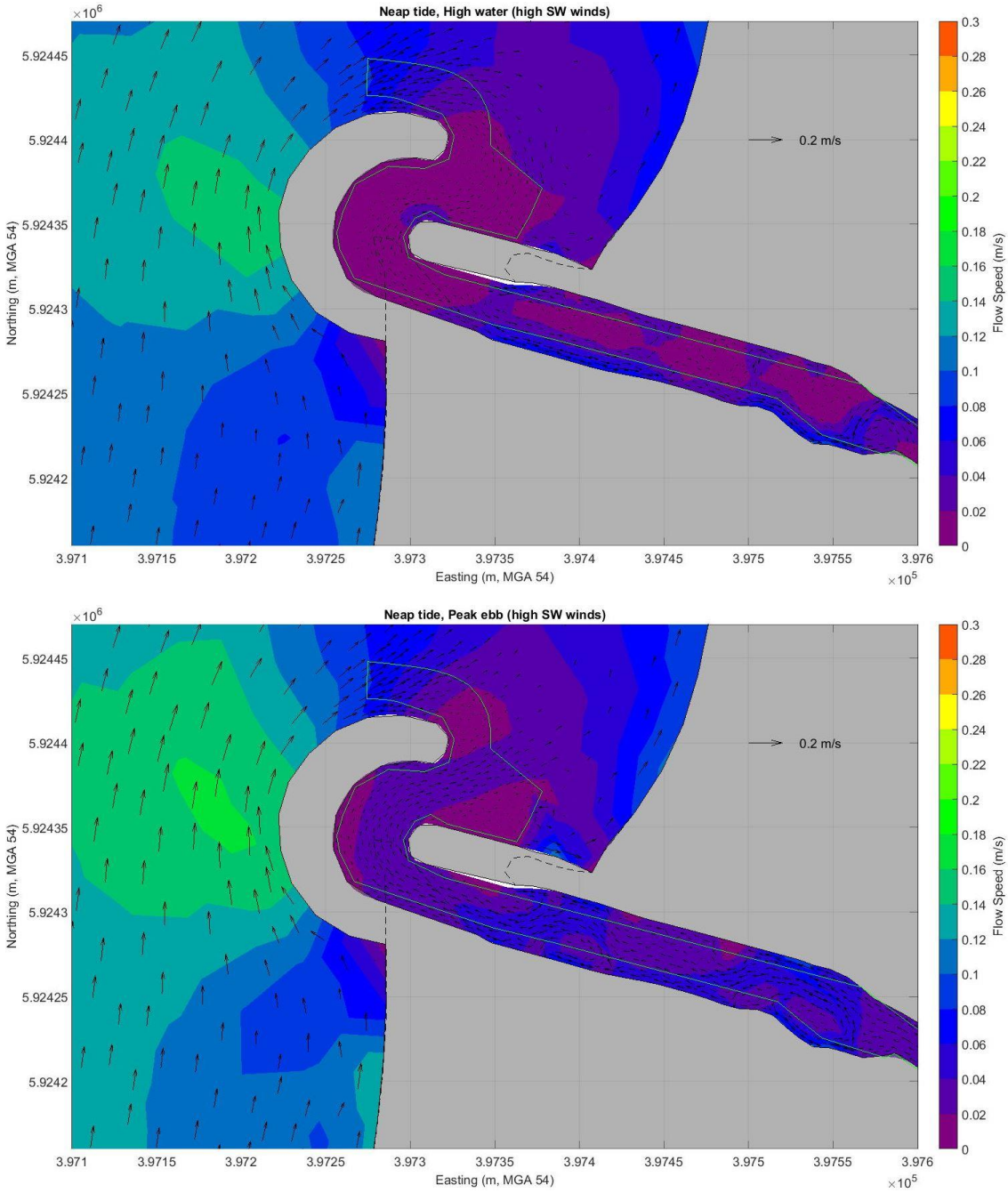

Figure A12. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a neap tide with high south westerly winds for Concept 2.

