SHUTE HARBOUR MARINA DEVELOPMENT

Coastal Processes Report to Support an EIS

Report Prepared for
Shute Harbour Marina Development Pty Ltd

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1. INTRODUCTION

This Coastal Processes Report has been prepared by Cardno Lawson Treloar Pty Ltd (CLT), specialist oceanic consultants, for Shute Harbour Marina Development Pty Ltd. This report has been completed to support the Environmental Impact Study (EIS) and address relevant sections of the Terms of Reference (ToR).

Shute Harbour Marina Development Pty Ltd propose developing their property at Shute Harbour – see Figure 1.1, as a commercial marina, including some residential and commercial property.

The works include dredging the marina basin to –5.2m AHD and the construction of a breakwater along three sides of the site (see Appendix A). The southern and eastern breakwaters would be formed from sheet-piled walls that may have toes imbedded to 1m below the sea bed with structural support being provided by pile bents at 25m centres. The original plan investigated the suitability of rubber tyre screens supported from horizontal plastic pipe clusters (understood to be 4 x 0.6m diameter. pipelines with draft of 0.6m). However, initial investigations demonstrated that that scheme would lead to unacceptable rates of siltation in the dredged marina basin. This siltation was caused by ebb tide currents flowing through the southern marina area where current speed reduced in the much deeper water, thereby reducing the sediment carrying capacity. Hence the proposed structures were changed to vertical walls that were impermeable to tidal flow.

These impermeable walls, although providing greater protection from wave penetration, then introduced other matters that needed to be addressed. They are:-

- Reflections from the walls may increase the rate of sediment re-suspension in Shute Bay and as a consequence the rate of siltation in other parts of Shute Bay
- Large wave forces on the vertical breakwaters

This report describes the detailed investigations on tidal/wind currents and waves in respect of siltation and local morphology, wave climate for marina design and operation and the impacts of the proposed works on Shute Bay.

Appendix B provides a glossary of terms used this report. Appendix C describes the key physical processes and how they are defined, including:

- Wave processes
- Currents
- Water levels
- Greenhouse impacts
- Coastline stability and bay sediment movement

The analysis techniques employed include long term geomorphologic modelling and incorporate the very latest techniques and modelling tools from Delft Hydraulics of the Netherlands, recognised world leaders in coastal management studies.

Note that the modelling results presented herein are the culmination of a series of studies undertaken as the proposed marina plans developed and they are based on Layout I.

2. DATA

For this study it has been necessary to apply numerical modelling methods to assess the wave climate and siltation rates at the proposed marina site for both existing and developed conditions. Appendix C describes the key physical processes and how they are defined, including:

- Wave processes
- Currents
- Water levels
- Greenhouse impacts
- Coastline stability and bay sediment movement

A range of data items were needed for that task.

2.1 Bathymetric Data

This information was required to describe the seabed and existing/future developments. This data came from three sources. They were:-

- Australian Chart 253 – Whitsunday Passage.
- Digitised soundings of the existing seabed in the immediate vicinity of the proposed marina site provided by Port Binnli.
- Development layout and plans (dredge depths) provided by Shute Harbour Marina Development – two variations.

2.2 Sediment Data

GHD (1999) provide some indicative data describing the general sediment character at the site and within Rooper Inlet. The data indicated there is considerable variation in seabed sediment composition. Sand to silt ratios varied between 70% sand – 30% silt to 20% sand – 80% silt. *FRC Environmental* prepared a spatial description of seabed sediment based on observations of the seabed during field surveys in Shute Harbour. Figure 2.1 presents a general description of sediment composition around Shute Harbour. As with the GHD data, sediments vary between sand dominated (sand greater than 50%) to silt dominated regions.

2.3 Water Levels

Tidal planes within Shute Harbour are presented in Table C.1 (Appendix C). Tides within Shute Harbour are generally semi-diurnal. The 100-years ARI water level has been estimated in previous storm tide studies (GHD, 2003) to be approximately 3.0mAHD. This value includes astronomical tides, wind set-up and wave set-up, but excludes climate change effects.

Predictions of global sea level rise due to the Greenhouse effect have been described in Section 4.4, with an allowance of 0.3 m adopted for this project.

2.4 Wave Data

Wave climate information for offshore and inshore regions within the Shute Harbour area was not available. To this end numerical wave modelling was required to determine the regional wave climate at the proposed site.

2.5 Wind Data

For wave climate and siltation assessment, operational and extreme wind conditions were required. A 3-hourly time series of wind speed and direction was obtained through the Bureau of Meteorology (BoM) from the Hamilton Island Airport Weather Station (33255). An analysis of this wind data showed that the vast majority of wind approached the region from a south-east direction. A wind rose of this data can be seen in Figure 2.2.

2.6 Cyclone Tracks

Extreme wind conditions as a result of cyclone events were approximated using historical cyclone track data obtained from the BoM. This data, which included a time-series of real world position and central pressure, were transformed into wind fields using the Holland wind algorithm, discussed in subsequent sections.

Infrequent tropical cyclones will cause the highest waves at the site. Because of the significant variability in cyclone tracks, cyclone waves propagate to the site from Rooper Inlet and from the Whitsunday Passage north of Repair Island.

This data was obtained from the Bureau of Meteorology web site.

3. WAVE CLIMATE INVESTIGATIONS

The location of the proposed Shute Harbour Marina is such that the complex offshore bathymetry and islands provide good protection from prevailing 'open sea' conditions. As such, locally generated wind waves are of primary concern when assessing the likely wave climate at the proposed site. Again, due to the near shore islands, fetch lengths are relatively short, and only extreme wind conditions, that is, cyclone events, would generate significant waves at the proposed marina site.

A sound knowledge of wave conditions at the site is needed for:-

- Preparation of design wave loads on the vertical wall breakwater structures
- Investigation of operational and design wave conditions within the marina
- Investigation of siltation issues; wave activity being a principal cause of seabed sediment re-suspension

Having no inshore wave data in close proximity to the site available, numerical wave modelling was required to assess operational and design wave conditions.

3.1 SWAN Wave Model

The numerical wave modelling system adopted for this study was SWAN, which was developed at the Delft Technical University and has been integrated into the Delft3D modelling system. SWAN provides a full spectral solution of the wave energy flux equations and includes refraction, diffraction, shoaling, bed friction, white capping, boundary wave input, wind input, directional spread, non-linear wave-wave interaction and current-wave interaction. It includes a nested model capability that overcomes the problem that arises when it is necessary to model a large offshore area using a coarse grid, yet fulfils the need for better resolution in selected areas where the rate of spatial variation in seabed character is high.

For this study a coarse grid area based on a 75m grid, that extends east to the Whitsundays, to the southern end of Long Island and north to North Molle Island, was adopted, see Figure 3.1. Additionally, a fine grid area (grid size of 25m) was adopted (Figure 3.1) that covers Shute Bay, Rooper Inlet and east to Shute Island. It is believed that this model set up provides a reliable modelling system for the Shute Harbour region. The model extent includes all appropriate fetch lengths from which waves can be generated to impact upon the proposed marina site – that is, all fetches are enclosed by natural land.

3.2 Cyclone Waves

The Shute Harbour-Whitsunday Islands region is affected by cyclones. Historical tracks defined by data obtained from the Australian Bureau of Meteorology are depicted in Figure 3.2. Major events likely to affect the Shute Harbour area were selected from a complete listing of historical cyclones

Only cyclones occurring since the mid-1950's were included in the selected data. Cyclone identification improved in that period as a result of the development of satellite imagery and over-the-horizon radar. A total of thirty historical cyclones were identified within the selection zone, see Figure 3.2, but not all of those would have caused notable waves in the study area. Fifty years of records were adopted on these bases. Table 3.1 identifies each of the cyclones considered and the date on which each passed within the defined Shute Harbour region.

Table 3.1 Cyclones Selected for Design Wave Analysis

Cyclone	Date	Cyclone	Date
Agnes	4/3/1956	Simon	22/2/1980
1958-Apr	31/3/1958	Freda	27/2/1981
Flora	6/12/1964	Elinor	1/3/1983
Dinah	26/1/1967	Grace	15/1/1984
Ada	17/1/1970	Pierre	21/2/1985
Althea	23/12/1971	Winifred	1/2/1986
Gertie	15/2/1971	Aivu	3/4/1989
Emily	31/3/1972	Joy	26/12/1990
Vera	17/1/1974	Fran	12/3/1992
Gloria	17/1/1975	Oliver	8/2/1993
David	17/1/1976	Rewa	3/1/1994
Dawn2nd	4/3/1976	Celeste	27/1/1996
Watorea	27/4/1976	Justin	9/3/1997
Gordon	11/1/1979	Erica	2/3/2003
Kerry	28/2/1979	Wati	22/3/2006

Each of these cyclone tracks was examined closely. About three track-time data points from each of the cyclones that were most likely to have caused high waves in the study area were selected for wave analysis. Spatially varying wind speeds and directions were determined using the windfield algorithm developed by Holland (1980) for the Australian Bureau of Meteorology and adopting a neutral pressure of 1005hPa with a radius to maximum winds of 30km. These parameters have been determined to be realistic in the Australian region as part of other Cardno Lawson Treloar studies related to design cyclone parameters. This windfield model is considered to be the most appropriate for the tropical Australian region.

Typical wave fields in terms of significant wave height (H_s) developed from two cyclone simulations, being the April 1958 and Cyclone Celeste events, are described in Figures 3.3 and 3.4. Results of the cyclone modelling show that for some cyclone events, April 1958 for example, waves may propagate to the site from an easterly direction along the channel between Repair Island and Coral Point.

The cyclone events were run for a high tide (3.3m LAT), mean water level and a low tide (0.5m LAT). Because this is a relatively shallow area, the largest waves develop at high tide.

The results from each of the cyclone events were then analysed using the Extreme Value Type 1 distribution and the maximum likelihood parameter estimate method to determine peak storm wave heights (H_s) for average recurrence intervals (ARI) between 10 and 500-years. The Extreme Value Type 1 distribution is commonly known as the Gumbel distribution and is generally appropriate for extremal analysis of ocean data, Lettenmaier and Burges (1982).

Results from these analyses are presented in Table 3.2. These design wave parameters have been determined for the seaward sides of the vertical wall breakwater structures. Locations of the selected output points can be seen in Figure 3.1.

Table 3.2 Design Wave Heights (H_s) for Breakwater Design

Analysed SWAN Output – H_s (m)						
ARI (years)	*1*	*2*	*3*	*4*	*5*	*6*
10	0.83	0.84	0.85	0.91	0.88	0.79
25	1.20	1.22	1.22	1.30	1.27	1.16
50	1.43	1.45	1.45	1.55	1.52	1.38
100	1.65	1.68	1.66	1.78	1.75	1.60
200	1.86	1.90	1.88	2.00	1.98	1.81
500	2.14	2.18	2.15	2.30	2.28	2.09
Coordinates - AMG Zone 54						
X	685282	685569	685846	686127	686143	686143
Y	7754673	7754672	7754669	7754669	7754799	7754898

Generally, for short fetches, the $H_{1/100}$ parameter is suitable for rigid structure design, for example, the proposed vertical, impermeable walls. This parameter is evaluated as:-

$H_{1/100} = 1.67 \times H_s$.

3.3 Wave Climate within the Marina

The proposed marina design includes three breakwater structures. The western side will be flanked by a land reclamation area. The southern and eastern sides of the marina will be protected by vertical walls with only two openings on the eastern side, see Figure 3.1. The south-eastern opening is required for navigational purposes and is about 40m wide at entrance to the marina berth area. Both entrances provide flushing capacity. Wave penetration into the marina will therefore be limited, however, there exists some potential for waves to penetrate along the marina entrance channel and around the northern end of the eastern breakwater.

Due to the large area encompassed by the marina it is also the case that wave climate at the berths will be influenced by wave generation within the marina. Under severe cyclonic wind conditions this is likely to be the greater influence on wave climate at the berths, where wind speeds can exceed 40 m/s.

In order to assess the extent of potential wave penetration into and wave generation within the marina, wave conditions and wind fields were applied to a local fine grid SWAN model of the marina. This model also included the effect of wave diffraction and reflection processes on wave propagation. Major structures and berths that influence wave propagation are included as obstacles in the model, all with representative transmission and reflection coefficients. The vertical wall breakwaters on the southern and eastern side are highly reflective structures with coefficients of between 0.5 and 0.9, depending on the incident wave period. That is, reflection of shorter period (and shorter crested) waves are not reflected to the same scale as longer period waves. The inner walls of the marina are included with a reflection coefficient of 0.5. Both these structures do not allow any wave transmission. The pontoons within the marina will also act to limit wave generation and propagation; again their influence is a function of the wave period. Wave transmission through the pontoons would likely be between 0.5 and 0.7 of the approaching wave height. Calculations of transmissity were based on typical widths and depths of floating pontoon structures.

To assess both the wave penetration and wave generation within the marina, two series of simulations were undertaken for all cyclone events described in Section 3.2. The first utilised results from the above cyclone wave modelling and applied wave conditions spatially along the boundary. These simulations excluded the influence of wind over the model so as to identify the amount of wave energy that is able to penetrate from Shute Bay into the marina area. The second series of cyclone simulations included both the boundary wave conditions and the spatially variable wind field over the model area to assess the amount of wave generation that may occur within the marina itself.

Wave height plots for two cyclone events, Celeste and Agnes, are presented in Figures 3.5 and 3.6, showing results of both types of simulations. Cyclone results were analysed using the Extreme Value Type 1 distribution and the maximum likelihood parameter estimate method to determine peak wind speeds for average recurrence intervals (ARI) between 10 and 500-years at the six locations around the marina site, see Table 3.3. Comparison of the two selected cyclone events suggests that Agnes is equivalent to a 50-years ARI event and Celeste to a 100 - 200-years ARI event, approximately, in terms of wind speed.

Table 3.3 Design Cyclonic Wind Speeds (m/s)

ARI (years)	Wind Speed					
	Output 1	Output 2	Output 3	Output 4	Output 5	Output 6
10	23.4	23.4	23.4	23.4	23.4	23.4
25	31.4	31.4	31.4	31.4	31.4	31.4
50	36.8	36.8	36.8	36.8	36.8	36.9
100	42.1	42.1	42.1	42.1	42.1	42.1
200	47.2	47.2	47.2	47.3	47.3	47.3
500	54.0	54.0	54.0	54.1	54.0	54.1

The cyclone simulations show that the breakwater and entrance alignment are extremely effective at blocking waves from penetrating into the marina from Shute Bay. Extremal analysis of these penetrated waves are presented in Table 3.4 and suggest that the 100yrARI wave is less than 0.2m for all areas of the marina. Reported output locations can be seen in Figure 3.1.

Table 3.4 Wave Penetration Design Wave Heights (H_s) (Cyclonic waves, no local wind over marina)

ARI (years)	Output Location						
	1	2	3	4	5	6	7
10	0.08	0.07	0.02	0.01	0.03	0.03	0.01
20	0.11	0.1	0.03	0.02	0.04	0.03	0.01
50	0.14	0.14	0.05	0.03	0.05	0.04	0.02
100	0.16	0.16	0.06	0.03	0.05	0.05	0.02
200	0.18	0.19	0.07	0.04	0.06	0.05	0.02
500	0.21	0.22	0.08	0.04	0.07	0.06	0.02

Wave generation within the marina under cyclonic conditions is more significant, though. Figures 3.5 and 3.6 show that wave heights of up to 0.9m may result at locations along the western wall if the extreme cyclonic winds occur coincident to the waves, and the wind is from due east. The high wave locality, however, is largely dependant on the cyclone track and the prevailing wind conditions (particularly direction) over the marina. Extremal analysis results for the cyclone wave conditions within the marina are presented in Table 3.5, which provides a 100-years ARI wave condition (H_s) of just over 0.9m where wind is from the east.

Table 3.5 Wave Penetration Design Wave Heights (H_s) (Cyclonic waves, with coincident local wind over marina)

ARI (years)	Output Location						
	1	2	3	4	5	6	7
10	0.32	0.4	0.45	0.36	0.31	0.34	0.41
20	0.42	0.55	0.62	0.47	0.42	0.47	0.57
50	0.54	0.71	0.8	0.59	0.55	0.61	0.74
100	0.62	0.82	0.93	0.67	0.64	0.71	0.86
200	0.69	0.93	1.06	0.75	0.72	0.8	0.98
500	0.8	1.07	1.22	0.86	0.84	0.93	1.13

Waves generated in Shute Bay will have longer periods than those generated within the marina due to the longer fetch lengths over which they are generated. Waves penetrating into the marina will have periods in the order of 4 to 5 seconds. Comparisons of the SWAN results suggest that the marina design is effective in minimising this longer period wave penetration, however, under severe wind conditions wave generation within the marina will be of greater concern for berth design. Associated wave periods for local sea conditions inside the marina are shorter and approximately 2s or less.

It is considered that the current layout design would generally comply with marina codes in terms of wave climate. Relevant code requirements for wave climates in small craft marinas include:-

- AS 3962 recommends that berth facilities are sheltered from a wave height (H_s) of 0.75m (50-years ARI) to achieve the criteria for a moderate wave climate. Furthermore the wave event exceeded once a year should be less than 0.38m. Inspection of Table 3.5 suggests that both criteria are fulfilled, generally.

- A wave height of 0.3m, which should not be exceeded more than 10% of the time for small-craft harbours (recommended by CEM, page V-5-74). This alternative condition would also be fulfilled. However, on rare occasions, those berths near the marina entrance would experience slightly higher waves.

The analysis of cyclone Celeste and Agnes also shows that:

- Cyclone Celeste is estimated to have been an ARI 50 year event within the marina site;

- With the proposed marina, internal wave conditions are generally less than 0.6m at the berths during a cyclone Celeste event;

- Cyclone Agnes is estimated to have been an ARI 100 to 200 year event within the marina site, with wind from due east;

Within the proposed marina, internal wave conditions are generally less than 0.8m at the berths during a cyclone Agnes event.

3.4 Post Development Wave Climate

An assessment of the effect of the development on the wave climate within Shute Bay to the south-west of the site was undertaken utilising the nested fine grid SWAN model. Operational and extreme wave events were simulated for pre- and post- development layouts. The modelling included reflections from the vertical walled breakwaters, assumed to have a reflection coefficient of 0.9 – some attenuation being caused by the roughness of the sheet piles.

The comparison between pre-and post- development conditions was made on the basis of the different bed shear stresses developed on the seabed in selected wave conditions. Bed shear stress (peak, under wave crests used) is a measure of the friction force exerted on the bed by propagating waves and hence the capacity for sediment re-suspension.

If the bed shear stress becomes sufficiently high, sediment re-suspension may be initiated, therefore an increase in the wave energy in Shute Bay could potentially increase the amount of sediment transport. For the silty sediments typically found within Shute Bay, a value of 0.3 to 0.4N/m^2 is considered to be a realistic threshold for sediment re-suspension, where there is no seagrass.

Four operational wave conditions were assessed; 5m/s, 7.5m/s, 10m/s and 12.5m/s winds all from the south-east. These wind conditions were defined by analysis of the Hamilton Island Airport wind data and are representative of typical operational and severe weather conditions. Under existing conditions, winds of these magnitudes are understood to not cause sediment re-suspension across the majority of the Bay; only at the shoreline.

The results of these operational cases can be seen in Figures 3.7 to 3.14. In all cases, wave heights and bed shear stresses are increased, particularly off the south-western corner of the marina, due to the reflective nature of the vertical wall breakwater. Note, for example, on Figure 3.7, the incident and reflected waves, shown as vectors, propagate from east to west along the southern wall. This is because only average wave direction can be plotted from the model results and this is the average direction of the incident and reflected waves. Each vector describes the local wave direction (towards) and height. Generally the results show areas of increased bed shear stress following construction south-west of the southern wall. However, only for south-easterly winds of about 12.5m/s does the shear stress in the area south-west of the wall approach the critical shear stress for sediment re-suspension. In some shoreline areas the presence of the marina would reduce shoreline shear stresses, see Figure 3.14.

The same assessment was undertaken under extreme wind conditions. Cyclone Celeste (27/1/1996) was selected as it was one of the more severe historical events for the Shute Harbour area. Figures 3.15 and 3.16 show the wave height and bed shear stress comparison between the pre- and post- development conditions. While it shows that the development would increase bed shear stresses, both pre- and post- development conditions are above the threshold for sediment re-suspension. The walls would cause sediment re-suspension at an earlier stage as the wave conditions develop; however, it would have a relatively insignificant impact on the overall effects of the cyclone event.

This matter is given further attention in Section 4.

3.5 Wave Forces

In order to provide design criteria for the southern and eastern vertical breakwater walls, the 200-years ARI wave heights, together with a wave period (T_p) of 4.2 seconds, were adopted. The details of these calculations are presented in Appendix D. Loads are in terms of the 200, 100, 10-years ARI design criteria.

Wave forces on a vertical wall result from the total hydrodynamic pressure distribution, which consists of two time varying components: the hydrostatic pressure component due to the instantaneous water depth at the wall, and the dynamic pressure component due to the accelerations of the water particles. Over a wave cycle, the force found from integrating the pressure distribution on the wall varies between a minimum value, when a wave trough is at the wall, to a maximum value, when a wave crest is at the wall. Wave forces have been based on $H_{1/100}$ which is calculated as:-

$$H_{1/100} = 1.67 \times H_s$$

When a wave crest impinges on the wall, the force on the seaward side is maximum and directed inward. Wave overtopping may occur, which provides a reduction in the total force and moment because the pressure distribution is truncated. When a wave trough is at the wall the force on the seaward side of the wall is a minimum, however the force due to the water level on the landward side (in this case inside the marina) may then be significant and cause a net outward force.

Non-breaking waves incident on smooth, impermeable vertical walls are completely reflected by the wall giving a reflection coefficient of ~1.0. Where walers, tiebacks, or other structural elements increase the wall surface roughness and retard the vertical water motion at the wall, the reflection coefficient will be slightly reduced. In order to determine the trough level, the reflection at the wall was calculated using a formula developed by Allsop et al (1994) for head-on waves. Generally, in this study, the reflection coefficient varied from 0.85 to 0.9 for these analyses, depending on the ratio between incident wave height and structure height above still water.

Loads have been calculated at the 100-years ARI storm tide level of 3.0m AHD for the 100 and 200-years ARI and at the Mean High Water Spring (MHWS) level of 1.5m AHD for the 10 years ARI, plus an allowance of 0.3m for possible MSL rise. Force calculation was also based on the available depths at each location.

In addition to single forces on each individual block there are spatially variable and changing oscillatory forces on the whole wall due to the successive passage of crests and troughs - more specifically the instantaneous action of crests and troughs along the sheet-piled wall. In order to develop an insight into the wave load distribution along the breakwater it was necessary to undertake an investigation into the wave crest and trough distribution along the breakwater.

For the southern section of the breakwater the incident wave angle is approximately 45 degrees. Each crest along the wall will then be separated by approximately $L_B = L_0 / \sin \alpha = 20m$, where L_0 is the wavelength based on design T_p of 4.2 seconds.

For the eastern section of the breakwater the waves would tend to hit head-on, which would imply that in theory a crest (or trough) is present at the same time along the whole length of the breakwater. However, this will not occur in reality due to the three-dimensional characteristics of the incoming waves – the directional spread of wave energy. An approximation of the wave crest length - that is, the distance along the crest or in the direction parallel to the ‘main’ wave crest, where the wave height varies from its maximum (peak crest height) to its minimum (trough), was undertaken. Considering a directional spreading of 20^o, which is representative of local sea waves; based on analyses of recorded wave data, the wave crest length was calculated by considering the superposition of sinusoidal wave trains varying from 0° wave angle of incidence to the breakwater (head-on) to 20° wave angle of incidence. The wave crest length was then 50m, which means that each crest (centroid of inward force), along the eastern wall will then be separated by approximately 50m – as a realistic, but idealized design wave load variation along the wall.

In summary, the southern breakwater will experience inward forces and outward forces every ~10m and the eastern breakwater will experience inward forces and outward forces every ~25m.

Considering a vertical breakwater design with pile bents spaced every 25m, then the southern breakwater may experience inward forces at a pile bent and outward forces in between the piles (or outward force at the pile bent and an inward force in between the pile bents), for example – or a range of other load forms as the waves effectively move along the wall.

The eastern breakwater may experience an inward force at a pile bent and outward force at the next pile bent (or an inward force in between two pile bents and outward forces at the next space between two pile bents).

Maximum wave loads may apply at the crest and trough; however, the force is spread over a section of the breakwater - that is, ~10m for the southern breakwater and 25m eastern breakwater. The design wave loads for the wave crest and trough were determined by calculating the mean wave loads that apply along the sections of the breakwater for the total sheet-piled height. Mean wave loads are equal to 2/π times the maximum wave load at the crest and trough. (i.e. $\frac{1}{\pi}\int_0^{\pi}\sin x dx = \frac{2}{\pi}$)

Maximum wave loads on the seawall for the wave crest and trough conditions were first calculated for 10, 100 and 200-years ARI conditions with a vertical wall of 5m AHD. Overtopping was determined using the overtopping formula developed by Franco and Franco (1999) for an impermeable vertical wall. Wave overtopping occurred for both 100 and 200-years ARI conditions.

Wave action in the marina beyond the vertical breakwaters may be caused by wave overtopping. Waves generated by the falling water from overtopping tend to have shorter periods than the incident waves. Generally the transmitted wave periods are about half those of the incident waves.

Wave transmission can be characterized by a transmission coefficient, C_t, defined as the ratio of transmitted to incident characteristic wave heights (for example, *Hs).* The ratio of the breakwater crest height above still water level (R_c) to the incident wave height (H_s) is the most important parameter in terms of wave overtopping.

Wave transmission into the marina was calculated using Goda's formula (1969) for head-on waves on plain vertical breakwaters (see Coastal Engineering Manual Table VI-5-16 EM 1110-2-1100 Part VI). The incident wave angle was not considered here because there are only a limited number of formulae available for plain vertical breakwaters similar to the proposed design. It is anticipated that the transmitted wave would be smaller with a greater incident wave angle.

