6 Modelling of Shoreline Processes

6.1 Introduction

The understanding of the Northern Moreton Bay study area shoreline processes has been extended using validated numerical models. These models facilitate description of complex interactions of processes, including those not able to be measured directly for practical and logistical reasons, and were used as the key method of sediment transport potential assessment. They have been shown in many previous studies to simulate the hydrodynamic processes reliably and in a manner suitable for assessments to support shoreline management decision making.

Spectral wave modelling based on the SWAN software system was used to describe the wave climate and wave propagation. SWAN is an industry standard modelling system and has been coupled with the hydrodynamic model TUFLOW FV to cater for interaction of wave, water level and current processes and their effects on sediment transport and shoreline processes. TUFLOW FV is a finite volume model that simulates hydrodynamic processes within a flexible mesh computational mesh format.

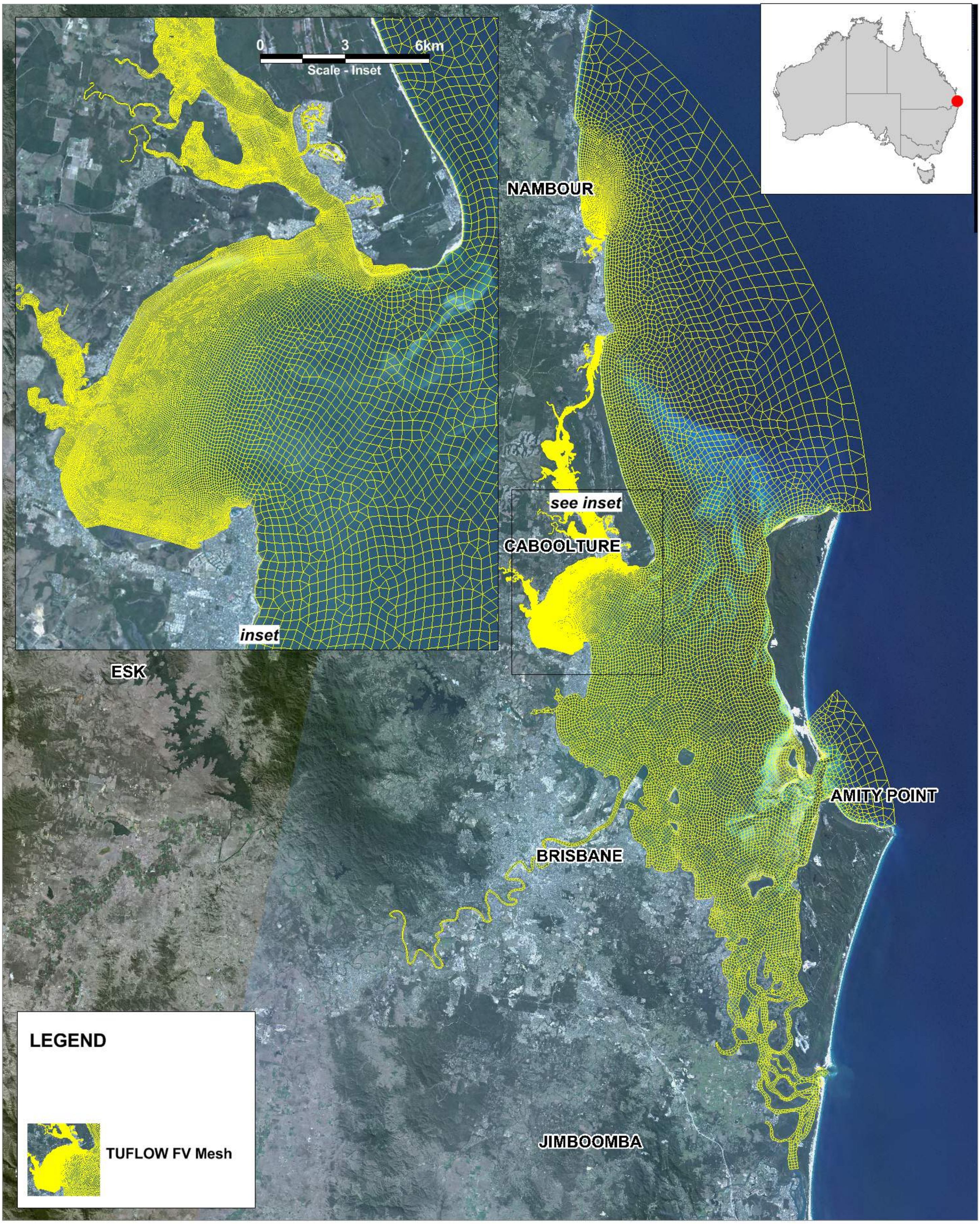
A key advantage of employing the flexible mesh model framework was its ability to adjust the spatial resolution of the computational network and, in particular, to increase resolution in areas of specific interest to the NMBSEMP study area. The hydrodynamic model mesh resolution has been reduced in areas away from the key locations. As such, simulation times and efficiencies were not constrained by the highest resolution required.

6.2 Hydrodynamic Modelling

The hydrodynamic model TUFLOW FV has been used to simulate flows in two-dimensional mode for the present study. An existing regional scale model of the Coral Sea was used to provide boundary conditions to the model developed specifically for the SEMP. The Coral Sea model has a main open boundary approximately 900km offshore of the Queensland coastline. The model requires prescribed tidal water levels along this boundary and the relatively smaller boundaries to the north and south (Torres Strait and extending seaward from northern NSW). Harmonic tidal constituents at 29 locations along the open boundaries were obtained from the National Tide Centre (NTC). Water level variation output from the Coral Sea model provides the open boundary conditions to the Moreton Bay model. Detail of the Moreton Bay model mesh is shown in Figure 6-1 with the inset showing the substantial mesh refinement specific to the present study.

A critical component of any hydrodynamic model development and calibration is the construction of a sufficiently accurate Digital Elevation Model (DEM) of the study area. In the case of the Moreton Bay model the following bathymetric data sources have been used:

- Northern Moreton Bay LiDAR survey (nearshore areas), Moreton Bay Regional Council (2013);
- Project 3DGBR bathymetry model, James Cook University (Beaman, 2010);
- Australian Bathymetry and Topography 250m Grid, Geoscience Australia (2009); and
- Hydrographic chart derived bathymetry (various AUS chart sources).



Title:

TUFLOW FV Model Extent

Figure:

6-1

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A

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0 15 30km

Scale - Main Map

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6.2.1 Hydrodynamic Model Validation

Formal calibration of the Moreton Bay model was undertaken as part of previous studies and is presented in Appendix A. Data used for calibration included MSQ tidal predictions, Acoustic Doppler Current Profiler (ADCP) velocity and water level measurements from previous studies (Brisbane Airport Corporation 2005 and CSIRO 2012). The locations for the various data sources are indicated in Figure A-1.

Recorded data specific to the NMBSEMP study areas was not available. Nevertheless, model validation to MSQ tidal predictions were undertaken at a number of “Secondary Places” relevant to the study area. Secondary Places at Beachmere, Bongaree, Toorbul and Donnybrook display a similar tidal pattern and are grouped with the Brisbane Bar Standard Port. MSQ provide sufficient data for calculating tide times and heights at the Secondary Places relative to the predictions at the Brisbane Bar.

Tidal validation results for the Brisbane Bar Standard Port and the various relevant Secondary Places within the hydrodynamic model domain are presented in Figure 6-2 to Figure 6-6. Generally the tidal phase and amplitude prediction is satisfactory at all locations. Predictive skill at the Deception Bay locations (Beachmere and Bongaree) is slightly better than at Toorbul and Donnybrook within Pumicestone Passage. The cause of the minor model inaccuracy at Toorbul and Donnybrook is expected to be associated with the relatively limited bathymetric information available for model input throughout these areas.

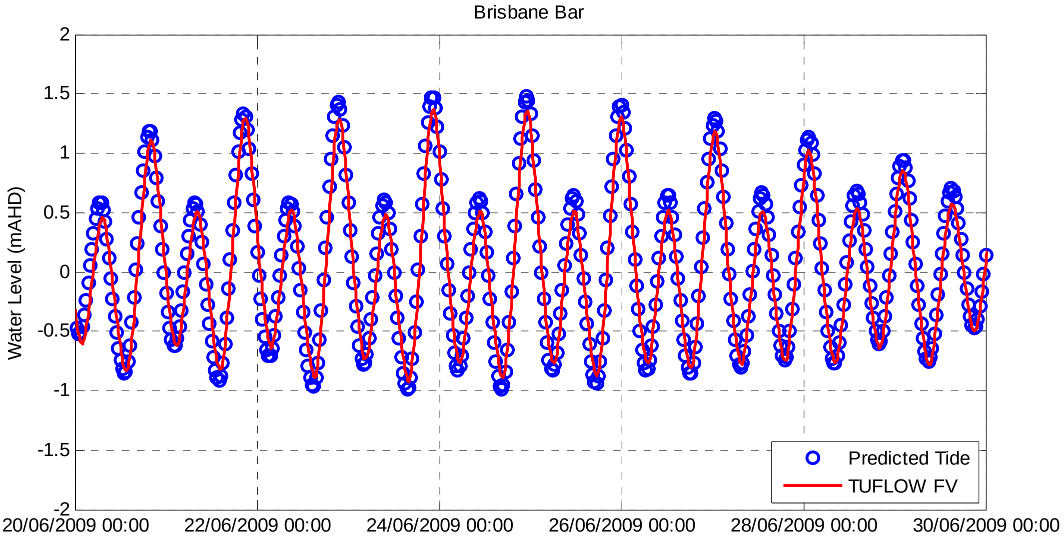


Figure 6-2 Hydrodynamic Model Validation at Brisbane Bar (Standard Port)

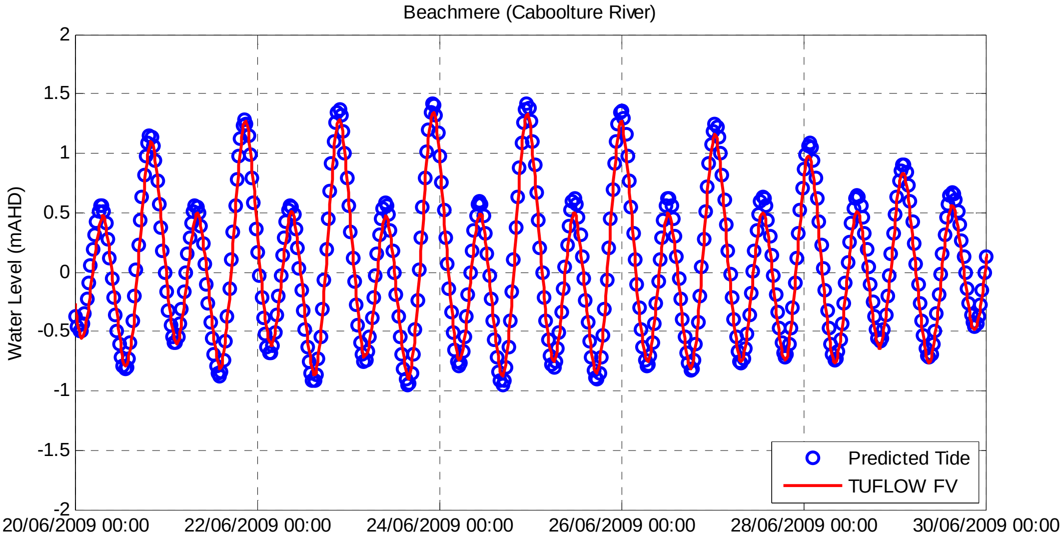


Figure 6-3 Hydrodynamic Model Validation at Beachmere (Secondary Place)