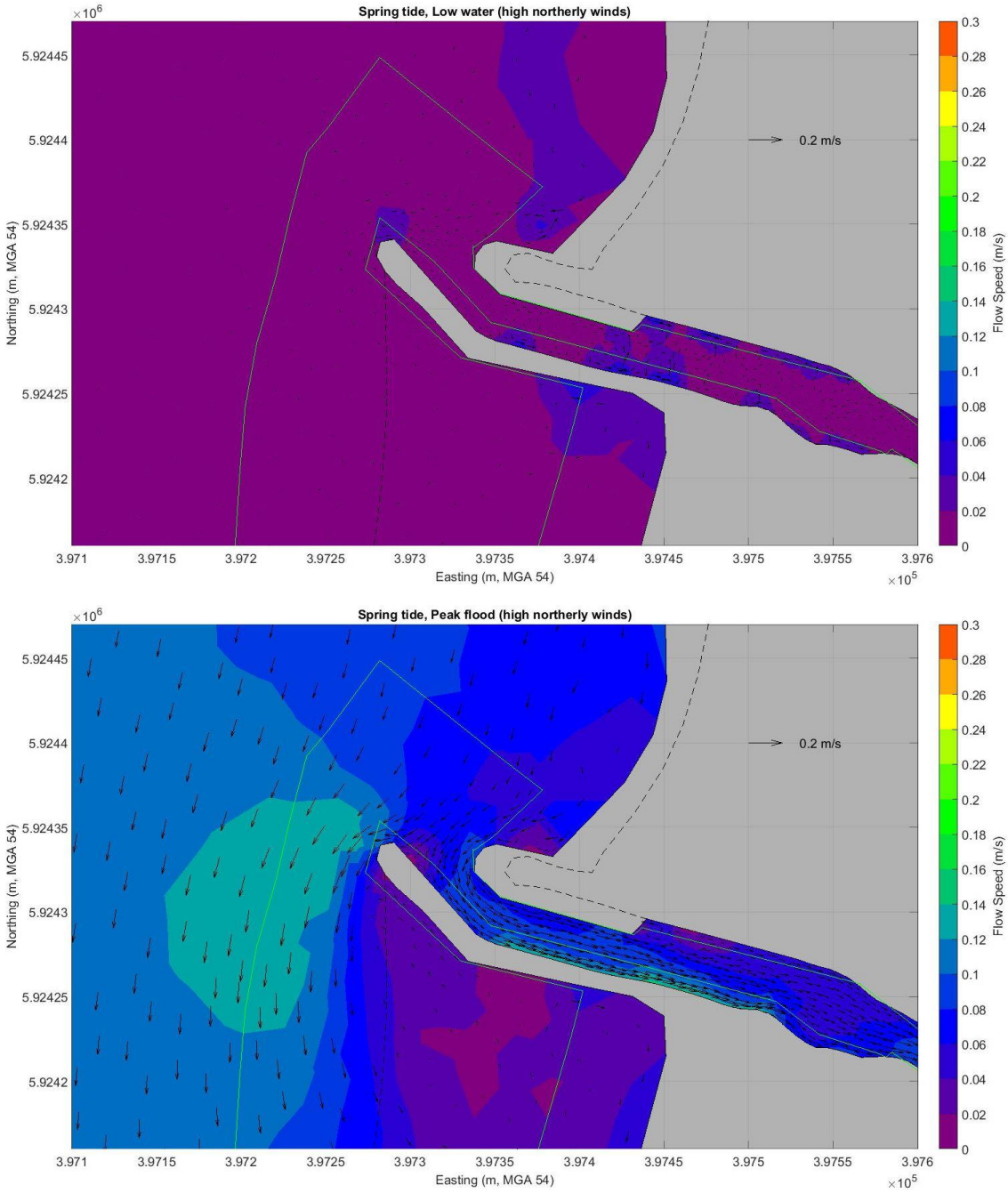

Figure A13. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a spring tide with high northerly winds for Concept 3.

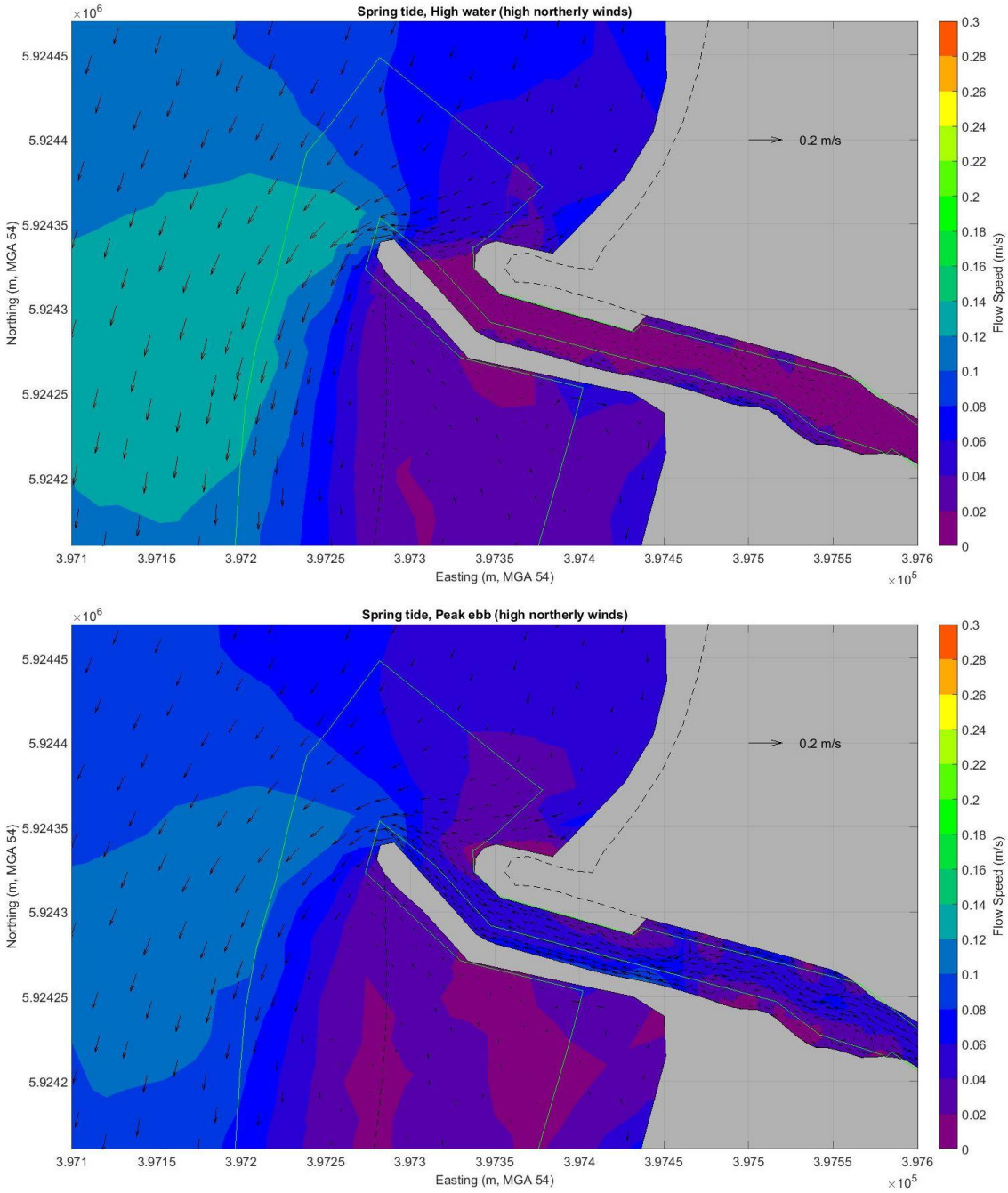

Figure A14. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a spring tide with high northerly winds for Concept 3.