The wave transmission coefficients, C_t and transmitted wave heights H_t were calculated for the 10, 100 and 200-years ARI significant wave heights and with seawall crest levels of 5, 4.5, 4 and 3.5 m AHD. For the 10-years ARI wave height there is no overtopping for all seawall crest levels investigated; therefore there is no change in the wave forces.
Maximum wave loads on vertical wall 1m blocks for the 10-years ARI wave and a wall crest at 5m AHD are presented in Table D-I-1 and Figure D-I-1. Wave loads (forces, moments and moment arm-heights above the seabed) for 10-years ARI wave conditions are presented in Table D-I-2. Transmitted significant wave height for the 100 and 200-years ARI waves for the selected seawall crests levels are presented in Tables D-II-1 and D-III-1.

A seawall crest level of 4.7m AHD was then adopted for the wave loads calculations. Maximum wave loads on vertical wall 1m blocks for the 100 and 200-years ARI wave and a wall crest at 4.7m AHD are presented in Tables D-II-2 and D-III-2 and Figures D-II-1 and D-III-1. Wave loads (forces, moments and moment arm-heights above the seabed) for 100 and 200-years ARI wave conditions are presented in Tables D-II-3 and D-III-3.

Maximum forces at the crest and trough could be determined by multiplying the forces presented in the Tables by π/2.

Note that wave forces on individual 1m x 1m blocks are provided as well as linear (along the wall) averaged loads for pile bent design. The latter assume that there is some lateral load distribution capacity in the wall so that the larger wave forces at wave crests are supported through lateral load distribution by more than the immediate section of wall at a wave crest. This is a realistic design position because of the short-crested nature of the waves (local sea) and the angle of impingement.

Impulsive forces caused by the collapse of a breaking wave crest on the wall will be reduced by making the face of the wall rough. This condition can be achieved using sheet pile sections – 300mm crest to crest of pile sections. Roughness prevents simultaneous Impulsive forces from occurring on a significant section of the wall.

Note that the forces are oscillatory and that it will be important to consider fatigue issues in design. In terms of wave conditions, there will be waves at the site for about 85% of the time. For a wave period of 2.5 seconds there will be about 12.5 million waves a year. Cardno Lawson Treloar's experience, post the fact, at three sites where there has been serious damage to vertical walls in not dissimilar conditions, is that fatigue failure is a major issue for walls of this type, and needs to be carefully considered in the structural design.

4. SILTATION AND LOCAL MORPHOLOGICAL ISSUES

Siltation can be a major factor for the economical and operational success of harbours and marinas. The proposed marina is in a silty/sand location and waves can frequently cause local sea of energy levels sufficient to re-suspend nearshore sediments, see for example, Figure 4.1. Wind conditions at the site are described in Figure 2.2 and show that the local wind conditions are dominated by winds from the south-easterly sector. Tidal and wind driven currents can transport these sediments to more tranquil areas, for example, the proposed marina, where they may settle on the seabed. Even more muddy conditions may develop in a severe cyclone.

There is little detailed data on seabed sediments in the study region. However, some descriptive data is provided in GHD (1999). The regional sediments can be described as silty-sands. Within Shute Harbour there is significant spatial variation in seabed sediment composition. Figure 2.1 presented a spatial description of seabed sediments based on field observation by FRC Environmental. There is insufficient quantitative data of sediment composition around Shute Harbour to prepare a detailed spatial distribution of silt and sand fractions. Therefore two general seabed sediment types were defined in the modelling. The general sediment composition has been described as a sand/silt mix. The sand fraction has been specified as 70% of the overall sediment composition and the silt fraction has been selected as 30% of the composition. Physical testing of similar seabed sediments with greater than 60% sand content indicate that although the silt component is only 30% of the seabed, the character and physical properties of the sediment are dominated by the silt/clay fraction rather than the sand fraction. Un-erodable areas have also been described in the model, for example, the Intertidal Reef in Figure 2.1.

The morphological modelling has not considered other sediment loads, for example, catchment sediment loads or ambient suspended sediment in tidal flows that would be transported into Shute Harbour from time-to-time. There is very little reliable data to quantify these processes; however the catchment area is limited and deeper waters beyond the entrance of the Bay are normally clear. The morphological modelling has focused on the potential for sediment redistribution inside the Harbour following construction of the proposed marina.

4.1 Model Set Up

Shute Harbour is located within the Whitsundays Region of Queensland, see Figure 1.1. The regional hydrodynamic processes in this area are complex due to the bathymetry and landform of the island group and the Great Barrier Reef, together with the tidal range in the area, which is up 4.3m at Shute Harbour. There was insufficient recorded data to develop boundary conditions (water levels or discharges) inside the Whitsundays. Therefore a regional model covering the whole of the Whitsundays extending from Bowen in the north to Mackay in the south was developed. This model was used to develop realistic water level and discharge boundary conditions inside the Whitsundays; to then apply to the boundaries of a finer scale model of the Shute Harbour study area. The regional Delft3D model was driven by predicted tidal water level boundaries. The five principal tidal constants (K_1, O_1, M_2, N_2, S_2) from the following tidal stations were utilised at the water level boundaries of the regional model (Australian Hydrographic Service, 2007):-

- Bowen (No. 59320),
- Unnamed Reef No. 2 (No. 59280),
- Mackay Outer Harbour (No. 59510), and
- Penrith Island (No. 59500).

The regional model featured 250m x 250m grid resolution and was run in 2D (depth-averaged) mode. Figure 4.2 describes the extents of the regional and fine grid models.

The regional model was verified at Shute Harbour using the predicted tide there for a selected simulation period. Figure 4.3 presents the predicted (solid black line) and modelled (dashed red line) water levels over a 28-days period at Shute Harbour. Overall the regional model is in good agreement with the predicted tide. Figure 4.4 presents a comparison of measured and modelled water levels at Shute Harbour for the period between 23 May 2007 and 06 June 2007. Again there is good agreement between modelled and measured water levels.

The finer scale model covers Shute Harbour and surrounding areas of the Whitsundays. It is designed to describe tidal and wind driven currents realistically, together with wave processes due to local sea, in the marina region. The model features a curvilinear grid which is aligned with the shoreline of Shute Harbour. Grid resolution near the marina is as fine as 15m by 15m horizontally. The model was run in 3D mode with 10 vertical sigma layers. The thickness of the sigma layers varied through the water column (2%, 5%, 8%, 15%, 20%, 20%, 15%, 8% and 2% - top to bottom) with thin layers at the top and bottom where process gradients are steep. For example, as caused by wind driven currents in the upper water column and the vertical shear structure in the bottom boundary layer.

A combination of water level time series at the northern boundaries, and discharge time series at the southern boundaries was applied to the fine grid model. This combination of boundary conditions enhances model stability. The time series boundary information was developed from the regional Delft3D model (see above). Bed friction in the model was described by the Chezy equation with a coefficient of $65m^{1/2}/s$.

Horizontal eddy viscosity and diffusivity coefficients were specified as $1m^2/s$, which is appropriate for the grid resolution of the model. Vertical viscosity and diffusivity were described by the k-ε model.

The silt fraction of the Delft3D model is based on the sediment description presented in GHD (1999), the sediment in the model was specified as cohesive. A fall velocity of 0.6mm/s was specified based on GHD (1999) and van Rijn (1993). A dry bed density of $500kg/m^3$ was specified, which reflects a sediment density exposed to an extended consolidation period. In areas of deposition, in the weeks to months following sediment deposition, the deposited sediment dry bed density would be significantly lower, for example only $200kg/m^3$ (van Rijn, 1993). Other sediment parameters were:-

- $\tau_{crit,e} = 0.4 N/m^2$,
- $\tau_{crit,d} = 0.1 N/m^2$, and
- Maximum Erosion Rate = $0.00015kg/m^2/s$.

During periods of high currents, modelled siltation and erosion is sensitive to the specified maximum erosion rate.

The sand fraction of the Delft3D model has been defined by a median sediment diameter of 200μm. In the following sections, the morphological changes in the Delft3D model are dominated by the silt fraction rather than the sand, which has much lower mobility. The van Rijn 1993 transport model has been prescribed in the model.

The hydrodynamics of the fine-scale Delft3D model were verified using the predicted water level at Shute Harbour. Figure 4.4 presents a time series plot of water levels at Shute Harbour based on the predicted tide (solid black line), regional Delft3D model (dashed red line) and the fine scale 3D model (dot-dash blue line). The agreement between the two Delft3D models is very good. The modelled water level shows good agreement with the measured data for both phase and tidal range.

For the post-marina layout, the design presented in Appendix A was specified in the model. The southern and eastern breakwaters were represented by thin dams which are 90%

reflective to incident wave conditions and block the tidal/wind driven flow, but reflect some wave energy incident from the south-east towards the south-west, for example.

4.2 Sediment Transport Model Verification

A suspended sediment data collection exercise was undertaken on 5 June 2007 to obtain suspended sediment data in terms of concentration with which to calibrate the Delft3D model. A total of three mid-depth suspended sediment samples were obtained at each of six locations in the western half of Shute Bay. Figure 4.5 presents a plan view of the suspended sediment sampling locations. The sites were selected to provide a description of mid-bay and nearshore suspended sediment concentrations. The data collection area is also where the current and wave conditions will be most altered following construction of the proposed marina.

The data collection period coincided with the SE trade wind season which is the prevailing wind condition at the site. Wind speeds were in the order of 15 knots from the south-east to south during the data collection period. During the data collection exercise, the Hamilton Island anemometer only recorded sporadic data rather than reporting hourly wind conditions, due to problems with the automatic weather station (AWS). Sediment transport rates in Shute Bay are very sensitive to wind speed and direction and part of the verification process involved adjusting the interpolation between the sporadic wind data records from the Hamilton Island AWS. Wind conditions from other nearby AWS were compared to the available Hamilton Island data to determine possible trends in the wind conditions throughout the day.

Figures 4.6 and 4.7 present time series comparison plots of modelled and measured mid-depth suspended sediment concentrations. In general, the modelled suspended sediment concentration agrees well with the measured data. At Location 6, the modelled results are lower than the field measurements. Location 6 is located along the southern shore in a region which is likely to feature seabed areas that have higher silt content than the uniform 30% prescribed in the Delft3D model. However, it is most critical that the modelled and measured suspended sediment concentrations are in good agreement near Location 3 because this is the region where sediment suspension rates are increased following the construction of the proposed marina (see Section4.3).

4.3 Annual Siltation Simulations

A large range of combined water level, wind and wave conditions could influence morphological processes at the marina site and in Shute Bay. An initial simulation was undertaken to investigate the influence of tides and wind (including local sea waves) on the siltation process. Under calm conditions, very little sediment becomes re-suspended – virtually zero. Local sea bottom stirring by waves is the primary sediment re-suspension mechanism and tidal/wind currents act to transport this material.

A series of simulations were developed to describe the long term siltation patterns in terms of the local wind climate. The following cases and their frequencies of occurrence were investigated:-

1. No wind (includes winds between 180 and 45 degrees),
2. 112.5 degrees @ 7m/s,
3. 146.25 degrees @ 7m/s, and
4. 146.25 degrees @ 12m/s (high and low water cases).

These four cases can be used to describe wind conditions approximately 75% of the time in the study area. A frequency – duration analysis of the Hamilton Island wind record was undertaken to determine the average persistence of wind speeds greater than 7m/s and 12m/s. The average persistence of wind speeds greater than 7m/s is 18 hours, and the average persistence of wind speeds greater than 12m/s is 6 hours. These durations were included in the modelling. The duration of wind speeds greater than 12m/s is less than the period of the semi-diurnal tide and as a result the 12m/s wind case was specified for two tide conditions; low water and high water.

An initial simulation using the existing bathymetry and a morphological factor of 26 was undertaken for a 14-day simulation period. The simulation produced a smoothed initial bathymetry with 1-year of equivalent morphological change (26 x 14 = 364 days). This process smoothed irregularities in the initial bathymetry due to the limited data and slight initial incompatibility between the modelled seabed and wind/waves. The marina layout was applied to the post 1-year bed change bathymetry.

Figures 4.8 to 4.11 present the spring tide velocity currents at 2-hourly intervals near the marina site for the existing condition. Figures 4.12 to 4.15 present similar plots for the post-marina layout. In Figures 4.8 to 4.15 a wind condition of 7m/s from south-east was applied, which is representative of median conditions during the SE Trade season.

Comparing the existing and post-marina cases, the marina acts as a flow constriction. During the simulated flood tide period in the Shute Harbour region (06:00 to 12:00 3 January 2006), the presence of the marina increases the currents along the southern shoreline opposite the marina. During the ebb tide, currents near the tip of the western breakwater are significantly increased.

Figure 4.16 presents the net sediment transport vectors for the existing and post-marina cases over 3 consecutive days (spring tide, median wind and wave conditions). The increase in currents along the southern shoreline results in a significant increase in the sediment suspension rate and mass transport. This increase in sediment transport is also due to waves reflecting from the southern wall to this area.

Figure 4.17 presents an estimate of the average annual siltation and erosion rates for the existing and post-marina cases for the first 5-years post-construction. In Figure 4.17, the four simulated wind conditions have been combined together proportionately, based on the long-term wind climate data for the site (see Figure 2.2). The increase in re-suspension of seabed material along the southern shoreline presented in Figure 4.16 is reflected in Figure 4.17 by the increased erosion in this area. Once the combined tidal and wind driven currents reach the upper section of Shute Cove, the additional material in suspension in the post-marina case will settle out of the water column. The model results for the existing case show that the model bathymetry is relatively stable and there is no significant erosion or deposition trend in Shute Bay. This is consistent with the observed seabed bathymetry in recent times being relatively stable

Figure 4.18 presents the initial seabed bathymetry for the post-marina layout and the post 5-years bathymetry. During this period, there has been deposition to the west of the western breakwater and erosion along the southern shore immediately opposite the marina. Figure 4.19 presents a similar 5-years bathymetric plot for the existing condition. The existing case shows very little change in the bathymetry over the 5-years period. Natural processes produce up to 15 cm siltation per annum and up to 10 cm scour in the existing bay.

Figure 4.20 presents average annual siltation rates for the 5 to 10 years post-construction period for the existing and post-marina layouts. Compared with Figure 4.18, the rate of annual morphological change has decreased. Figure 4.21 presents the initial and post-10-years bathymetries for the post-marina case. For the post development case, there is a limited accretion zone predicted immediately west of the western wall and some additional scouring of the southern bay close to the shoreline. Figure 4.22 presents a similar plot for the existing layout. Compared with the post-marina case, there has been little to no change in the bathymetry of Shute Bay in the existing case.

Inside the marina basin, over a 10-years period there has been up to 0.3m of deposition in the eastern end of the basin. This has raised the seabed levels in this region from -5.2m AHD to -4.9m AHD. In the western half of the basin, total siltation depths are in the order of 0.03 to 0.06m over a 10-years period. In the access channel, siltation rates over a 10-years period are up to 0.2m (western end). The average annual siltation volume in the marina basin is approximately 3,000m^3, which corresponds to a dry mass of 1500 tonnes per year.

The siltation investigations described above have only considered the potential for transport of seabed material inside Shute Bay and have not considered external sediment sources such as catchment run-off and sediments coming from beyond the bay; however the catchment extent is limited and totally vegetated, and deeper water beyond the bay is generally clear, indicating low sediment transport..

4.4 Siltation During Cyclone Conditions

Cyclone events have the potential to change the morphology of Shute Bay due to a number of factors including sediment influx, storm surge and associated currents and high wave conditions. Section 4.2 presented a hindcast of cyclone wave conditions. A hindcast of potential morphological changes in Shute Bay during tropical cyclones has been undertaken using the track of Tropical Cyclone Celeste. Celeste was an intense cyclone (central pressure = 965 hPa), which generated peak (hindcast) wind speeds up to 34m/s between east and south at Shute Bay.

Figure 4.23 presents a comparison between modelled siltation rates for the existing and post-marina layouts after a severe cyclone event – Cyclone Celeste. In the post-marina case, the most significant morphological change has been the erosion to the south-east of the marina entrance of up to 0.25m and deposition of up to 0.2m in the access channel. An unusual outcome in this case has been the lack of any deposition behind the western breakwater. During storm surge conditions, the storm tide currents and larger waves result in the shear stress behind the western wall being above the critical shear stress for deposition for most of the cyclone event. It is possible that, in such an event, some recently deposited silts adjacent to the western marina wall would be resuspended and distributed across the bay.

5. TIDAL FLUSHING

In order to assess the tidal flushing performance of the proposed marina layout a hydrodynamic simulation for a neap-spring tidal cycle was undertaken using the same D3D model system (in 3D) applied to the morphological and sedimentation assessments. This was undertaken using an inert tracer having no density – simulating a dissolved contaminant. Initially the entire marina was filled with this conservative tracer at concentration 100.

Conservative tracer testing can provide a measure of the flushing rate of a particular section of a water-body (estuary, canal etc). A useful measure for quantification of the flushing time is to determine the e-folding time. The e-folding time is the time taken for the tracer to reach the concentration in Equation 5.1:-

$$C = \frac{C_0}{e} \approx 0.37 \times C_0 \qquad \text{where } C_0 \text{ is the initial concentration} \qquad (5.1)$$

Figure 5.1 shows a time series plot of the tracer concentration at five locations within the marina. E-folding times were calculated as the time taken for the tracer concentration to fall below the level indicated by the solid red line. The approximate e-folding times during a neap-spring tide cycle are presented in Table 5.1 (see Figure 5.1 for locations).

Table 5-1: E-Folding Times for Proposed Marina during a Neap-Spring Tide Cycle (18/01/06-27/01/06)

Location	e-Folding Time (days)
Marina-1	5.5
Marina-2	5.4
Marina-3	5.2
Marina-4	4.6
Marina-5	2.0

With a maximum e-folding time of 5.5 days it can be said that the marina layout is likely to perform well in terms of tidal flushing and is not likely to have any associated water quality issues. This position requires that no contaminants are allowed to enter the marina from boat-waste and stormwater runoff. If there were to be a significant contaminant influx, flushing times of about 2-days may lead to increased concentrations within the marina and eutrophic conditions. It is noted that the Department of Planning and Infra-structure, Western Australia, specify a desirable flushing time of 10 days or less at neap tide and with no wind in their design guidelines, and the proposal more than satisfies this requirement.

6. MAINTENANCE DREDGING REQUIREMENTS

As discussed in Chapter 4, siltation rates over a 10-years period are estimated to be:

- 0.03 m / annum in the eastern end of the marina.
- 0.006 m / annum in the western end of the basin.
- 0.02 m / annum in the access channel.

The maintenance dredging strategy involves use of a Geotube dewatering system that is to be located in the park area on the western side of the western revetment wall (refer the Master Plan in Appendix A)

Dredging campaigns would be conducted every 2 to 3 years over a 2 to 3 months period. Once de-watered, the dry silt would be removed from site.

Approximately 6000m^3 of in-place settled sediment would be removed on each campaign, from the marina basin.

7. MONITORING

To ensure that siltation predictions are reliable and to enable the dredging strategy to be adjusted to suit actual conditions, it is recommended that settlement boxes be installed after construction in the following locations:-

- Eastern end of the marina.
- Central marina.
- Western end of marina.
- three or four other distributed marina locations.
- In three or four locations west of the western breakwater.
- At two locations in the access channel.

Annual diver measurement of siltation rates would then be carried out to allow an on-going assessment of dredging requirements. If necessary, dredging frequency would be varied to suit the nominated disposal strategy.

Settlement boxes have been successfully developed, tested and implemented by Cardno Lawson Treloar in Port Phillip Bay as part of The Rip dredging program.

8. CONCLUSIONS

Based on detailed analysis of coastal processes, we conclude that in relation to physical processes, the proposed development is feasible and manageable with regard to engineering standards and safety requirements and marina operational requirements, including maintenance dredging. The project has been found to be adequate to withstand severe cyclonic conditions with appropriate reclamation and habitable floor levels provided.

Predicted siltation rates are modest and manageable, at 3000 m^3/annum.

The marina will remain well flushed, with flushing times well within recognised guidelines.

Some accretion of sediment is predicted immediately in the lee of the western breakwater, though annual accretion rates are low.

Scouring along the southern bay is also predicted, again at a low annual rate.

9. QUALIFICATIONS

The analyses and modelling results presented in this report are subject to a number of qualifications, including:-

Bathymetric data for Shute Bay has been obtained from available navy charts and hydro survey data. The hydro survey is generally limited to the north-east area of Shute Bay and was provided by the Client . The resolution of the hydrographic charts is coarse in the inter-tidal areas.

Data available from FRC Environmental and GHD has been used to characterize the spatial seabed sediment variation in the study area. As a result of the coarse resolution of the sediment data, a broad sediment description has been applied in the Delft3D model for the whole of Shute Bay.

Sediment loads which may originate from outside Shute Bay or from the catchment have not been considered in this study.

Appropriate sediment parameters, for example shear stresses for deposition and erosion, have been based on available literature for similar sediment. The actual sediment properties at Shute Bay may vary from these parameter values.

Wind conditions have been based on observed wind conditions at Hamilton Island airport. Actual wind conditions at Shute Harbour may vary from these observations. That was the closest available site.

Typical 5 and 10-years scenarios are based on the long-term wind conditions at the site (measured at Hamilton Island). Actual siltation patterns may vary from the modelling results if the climate conditions vary from the long-term average conditions analysed at Hamilton Island.

There is insufficient historical survey data to spatially verify existing siltation and erosion rates and patterns inside Shute Bay; hence, future observed siltation and erosion patterns may vary from those predicted.

It has not been possible to verify wave modelling undertaken as part of this study. However the SWAN wave model is recognized as an advanced wave generation and propagation model. Cardno Lawson Treloar have verified the model system to measured data at numerous sites including Botany Bay and Shellcove (NSW).

Wave forces calculated for the proposed walls assume significant horizontal load distribution from one panel to another and that vertical faces on the walls will be rough to reduce impulsive loads

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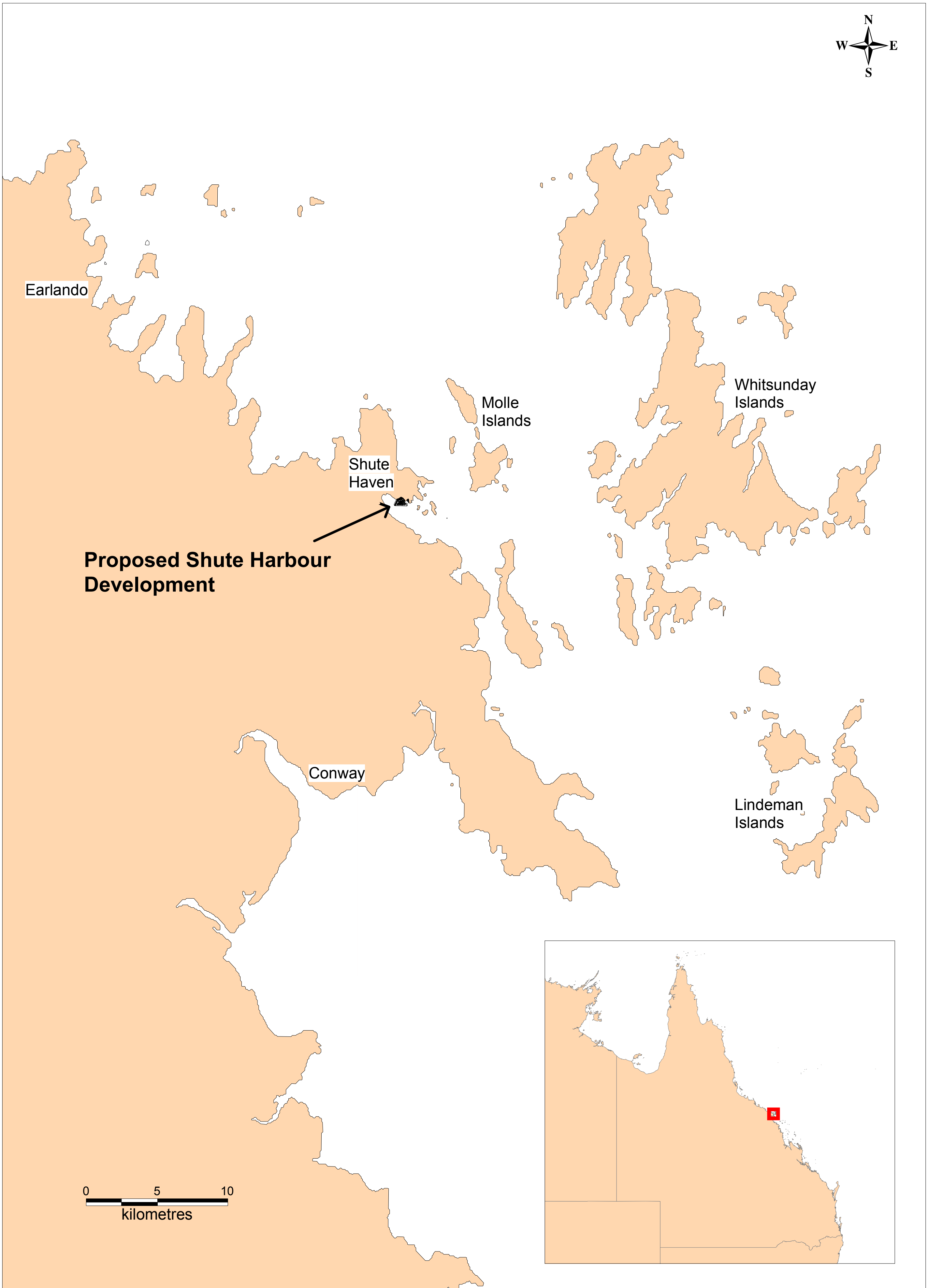
FIGURES