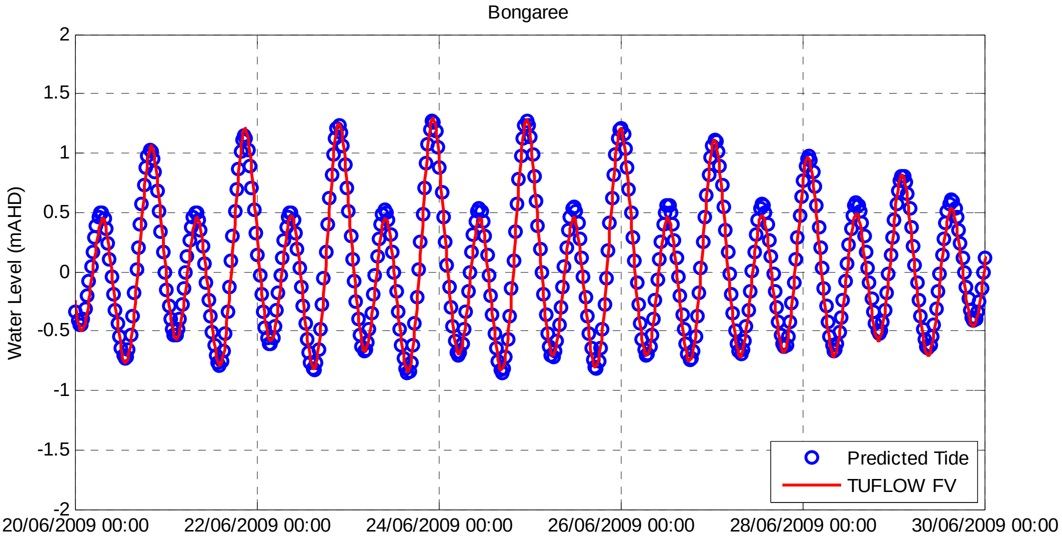


Figure 6-4 Hydrodynamic Model Validation at Bongaree (Secondary Place)

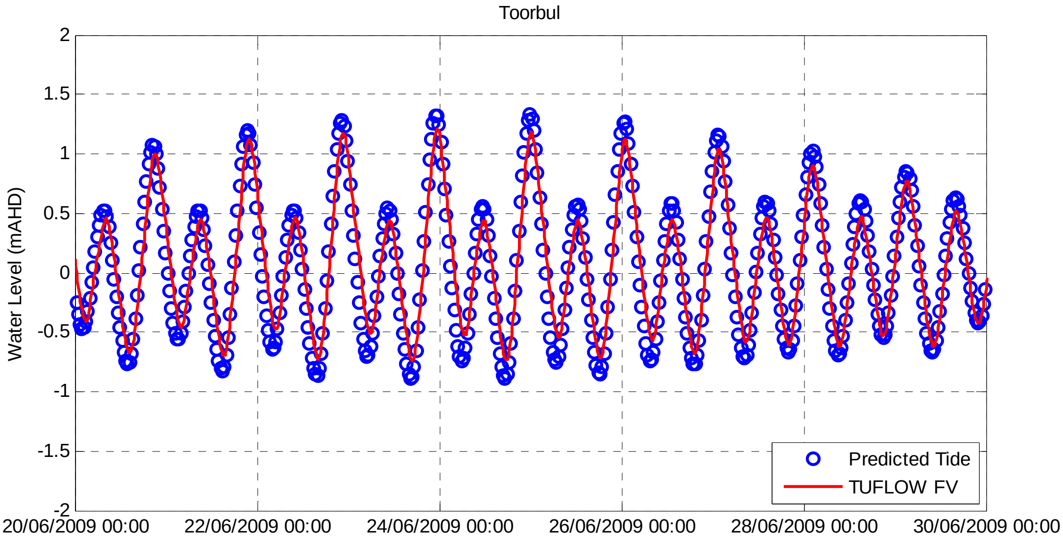


Figure 6-5 Hydrodynamic Model Validation at Toorbul (Secondary Place)

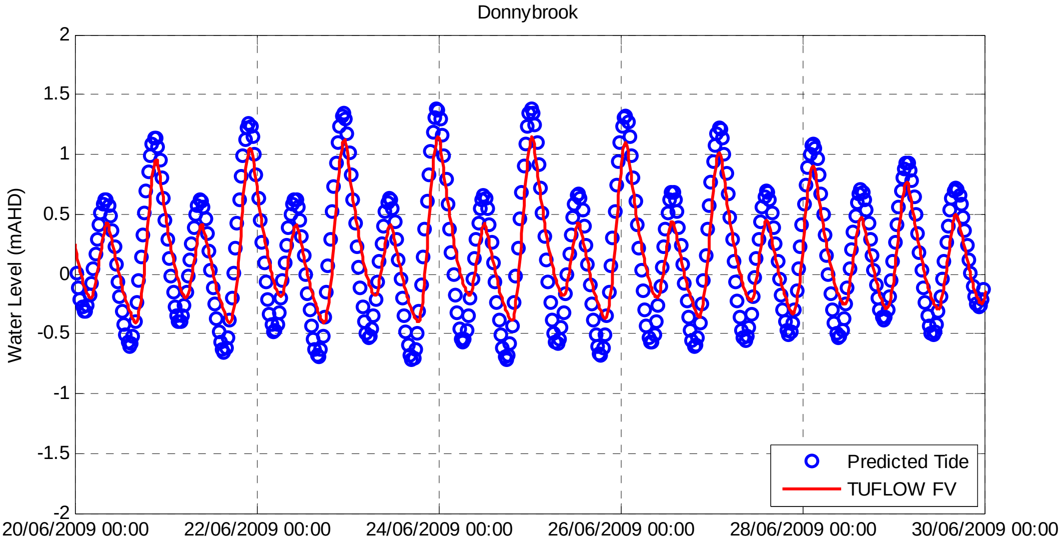


Figure 6-6 Hydrodynamic Model Validation at Donnybrook (Secondary Place)

6.2.2 Modelling of Tide Related Processes

The hydrodynamic model applied to this study has been used as a tool for providing both qualitative insights and quantitative information about the processes taking place. The level of model validation and analysis undertaken is considered sufficient for the purpose of guiding shoreline management decisions. It is reiterated that the no formal model calibration specific to this study's areas of interest has been completed and the current patterns described throughout the NMBSEMP study area have not been validated. More detailed investigations that include measurements of currents are often undertaken as part of the detailed design of a specific shoreline management option.

Regional peak ebb and peak flood tide current patterns are shown in Figure 6-7 and Figure 6-8. Current speeds exceeding 1m/s occur near the entrance to Deception Bay at Skirmish Point (southern tip of Bribie Island) with significantly lower currents typically less than 0.5m/s predicted along the shoreline of Deception Bay. Peak currents close to 1m/s are predicted at the southern entrance to Pumicestone Passage.

Timeseries plots of tidal current speed and direction are provided in Figure 6-9 to Figure 6-20 for the same period as the water level validation plots shown in Section 6.2.1. Note that the direction convention is Cartesian and corresponds to the direction the current is going (measured counter-clockwise from the positive x-axis). Throughout Deception Bay the model output point locations are slightly seaward of the tidal flat that is exposed at low tide. The tidal current climate differs at each study area location with key features summarised below:

- Generally the tidal current speeds are low at all locations and are not expected to drive significant sediment transport at Deception Bay, Beachmere and Godwin Beach in the absence of wind and wave forcing.

- At Deception Bay the flood tide currents align approximately 200deg and the ebb tide currents align at 20deg. Peak current speeds occasionally exceed 0.1m/s during the spring flood tides with a mean current speed of approximately 0.05m/s.
- Beachmere has the lowest mean current speed (less than 0.04m/s) with peaks not exceeding 0.1m/s.
- Godwin Beach is sheltered by adjacent mangrove habitats which complicate the tidal current signal. The predicted peak currents that occasional exceed 0.1m/s are associated with the transition from ebb to flood tide.
- The current climate near Sandstone Point is influenced by the flow entering and exiting Pumicestone Passage. Peak current speeds exceeding 0.3m/s are associated with flows exiting the Passage during the ebb tide. The flood tide direction aligns approximately 270deg and ebb aligns between 90deg and 20deg. Tidal currents of this magnitude exceed the threshold for the transport of sand (e.g. Soulsby, 1997). Wind and wave forcing will further enhance the sand transport potential.
- At Toorbul and Donnybrook the current directions are clearly defined by the alignment of Pumicestone Passage. During the flood tide the currents align between 120deg and 135deg and the ebb currents between 300deg and 315deg. The tidal current speeds at Toorbul and Donnybrook are up to 0.35m/s and sufficient to mobilise fine sediments.