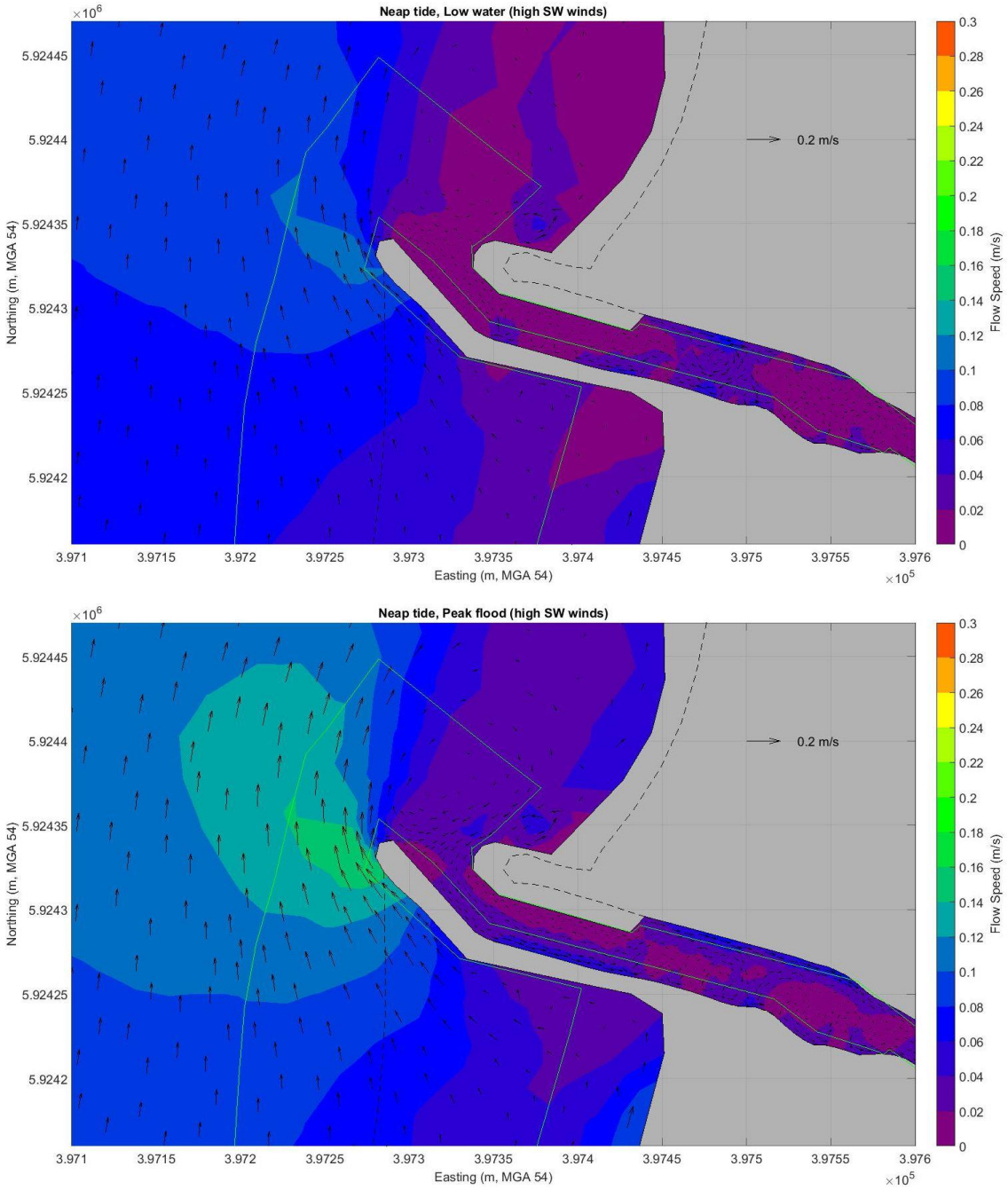

Figure A15. Modelled tidal current speeds around Maria Creek at low Water (top) and peak flood (bottom) for a neap tide with high south westerly winds for Concept 3.