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Shute Harbour Marina Development - Coastal Process Issues

LOCALITY PLAN

Figure 1.1

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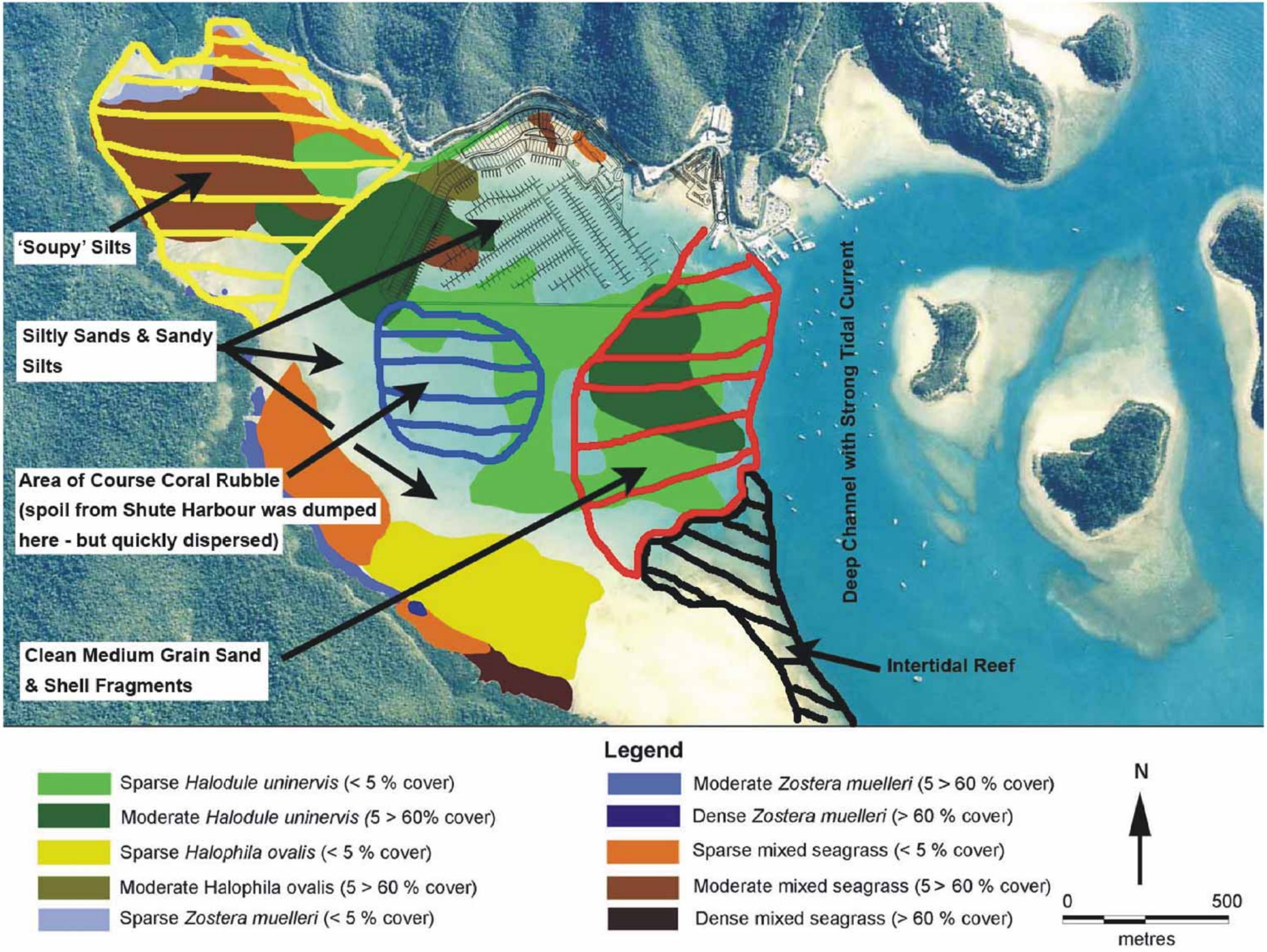
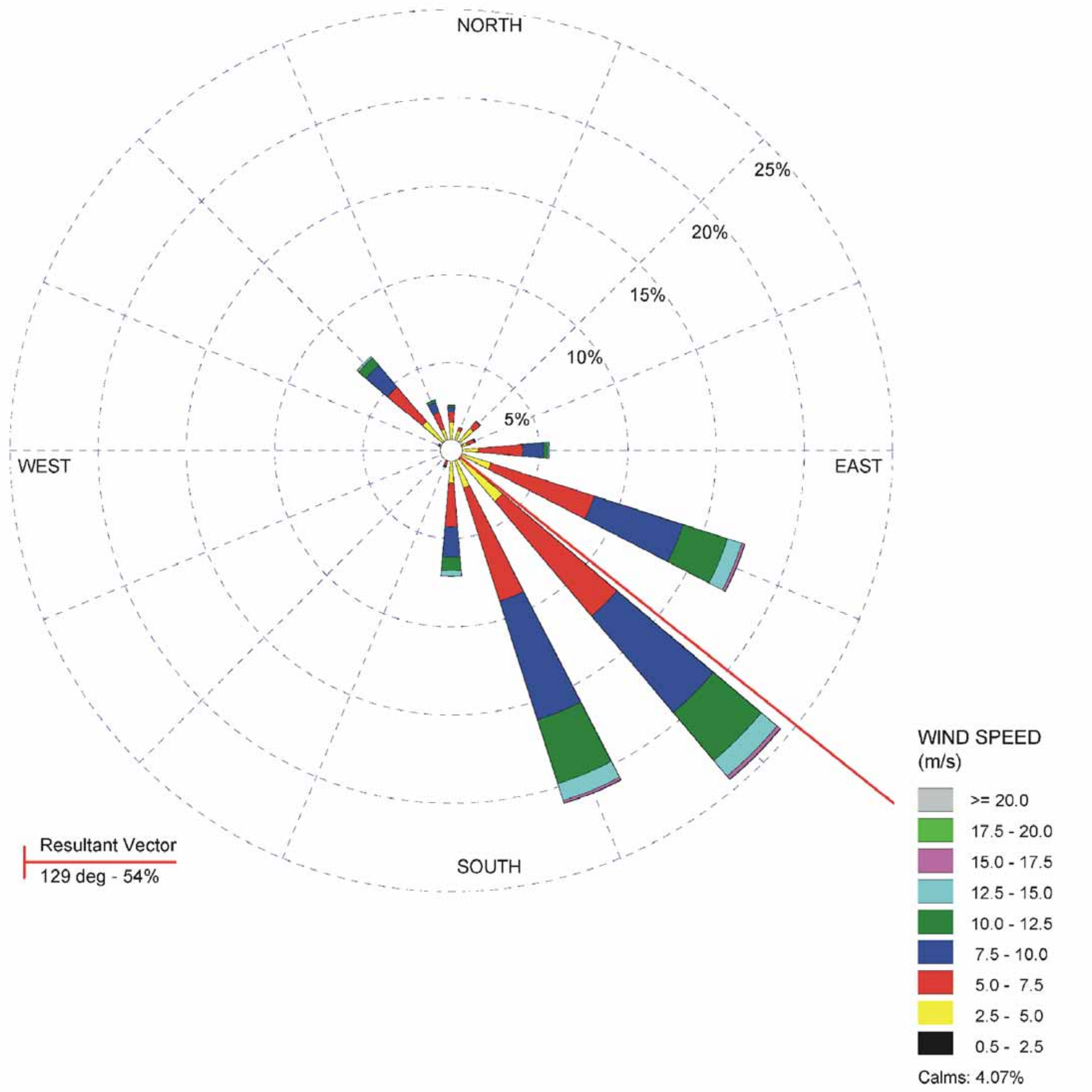


Shute Harbour Marina Development - Coastal Process Issues
SHUTE BAY SEABED SEDIMENTS
frc environmental
Figure 2.1



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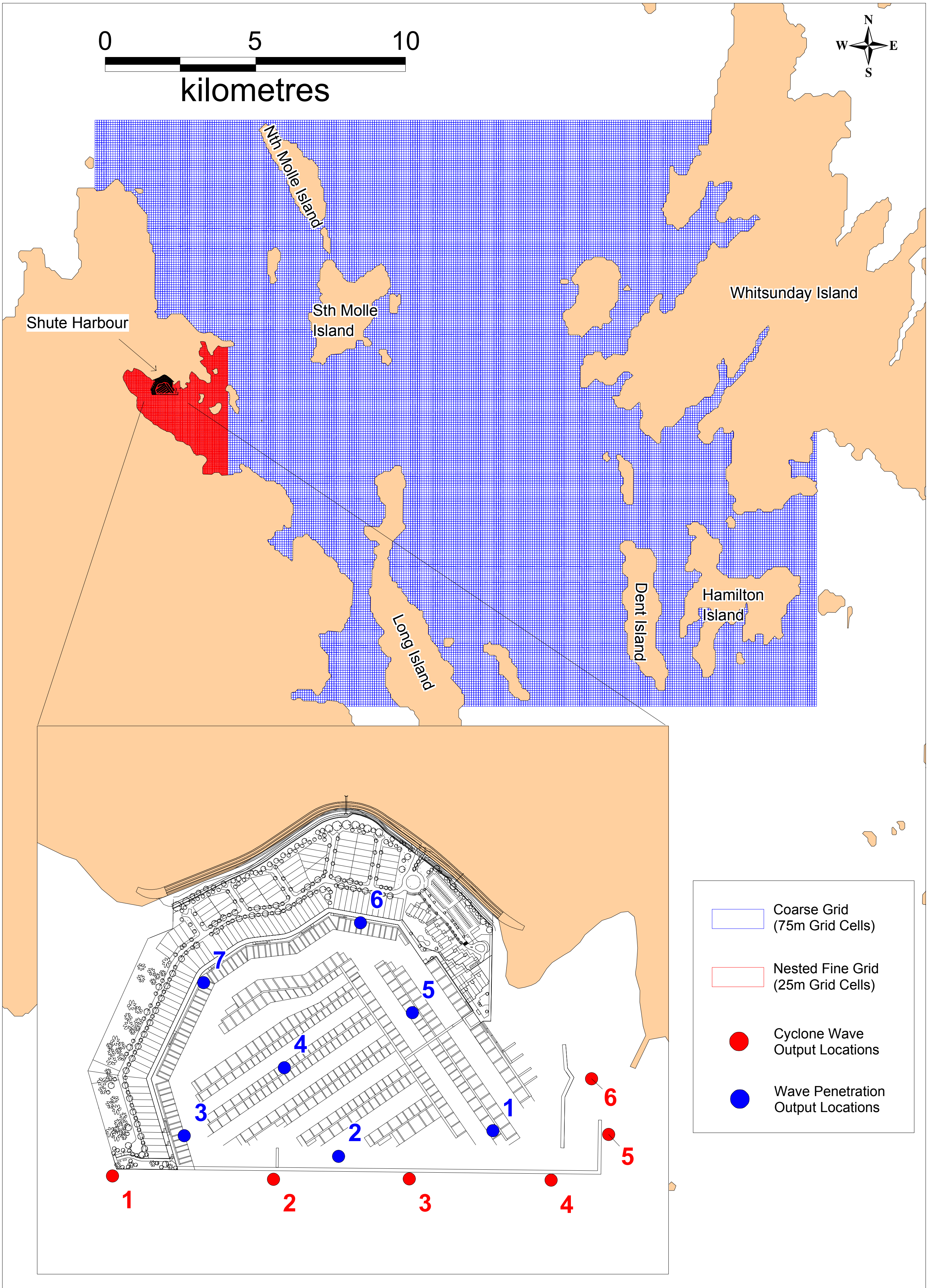
Shute Harbour Marina Development - Coastal Process Issues
HAMILTON ISLAND AIRPORT WIND ROSE
WIND SPEED - DIRECTION (BLOWING FROM)
Figure 2.2



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Shute Harbour Marina Development - Coastal Process Issues

SWAN MODEL GRIDS AND OUTPUT LOCATIONS

Figure 3.1

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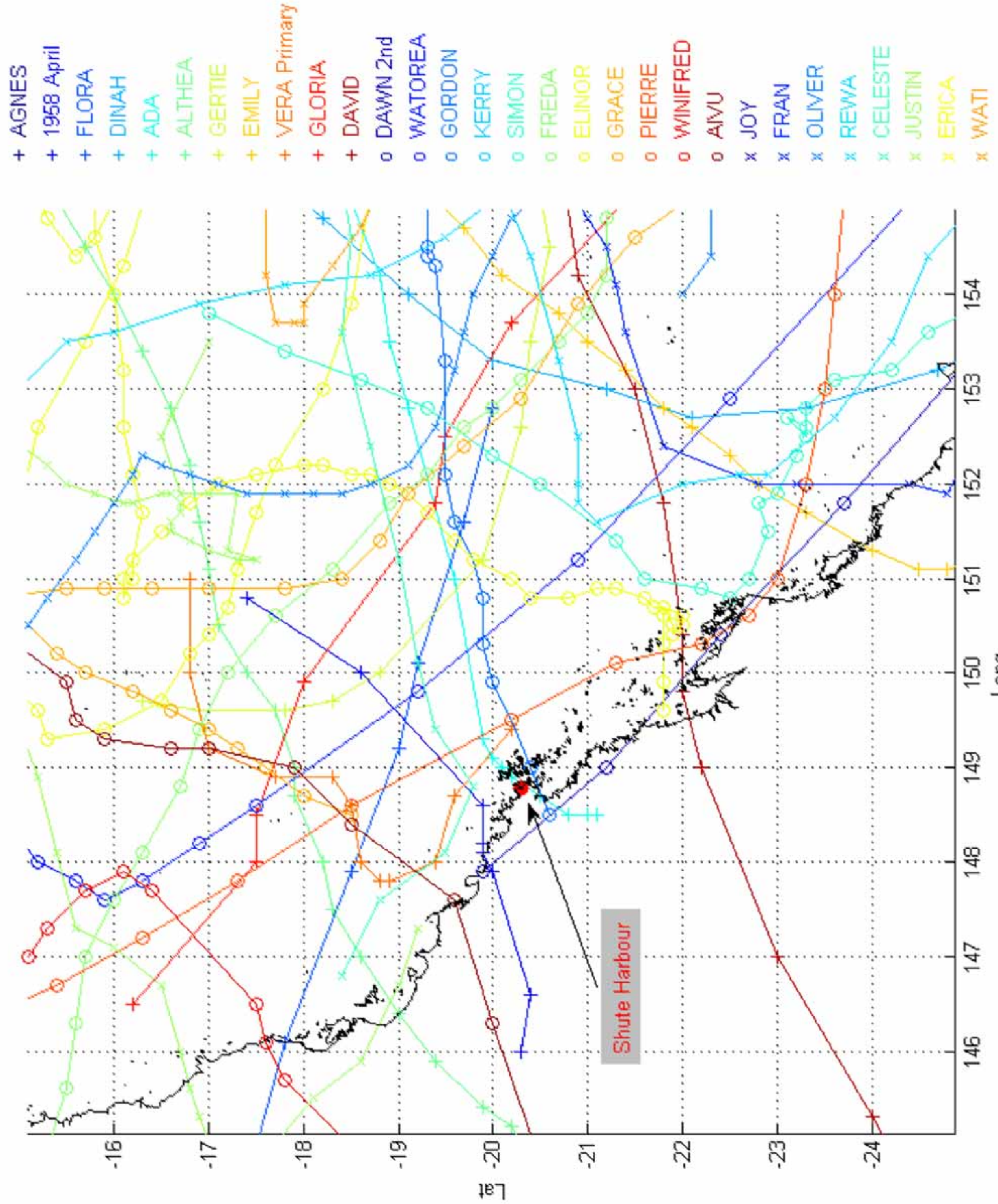

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Shute Harbour Marina Development - Coastal Process Issues
HISTORICAL CYCLONE TRACKS

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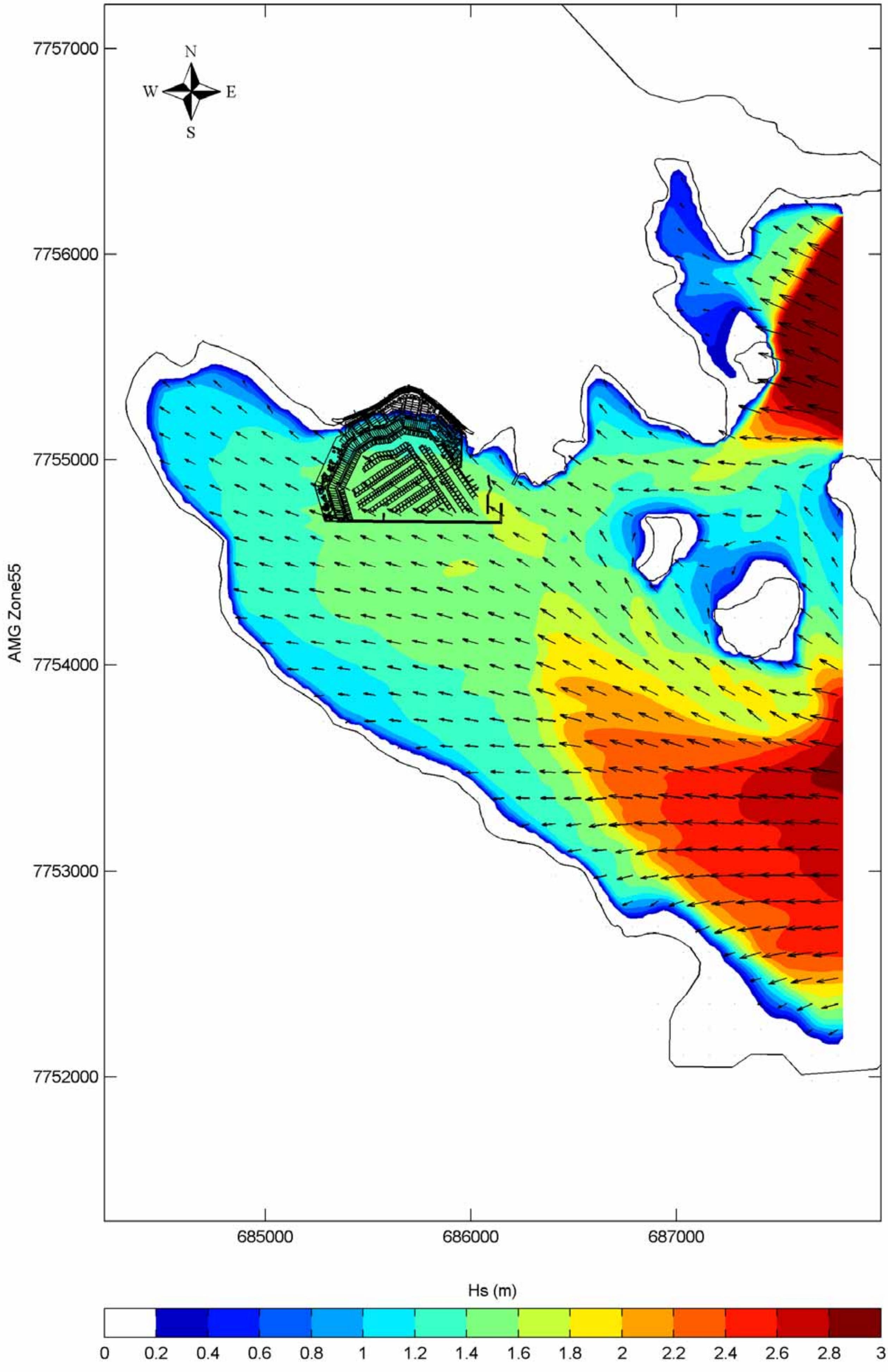
Figure 3.2



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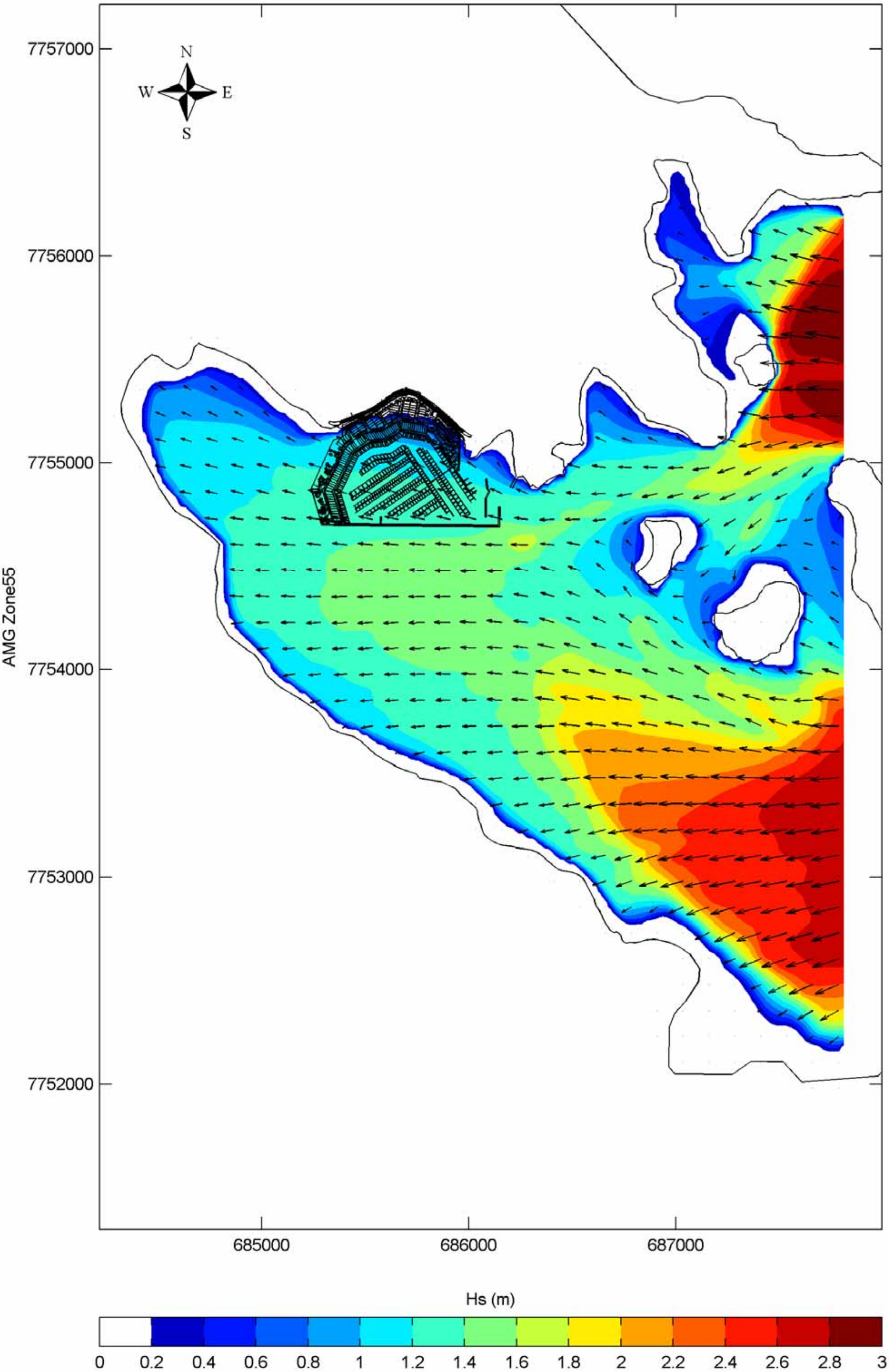
Shute Harbour Marina Development - Coastal Processes Issues

SIGNIFICANT WAVE HEIGHT (m)

CYCLONE Celeste 27/01/1996 06:00

Figure 3.3



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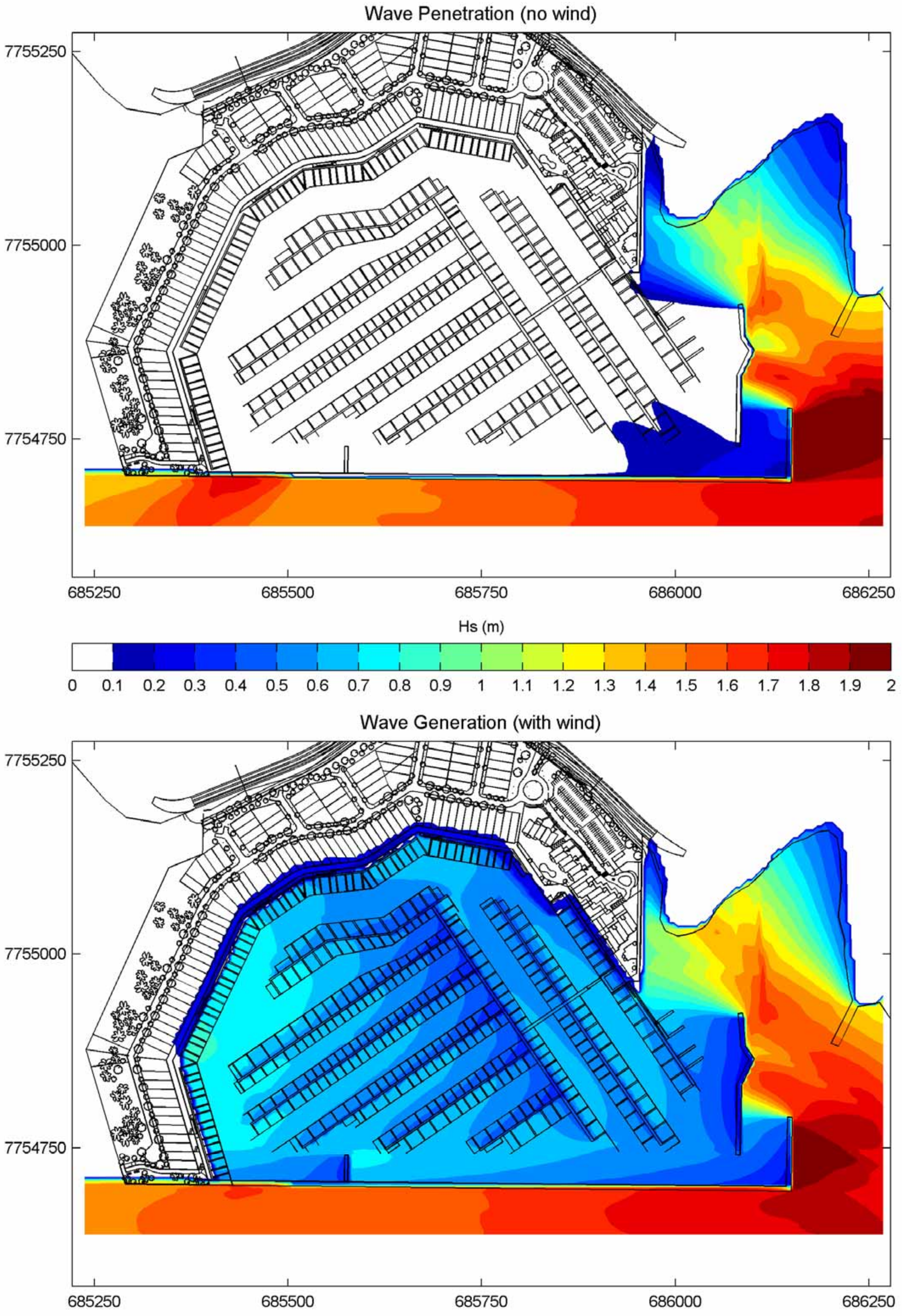
Shute Harbour Marina Development - Coastal Processes Issues