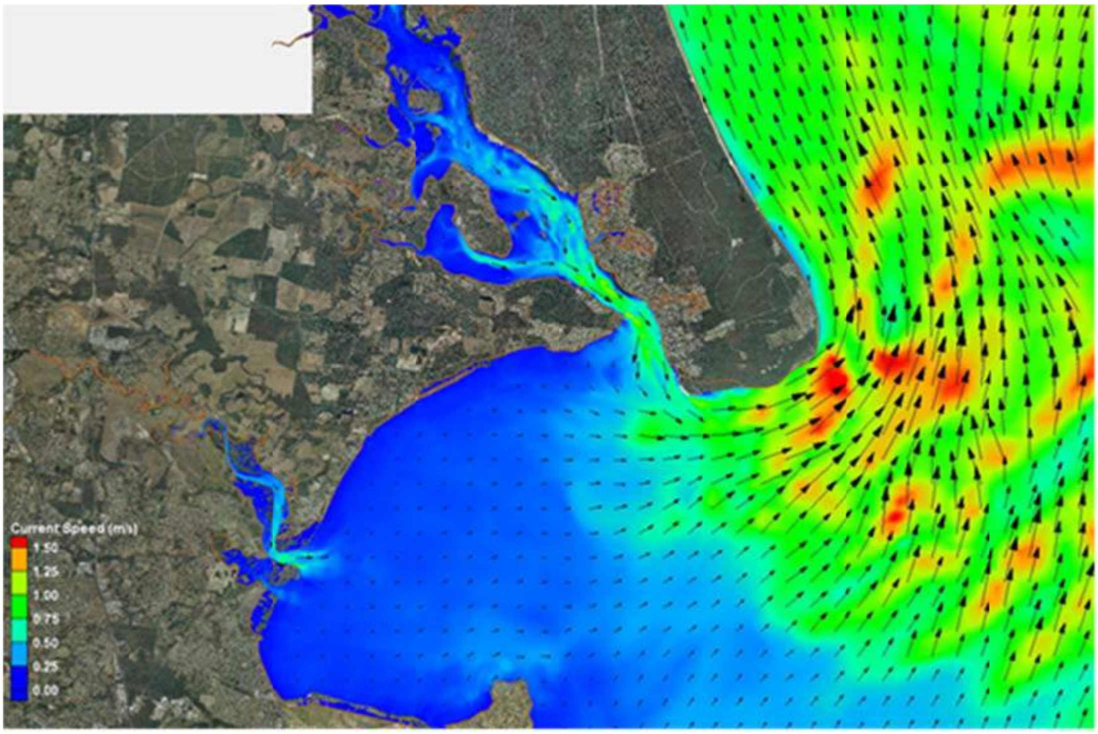


Figure 6-7 Regional Peak Ebb Tide Current Patterns

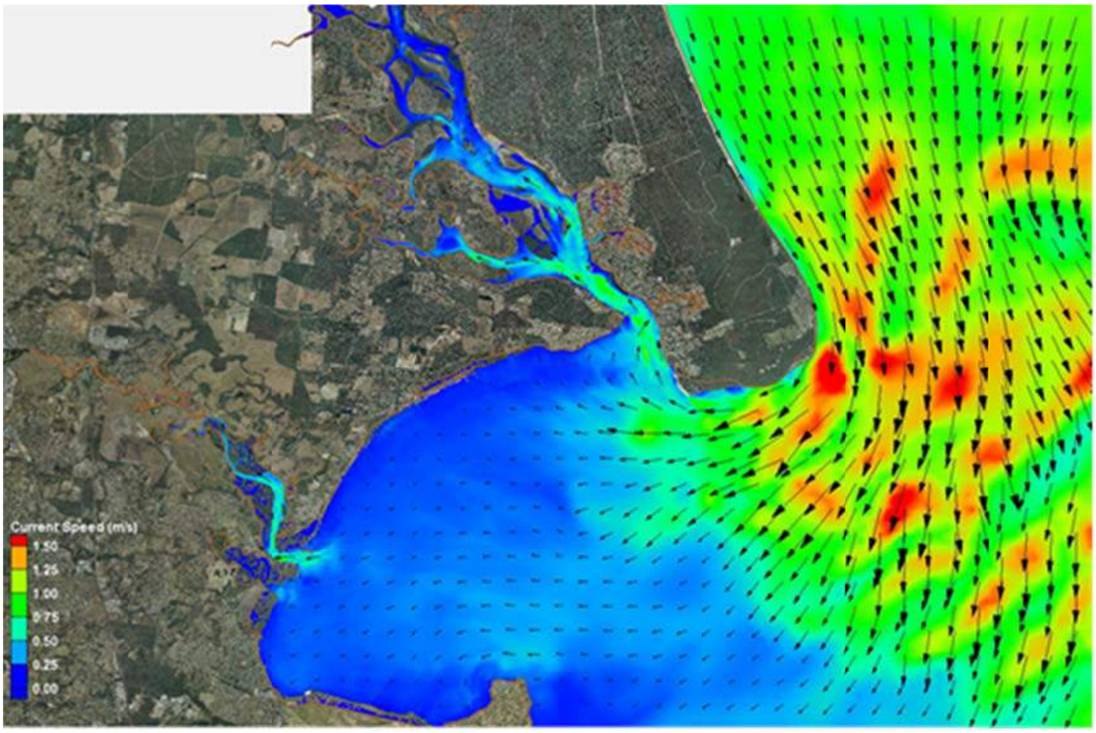


Figure 6-8 Regional Peak Flood Tide Current Patterns

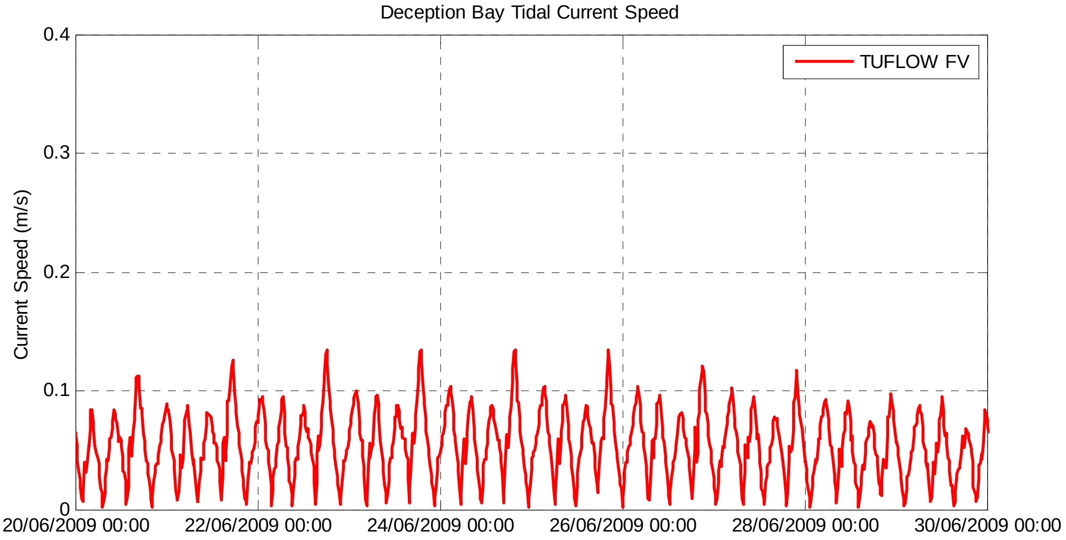


Figure 6-9 Deception Bay Tidal Current Speed Timeseries

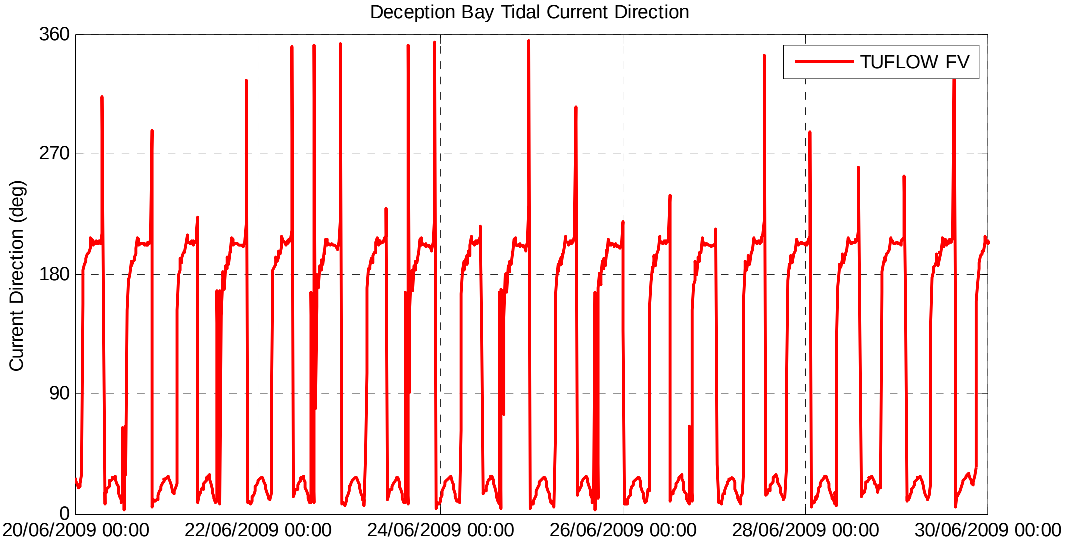


Figure 6-10 Deception Bay Tidal Current Direction Timeseries

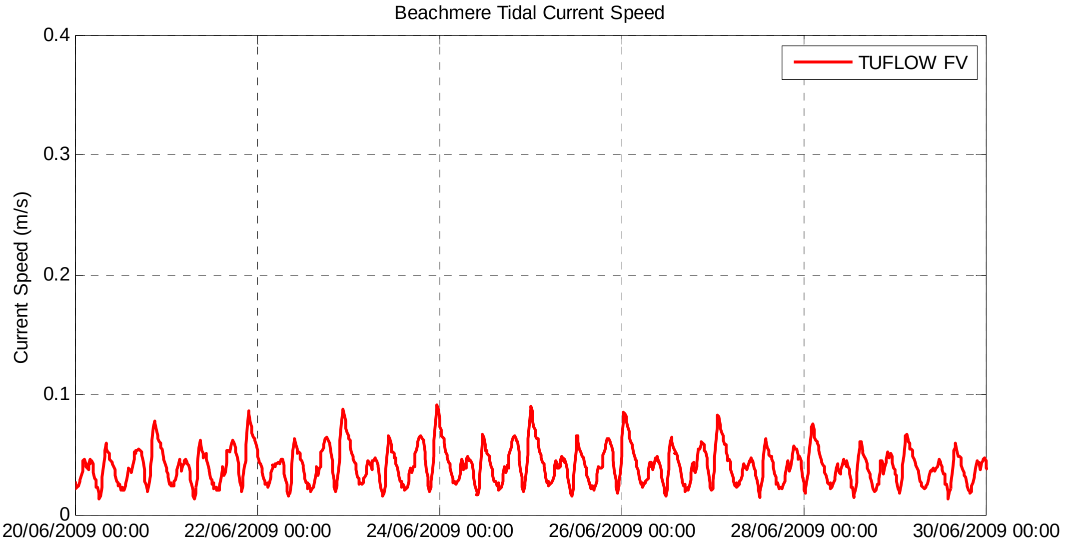


Figure 6-11 Beachmere Tidal Current Speed Timeseries

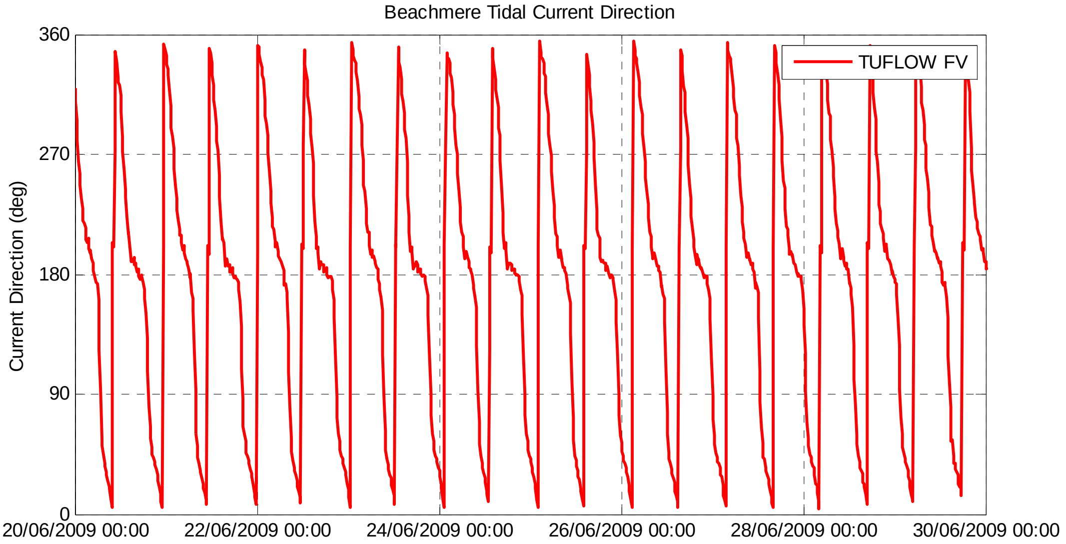


Figure 6-12 Beachmere Tidal Current Direction Timeseries

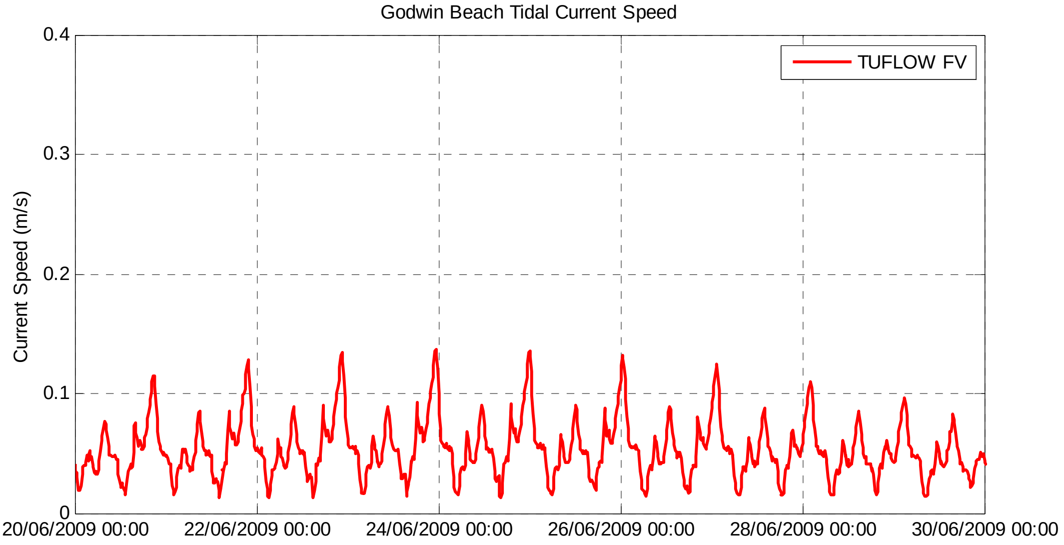


Figure 6-13 Godwin Beach Tidal Current Speed Timeseries

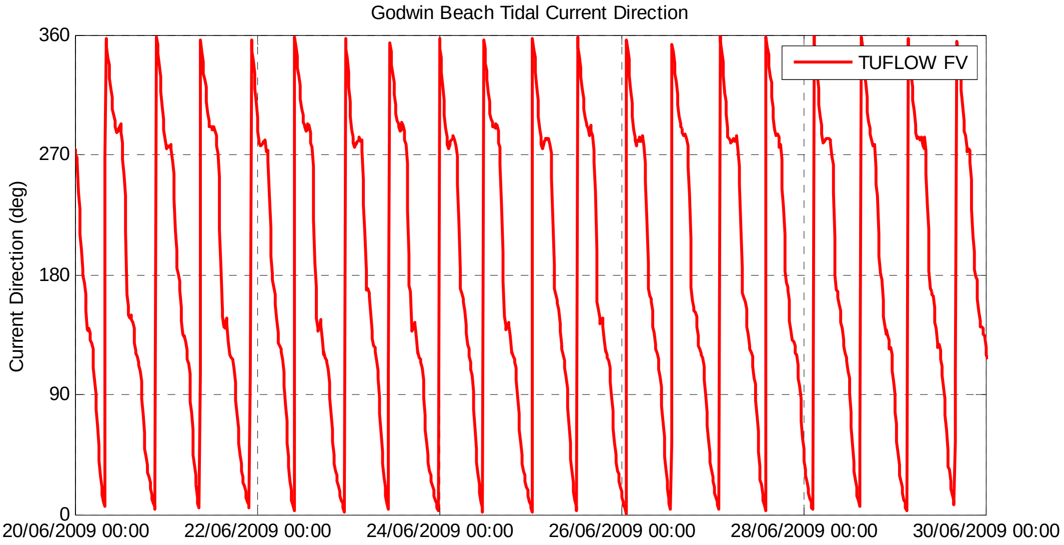


Figure 6-14 Godwin Beach Tidal Current Direction Timeseries

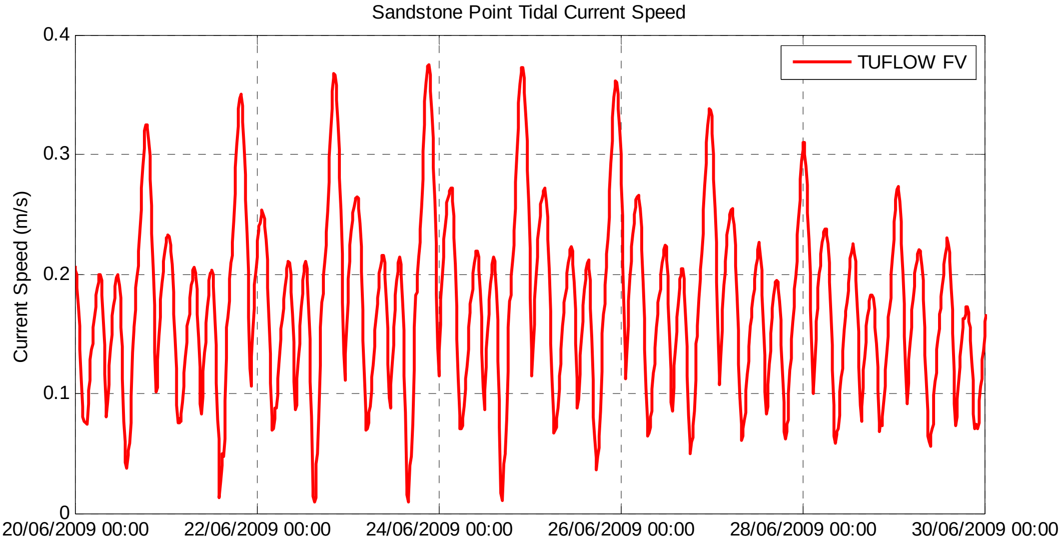


Figure 6-15 Sandstone Point Current Speed Timeseries