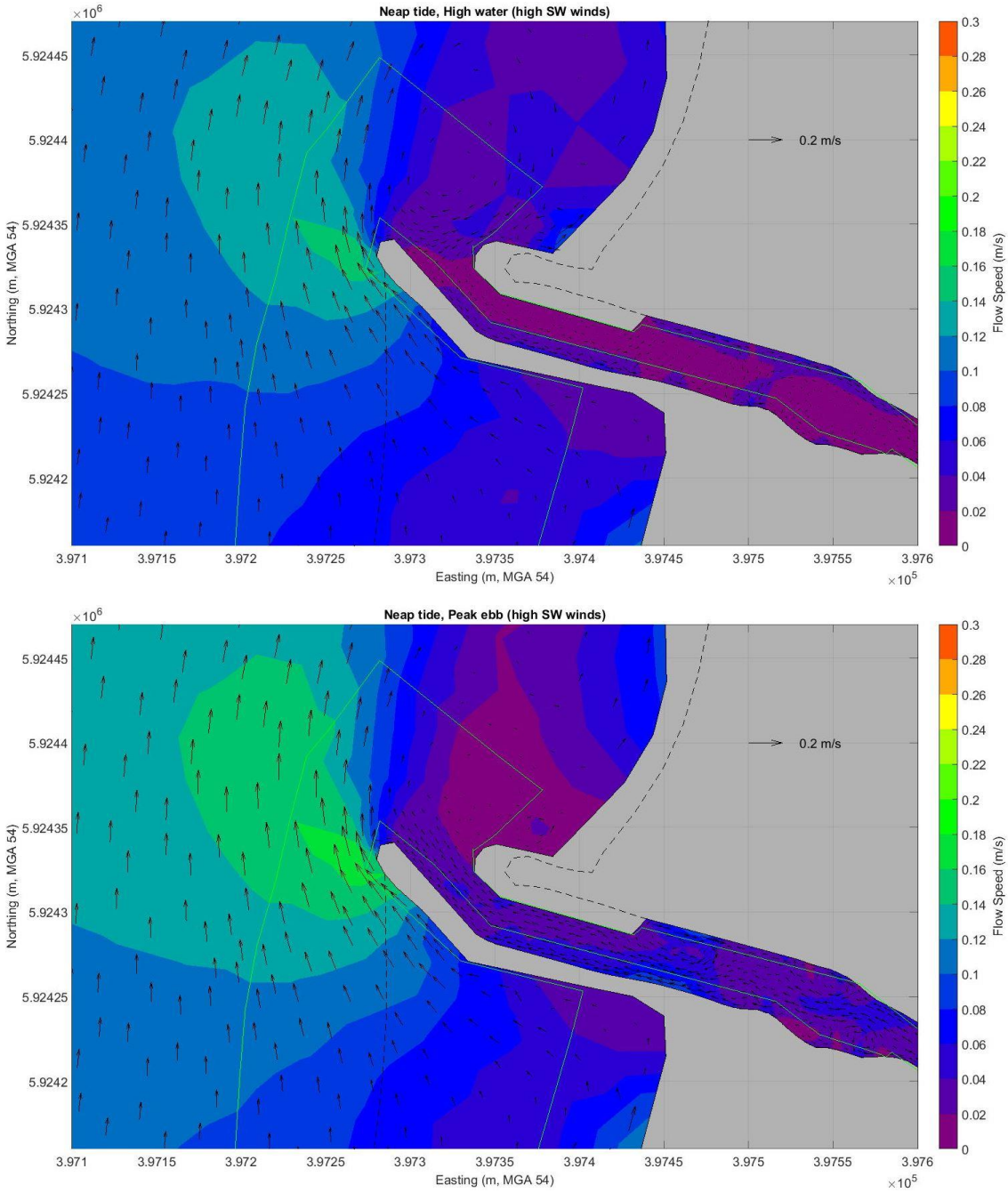

Figure A16. Modelled tidal current speeds around Maria Creek at high water (top) and peak ebb (bottom) for a neap tide with high south westerly winds for Concept 3.

Appendix E Maria Creek concept cost estimates and NPV calculations

Concept 1 - Breakwater Repairs

Item No.	Description	Unit	Quantity	Rate, $	Amount, $
1	**Preliminaries**	Item			$ 239,014
2	**South Breakwater Repairs**				
2.1	Remove external armour	t	5,906	$ 30	$ 177,188
2.2	Sort armour	t	5,906	$ 10	$ 59,063
2.3	Supply core	m^3	6,480	$ 47	$ 304,560
2.4	Place core	m^3	6,480	$ 30	$ 194,400
2.5	Supply 6t -8t armour	t	10,395	$ 60	$ 623,700
2.6	Place 6t -8t armour	t	10,395	$ 60	$ 623,700
2.7	Place layer of internal 1.5t armour	t	2,363	$ 50	$ 118,125
2.8	Supply external armour 50m length (3xlayers 1.5t)	t	2,756	$ 55	$ 151,594
2.9	Place external armour 50m length (3xlayers 1.5t)	t	2,756	$ 50	$ 137,813
	Sub-total (exc preliminaries)				$ 2,390,141
	Construction Total (inc preliminaries)				$ 2,629,155
	Contingency	%		20%	$ 525,831
	Approvals				$ 50,000
	Management	%		5%	$ 131,458
	Project Total (exc GST)				**$ 3,336,444**