SIGNIFICANT WAVE HEIGHT (m)

CYCLONE Apr 1958 01/04/1958 08:00

Figure 3.4

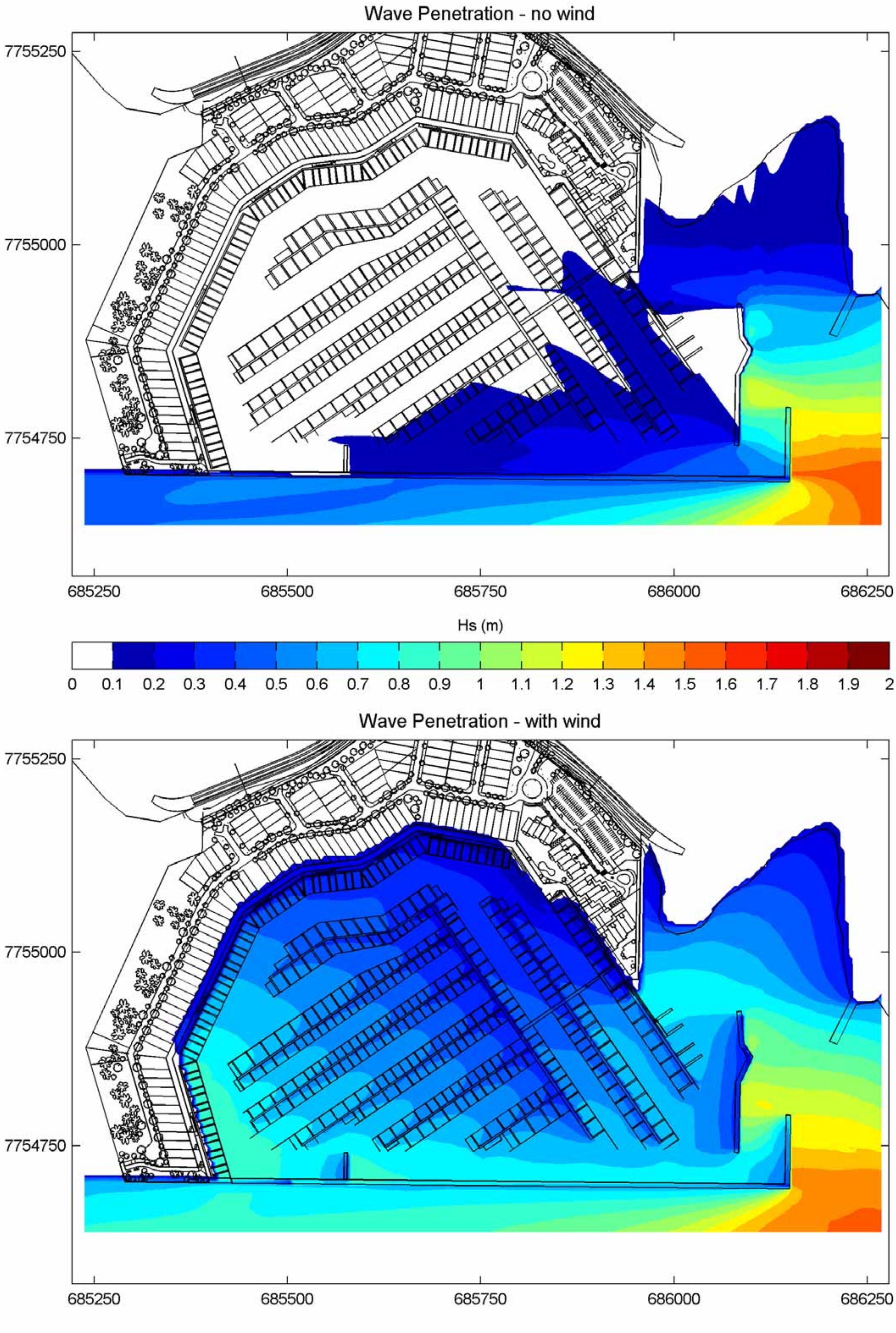


Shute Harbour Marina Development - Coastal Processes Issues

WAVE PENETRATION INVESTIGATION

CYCLONE CELESTE (27/1/1996 06:00)

Figure 3.5



Cardno Lawson Treloar

LJ8779/R1 November 2007

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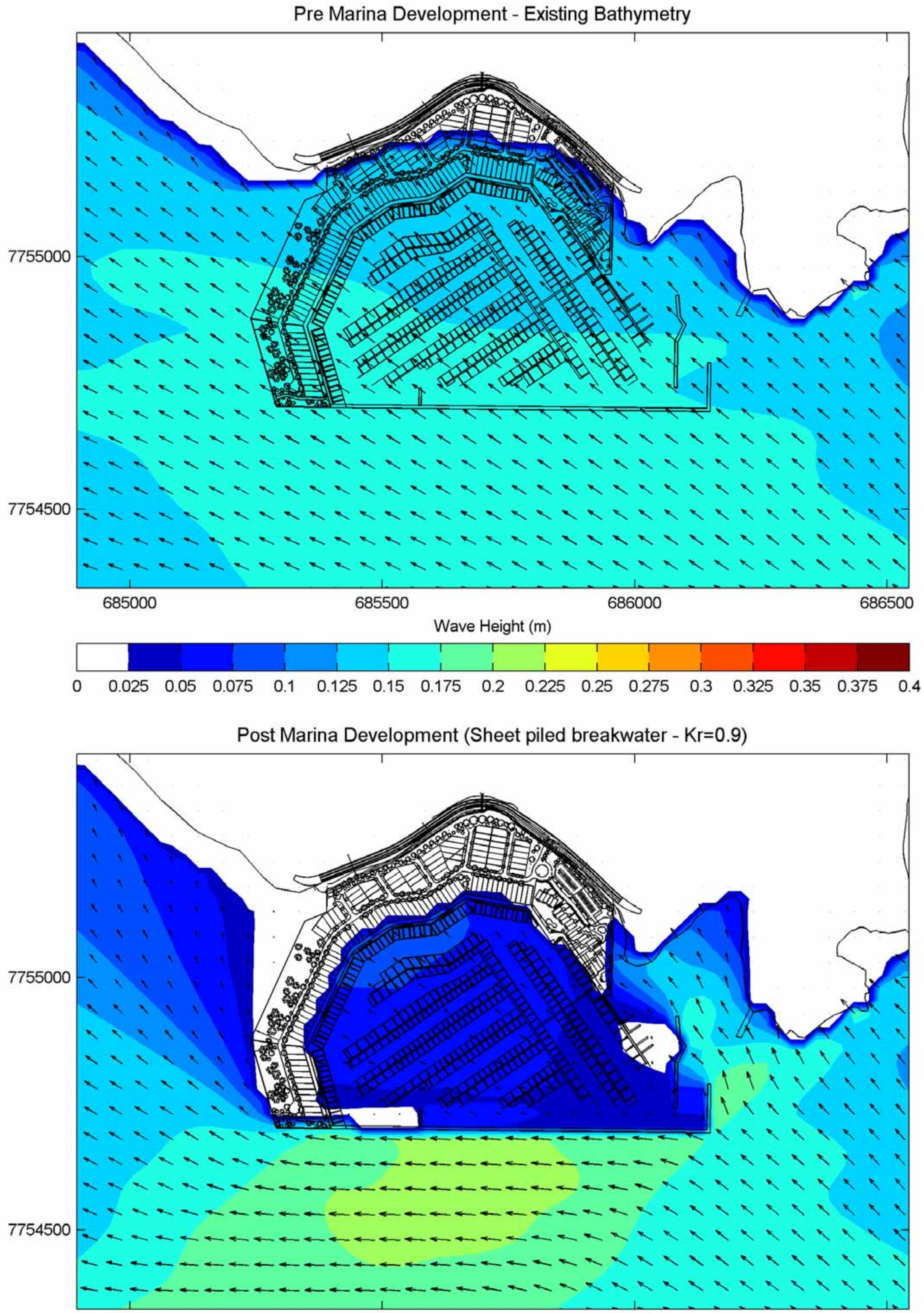
Shute Harbour Marina Development - Coastal Processes Issues

WAVE PENETRATION INVESTIGATION

CYCLONE AGNES (05/03/1956 18:00)

Figure 3.6



Cardno Lawson Treloar

LJ8779/R1 November 2007

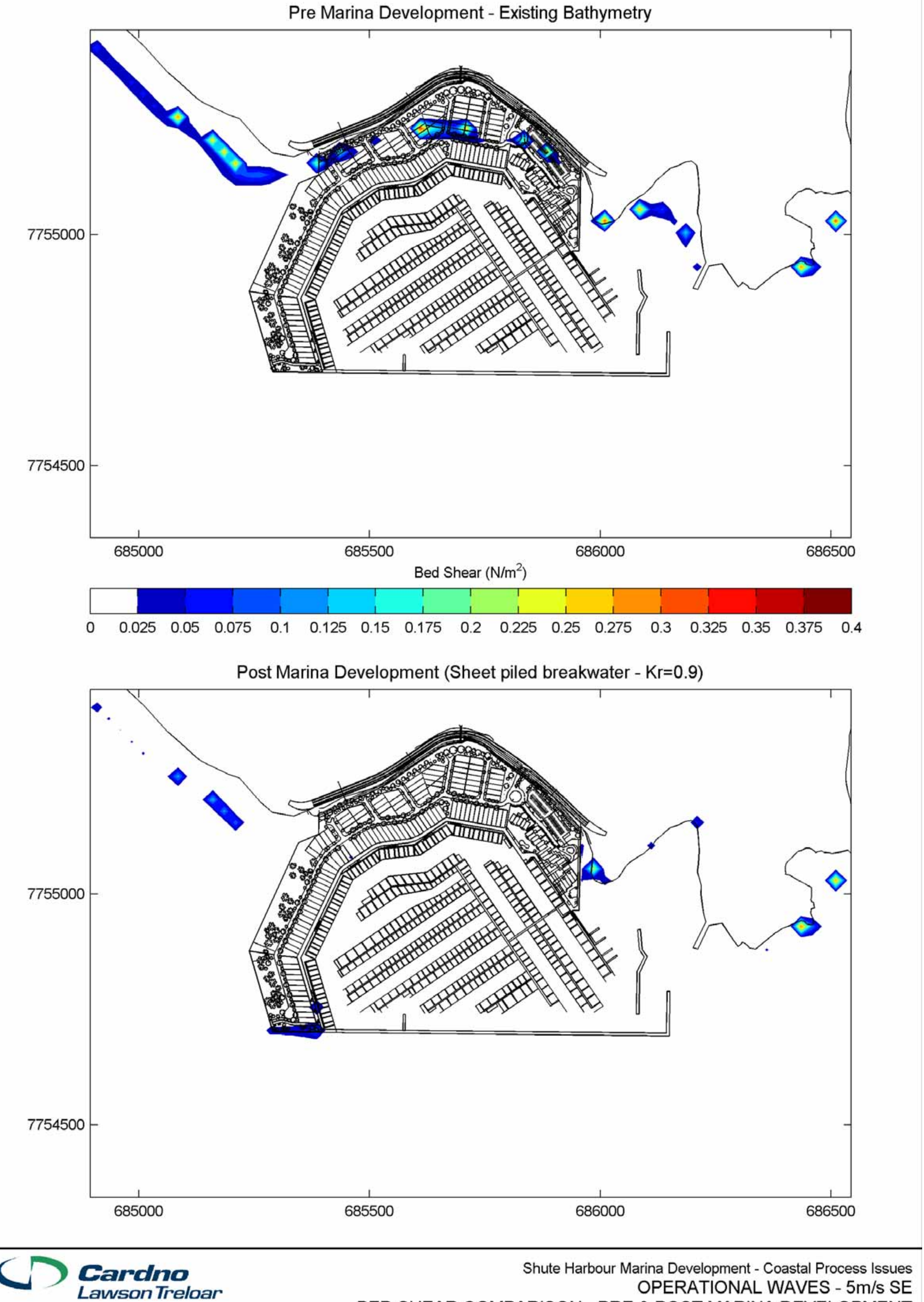
File: C0266\D:\jobsd3d\790048-ShuteHarbour\bed_shear\5mps\wave_height.mup

Shute Harbour Marina Development - Coastal Process Issues

OPERATIONAL WAVES - 5m/s SE

WAVE HEIGHT COMPARISON - PRE & POST MARINA DEVELOPMENT

Figure 3.7



Cardno Lawson Treloar

LJ8779/R1 November 2007

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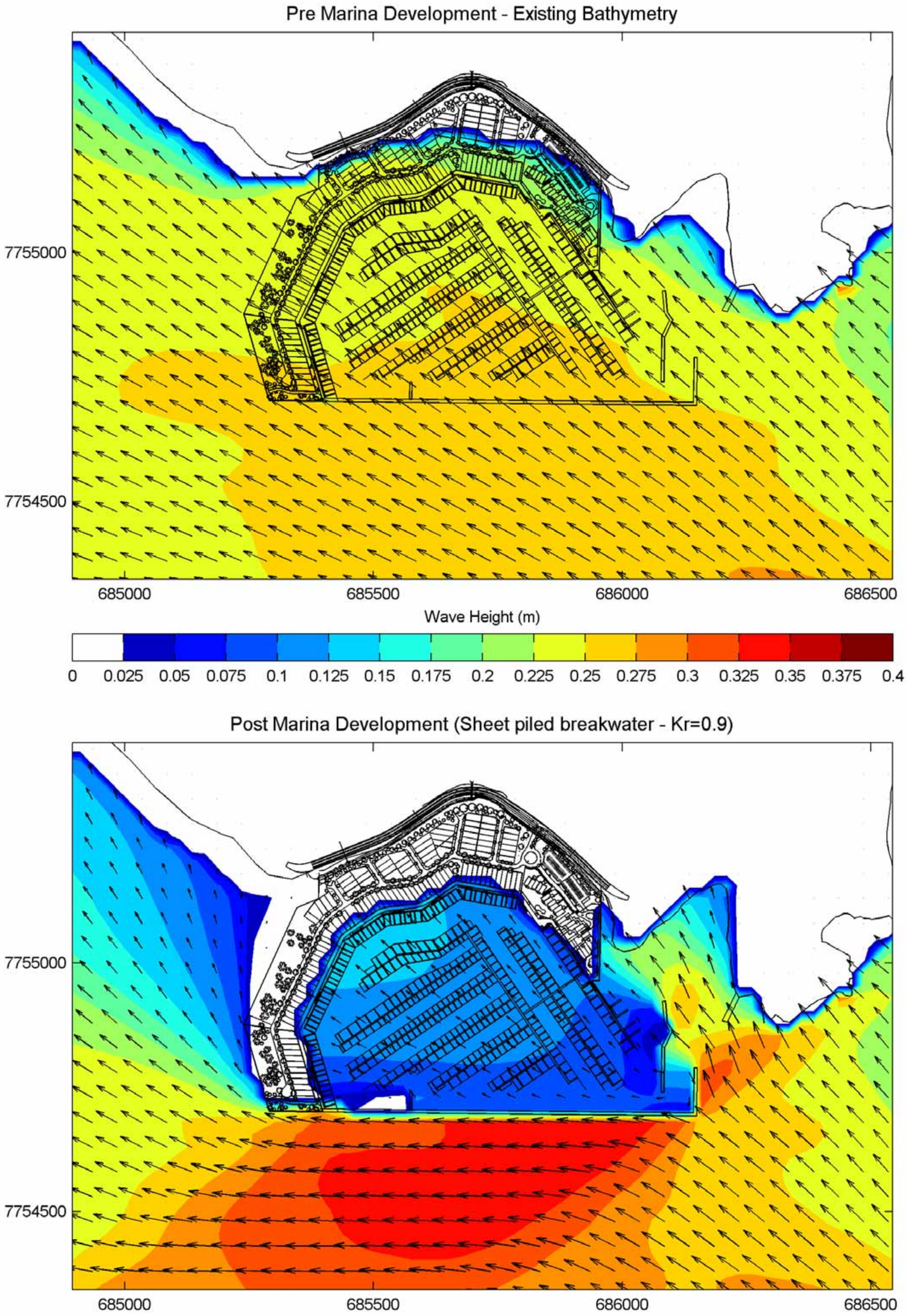
Shute Harbour Marina Development - Coastal Process Issues

OPERATIONAL WAVES - 5m/s SE

BED SHEAR COMPARISON - PRE & POST MARINA DEVELOPMENT

Figure 3.8



Cardno Lawson Treloar

LJ8779/R1 November 2007

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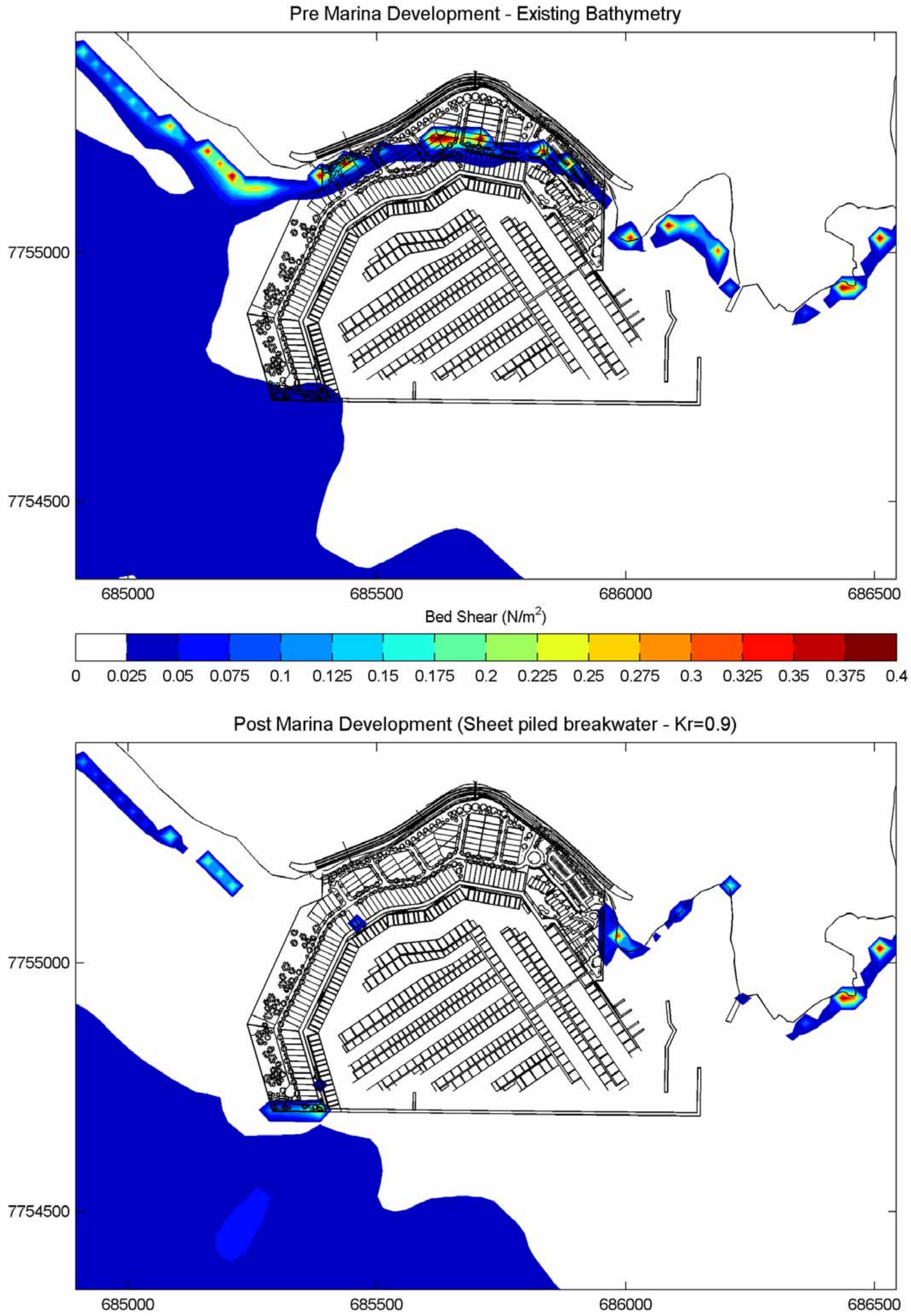
Shute Harbour Marina Development - Coastal Process Issues

OPERATIONAL WAVES - 7.5m/s SE

WAVE HEIGHT COMPARISON - PRE & POST MARINA DEVELOPMENT

Figure 3.9



Cardno Lawson Treloar

LJ8779/R1 November 2007

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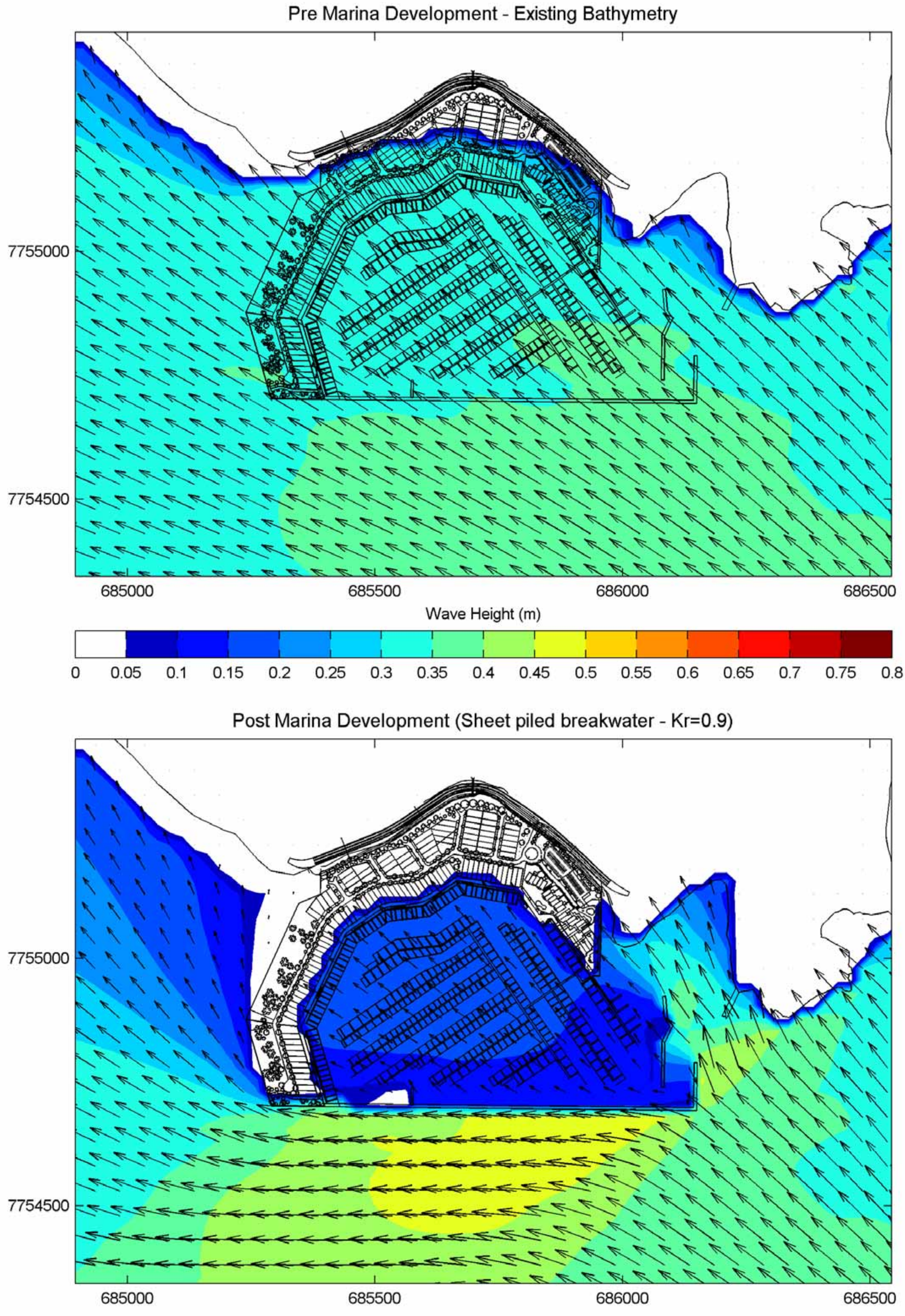
Shute Harbour Marina Development - Coastal Process Issues