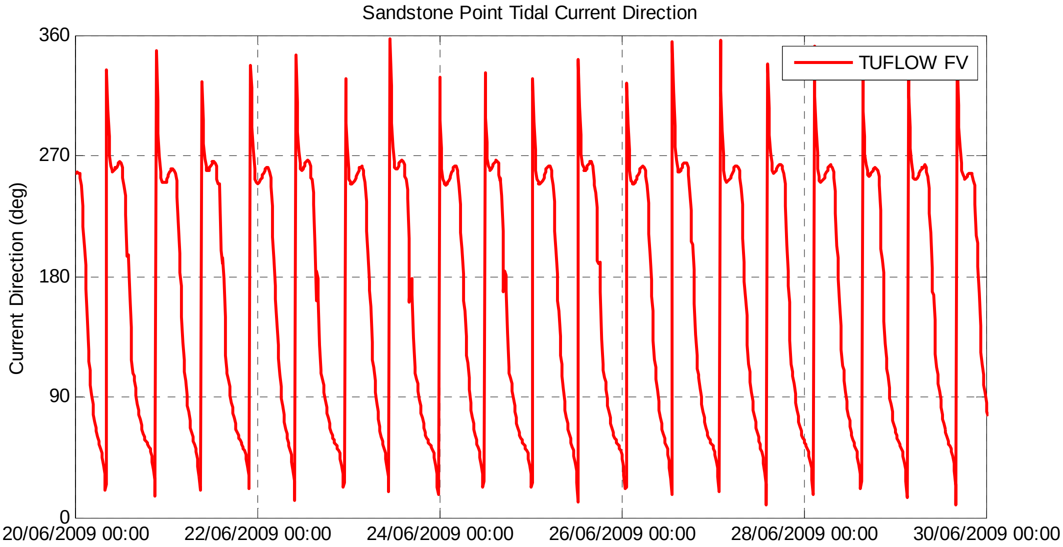


Figure 6-16 Sandstone Point Current Direction Timeseries

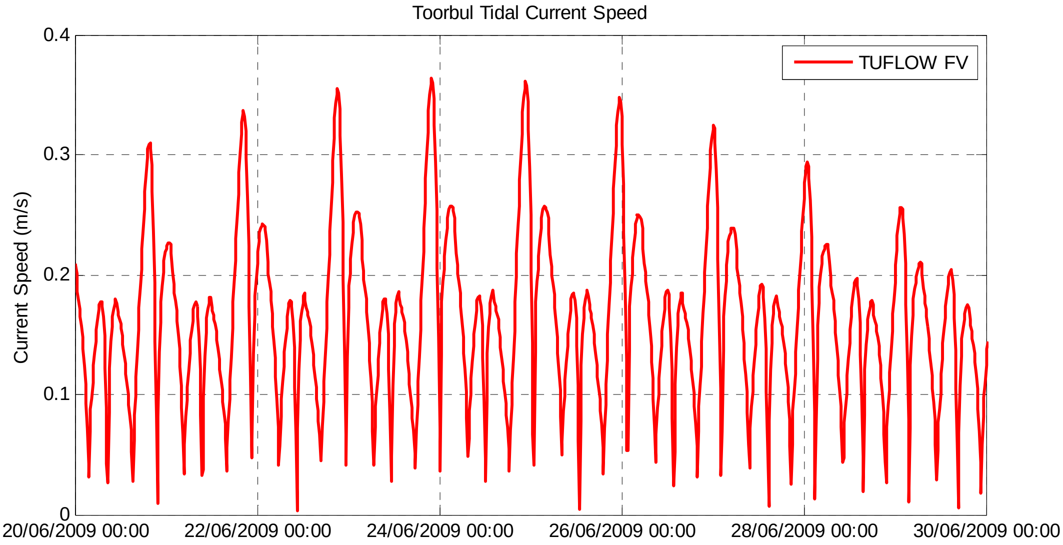


Figure 6-17 Toorbul Tidal Current Speed Timeseries

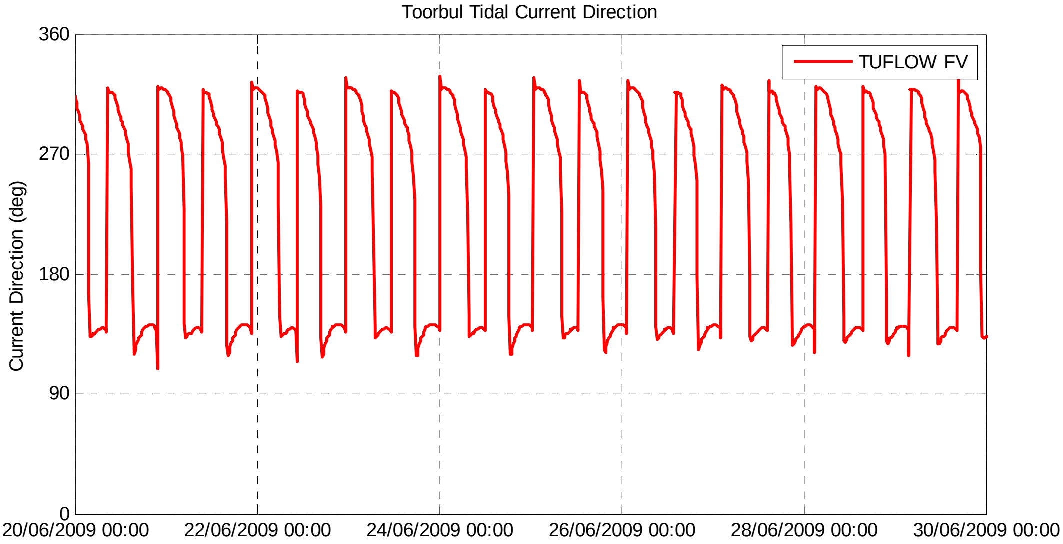


Figure 6-18 Toorbul Tidal Current Direction Timeseries

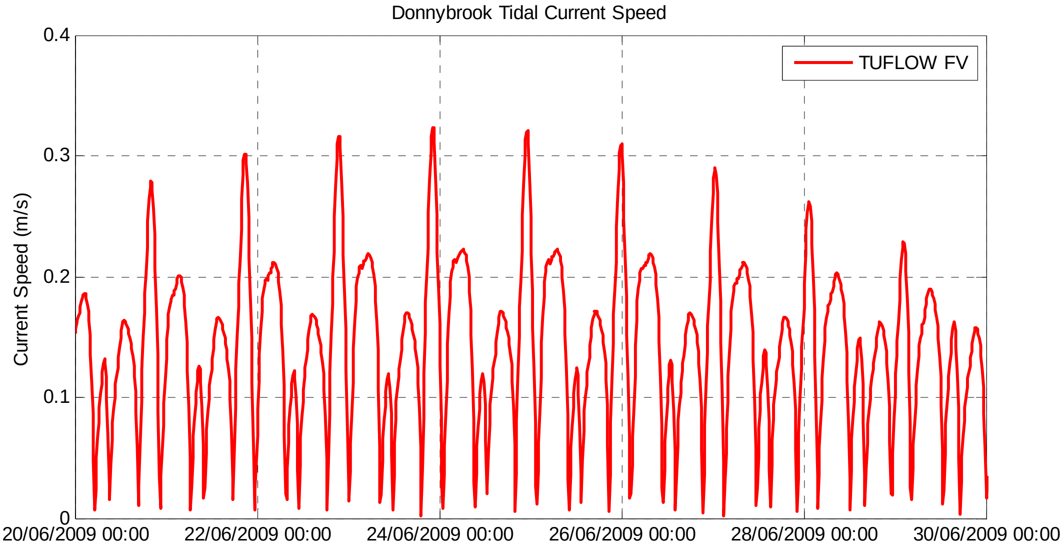


Figure 6-19 Donnybrook Tidal Current Speed Timeseries

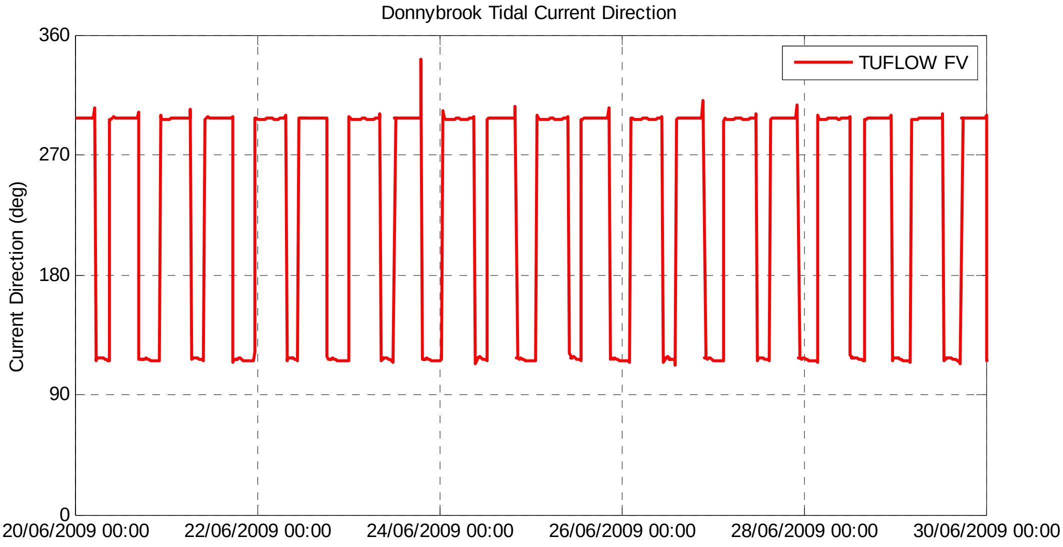


Figure 6-20 Donnybrook Tidal Current Direction Timeseries

6.3 Wave Modelling

Comprehensive spectral wave models covering the broader region surrounding and within Moreton Bay were established to assess the wave climate and wave propagation in the context of the NMBSEMP study area. The detailed wave modelling results were used to guide the assessment of shoreline processes and for coupling with the hydrodynamic model.