Concept 1 - On-going Management NPV

Discount Rate 5

Years from Present	Discount Factor	Item	Nominal Cash Flow				Net Present Value			
			BW Repairs	Capital Dredging	BW Mtce	Sand & Wrack	Capital	Capital Dredging	BW Mtce	Sand & Wrack
0	1.00000	Option 1 - Capital construction	$ 3,336,444	$ 2,660,000		$ 502,500	$ 3,336,444	$ 2,660,000	$ -	$ 502,500
1	0.95238					$ 502,500	$ -	$ -	$ -	$ 478,571
2	0.90703					$ 502,500	$ -	$ -	$ -	$ 455,782
3	0.86384					$ 502,500	$ -	$ -	$ -	$ 434,078
4	0.82270					$ 502,500	$ -	$ -	$ -	$ 413,408
5	0.78353					$ 502,500	$ -	$ -	$ -	$ 393,722
6	0.74622					$ 502,500	$ -	$ -	$ -	$ 374,973
7	0.71068					$ 502,500	$ -	$ -	$ -	$ 357,117
8	0.67684					$ 502,500	$ -	$ -	$ -	$ 340,112
9	0.64461					$ 502,500	$ -	$ -	$ -	$ 323,916
10	0.61391	Repairs for settlement of rock armour			$ 183,290	$ 502,500	$ -	$ -	$ 112,524	$ 308,491
11	0.58468					$ 502,500	$ -	$ -	$ -	$ 293,801
12	0.55684					$ 502,500	$ -	$ -	$ -	$ 279,811
13	0.53032					$ 502,500	$ -	$ -	$ -	$ 266,486
14	0.50507					$ 502,500	$ -	$ -	$ -	$ 253,797
15	0.48102					$ 502,500	$ -	$ -	$ -	$ 241,711
16	0.45811					$ 502,500	$ -	$ -	$ -	$ 230,201
17	0.43630					$ 502,500	$ -	$ -	$ -	$ 219,239
18	0.41552					$ 502,500	$ -	$ -	$ -	$ 208,799
19	0.39573					$ 502,500	$ -	$ -	$ -	$ 198,856
20	0.37689	Repairs for storm damage			$ 183,290	$ 502,500	$ -	$ -	$ 69,080	$ 189,387
21	0.35894					$ 502,500	$ -	$ -	$ -	$ 180,369
22	0.34185					$ 502,500	$ -	$ -	$ -	$ 171,780
23	0.32557					$ 502,500	$ -	$ -	$ -	$ 163,600
24	0.31007					$ 502,500	$ -	$ -	$ -	$ 155,809
25	0.29530					$ 502,500	$ -	$ -	$ -	$ 148,390
			$ 3,336,444	**$ 2,660,000**	**$ 366,580**	**$ 13,065,000**	**$ 3,336,444**	**$ 2,660,000**	**$ 181,604**	**$ 7,584,707**
							$			**13,762,755**

Concept 2 Breakwater Cost Estimate

Item No.	Description	Unit	Quantity	Rate, $	Amount, $
1	**Preliminaries**	Item			$ 711,296
2	**Removal & Repair Works**				
2.1	Remove Armour & stockpile	t	11,183	$ 30	$ 335,475
2.2	Remove Core & stockpile	m^3	4,950	$ 30	$ 148,500
2.3	Sort armour	t	11,183	$ 10	$ 111,825
2.4	Sort Core	m^3	4,950	$ 10	$ 49,500
2.6	Place core	m3	2,880	$ 30	$ 86,400
2.7	Supply 6t armour	t	4,410	$ 60	$ 264,600
2.8	Place 6t armour	t	4,410	$ 60	$ 264,600
2.9	Place layer of internal 1.5t armour	t	1,260	$ 50	$ 63,000
3.1	Place external armour 50m length (3xlayers 1.5t)	t	2,756	$ 50	$ 137,813
3	**South Breakwater**				
3.1	Supply Core for breakwater	m^3	25,075	$ 47	$ 1,178,525
3.2	Place Core for breakwater	m^3	25,075	$ 20	$ 501,500
3.3	Supply 0.5t armour	t	8,820	$ 55	$ 485,100
3.4	Place 0.5t armour	t	8,820	$ 50	$ 441,000
3.5	Supply 6t -8t granite armour	t	17,561	$ 60	$ 1,053,675
3.6	Place 6t -8t granite armour	t	17,561	$ 60	$ 1,053,675
3	**North Breakwater**				
3.1	Supply Core for breakwater	m^3	5,950	$ 47	$ 279,650
3.2	Place Core for breakwater	m^3	5,950	$ 20	$ 119,000
3.3	Supply 0.5t armour	t	1,418	$ 55	$ 77,963
3.4	Place 0.5t armour	t	1,418	$ 50	$ 70,875
3.5	Supply 4t armour	t	3,717	$ 55	$ 204,435
3.6	Place 4t armour	t	3,717	$ 50	$ 185,850
	Sub-total (exc preliminaries)				$ 7,112,960
	Construction Total (inc preliminaries)				$ 7,824,256
	Contingency	%		20%	$ 1,564,851
	Approvals				$ 100,000
	Management	%		5%	$ 391,213
	Project Total (exc GST)				**$ 9,880,320**