OPERATIONAL WAVES - 7.5m/s SE

BED SHEAR COMPARISON - PRE & POST MARINA DEVELOPMENT

Figure 3.10



Cardno Lawson Treloar

LJ8779/R1 November 2007

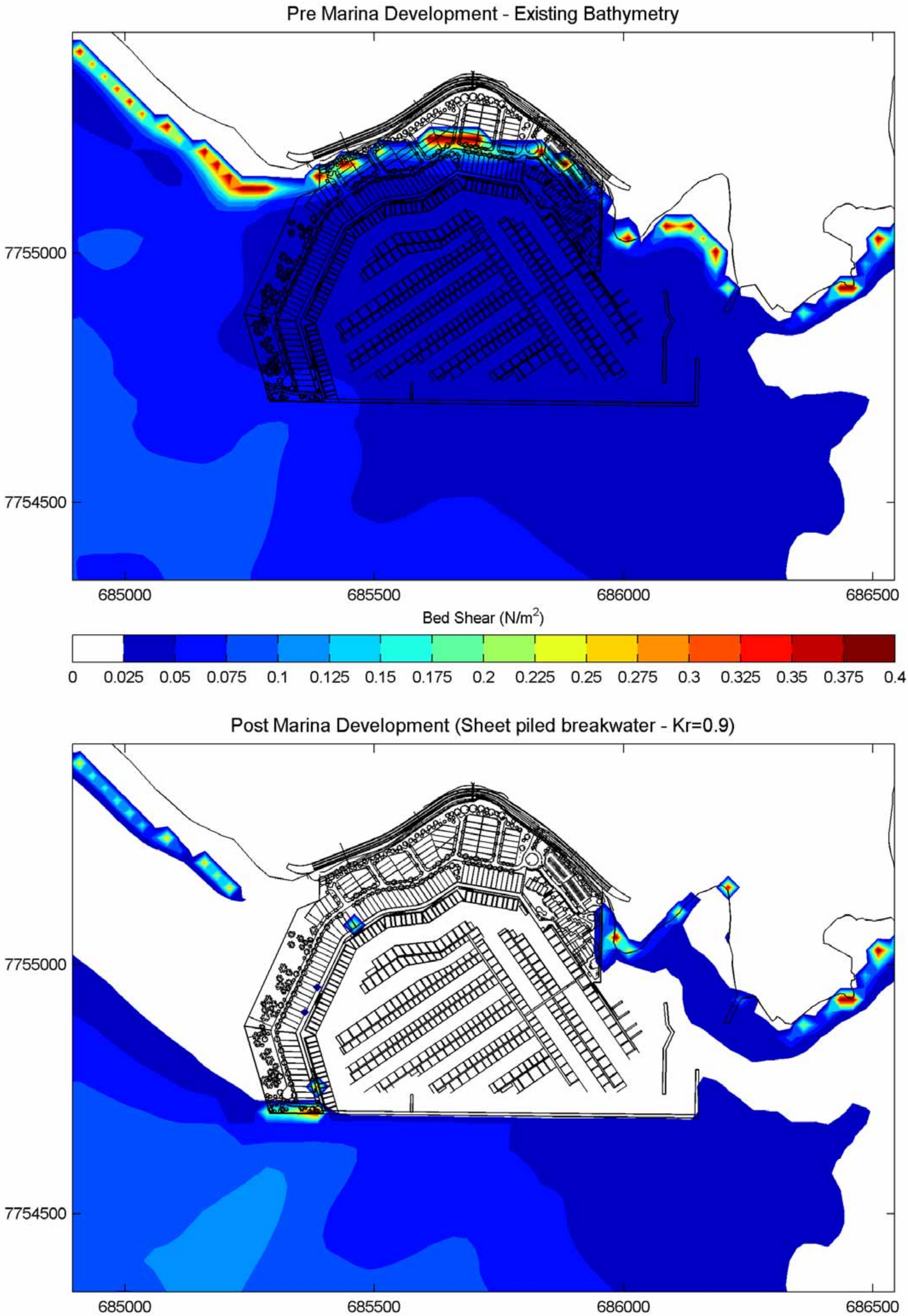
File: C0266\D:\jobsd3d\790048-ShuteHarbour\bed_shear\10mps\wave_height.mup

Shute Harbour Marina Development - Coastal Process Issues

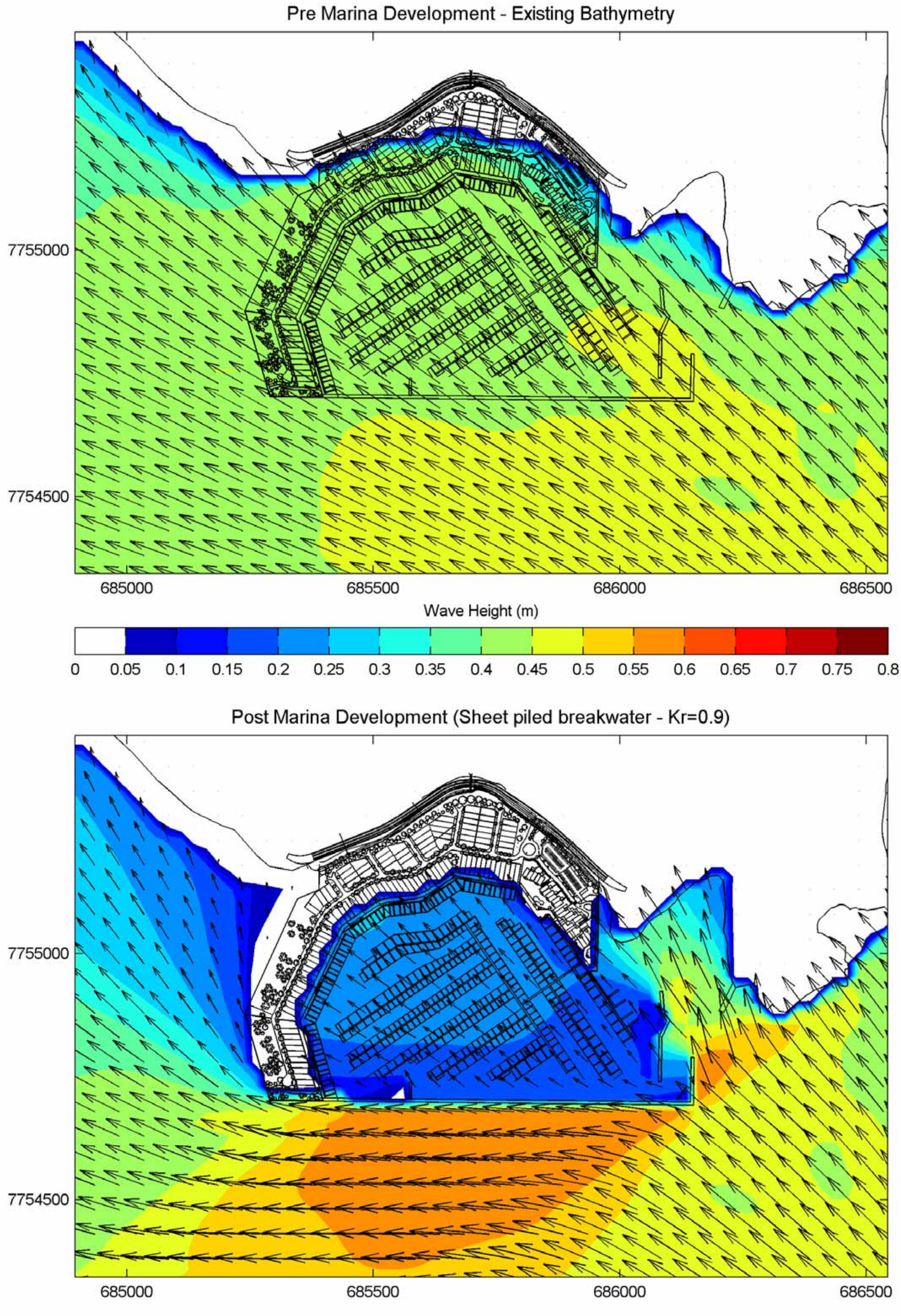
OPERATIONAL WAVES - 10m/s SE

WAVE HEIGHT COMPARISON - PRE & POST MARINA DEVELOPMENT

Figure 3.11



Shute Harbour Marina Development - Coastal Process Issues
OPERATIONAL WAVES - 10m/s SE
BED SHEAR COMPARISON - PRE & POST MARINA DEVELOPMENT
Figure 3.12



Cardno Lawson Treloar

LJ8779/R1 November 2007

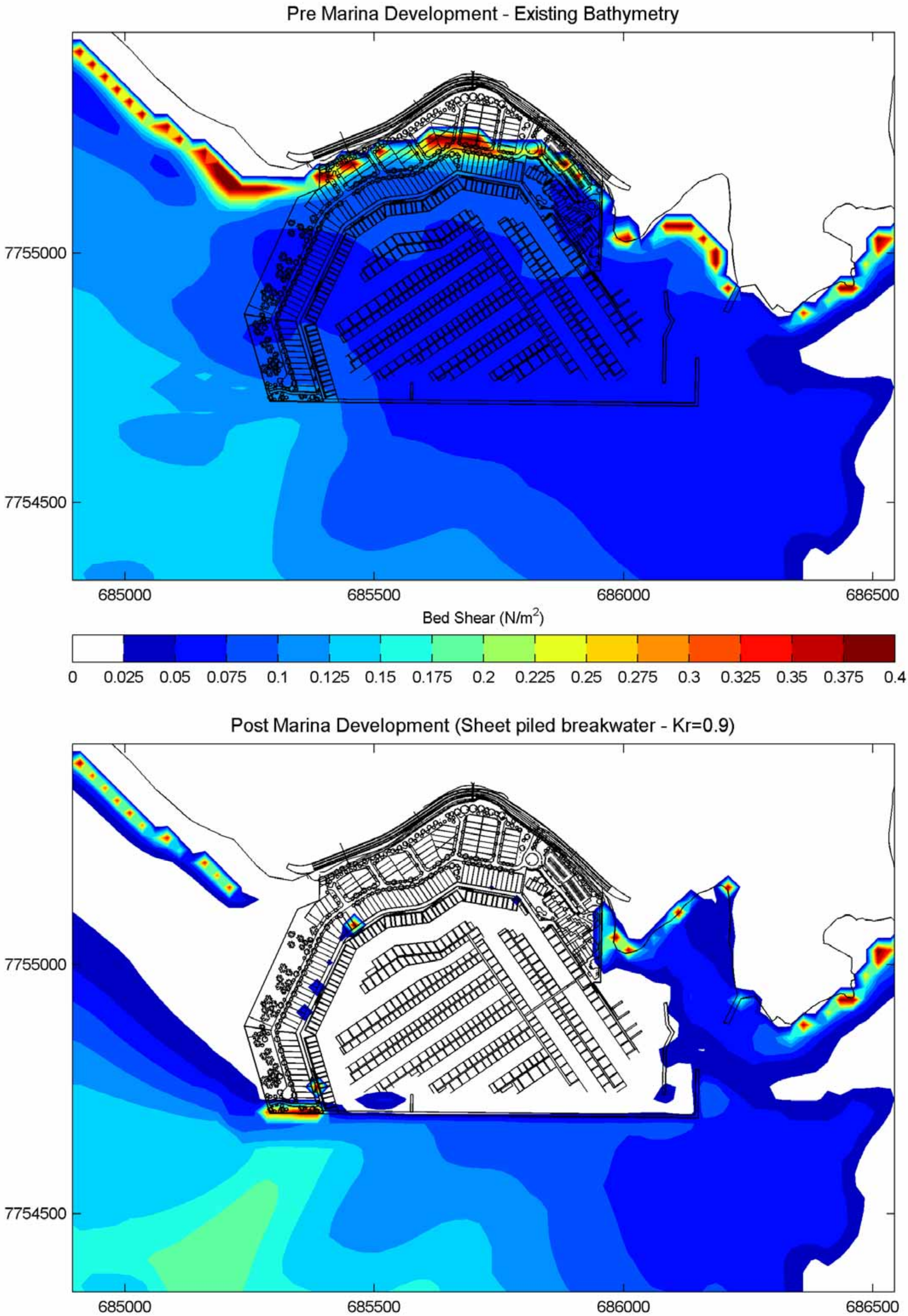
File: C0266\D:\jobsd3d\790048-ShuteHarbour\bed_shear\12.5mps\wave_height.mup

Shute Harbour Marina Development - Coastal Process Issues

OPERATIONAL WAVES - 12.5m/s SE

WAVE HEIGHT COMPARISON - PRE & POST MARINA DEVELOPMENT

Figure 3.13



Cardno Lawson Treloar

LJ8779/R1 November 2007

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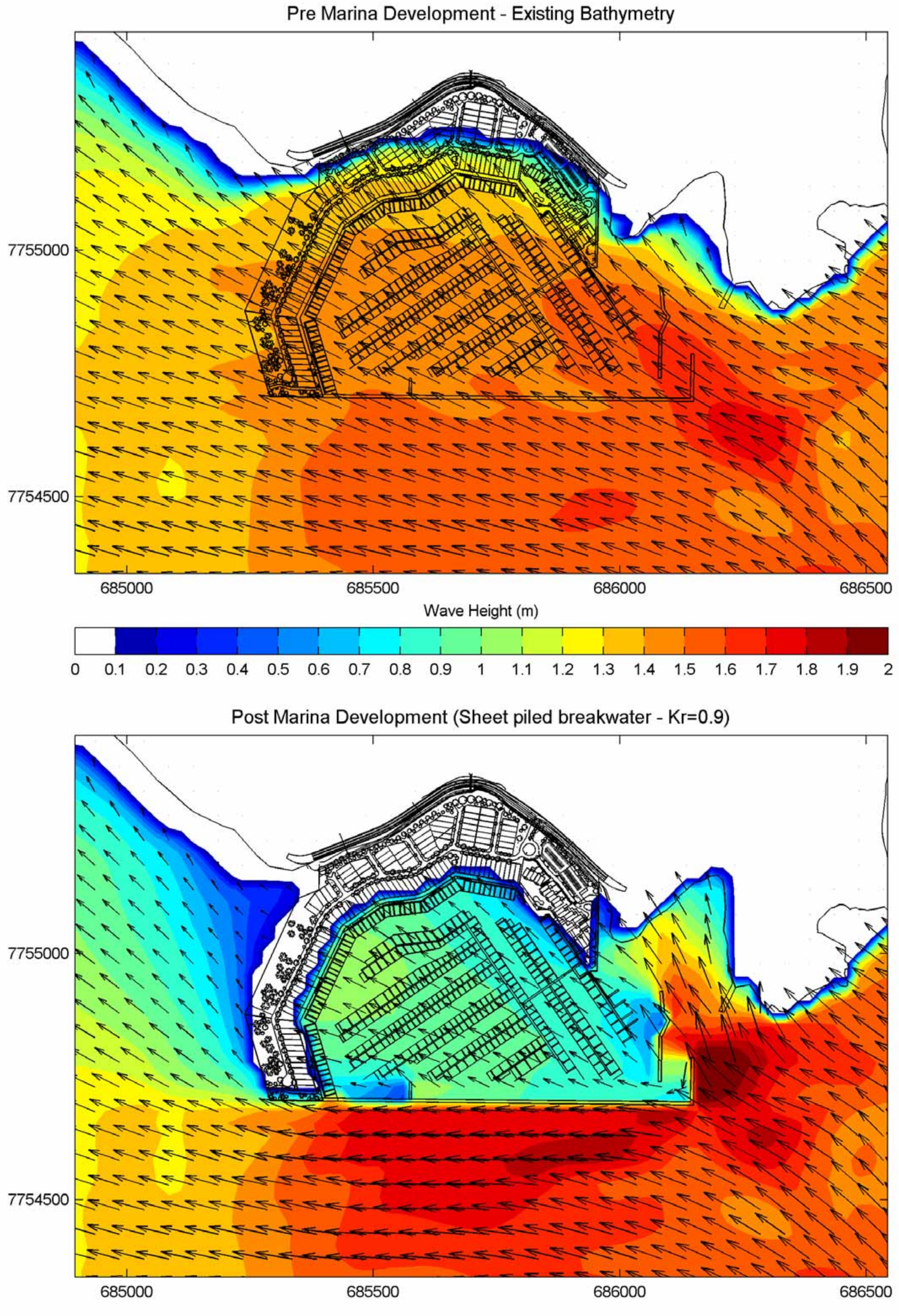
Shute Harbour Marina Development - Coastal Process Issues

OPERATIONAL WAVES - 12.5m/s SE

BED SHEAR COMPARISON - PRE & POST MARINA DEVELOPMENT

Figure 3.14



Cardno Lawson Treloar

LJ8779/R1 November 2007

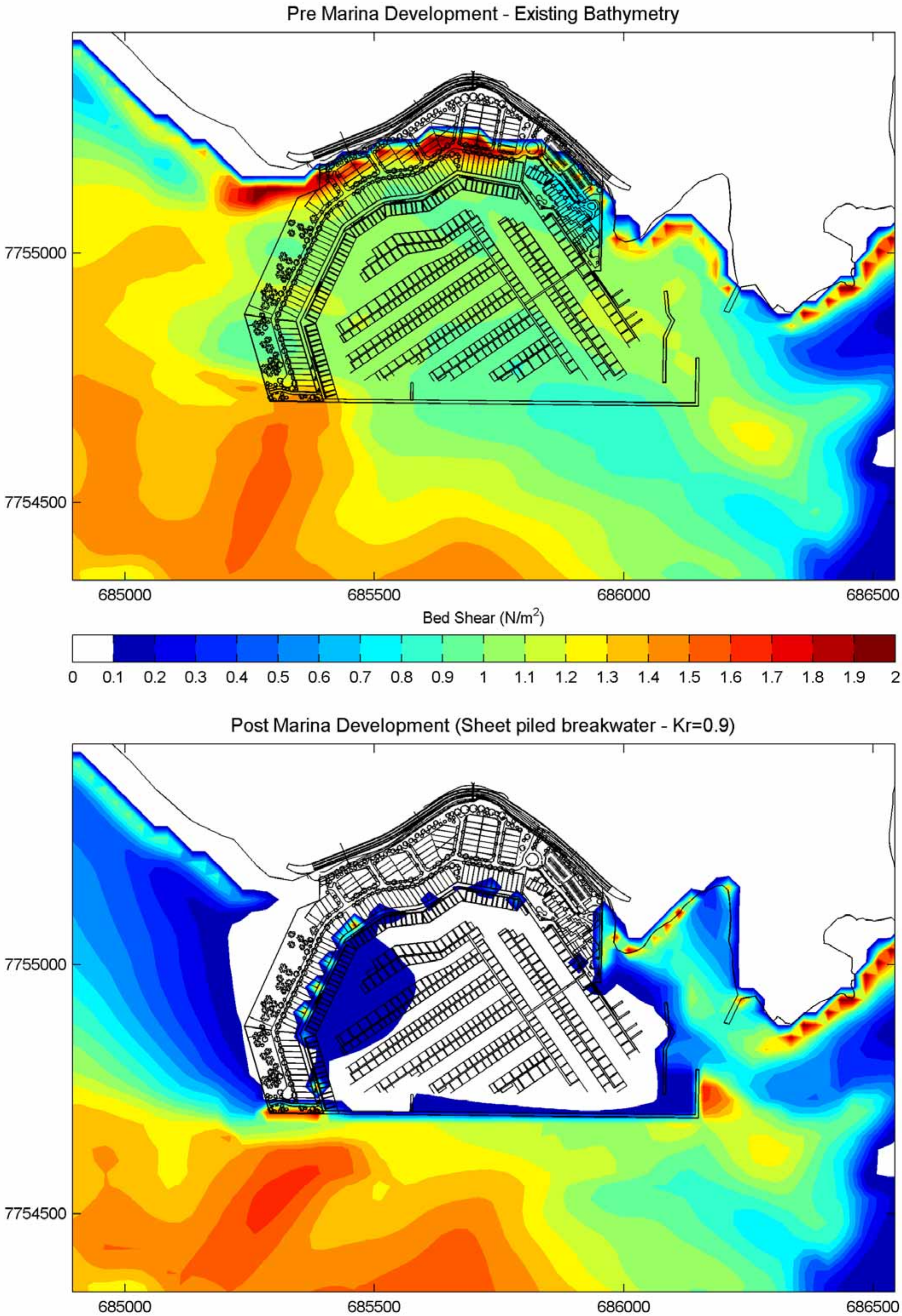
File: C0266\D:\jobsd3d\790048-ShuteHarbour\SWAN\Cyclone_Runs\SWAN\Ce2_hw\breakwater\wave_height.mup

Shute Harbour Marina Development - Coastal Processes Issues

WAVE HEIGHT COMPARISON - PRE & POST MARINA DEVELOPMENT

CYCLONE CELESTE (27/01/1996 06:00)

Figure 3.15



Cardno Lawson Treloar

Shute Harbour Marina Development - Coastal Processes Issues

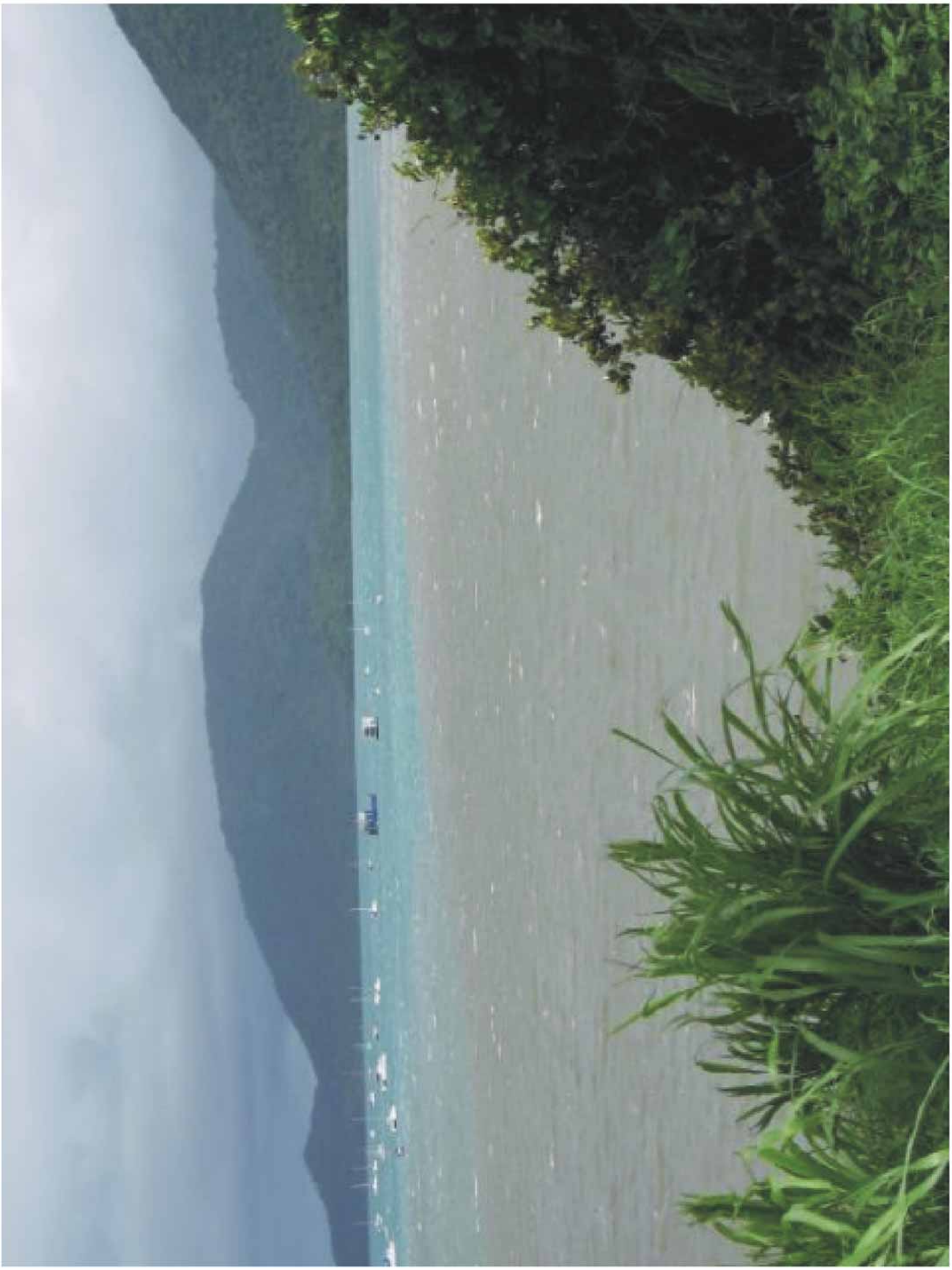
BED SHEAR COMPARISON - PRE & POST MARINA DEVELOPMENT

CYCLONE CELESTE (27/01/1996 06:00)

Figure 3.16

LJ8779/R1 November 2007

File: C0266\D:\jobsd3d\790048-ShuteHarbour\SWAN\Cyclone_Runs\SWAN\Ce2_hw\breakwater\bed_shear.mup