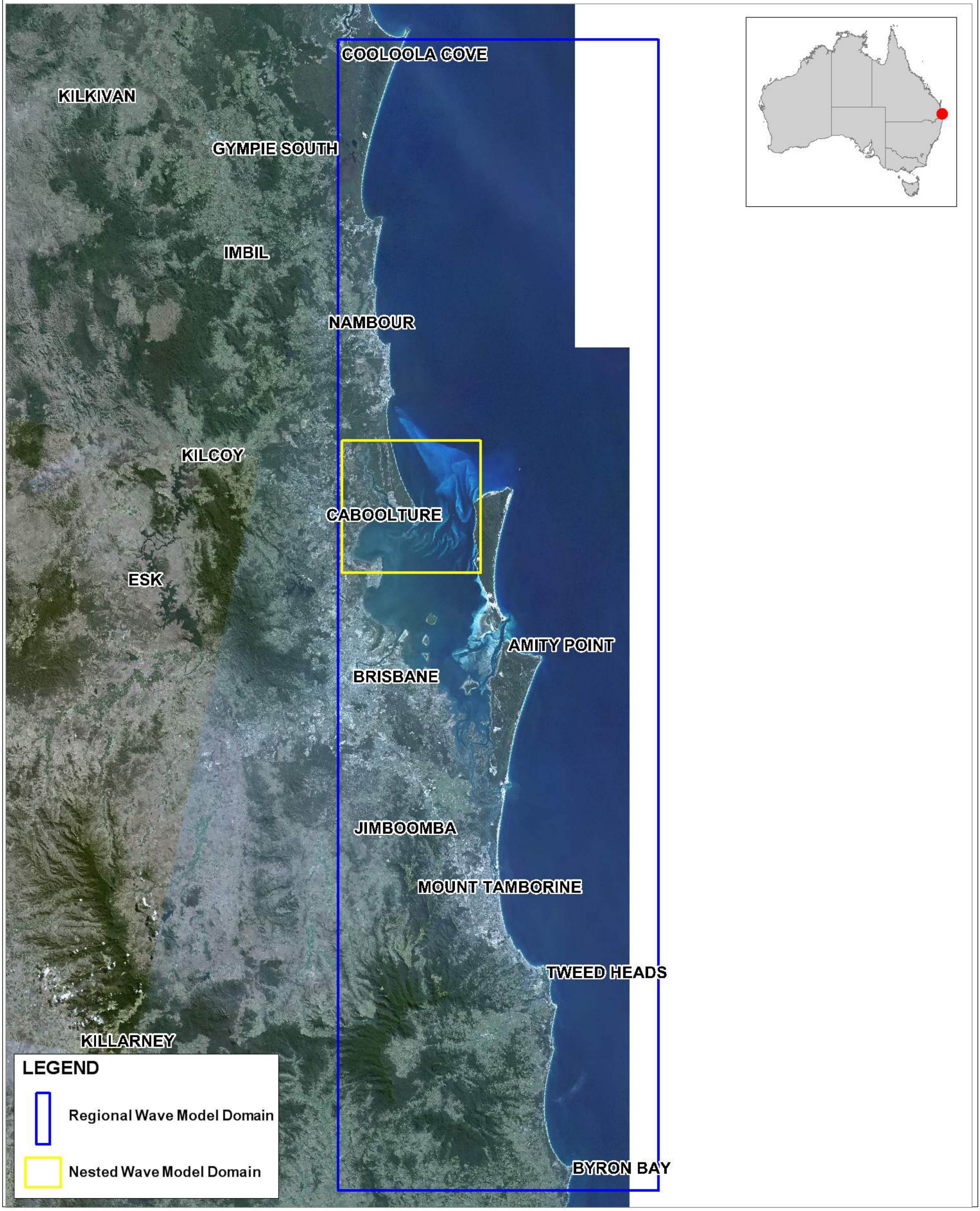
Wave conditions were simulated using SWAN models of the study area. SWAN is a third generation spectral wave model that estimates wave parameters in coastal regions from given wind, wave and current conditions. SWAN is developed by Delft University of Technology and is widely used by the coastal engineering community.

The SWAN input parameters employed in this study are considered to be realistic and are based upon previous experience with similar models. Default values for the whitecapping dissipation coefficient and wave steepness parameter were used for the Komen et al (1984) calculations. The bottom friction formulation of Collins (1972) was implemented with a coefficient of 0.025. The first order Backward Space Backward Time (BSBT) scheme was used for the numerical propagation scheme. A mid-range refraction coefficient was chosen to achieve an accurate result without spurious oscillations.

A nested grid system was used to maximise wave model efficiency while minimising inaccuracies associated with the model boundary definitions. Following this approach, the finest-scale grid surrounds the study area and its boundary conditions are obtained from the encompassing coarser grid. The nested wave model extents are shown in Figure 6-21 and described as:

- Regional scale (400m grid resolution) model extending from Cape Byron to Double Island Point and offshore to the continental shelf; and
- Local scale (100m grid resolution) model representing Deception Bay and the NMBSEMP study area.

Wave conditions at the offshore boundary of the regional domain were derived by transforming measured bulk wave parameters from the Stradbroke Island wave rider buoy (operated by EHP) to deep water offshore values. This procedure was accomplished by using an existing BMT WBM SWAN model to construct transformation tables for representative swell conditions as a function of significant wave height (Hsig) and the spectral peak wave direction. Recorded wave data for the four years from June 2006 to April 2010 were then converted to the corresponding deep water wave conditions using these transformation tables. The spectral peak period (Tp) and spectral peak wave direction in conjunction with the significant wave height were used as the best estimate bulk wave parameters describing the dominant sea state. A spatially interpolated wind field was also applied to the model based on recorded wind data from the Bureau of Meteorology (BOM).



Title:

Wave Model Extents

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6-21

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Approx. Scale

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6.3.1 Wave Model Validation

Wave model output was compared to measurements from a non-directional Waverider buoy located approximately 10km offshore from the Redcliffe Peninsula (location indicated in Figure A-1 and operated by the former Department of Environment and Resource Management (DERM). It is noted that this instrument is no longer in this position. A comparison of the model results with the measured bulk wave parameters for August to September 2007 are shown in Figure 6-22. The model reproduces the temporal variation and magnitude of the significant wave height and peak period recorded at the buoy location very accurately.

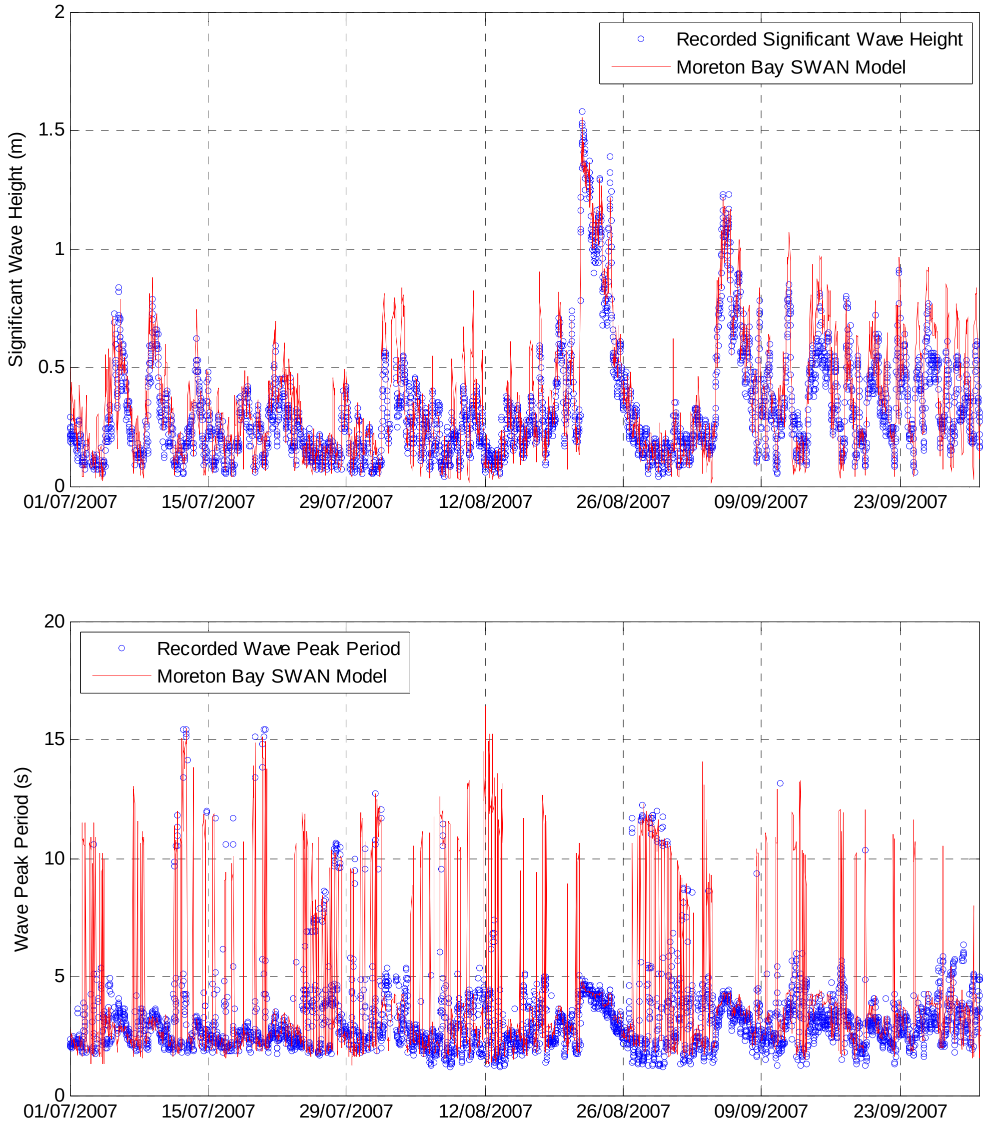


Figure 6-22 Moreton Bay SWAN Model Validation

6.3.2 Wave Climate Analysis

As described in Section 5.3, the study area is sheltered from the prevailing southerly swell waves by North Stradbroke, Moreton and Bribie Islands. The ocean swell energy that enters Moreton Bay and reaches the bay shorelines is substantially attenuated by the processes of refraction, diffraction, bed friction and breaking across the shallow shoals at the bay entrance. This is demonstrated using predicted wave patterns in Figure 6-23 and Figure 6-24 which show very limited long period swell penetration for south easterly and north easterly swell directions.

The Deception Bay shorelines are generally most affected by the local wind generated sea waves that are of shorter period (generally less than 5 seconds). The locally generated waves have a direction and angle at the shoreline that is determined predominantly by the wind direction. South easterly and north easterly wind wave patterns are shown in Figure 6-25 and Figure 6-26.

The SWAN wave modelling system has been used to develop nearshore wave climates for each study area location. The average wave climate (based on a hindcast period from June 2006 to April 2010) for each Deception Bay study area location is presented below as a wave rose and wave frequency recurrence table. Toorbul and Donnybrook have been removed from this assessment due to the relatively low wave energy at these locations. The nearshore wave climate analysis indicates the following:

- The Deception Bay beach unit is most exposed to waves from the north easterly sector with the Redcliffe peninsula providing some additional shelter from south easterly wind waves. Significant wave heights less than 0.5m occur more than 90% of the time.
- Moving from south to north along the shoreline between Beachmere and Godwin Beach, the nearshore area is progressively more exposed to south easterly conditions and less exposed to north easterly conditions.
- Significant wave heights at Beachmere exceed 0.5m approximately 12% of the time. During severe wind conditions the significant wave heights occasionally exceed 1m in the nearshore zone.
- Godwin Beach is more exposed to south easterly wind wave conditions however experiences lower nearshore wave energy compared to the Beachmere beach unit. The southern tip of Bribie Island shelters Godwin Beach from north easterly wave conditions. For close to 90% of the year the significant wave height is less than 0.5m.
- Similarly to Godwin Beach, the southern side of Sandstone Point experiences very little wave energy from the north easterly sector due to the southern tip of Bribie Island. The majority of wave energy is from the south easterly sector with significant wave heights exceeding 0.5m approximately 14% of the time.

The spatially varying output from the wave model has been coupled with the TUFLOW FV hydrodynamic model and used to estimate to total sediment transport rates throughout the NMBSEMP study area. This assessment is presented in Section 6.4.1.