Concept 2 - Extend Breakwaters NPV

Discount Rate 5

Years from Present	Discount Factor	Item	Nominal Cash Flow				Net Present Value			
			Breakwater Capital	Dredging Capital	BW Mtce	Sand & Wrack	Breakwater Capital	Dredging Capital	BW Mtce	Sand & Wrack
0	1.00000	Option 1 - Capital construction	$ 9,880,320	$ 1,080,000		$ 421,500	$ 9,880,320	$ 1,080,000	$ -	$ 421,500
1	0.95238					$ 421,500	$ -	$ -	$ -	$ 401,429
2	0.90703					$ 421,500	$ -	$ -	$ -	$ 382,313
3	0.86384					$ 421,500	$ -	$ -	$ -	$ 364,108
4	0.82270					$ 421,500	$ -	$ -	$ -	$ 346,769
5	0.78353					$ 421,500	$ -	$ -	$ -	$ 330,256
6	0.74622					$ 421,500	$ -	$ -	$ -	$ 314,530
7	0.71068					$ 421,500	$ -	$ -	$ -	$ 299,552
8	0.67684					$ 421,500	$ -	$ -	$ -	$ 285,288
9	0.64461					$ 421,500	$ -	$ -	$ -	$ 271,703
10	0.61391	Repairs for settlement of rock armour			$ 302,229	$ 421,500	$ -	$ -	$ 185,543	$ 258,764
11	0.58468					$ 421,500	$ -	$ -	$ -	$ 246,442
12	0.55684					$ 421,500	$ -	$ -	$ -	$ 234,707
13	0.53032					$ 421,500	$ -	$ -	$ -	$ 223,530
14	0.50507					$ 421,500	$ -	$ -	$ -	$ 212,886
15	0.48102					$ 421,500	$ -	$ -	$ -	$ 202,749
16	0.45811					$ 421,500	$ -	$ -	$ -	$ 193,094
17	0.43630					$ 421,500	$ -	$ -	$ -	$ 183,899
18	0.41552					$ 421,500	$ -	$ -	$ -	$ 175,142
19	0.39573					$ 421,500	$ -	$ -	$ -	$ 166,802
20	0.37689	Repairs for storm damage			$ 302,229	$ 421,500	$ -	$ -	$ 113,907	$ 158,859
21	0.35894					$ 421,500	$ -	$ -	$ -	$ 151,294
22	0.34185					$ 421,500	$ -	$ -	$ -	$ 144,090
23	0.32557					$ 421,500	$ -	$ -	$ -	$ 137,228
24	0.31007					$ 421,500	$ -	$ -	$ -	$ 130,694
25	0.29530					$ 421,500	$ -	$ -	$ -	$ 124,470
			$ 9,880,320	**$ 1,080,000**	**$ 604,459**	**$ 10,959,000**	**$ 9,880,320**	**$ 1,080,000**	**$ 299,450**	**$ 6,362,098**
							$			**17,621,867**

Concept 3 - Breakwater Cost Estimate

Item No.	Description	Unit	Quantity	Rate, $	Amount, $
1	**Preliminaries**	Item			$ 285,313
2	**South Breakwater Repairs**				
2.1	Remove external armour	t	5,906	$ 30	$ 177,188
2.2	Sort armour	t	5,906	$ 10	$ 59,063
2.3	Supply core	m^3	6,480	$ 47	$ 304,560
2.4	Place core	m^3	6,480	$ 30	$ 194,400
2.5	Supply 6t armour	t	9,923	$ 60	$ 595,350
2.6	Place 6t armour	t	9,923	$ 60	$ 595,350
2.7	Place layer of internal 1.5t armour	t	2,363	$ 50	$ 118,125
2.8	Supply external armour 50m length (3xlayers 1.5t)	t	2,756	$ 55	$ 151,594
2.9	Place external armour 50m length (3xlayers 1.5t)	t	2,756	$ 50	$ 137,813
3	**North Breakwater**				
3.1	Remove Armour & stockpile	t	2,363	$ 30	$ 70,875
3.2	Supply Core for breakwater	m^3	3,450	$ 47	$ 162,150
3.3	Place Core for breakwater	m^3	3,450	$ 20	$ 69,000
3.4	Supply 0.5t armour	t	0	$ 55	$ -
3.5	Place 0.5t armour	t	1,575	$ 50	$ 78,750
3.6	Supply 4t armour	t	1,323	$ 55	$ 72,765
3.7	Place 4t armour	t	1,323	$ 50	$ 66,150
	Sub-total (exc preliminaries)				$ 2,853,131
	Construction Total (inc preliminaries)				$ 3,138,444
	Contingency	%		20%	$ 627,689
	Approvals				$ 100,000
	Management	%		5%	$ 156,922
	Project Total (exc GST)				**$ 4,023,055**

Concept 3 - Narrow Entrance NPV

Discount Rate 5

Years from Present	Discount Factor	Item	Nominal Cash Flow				Net Present Value			
			Breakwater Capital	Dredging Capital	BW Mtce	Sand & Wrack	Breakwater Capital	Dredging Capital	BW Mtce	Sand & Wrack
0	1.00000	Option 1 - Capital construction	$ 4,023,055	$ 2,652,000		$ 502,500	$ 4,023,055	$ 2,652,000	$ -	$ 502,500
1	0.95238					$ 502,500	$ -	$ -	$ -	$ 478,571
2	0.90703					$ 502,500	$ -	$ -	$ -	$ 455,782
3	0.86384					$ 502,500	$ -	$ -	$ -	$ 434,078
4	0.82270					$ 502,500	$ -	$ -	$ -	$ 413,408
5	0.78353					$ 502,500	$ -	$ -	$ -	$ 393,722
6	0.74622					$ 502,500	$ -	$ -	$ -	$ 374,973
7	0.71068					$ 502,500	$ -	$ -	$ -	$ 357,117
8	0.67684					$ 502,500	$ -	$ -	$ -	$ 340,112
9	0.64461					$ 502,500	$ -	$ -	$ -	$ 323,916
10	0.61391	Repairs for settlement of rock armour			$ 183,290	$ 502,500	$ -	$ -	$ 112,524	$ 308,491
11	0.58468					$ 502,500	$ -	$ -	$ -	$ 293,801
12	0.55684					$ 502,500	$ -	$ -	$ -	$ 279,811
13	0.53032					$ 502,500	$ -	$ -	$ -	$ 266,486
14	0.50507					$ 502,500	$ -	$ -	$ -	$ 253,797
15	0.48102					$ 502,500	$ -	$ -	$ -	$ 241,711
16	0.45811					$ 502,500	$ -	$ -	$ -	$ 230,201
17	0.43630					$ 502,500	$ -	$ -	$ -	$ 219,239
18	0.41552					$ 502,500	$ -	$ -	$ -	$ 208,799
19	0.39573					$ 502,500	$ -	$ -	$ -	$ 198,856
20	0.37689	Repairs for storm damage			$ 183,290	$ 502,500	$ -	$ -	$ 69,080	$ 189,387
21	0.35894					$ 502,500	$ -	$ -	$ -	$ 180,369
22	0.34185					$ 502,500	$ -	$ -	$ -	$ 171,780
23	0.32557					$ 502,500	$ -	$ -	$ -	$ 163,600
24	0.31007					$ 502,500	$ -	$ -	$ -	$ 155,809
25	0.29530					$ 502,500	$ -	$ -	$ -	$ 148,390
			$ 4,023,055	**$ 2,652,000**	**$ 366,580**	**$ 13,065,000**	**$ 4,023,055**	**$ 2,652,000**	**$ 181,604**	**$ 7,584,707**
							$			**14,441,367**