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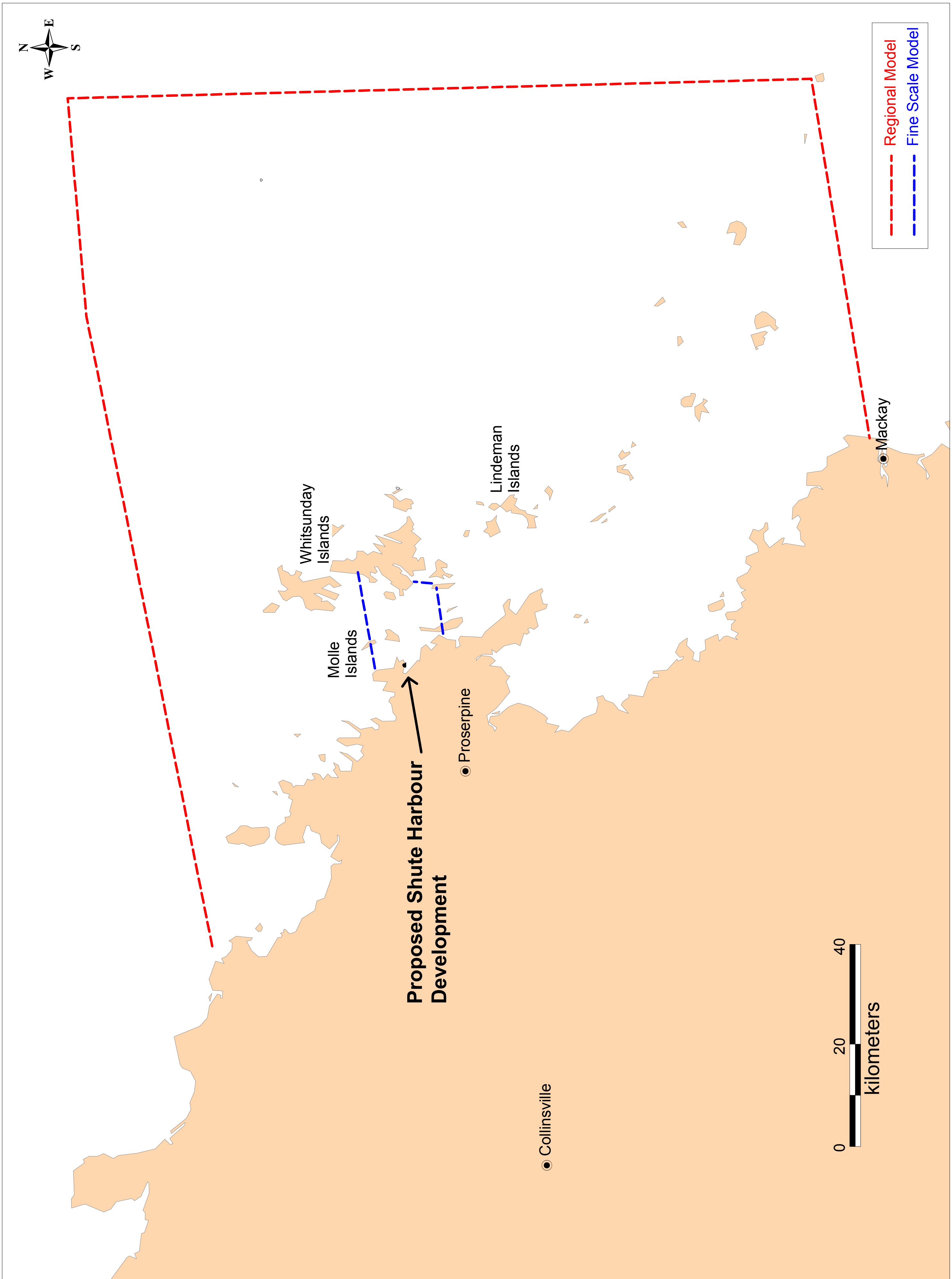
Shute Harbour Marina Development - Coastal Process Issues

NEARSHORE SUSPENDED SEDIMENT PLUMES UNDER LOW SEA CONDITIONS FEBRUARY 2007

Figure 4.1

LJ8779/R1 November 2007

File: J:\CM\790048-Shute\Figures\LJ8779\Fig4.1_Sus_Sed.cdr



Cardno Lawson Treloar

Shute Harbour Marina Development - Coastal Processes Issues

DELFT3D MODEL EXTENTS

LJ8779/R1 November 2007
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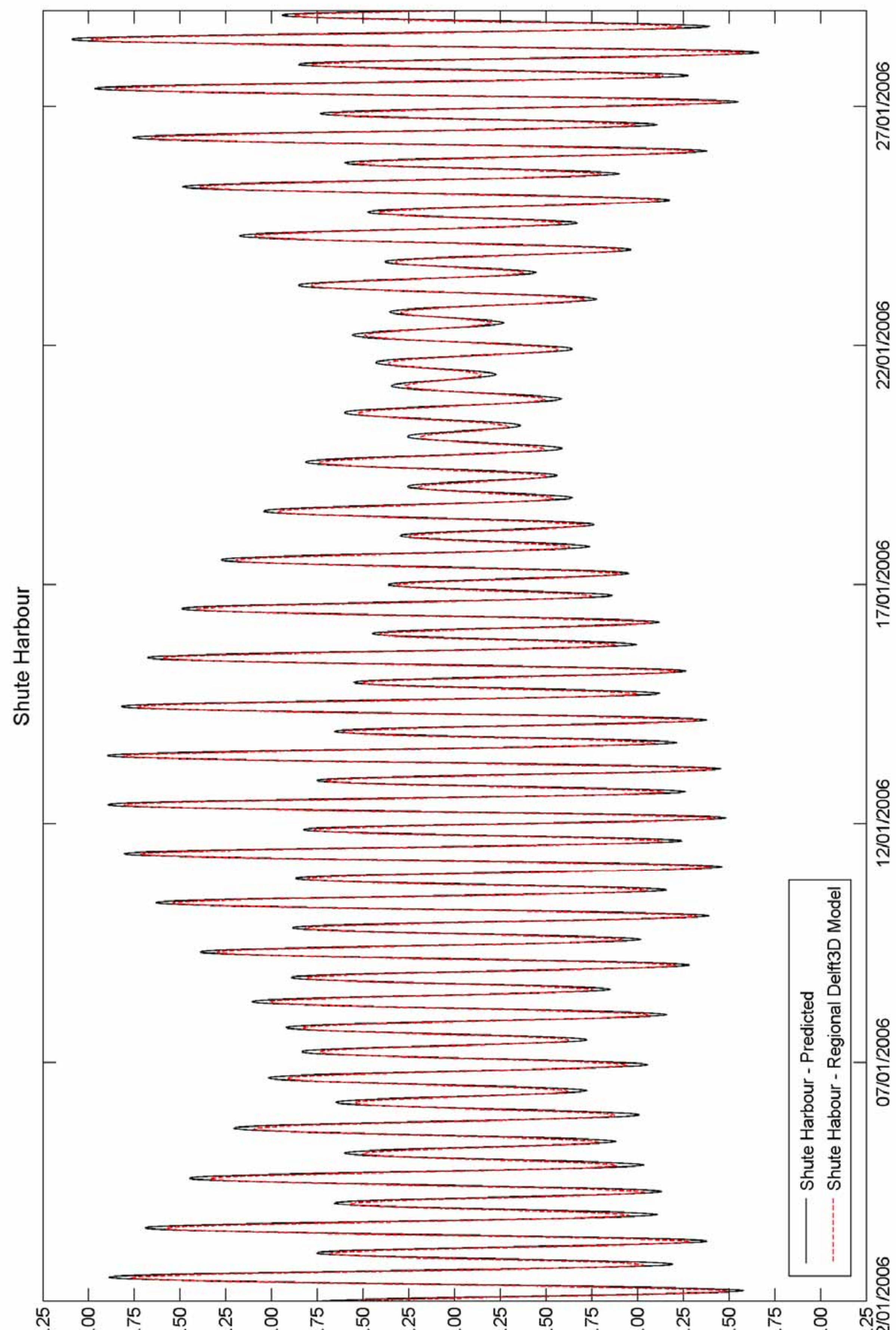
Figure 4.2



Cardno Lawson Treloar

LJ8779/R1 November 2007

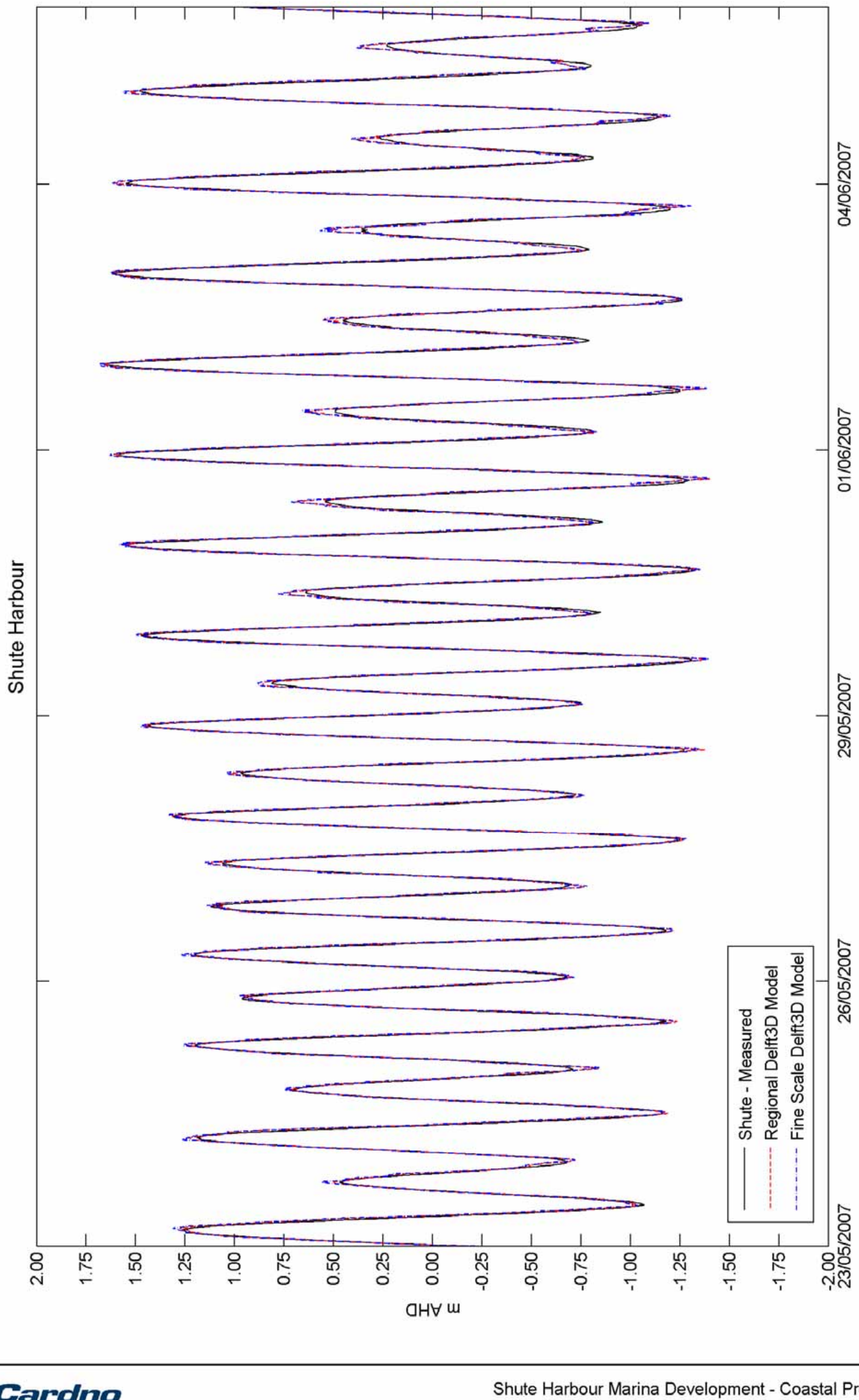
File: J:/CM/790048-Shute/Figures/R2344/Figure4.3.png

Shute Harbour Marina Development - Coastal Processes Issuse

REGIONAL HYDRODYNAMIC MODEL VERIFICATION

WATER LEVEL - JANUARY 2006

Figure 4.3



Cardno Lawson Treloar

LJ8779/R1 November 2007

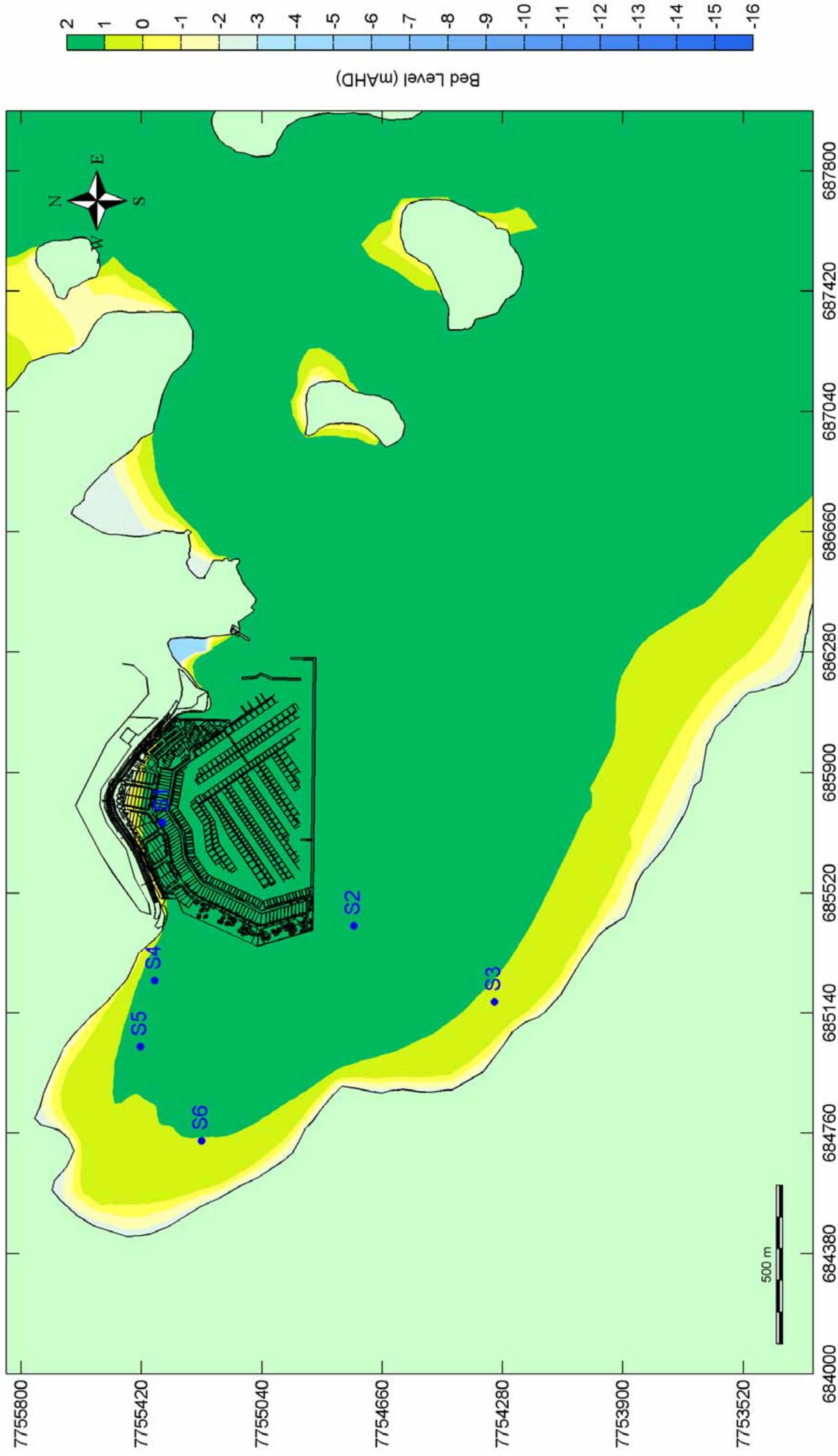
File: J:/CM/790048-Shute/Figures/R2344/Figure4.3.png

Shute Harbour Marina Development - Coastal Processes Issuse

REGIONAL HYDRODYNAMIC MODEL VERIFICATION

WATER LEVEL - MAY-JUNE 2006

Figure 4.4



Cardno Lawson Treloar

LJ8779/R1 November 2007

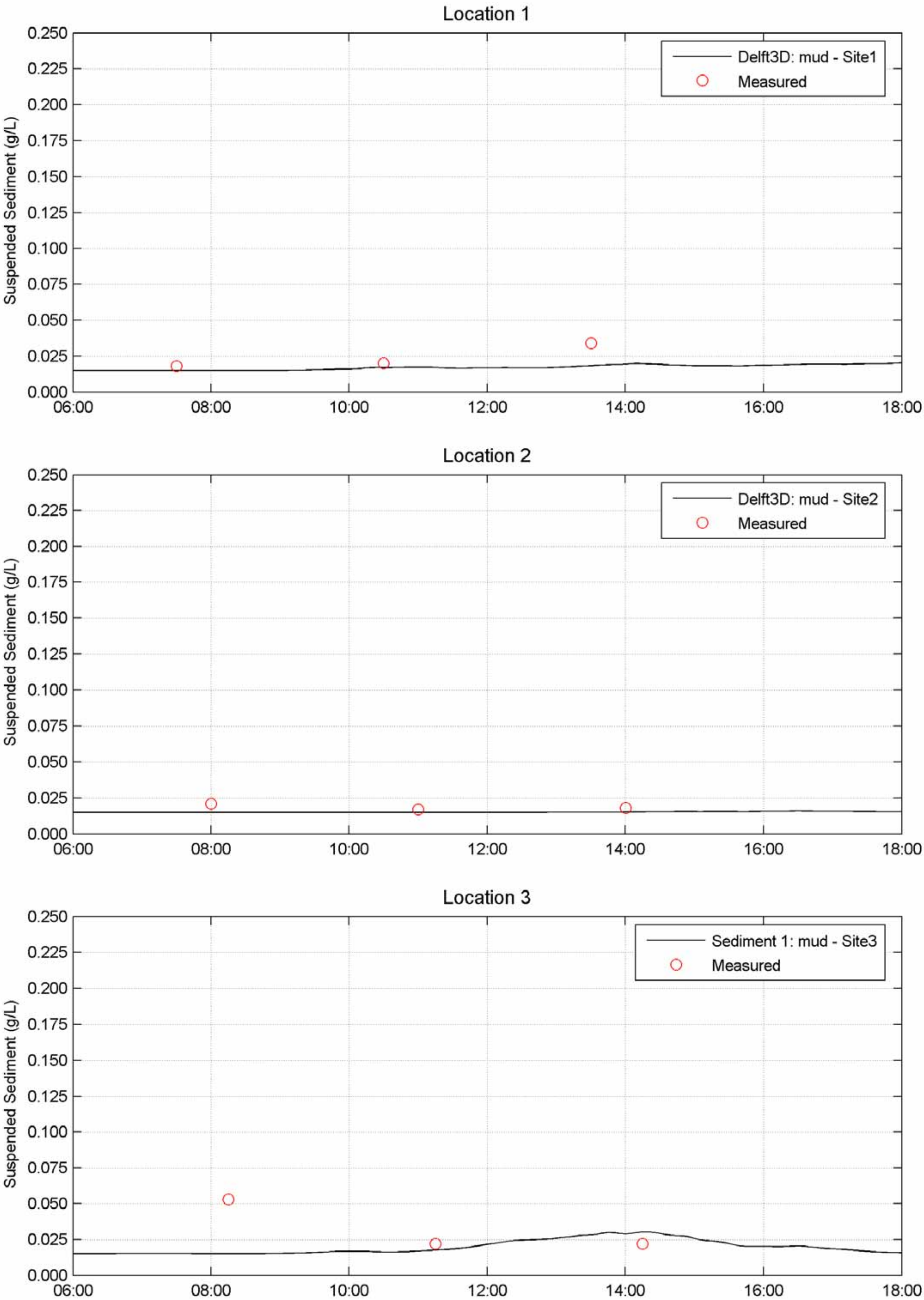
File: J:/CM/790048-Shute/Figures/R2344/Figure4.5.png