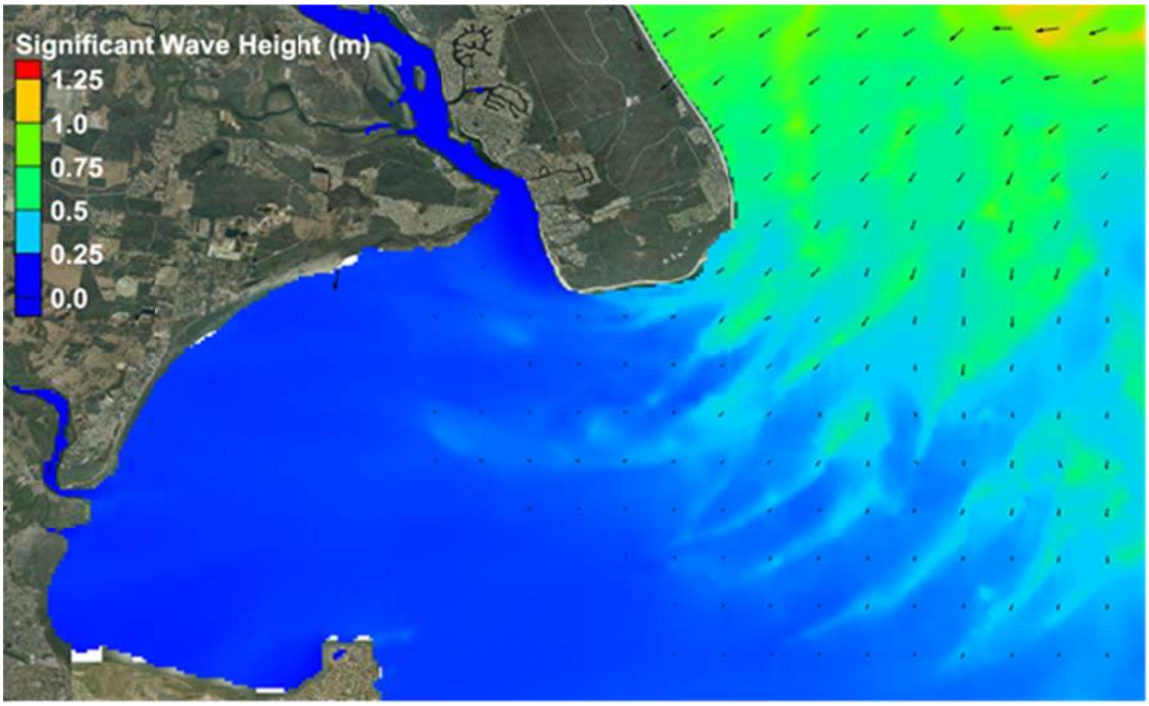


Figure 6-23 South Easterly Swell Wave Patterns

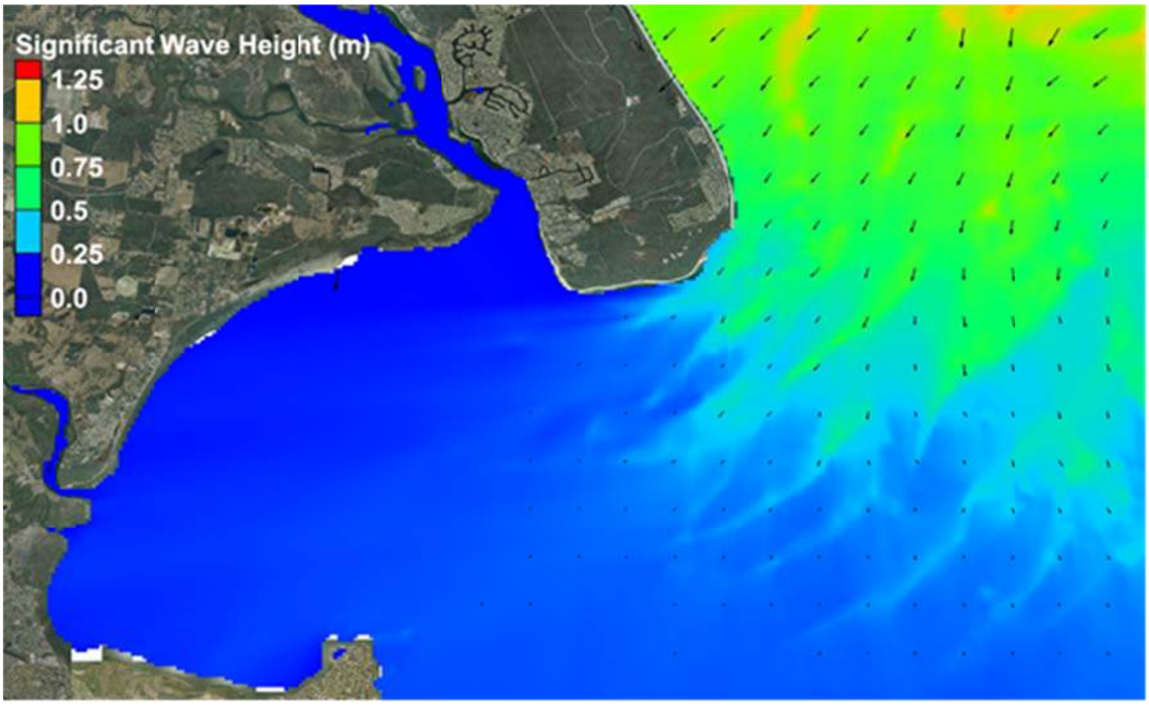


Figure 6-24 North Easterly Swell Wave Patterns

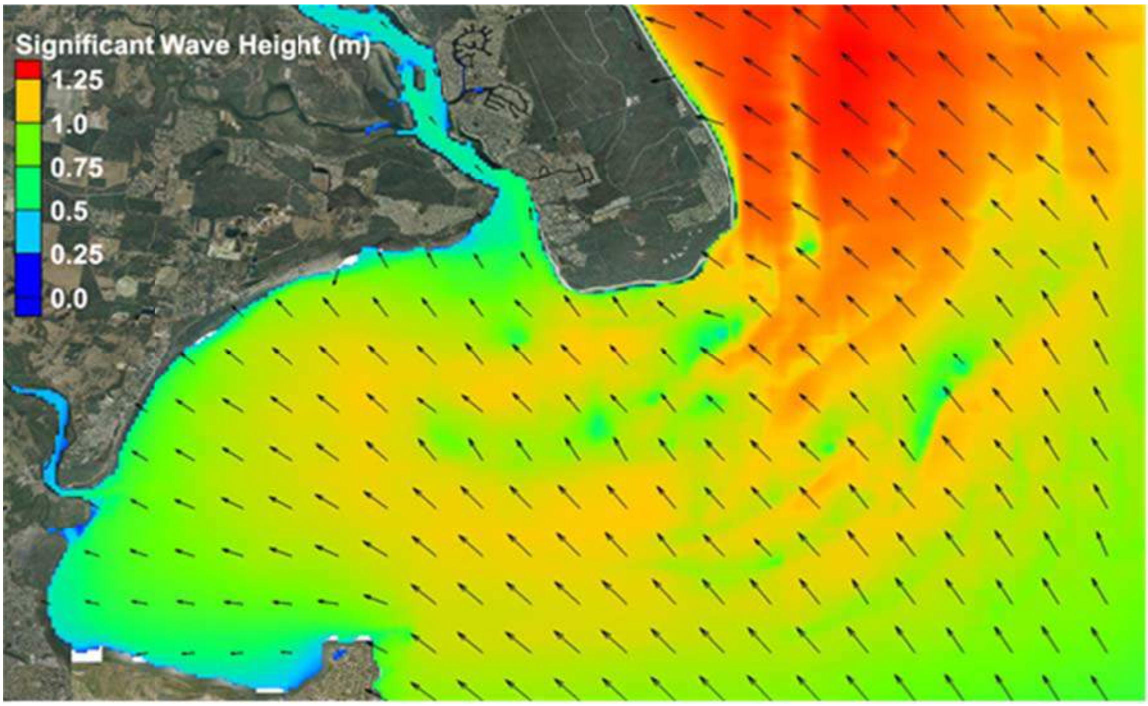


Figure 6-25 South Easterly Wind Wave Patterns

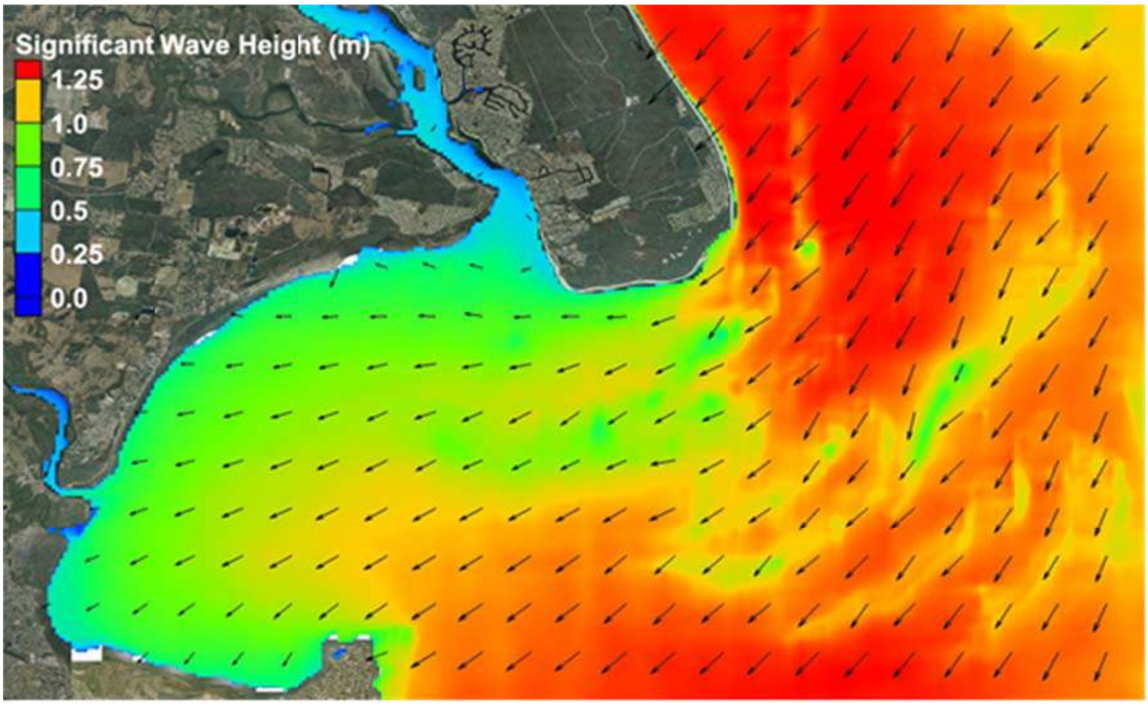


Figure 6-26 North Easterly Wind Wave Patterns

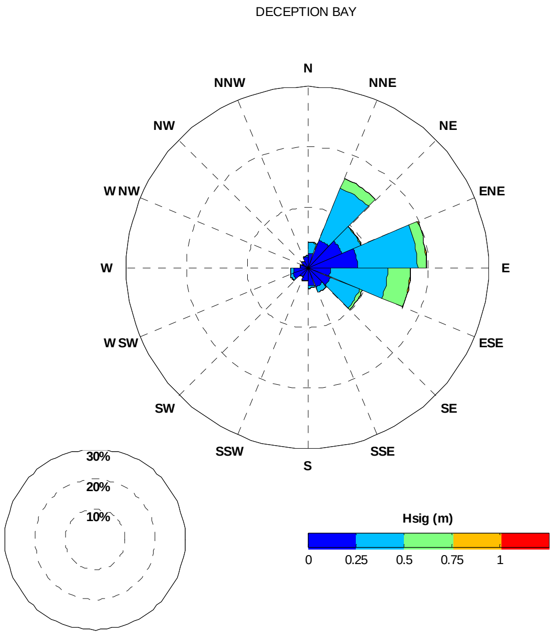


Figure 6-27 Deception Bay Wave Rose

Table 6-1 Deception Bay Wave Frequency Recurrence (% of time)

	Directional Bin (deg)																
Hs (m)	**0**	**22.5**	**45**	**67.5**	**90**	**112.5**	**135**	**157.5**	**180**	**202.5**	**225**	**247.5**	**270**	**292.5**	**315**	**337.5**	**Total**
0.00-0.25	2.4	4.9	6.2	8.3	3.8	4.1	3.5	3.1	2.2	1.7	2.6	2.3	1.2	0.9	1.2	1.7	50.3
0.25-0.50	1.7	9.4	3.4	10.0	9.4	5.3	1.0	0.3	0.1	0.1	0.4	0.5	0.1	0.0	0.0	0.2	41.9
0.50-0.75	0.1	1.7	0.1	1.5	3.8	0.5	0.0	0.0									7.7
0.75-1.00				0.0	0.1	0.0											0.1
>1.00																	0.0
Total	4.3	16.0	9.7	19.8	17.2	9.9	4.5	3.4	2.3	1.7	3.1	2.9	1.3	1.0	1.2	1.9	100.0