Appendix F Kingston Jetty and Foreshore concept cost estimates and NPV calculations

Concept 1 - On-going Management Additional Dredging for Jetty NPV

Discount Rate 5

Years from Present	Discount Factor	Item	Nominal Cash Flow	Net Present Value
			Jetty Additional Dredging	Jetty Additional Dredging
0	1.00000			$ -
1	0.95238	120,000 m3 dredging	$ 960,000	$ 914,286
2	0.90703	60,000 m3 dredging	$ 480,000	$ 435,374
3	0.86384	30,000 m3 dredging	$ 240,000	$ 207,321
4	0.82270	30,000 m3 dredging	$ 240,000	$ 197,449
5	0.78353			$ -
6	0.74622			$ -
7	0.71068			$ -
8	0.67684			$ -
9	0.64461			$ -
10	0.61391			$ -
11	0.58468			$ -
12	0.55684			$ -
13	0.53032			$ -
14	0.50507			$ -
15	0.48102			$ -
16	0.45811			$ -
17	0.43630			$ -
18	0.41552			$ -
19	0.39573			$ -
20	0.37689			$ -
21	0.35894			$ -
22	0.34185			$ -
23	0.32557			$ -
24	0.31007			$ -
25	0.29530			$ -
			$ 1,920,000	**$ 1,754,429**

Concept 2 - Extend Breakwaters Additional Dredging for Jetty NPV

Discount Rate 5

Years from Present	Discount Factor	Item	Nominal Cash Flow	Net Present Value
			Jetty Additional Dredging	Jetty Additional Dredging
0	1.00000	200,000 m3 dredging	$ 1,600,000	$ 1,600,000
1	0.95238	120,000 m3 dredging	$ 960,000	$ 914,286
2	0.90703	60,000 m3 dredging	$ 480,000	$ 435,374
3	0.86384	30,000 m3 dredging	$ 240,000	$ 207,321
4	0.82270	30,000 m3 dredging	$ 240,000	$ 197,449
5	0.78353			$ -
6	0.74622			$ -
7	0.71068			$ -
8	0.67684			$ -
9	0.64461			$ -
10	0.61391			$ -
11	0.58468			$ -
12	0.55684			$ -
13	0.53032			$ -
14	0.50507			$ -
15	0.48102			$ -
16	0.45811			$ -
17	0.43630			$ -
18	0.41552			$ -
19	0.39573			$ -
20	0.37689			$ -
21	0.35894			$ -
22	0.34185			$ -
23	0.32557			$ -
24	0.31007			$ -
25	0.29530			$ -
			$ 3,520,000	**$ 3,354,429**

Concept 3 - Narrow Entrance Additional Dredging for Jetty NPV

Discount Rate 5

Years from Present	Discount Factor	Item	Nominal Cash Flow	Net Present Value
			Jetty Additional Dredging	Jetty Additional Dredging
0	1.00000			$ -
1	0.95238	120,000 m3 dredging	$ 960,000	$ 914,286
2	0.90703	60,000 m3 dredging	$ 480,000	$ 435,374
3	0.86384	30,000 m3 dredging	$ 240,000	$ 207,321
4	0.82270	30,000 m3 dredging	$ 240,000	$ 197,449
5	0.78353			$ -
6	0.74622			$ -
7	0.71068			$ -
8	0.67684			$ -
9	0.64461			$ -
10	0.61391			$ -
11	0.58468			$ -
12	0.55684			$ -
13	0.53032			$ -
14	0.50507			$ -
15	0.48102			$ -
16	0.45811			$ -
17	0.43630			$ -
18	0.41552			$ -
19	0.39573			$ -
20	0.37689			$ -
21	0.35894			$ -
22	0.34185			$ -
23	0.32557			$ -
24	0.31007			$ -
25	0.29530			$ -
			$ 1,920,000	**$ 1,754,429**

Concept 4 - Remove Breakwater Cost Estimate

Item No.	Description	Unit	Quantity	Rate, $	Amount, $
1	**Preliminaries**	Item			$ 163,335
2	**Removal Works**				
2.1	Remove Armour & stockpile	t	32,445	30.00	$ 973,350
2.2	Remove Core & stockpile	m^3	22,000	30.00	$ 660,000
	Sub-total (exc preliminaries)				$ 1,633,350
	Construction Total (inc preliminaries)				$ 1,796,685
	Contingency	%		20%	$ 359,337
	Approvals				$ 50,000
	Shoreline Evolution Modelling and wave measurements				$ 50,000
	Management	%		5%	$ 89,834
	Project Total (exc GST)				**2,345,856**