Shute Harbour Marina Development - Coastal Processes Issuse

SUSPENDED SEDIMENT SAMPLING LOCATIONS

5 JUNE 2007

Figure 4.5



Cardno Lawson Treloar

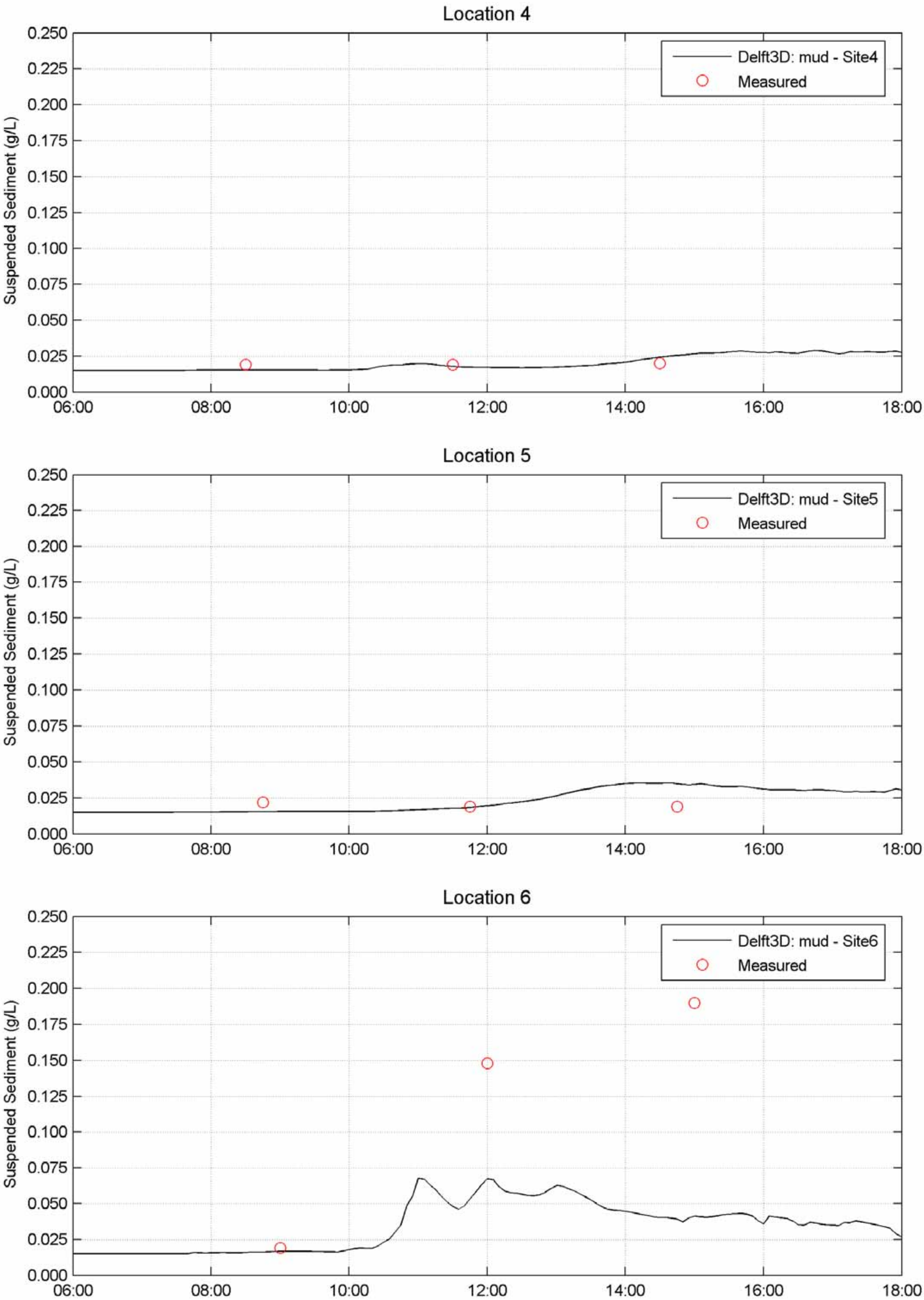
LJ8779/R1 November 2007

File: J:/CM/790048-Shute/Figures/R2344/Figure4.6.png

Shute Harbour Marina Development - Coastal Processes Issuse

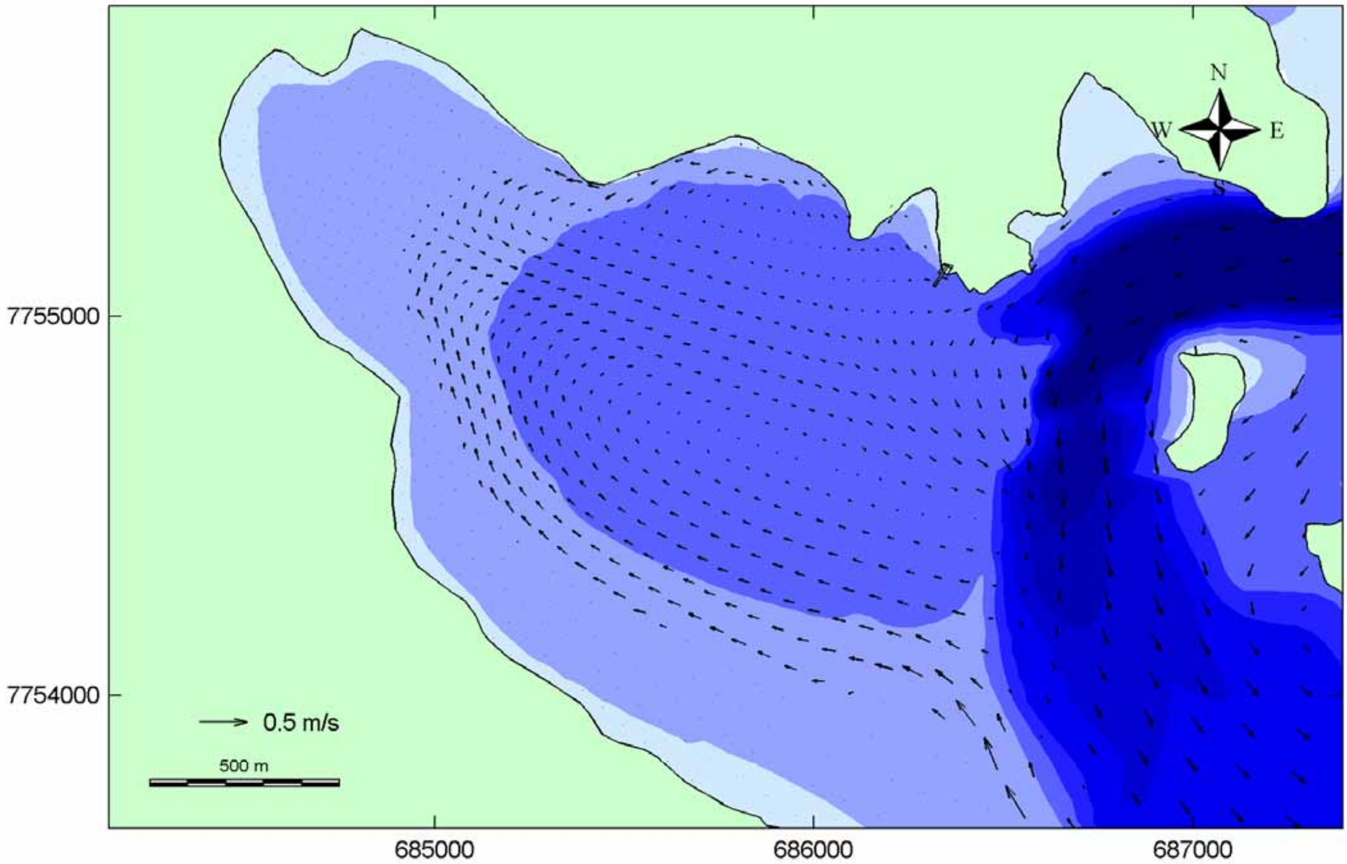
SHUTE HARBOUR DELFT3D MORPHOLOGICAL MODEL CALIBRATION - JUNE 2007

Figure 4.6

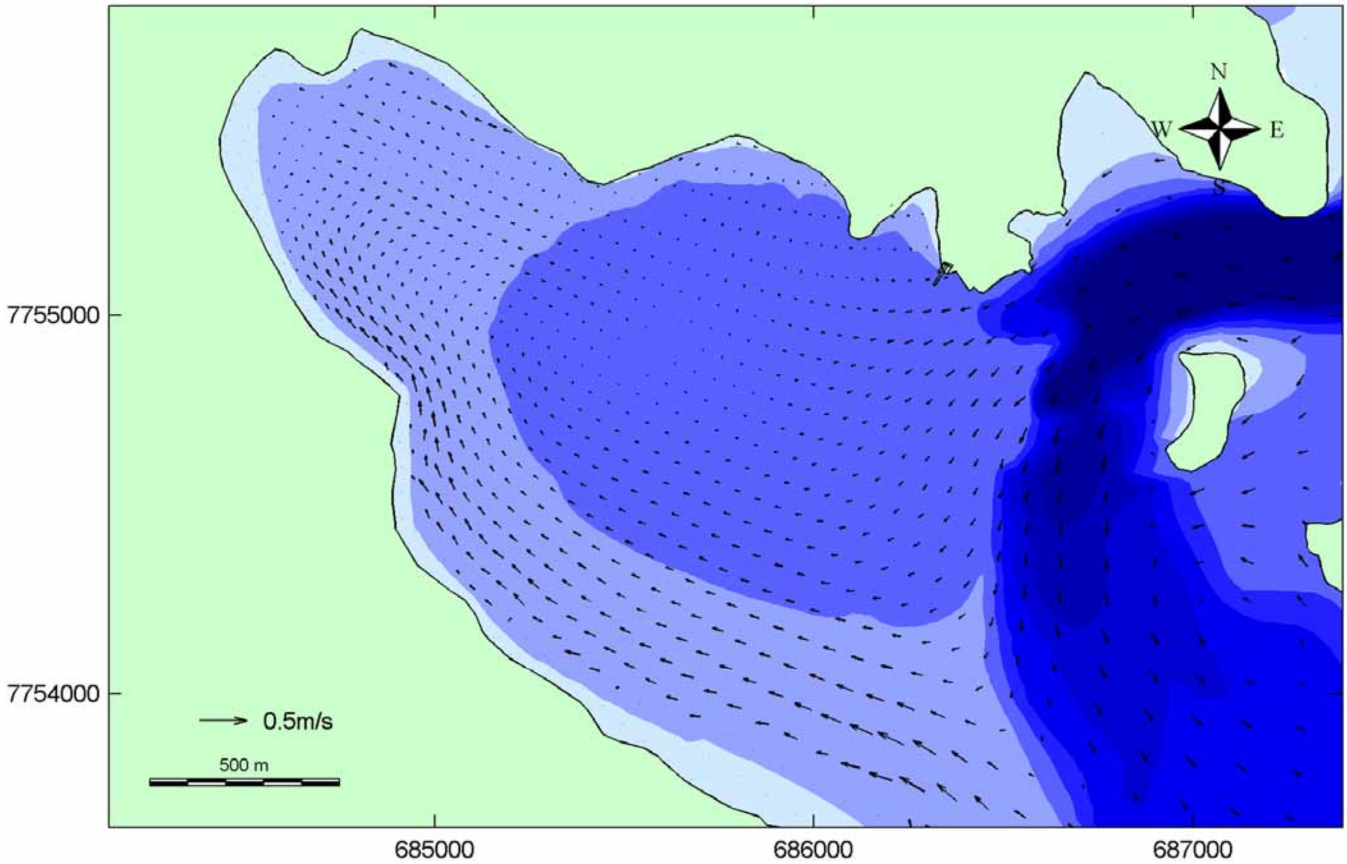


Shute Harbour Marina Development - Coastal Processes Issuse
SHUTE HARBOUR DELFT3D MORPHOLOGICAL MODEL
CALIBRATION - JUNE 2007
Figure 4.7

16:00 30/05/2007 - Low Water



18:00 30/05/2007 - Flood Tide



Cardno Lawson Treloar

LJ8779/R1 November 2007

File: J:/CM/790048-Shute/Figures/R2344/Figure4.8.png

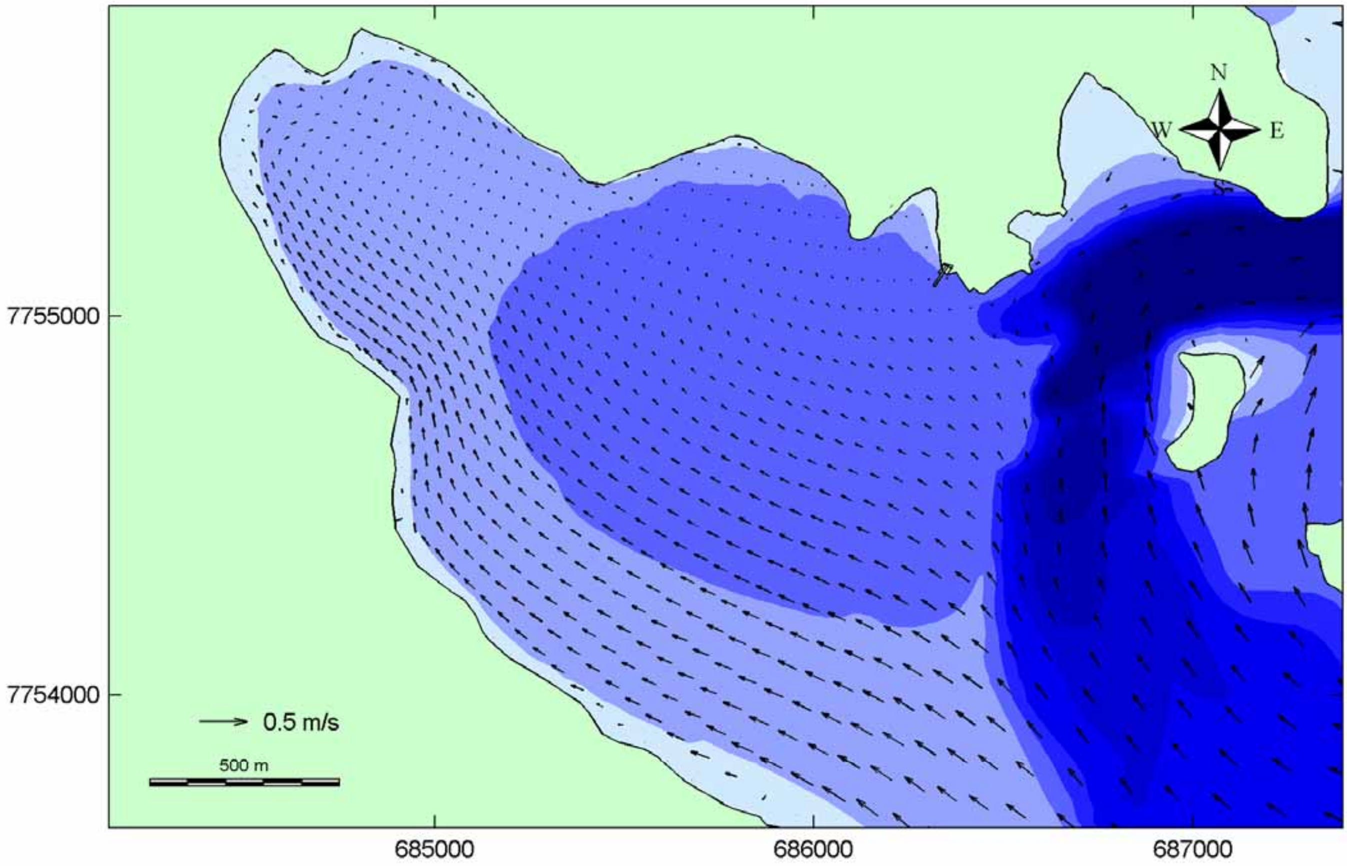
Shute Harbour Marina Development - Coastal Processes Issuse

CURRENTS - DEPTH-AVERAGED (SE Wind @ 7m/s)

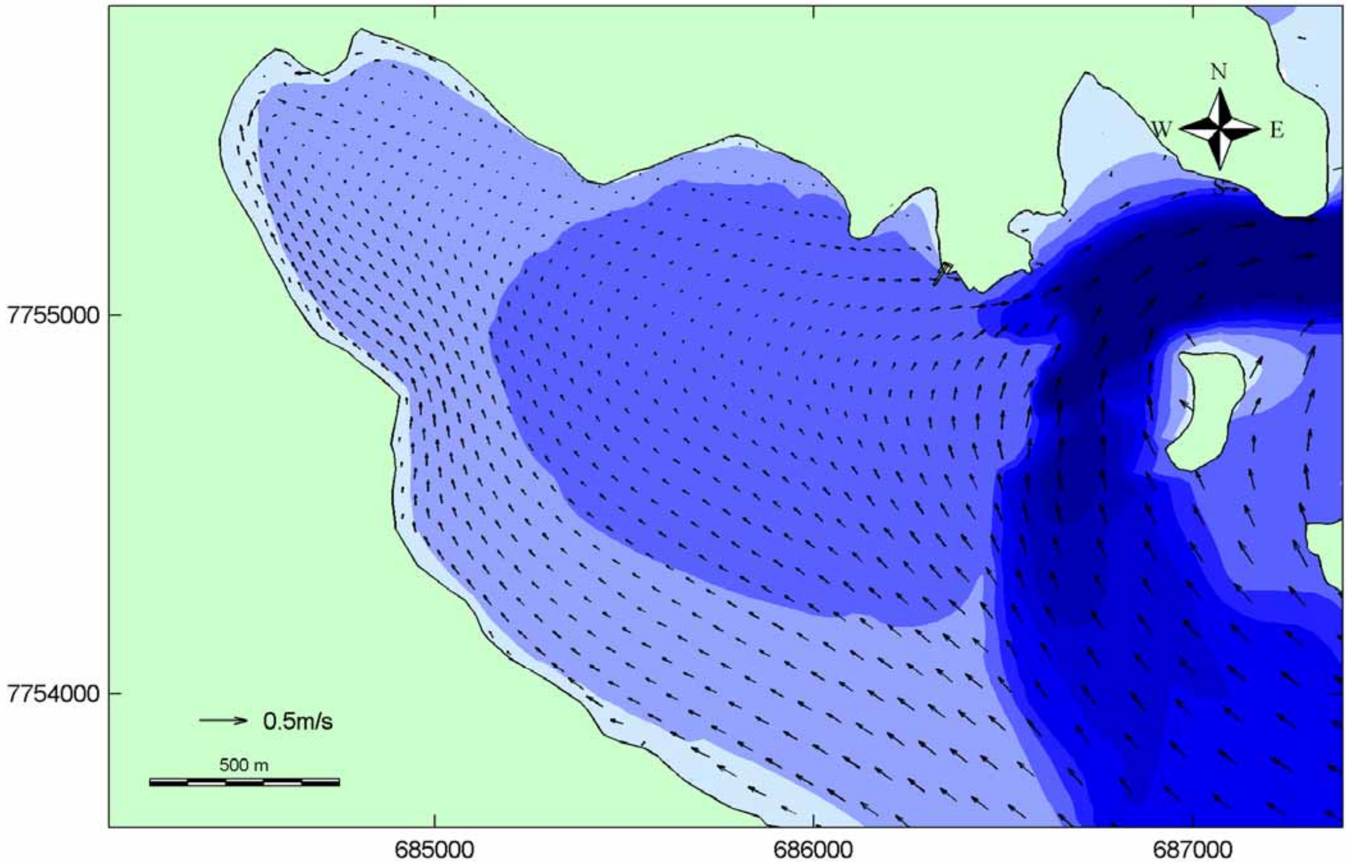
EXISTING

Figure 4.8

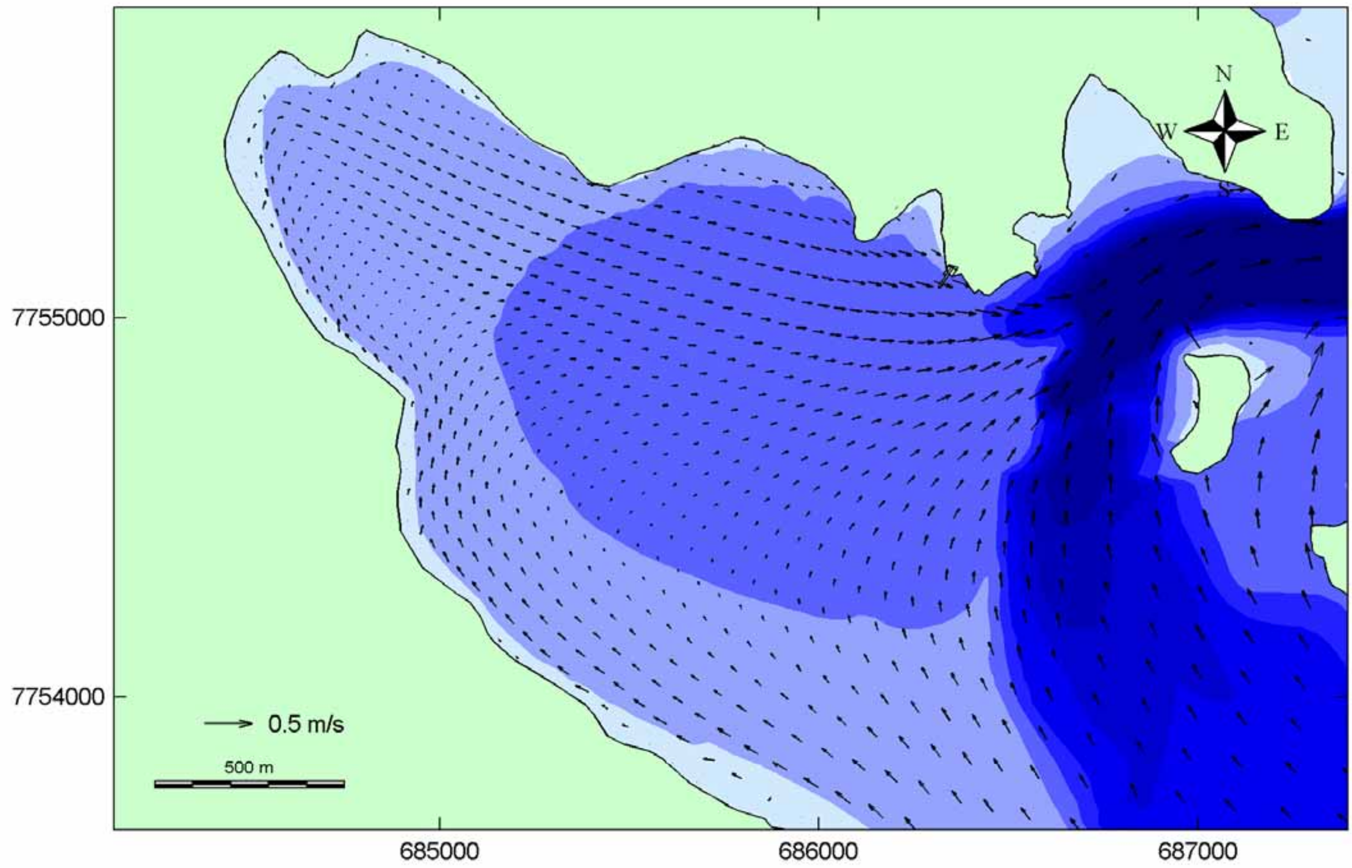
20:00 30/05/2007 - Flood Tide



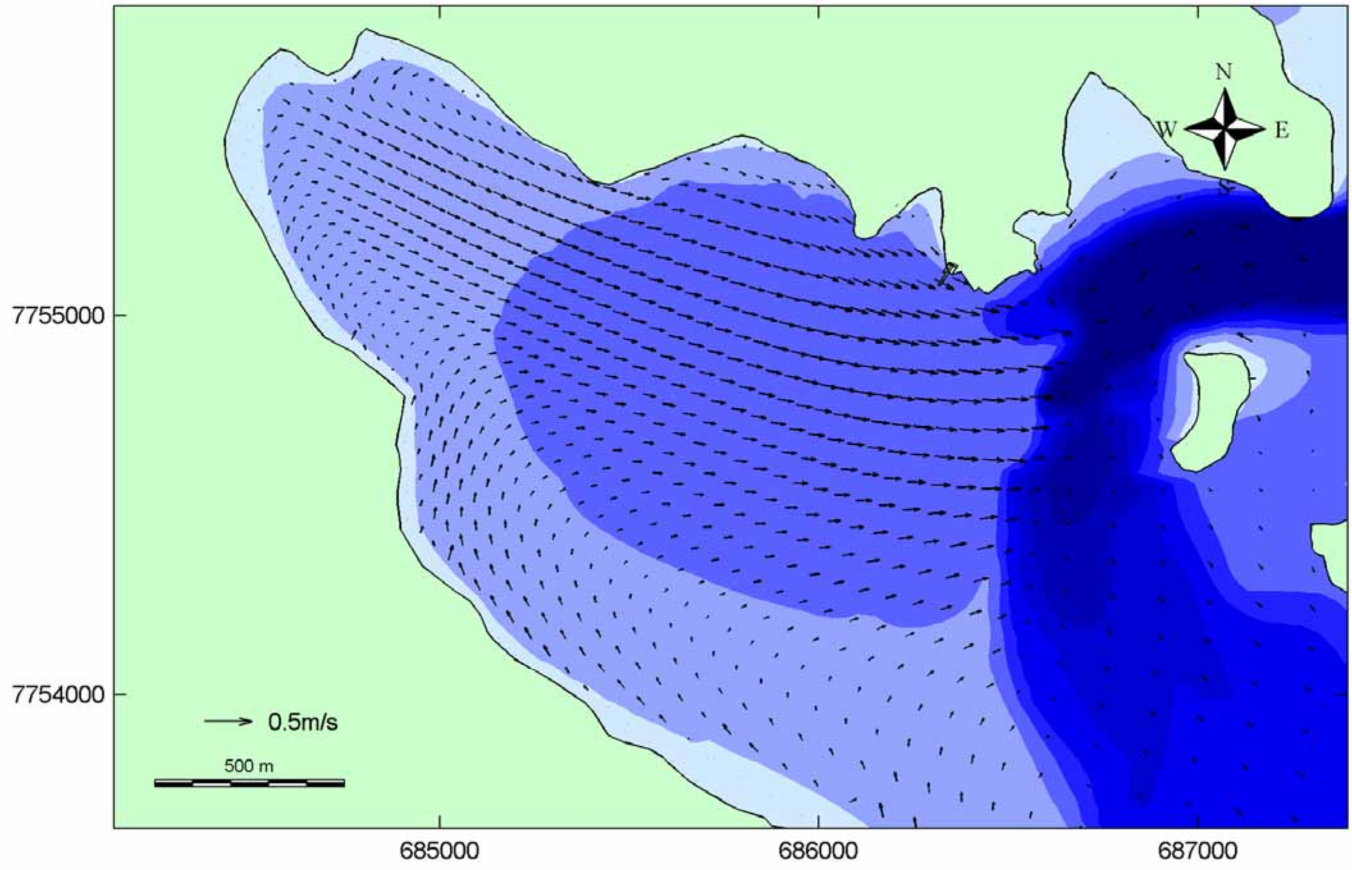
22:00 30/05/2007 - High Water



00:00 31/05/2007 - Ebb Tide



02:00 31/05/2007 - Ebb Tide



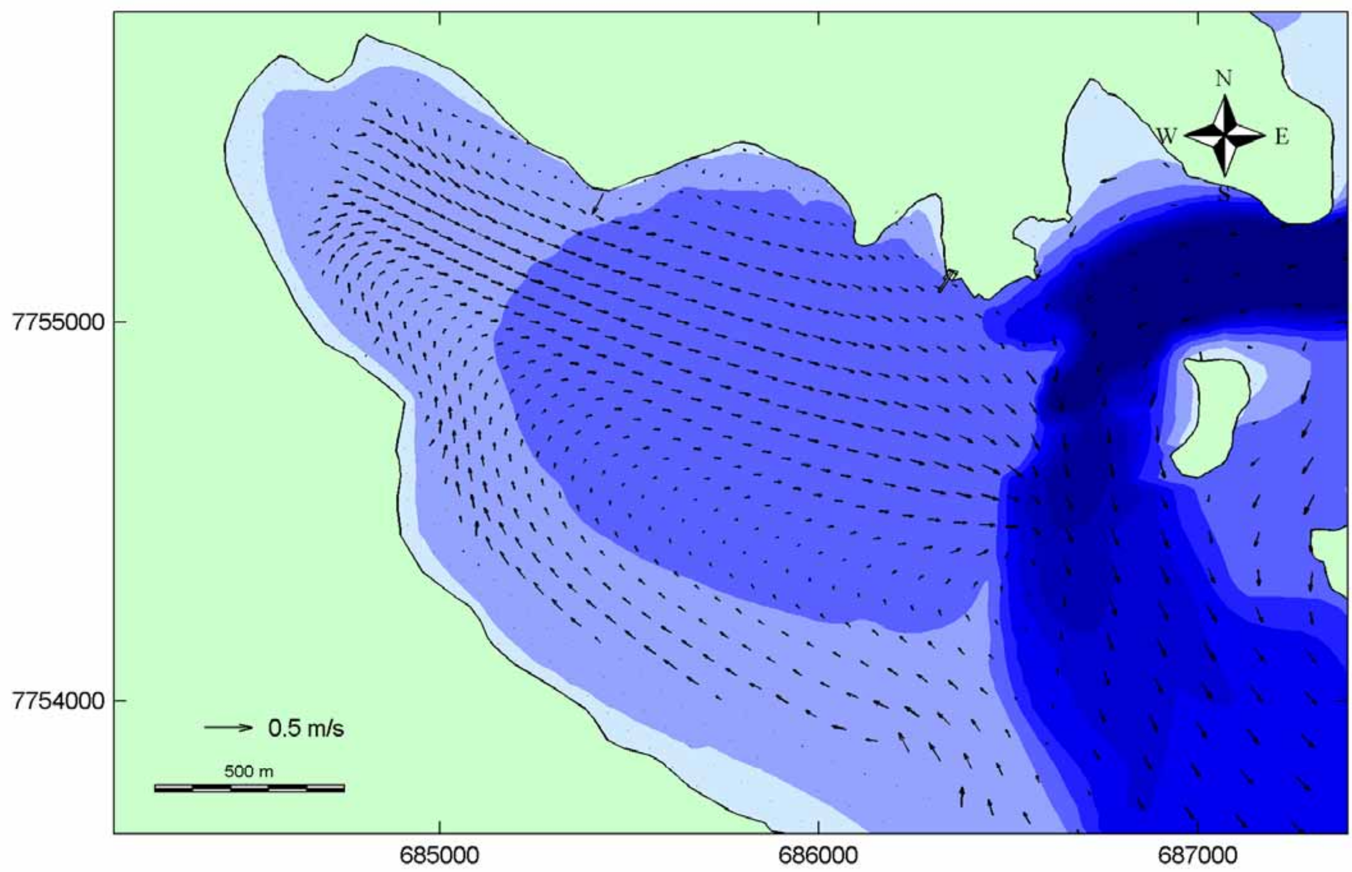
Shute Harbour Marina Development - Coastal Processes Issuse

CURRENTS - DEPTH-AVERAGED (SE Wind @ 7m/s)