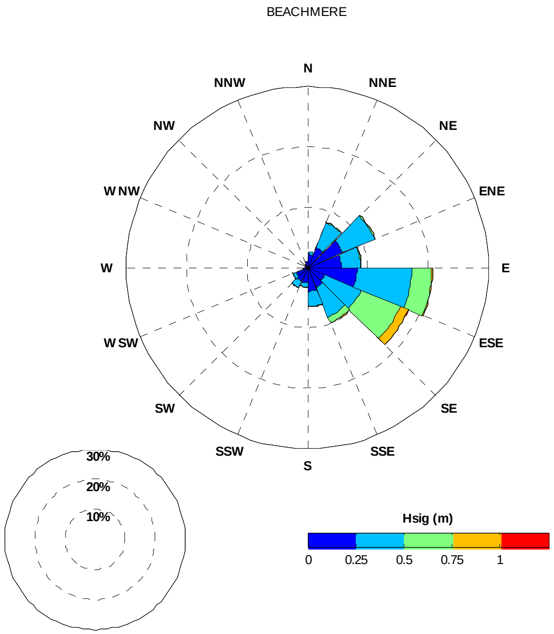


Figure 6-28 Beachmere Wave Rose

Table 6-2 Beachmere Wave Frequency Recurrence (% of time)

	Direction Bin (degN)															
Hs (m)	**0**	**22.5**	**45**	**67.5**	**90**	**112.5**	**135**	**157.5**	**180**	**202.5**	**225**	**247.5**	**270**	**292.5**	**315**	**337.5**	**Total**
0.00-0.25	2.2	3.6	6.4	5.6	8.2	3.1	3.5	4.0	2.5	2.4	2.0	0.8	0.4	0.4	0.5	1.0	46.7
0.25-0.50	0.4	4.4	5.7	3.2	9.1	6.6	5.4	2.5	0.7	1.2	0.6	0.0		0.0	0.0	0.0	39.8
0.50-0.75		0.1	0.2	0.2	3.3	7.0	1.0	0.0	0.0	0.0	0.0						11.9
0.75-1.00					0.2	1.4	0.0										1.6
>1.00					0.0	0.0											0.0
Total	2.7	8.1	12.2	9.0	20.8	18.1	10.0	6.5	3.2	3.7	2.6	0.8	0.4	0.4	0.5	1.0	100.0

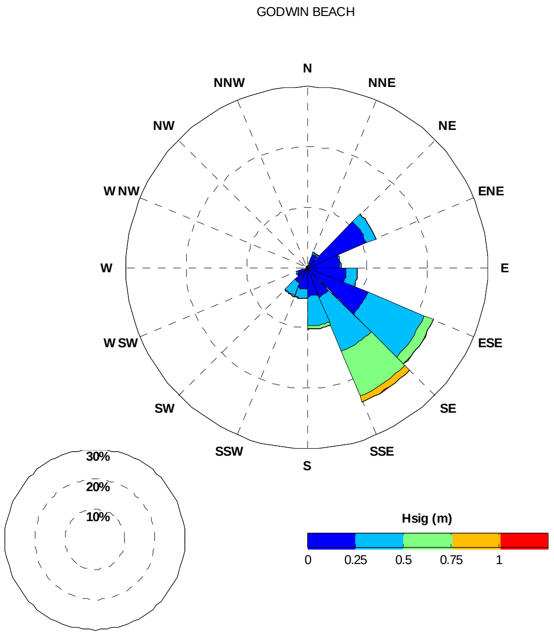


Figure 6-29 Godwin Beach Wave Rose

Table 6-3 Godwin Beach Wave Frequency Recurrence (% of time)

	Direction Bin (degN)															
Hs (m)	**0**	**22.5**	**45**	**67.5**	**90**	**112.5**	**135**	**157.5**	**180**	**202.5**	**225**	**247.5**	**270**	**292.5**	**315**	**337.5**	**Total**
0.00-0.25	0.5	2.4	10.7	5.4	6.6	10.9	5.2	4.7	3.6	2.8	1.7	0.3	0.1	0.1	0.1	0.1	55.4
0.25-0.50	0.0	0.4	1.7	0.2	1.9	10.1	9.8	5.0	1.6	2.3	0.2						33.2
0.50-0.75					0.0	1.6	8.0	0.6	0.1	0.1							10.4
0.75-1.00						0.0	1.1	0.0									1.1
>1.00							0.0										0.0
Total	0.5	2.7	12.5	5.6	8.4	22.7	24.1	10.3	5.2	5.2	2.0	0.3	0.1	0.1	0.1	0.1	100.0

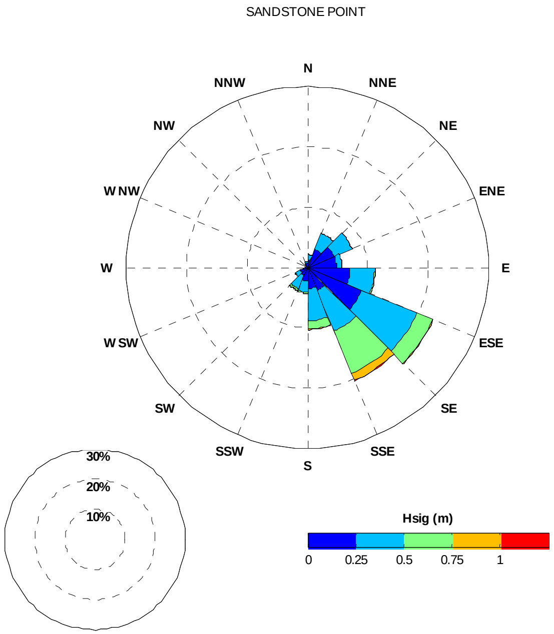


Figure 6-30 Sandstone Point Wave Rose

Table 6-4 Sandstone Point Wave Frequency Recurrence (% of time)

	Direction Bin (degN)															
Hs (m)	**0**	**22.5**	**45**	**67.5**	**90**	**112.5**	**135**	**157.5**	**180**	**202.5**	**225**	**247.5**	**270**	**292.5**	**315**	**337.5**	**Total**
0.00-0.25	1.3	3.2	4.8	4.8	7.1	9.7	4.1	3.5	2.2	1.4	1.2	0.6	0.4	0.4	0.5	0.7	46.1
0.25-0.50	0.8	2.9	3.3	0.9	4.0	9.9	7.3	5.3	1.8	2.4	0.9	0.2	0.0	0.0	0.0	0.1	40.0
0.50-0.75					0.1	2.8	7.5	1.4	0.2	0.4	0.0						12.5
0.75-1.00						0.1	1.3	0.1									1.5
>1.00							0.0										0.0
Total	2.2	6.1	8.0	5.7	11.3	22.5	20.3	10.3	4.2	4.1	2.2	0.8	0.4	0.4	0.5	0.9	100.0

6.3.3 Storm Wave Assessment

Storm wave conditions throughout the study area were assessed using the local scale SWAN model. The assessment considered conditions representative of those observed during ex-TC Oswald (refer Section 5.6.1), namely:

- •Water level 2.5m (approximately equivalent to HAT at Beachmere).
- Offshore wave conditions (applied as swell at the northern and eastern model boundary):
  - Significant wave height = 6.0m
  - Wave period = 12s
  - Wave direction = 60deg (from the ENE)
- Wind conditions (applied across model domain):
  - Wind speed = 60km/h
  - Wind direction = ENE

As discussed in Section 5.6.1, the storm tide and wave conditions associated with ex-TC Oswald caused notable damage to seawalls at Deception Bay and Beachmere. Figure 6-31 shows significant wave height contours and vectors considered representative of the peak wave conditions during ex-TC Oswald. The results suggests the Deception Bay and Beachmere beach units were most exposed to the conditions generated by ex-TC Oswald with peak significant wave heights between 0.75m and 1.0m predicted at the shoreline for these locations. Peak significant wave heights less than 0.75m were predicted at other shoreline locations within the study area (wave height less than 0.5m is not shown in Figure 6-31). The predicted wave heights associated with ex-TC Oswald are approximately consistent with the non-cyclonic design wave heights reported by Cardno Lawson Treloar (2009) which are summarised in Section 5.3.1.

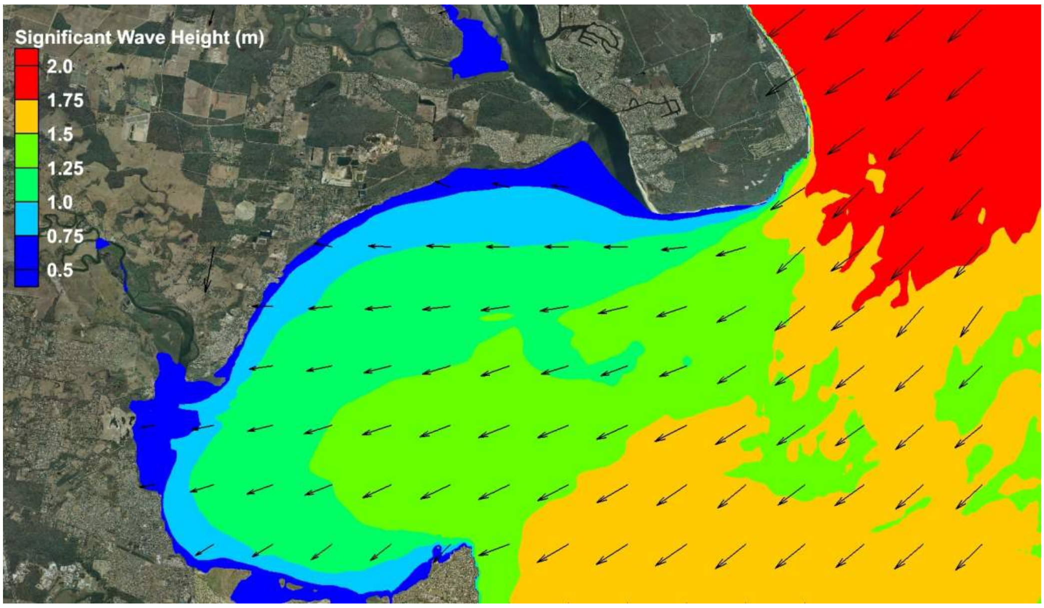


Figure 6-31 Storm Wave Assessment Results - Significant Wave Height

The ex-TC Oswald representative storm wave assessment highlights the exposure of the Deception Bay and Beachmere beach units to wave conditions from the north easterly directional sector. Storm waves entering Moreton Bay from this sector and driven by strong onshore winds are likely to present the greatest wave-induced erosion threat for the southern section of the NMBSEMP study area.

6.4 Sediment Transport Potential Modelling

6.4.1 Total Sediment Transport

Sediment transport under tidal currents only and under the combined action of currents and waves has been explored using the numerical modelling tools described above. For the tidal currents only scenario, the TUFLOW FV hydrodynamic was coupled with van Rijn's model for total sediment transport (van Rijn et al., 2004). For the combined the current and wave case, additional forcing from the SWAN wave model and seasonal winds was applied.

Insignificant transport rates were predicted under tidal forcing only and have not been reported. Table 6-5summaries the total sediment transport assessment results for summer and winter cases with positive values representing northerly transport potential.

Table 6-5 Predicted Total Sediment Transport Rates

	Net Sediment Transport (m3/m year)	
Beach Unit	**Current and Wave Summer**	**Current and Wave Winter**
Deception Bay	0.02	-0.04
Beachmere	0.25	-0.74
Godwin Beach	0.02	-0.03
Sandstone Point	-0.08	-0.24
Toorbul	-0.03	-0.31
Donnybrook	-0.05	-0.47

Note: Positive values indicate northerly transport, negative values indicate southerly transport

The calculated transport rates are based on the volume of sediment that passes through the nearshore zone per meter offshore. While it is not possible to verify the predicted transport rates, the results provide important information about general transport patterns, including:

- Extremely low transport rates are predicted at all locations under tidal current forcing only. No significant trends could be identified at Deception Bay, Beachmere, Godwin Beach or Donnybrook where the thresholds for sediment transport were not exceeded.
- The sediment transport direction at Deception Bay, Beachmere and Godwin Beach display is seasonal with weak northerly transport during the summer months and southerly transport during the winter months. The winter transport rates dominant and therefore a net annual transport to the south is assumed. This is consistent with descriptions of sediment transport pathways throughout these areas (e.g. Flood, 1981) and sand spit evolution observed at the north bank of the Caboolture River (refer Figure 5-8).