Concept 4 - Remove Breakwaters NPV

Discount Rate 5

Years from Present	Discount Factor	Item	Nominal Cash Flow		Net Present Value	
			Capital	Sand & Wrack	Capital	Sand & Wrack
0	1.00000	Concept 4 remove breakwaters	$ 2,345,856	$ 22,500	$ 2,345,856	$ 22,500
1	0.95238			$ 22,500	$ -	$ 21,429
2	0.90703			$ 22,500	$ -	$ 20,408
3	0.86384			$ 22,500	$ -	$ 19,436
4	0.82270			$ 22,500	$ -	$ 18,511
5	0.78353			$ 22,500	$ -	$ 17,629
6	0.74622			$ 22,500	$ -	$ 16,790
7	0.71068			$ 22,500	$ -	$ 15,990
8	0.67684			$ 22,500	$ -	$ 15,229
9	0.64461			$ 22,500	$ -	$ 14,504
10	0.61391			$ 22,500	$ -	$ 13,813
11	0.58468			$ 22,500	$ -	$ 13,155
12	0.55684			$ 22,500	$ -	$ 12,529
13	0.53032			$ 22,500	$ -	$ 11,932
14	0.50507			$ 22,500	$ -	$ 11,364
15	0.48102			$ 22,500	$ -	$ 10,823
16	0.45811			$ 22,500	$ -	$ 10,308
17	0.43630			$ 22,500	$ -	$ 9,817
18	0.41552			$ 22,500	$ -	$ 9,349
19	0.39573			$ 22,500	$ -	$ 8,904
20	0.37689			$ 22,500	$ -	$ 8,480
21	0.35894			$ 22,500	$ -	$ 8,076
22	0.34185			$ 22,500	$ -	$ 7,692
23	0.32557			$ 22,500	$ -	$ 7,325
24	0.31007			$ 22,500	$ -	$ 6,977
25	0.29530			$ 22,500	$ -	$ 6,644
			$ 2,345,856	**$ 585,000**	**$ 2,345,856**	**$ 339,614**
					$	**2,685,470**

Appendix G DPTI SA Boat Ramp Service Level Categories

Boatramp Performance Rating			Service Level
1	Beach launch and retrieval		<u>Minimum levels of service</u> • Safe vehicle access • Unsealed ramp, beach access or sound sand foundation • Tidal access only • Limited manoeuvring capabilities • Limited protection from sea conditions (use by experienced mariners only) *Examples: Farm Beach, Hardwicke Bay.*
2A Marine	Ramp launch and retrieval		• Safe vehicle access • Sealed ramp (concrete) up to the low water mark • Tidal access only • Appropriate manoeuvring area • Limited rigging/de-rigging and parking facilities • Limited protection from sea conditions *Examples: Emu Bay, Anxious Bay, Southend.*
2B Inland Waters			• Safe vehicle access • Sealed ramp (concrete) up to pool level • Access to river pool level • Appropriate manoeuvring area • Limited rigging/de-rigging and parking facilities *Examples: Qualco, Rilli, Greenways Landing*
3A Marine	Floating pontoon or landing structures launch and retrieval		• Safe vehicle access • Sealed ramp (concrete) • Tidal access only • Limited manoeuvring area • Limited rigging/de-rigging and parking facilities • Limited protection from sea conditions • All tide safe launch and retrieval of boats (pontoon or fixed landings) *Examples: Blackfellows Caves*
3B Inland Waters			• Safe vehicle access • Sealed ramp (concrete) • Access to river pool level • Limited manoeuvring area • Limited rigging/de-rigging and parking facilities • All water level safe launch and retrieval of boats (pontoon or fixed landings) *Examples: Bruno Bay (Cobdogla)*
4			• Safe vehicle access • Sealed ramp (concrete) • All-weather, all tide ramp with weather protection • Manoeuvring area • Rigging/de-rigging and parking facilities • Safe launch and retrieval of boats (pontoon or fixed landings) *Examples: Stansbury, Point Turton, Adelaide Shores, O'Sullivans Beach*
5			• Safe vehicle access • Sealed ramp (concrete) • All-weather, all tide concrete ramp with weather protection • Manoeuvring area • Rigging/de-rigging and parking facilities • Safe launch and retrieval of boats (pontoon or fixed landings) • Servicing of boats available (refuelling, lay-by wharf) *Examples: Port MacDonnell, Coffin Bay, Lincoln Cove Marina*

Boatramp Performance Rating	Ramp	Manoeuvring	Launch and Retrieval	Car and Trailer Parking	Rigging and De-Rigging	Wave Protection	Services
1	■	■					
2	✓	■	■ (non tidal, fixed structure)	■	■	■	
3	✓	■	✓	■	■	■	
4	✓	✓	✓	✓	✓	✓	■
5	✓	✓	✓	✓	✓	✓	✓

Legend

■	**Partial Capability**
✓	**Full Capability** Ramp – design lane width, slope, head and toe level characteristics to SABFAC* design guidelines. Car and Trailer Parking – formalised car and trailer parking area with circulation lanes. Manoeuvring – area extends 30m landward beyond top of ramp and 20 m wide (minimum). Wave Protection – all tide all weather access with minimal restrictions due to adverse wave climate. Launch and retrieval – access boat from floating pontoon or fixed landing system. Services - provision within reach of lay-by berth, refuelling, power, water services.

** SA Boating Facilities Advisory Committee*