- Southerly directed transport is predicted for both the summer and winter cases at Sandstone Point, Toorbul and Donnybrook. This is likely to be balanced by the supply of sand moving to these areas from the southern tip of Bribie Island (e.g. BMT WBM, 2007). It is noted that sand accumulation trends identified during Toorbul site visits suggest weak, net northerly transport for locations north of the public boat ramp.
- Considering the average of the predicted summer and winter transport rates it is inferred that Beachmere experiences the highest annual rate. Considering a 500m profile offshore approximately 125m3 of material moves toward the south. It is noted that this volume is expected to be highly variable annually and could be substantially larger during particularly stormy seasons.

The average of the predicted summer and winter transport rates for each location are considered further in Section 7 as part of the shoreline erosion risk assessment.

6.5 Short Term Storm Erosion Potential

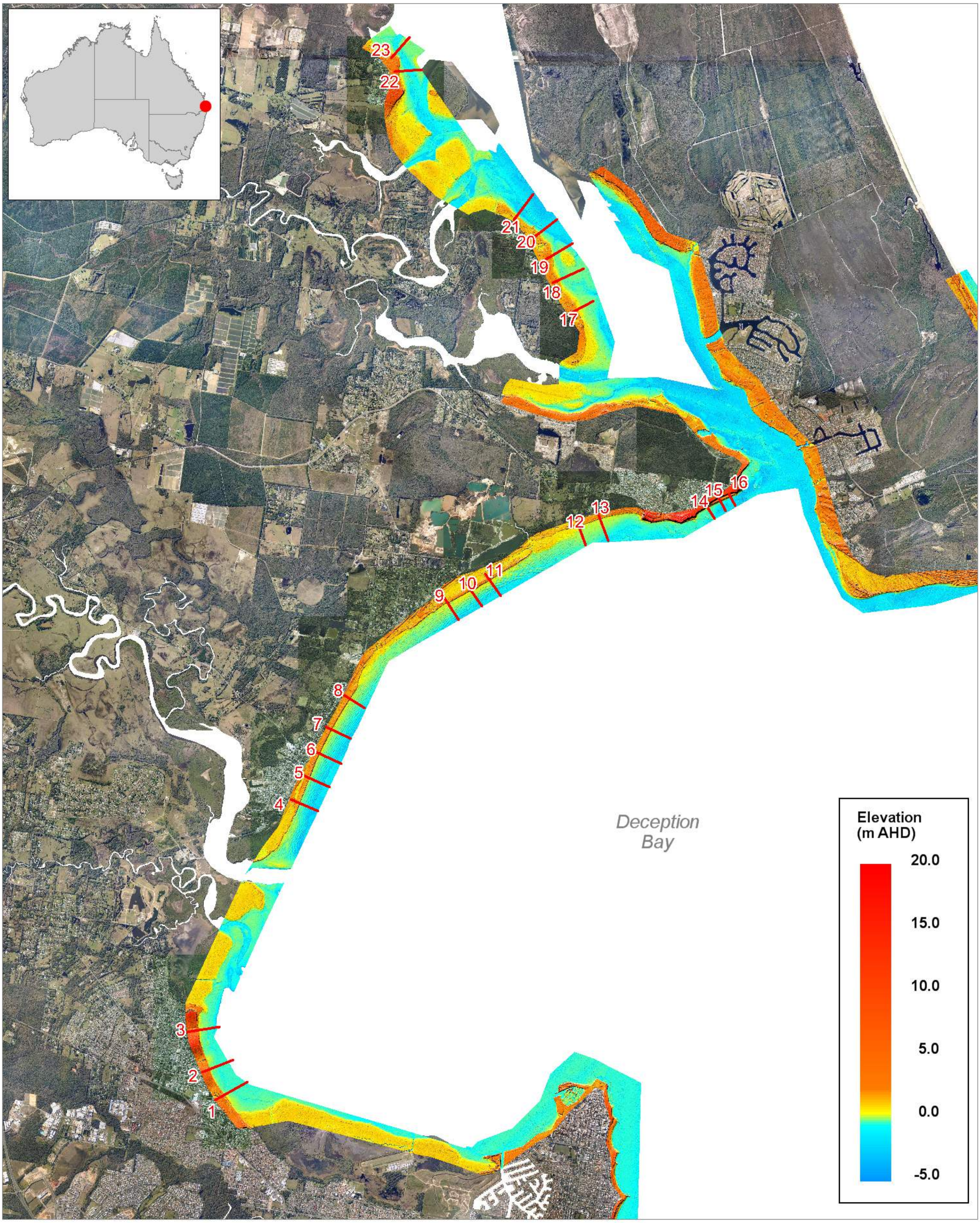
Storm erosion occurs when increased wave heights and water levels result in the erosion of material from the upper shoreline. On open coasts, the eroded material is taken offshore where it is deposited as a sand bar located in the vicinity of the wave break area. After the storm event the sediment is slowly transported onshore, often over many months or several years, rebuilding the beach.

The potential for short-term storm erosion due to severe wave and elevated sea water levels (surge conditions) has been predicted using the simple cross-shore equilibrium profile model of Vellinga (1983). This empirical model calculates upper shoreline erosion associated with storm induced surge and wave conditions. The amount of shoreline recession is determined from the significant wave height, the storm surge plus tide level and the initial beach profile shape. The model assumes the volume of material eroded from the upper shoreline and deposited offshore is balanced by a setback of the shoreline.

Storm erosion assessment was performed at locations where sufficient offshore profile data was available. This information was extracted from a DEM created using a 2013 LiDAR survey of the study area. The survey was flown at low tide and provided bed elevation data across the tidal flats to approximately 1m below AHD. It was necessary to combine this data with information from nautical charts in order to extend the profile beyond the active surf zone (a requirement for the Vellinga model).

Design water level and wave conditions for each beach unit were obtained from the Moreton Bay Regional Council Storm Tide Hazard Study (Cardno Lawson Treloar, 2009). The design storm defined for this assessment combined the 100 year ARI cyclonic design water level with the 20 year ARI design wave height. This design storm definition is recommended for storm erosion assessments as part of the Queensland Coastal Hazard Technical Guide (Queensland Government, 2013). For assessment locations within Deception Bay, the 20 year cyclonic design wave height was adopted (refer Table 5-5). For assessment locations within Pumicestone Passage, the 20 year non-cyclonic design wave height was adopted (refer Table 5-4). This decision was made based on knowledge that the SWAN wave model (the basis for deriving design wave heights described in Cardno Lawson Treloar, 2009) is likely to over-predict extreme waves close to the land-sea transition (e.g. Bottema and van Vledder, 2009). Within the fetch-limited reaches of Pumicestone Passage waves generated by extreme winds are not expected to reach a fully-developed state and therefore the smaller, non-cyclonic design wave heights were considered appropriate.

Figure 6-32 shows the storm erosion assessment locations and Table 6-6 lists the assessment input parameters and the predicted erosion result. The storm erosion distance is measured landward from the position where the design water level intersects the beach profile and varies primarily due to the initial beach profile and volume of material assumed available in the upper shoreline. The relatively minor differences in design storm conditions at each location also influence the estimated erosion distance. Vellinga (1980) predicts more setback for steeper initial profiles since a greater volume of sand is required to achieve the ultimate storm profile. An example storm erosion assessment result for a location at Beachmere is shown in Figure 6-33. The estimated storm profiles for all locations are presented in Appendix B.



Title:

Storm Erosion Assessment Locations

Figure:

6-32

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Approx. Scale

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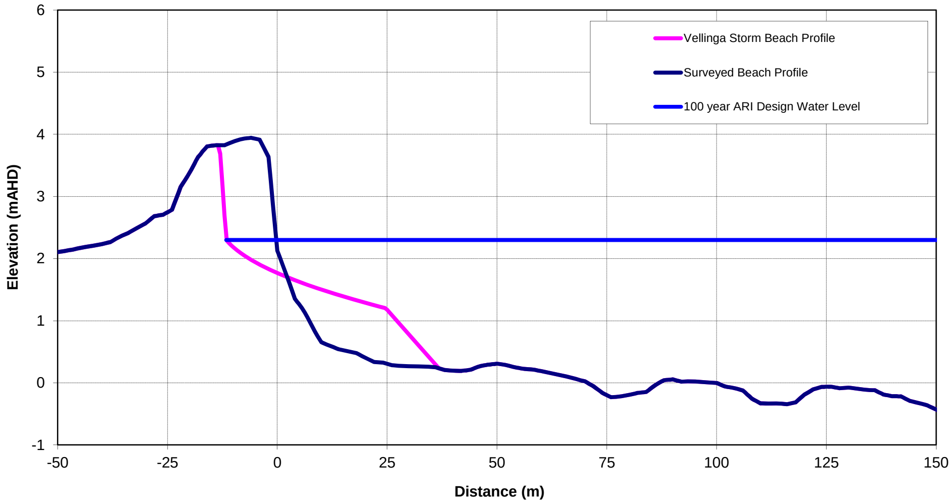

Figure 6-33 Example Storm Erosion Assessment Results at Beachmere

It is noted that no attempt to verify the Vellinga (1983) model estimates has been undertaken and the assessment is assumed to provide conservative erosion potentials. The calculations consider the upper shoreline to consist of erodible material only and therefore erosion will be overestimated in areas where rock, dense vegetation and/or manmade structures exist. Furthermore, the adopted 100 year ARI design water level overtops existing shoreline structures at many assessment locations and the Vellinga (1983) model is not expected to provide reliable estimates of erosion associated with coastal barrier overtopping.

The mean storm erosion width estimate across the entire study area is approximately 16m, with higher than average erosion widths predicted at some Beachmere, Godwin Beach and Toorbul assessment locations. Storm erosion hazard area mapping is presented in Appendix C, together with the 50-year horizon erosion prone area estimate (calculation described in Section 7). It is reiterated that these erosion width estimates are not expected to be realised at shorelines with terminal protection. Nevertheless, the erosion potential results help to identify assets potentially at risk and areas where existing structures may be vulnerable due to relatively high erosion pressure.

Table 6-6 Vellinga Model Inputs and Erosion Estimates

Beach Unit	Profile ID	20yr ARI Offshore Design Wave Height (m)	100yr ARI Design Water Level (mAHD)	Vellinga (1980) Design Storm Erosion Width Potential (m)
Deception Bay	1	0.9	2.2	18.4
Deception Bay	2	0.9	2.2	13.6
Deception Bay	3	0.9	2.2	12.2
Beachmere	4	1.1	2.3	33.1
Beachmere	5	1.1	2.3	20.2
Beachmere	6	1.1	2.3	11.6
Beachmere	7	1.1	2.3	13.5
Beachmere	8	1.1	2.3	14.9
Godwin Beach	9	1.3	2.2	18.6
Godwin Beach	10	1.3	2.2	12.9
Godwin Beach	11	1.3	2.2	16.4
Godwin Beach	12	1.3	2.2	7.4
Godwin Beach	13	1.3	2.2	15.6
Sandstone Point	14	1.0	2.1	10.1
Sandstone Point	15	1.0	2.1	12.5
Sandstone Point	16	1.0	2.1	13.4
Toorbul	17	0.5	1.9	6.5
Toorbul	18	0.5	1.9	4.0
Toorbul	19	0.5	1.9	6.4
Toorbul	20	0.5	1.9	16.2
Toorbul	21	0.5	1.9	18.78
Donnybrook	22	0.5	2.3	13.6
Donnybrook	23	1.1	2.3	9.2

6.5.1 Scour at Terminal Structures

The existing seawalls throughout the study area are expected to limit landward shoreline erosion. However, restricting landward erosion typically transfers the erosion pressure to the toe of the structure where more erodible material is usually present. Scour at the toe of a seawall or revetment can undermine and ultimately lead to a sliding failure of the structure. Toe protection of a terminal shoreline structure is typically provided to a depth that exceeds the predicted scour depth.

The Shore Protection Manual (CERC, 1984) suggests the potential depth of scour is equivalent to the maximum unbroken wave height. Considering the 20yr ARI offshore significant wave heights used in the storm erosion assessment (refer Table 6-6), and that the maximum wave height may be up to twice the significant wave height, the potential depth of scour at the toe of existing seawalls is approximately 2m. Reductions in scour depths can be achieved through careful design, such as adopting a shallower slope angle (e.g. McConnell and Allsop, 1998).