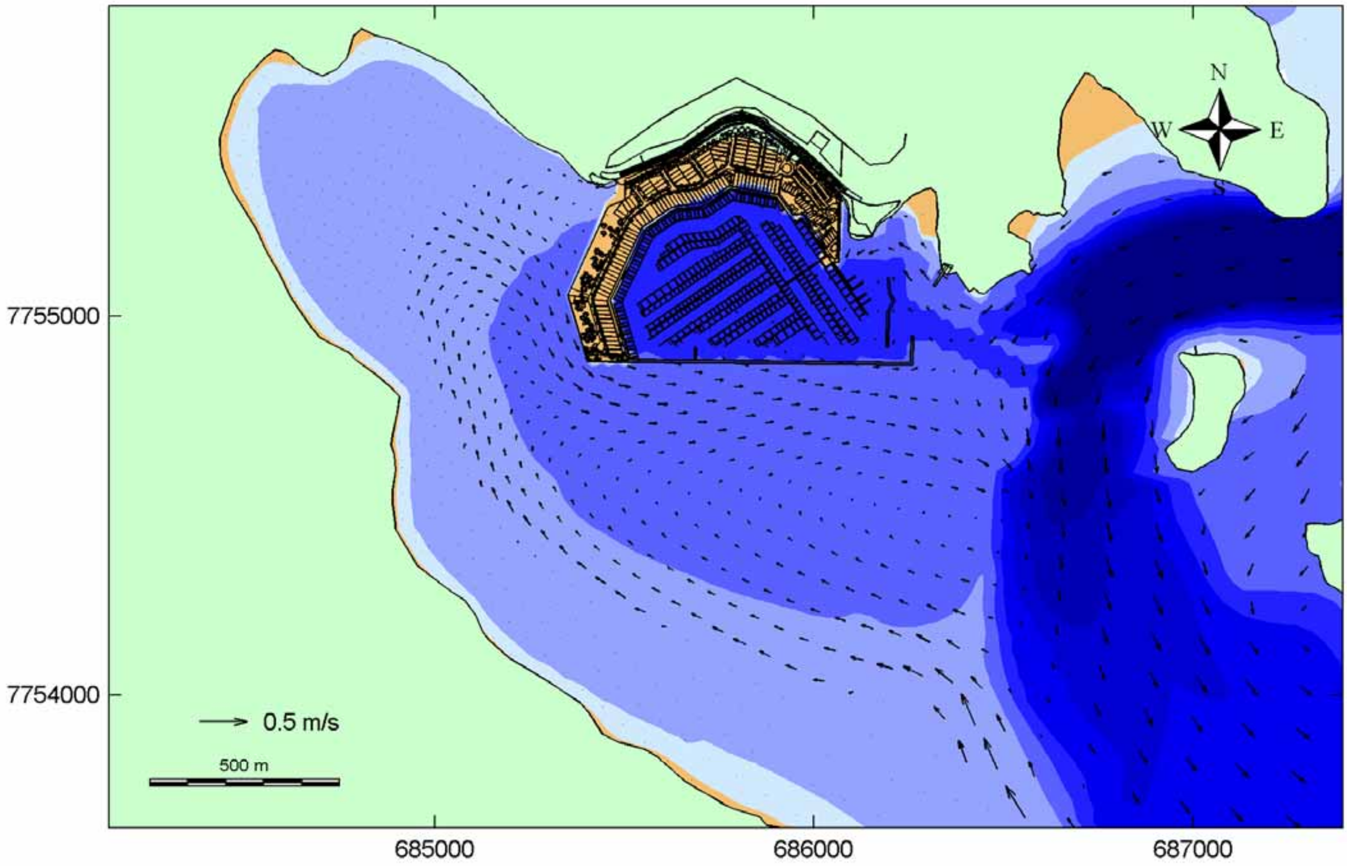
EXISTING

Figure 4.10

04:00 31/05/2007 - Low Water



16:00 30/05/2007 - Low Water



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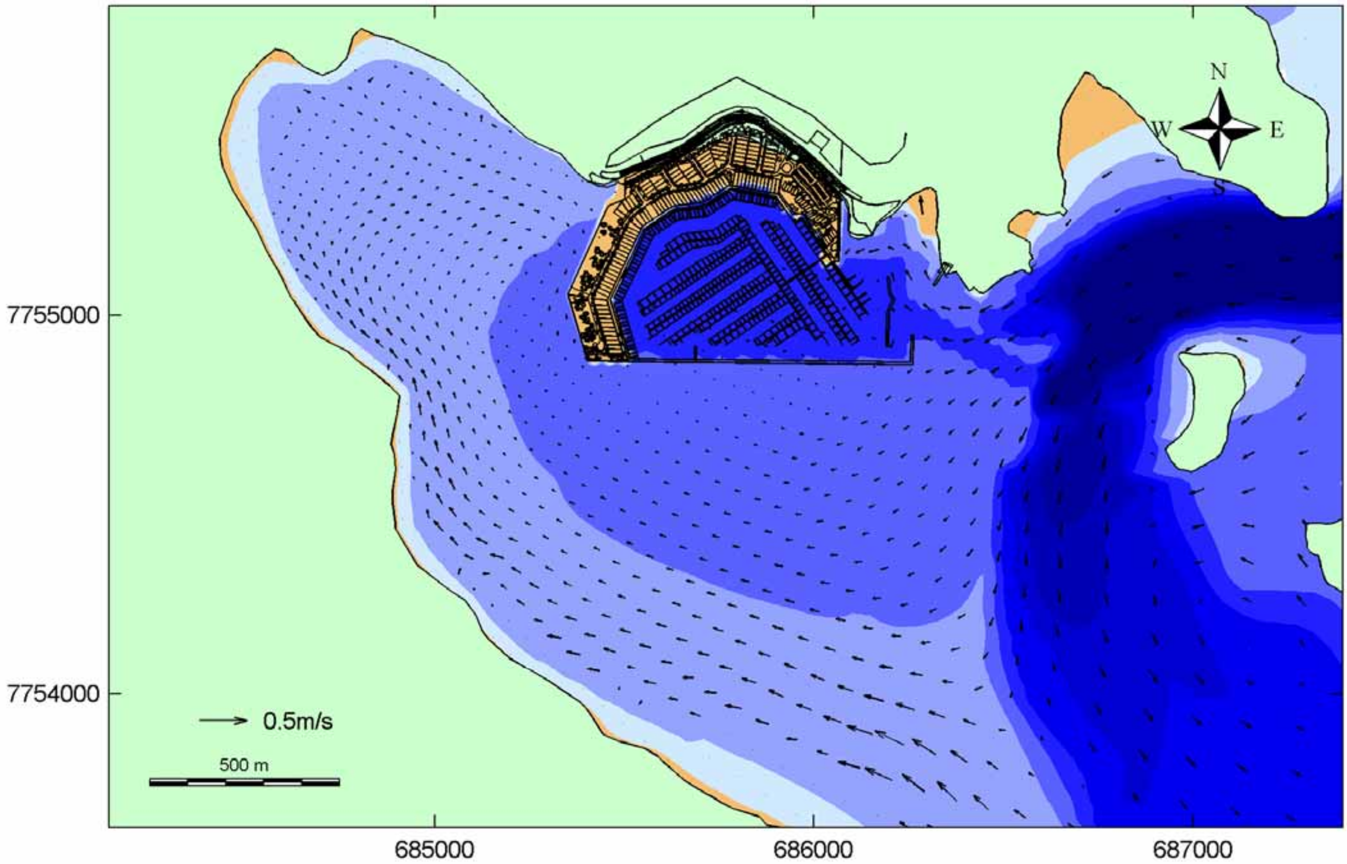

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LJ8779/R1 November 2007

File: J:/CM/790048-Shute/Figures/R2344/Figure4.12.png

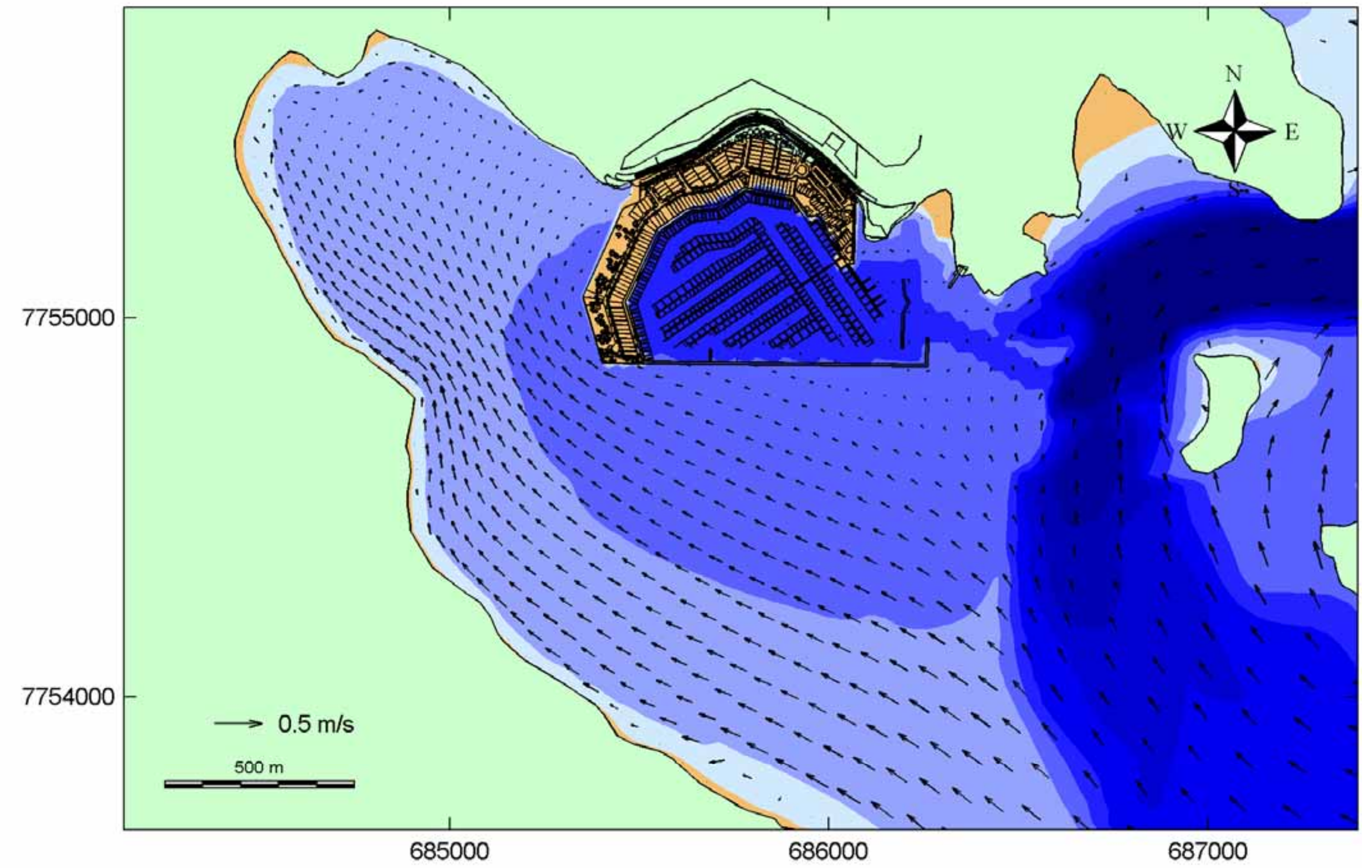
Shute Harbour Marina Development - Coastal Processes Issuse

CURRENTS - DEPTH-AVERAGED (SE Wind @ 7m/s)

POST-MARINA

Figure 4.12

20:00 30/05/2007 - Flood Tide



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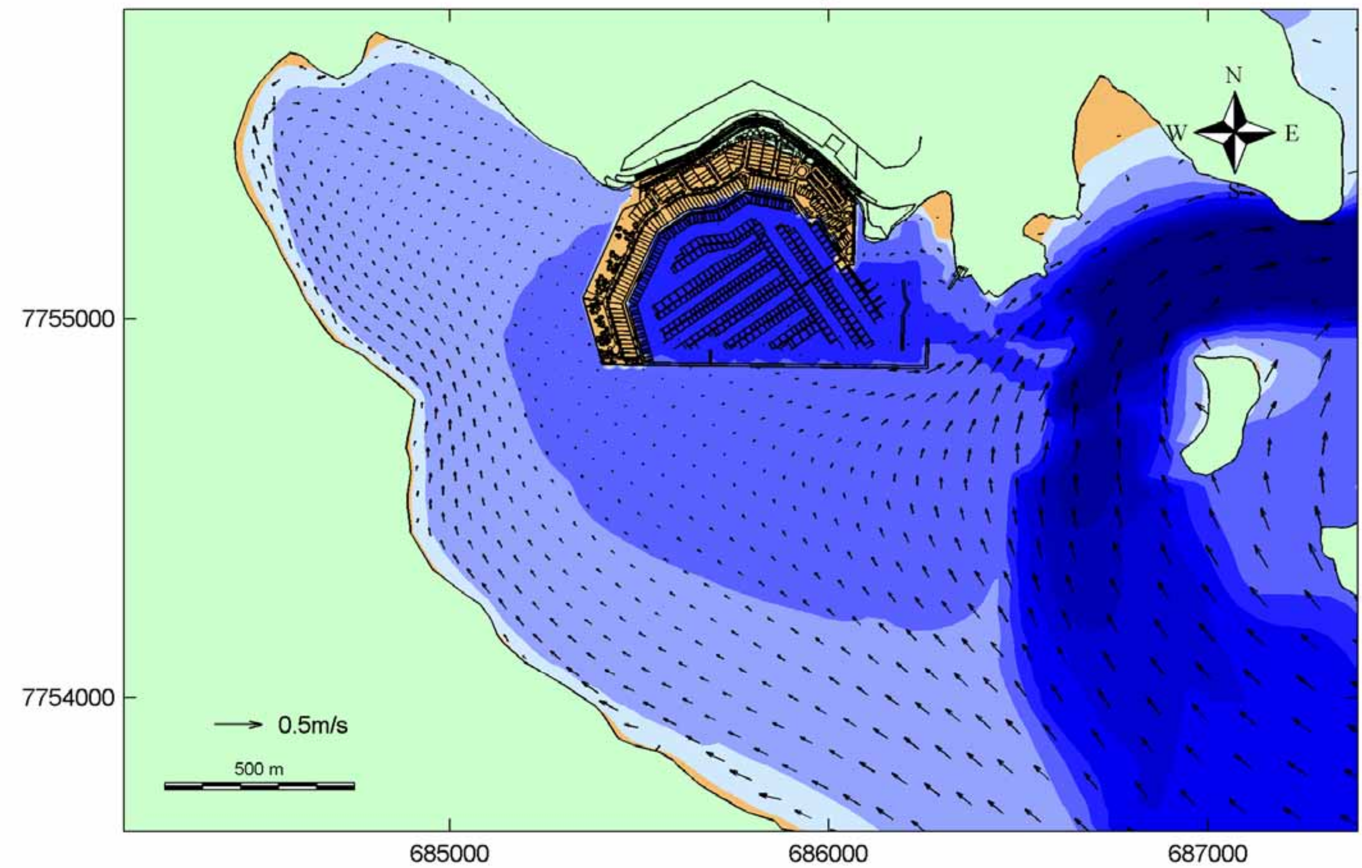
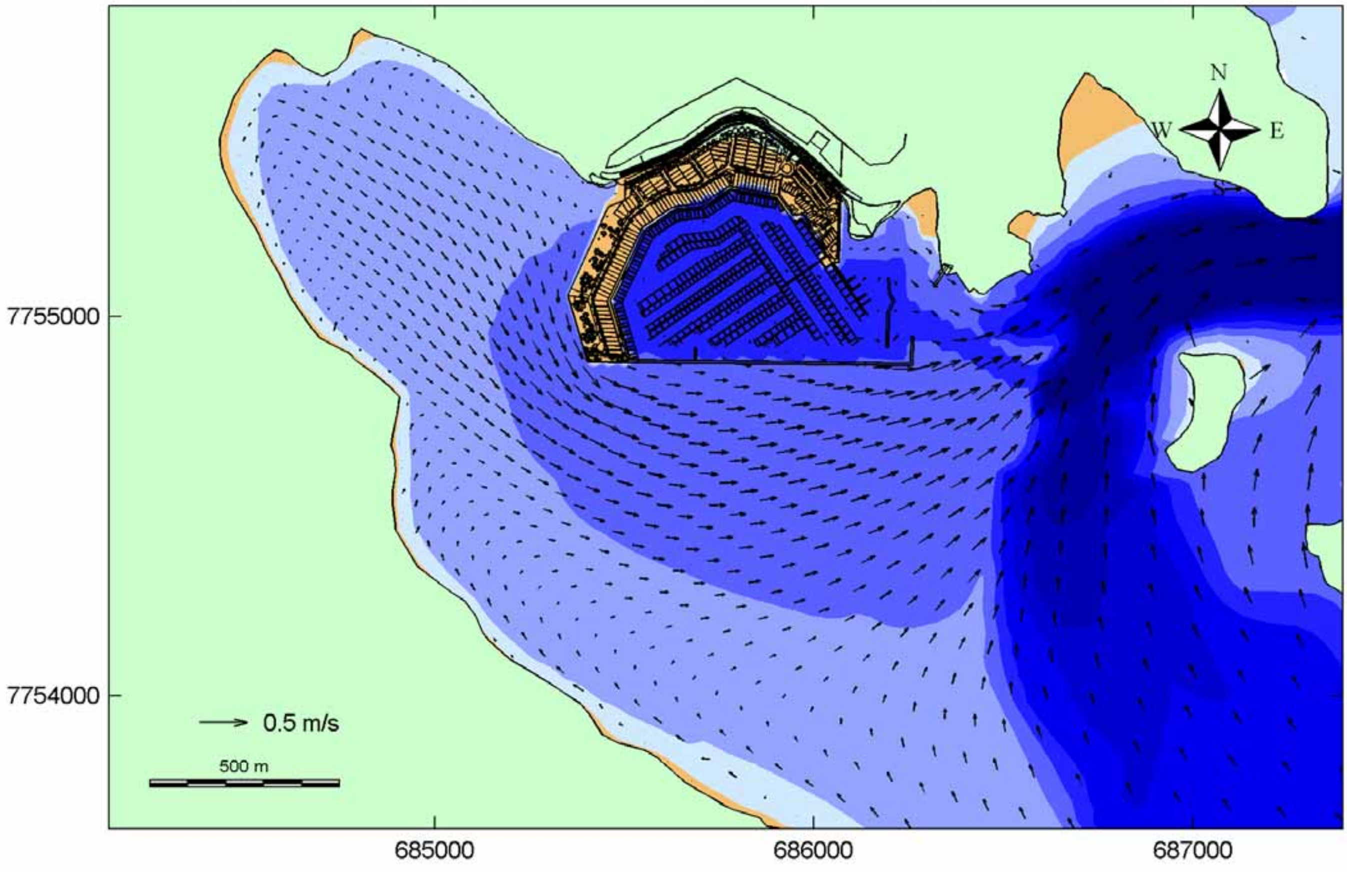


00:00 31/05/2007 - Ebb Tide



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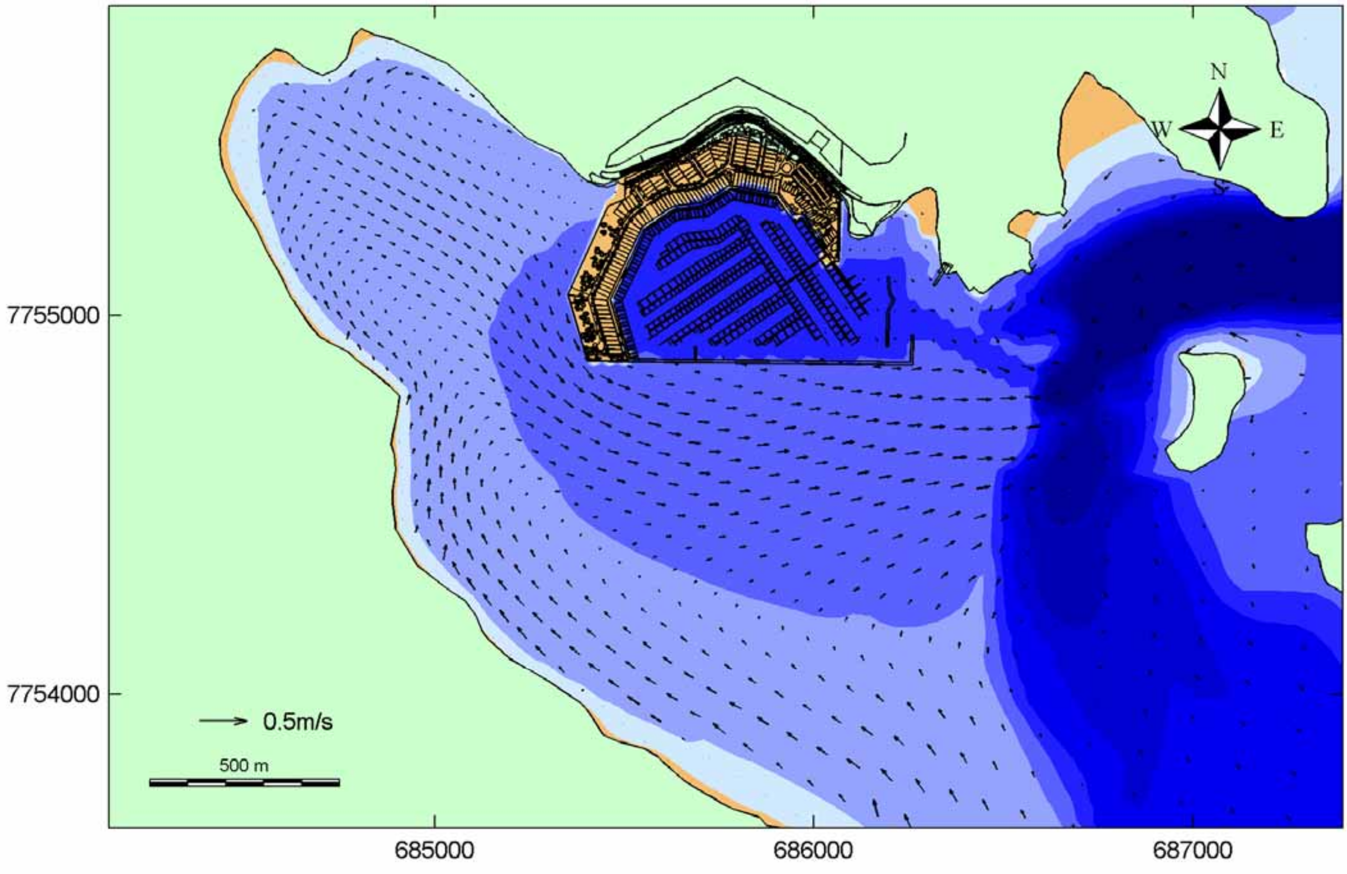

Cardno Lawson Treloar

Shute Harbour Marina Development - Coastal Processes Issuse

CURRENTS - DEPTH-AVERAGED (SE Wind @ 7m/s)

POST-MARINA

Figure 4.14

LJ8779/R1 November 2007

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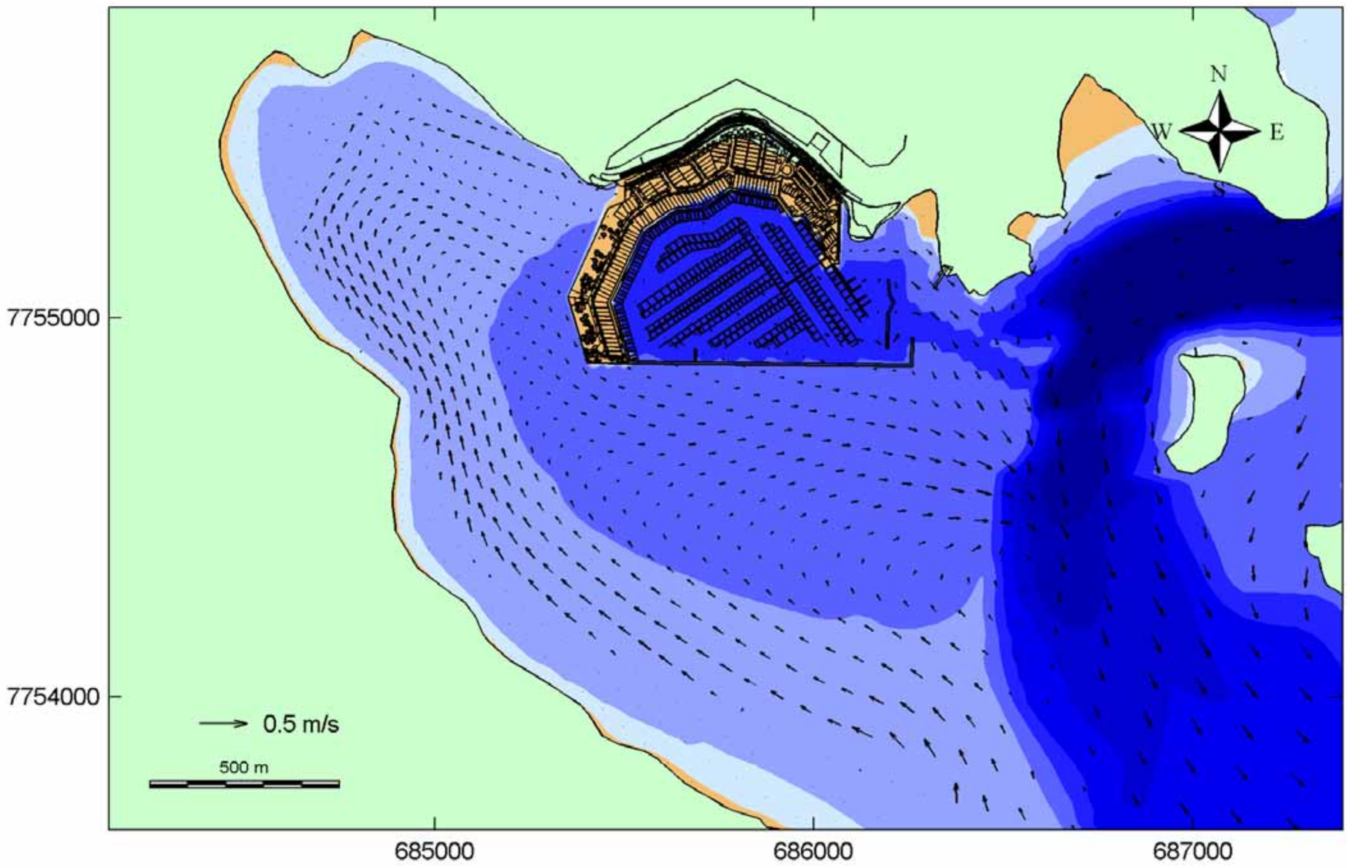
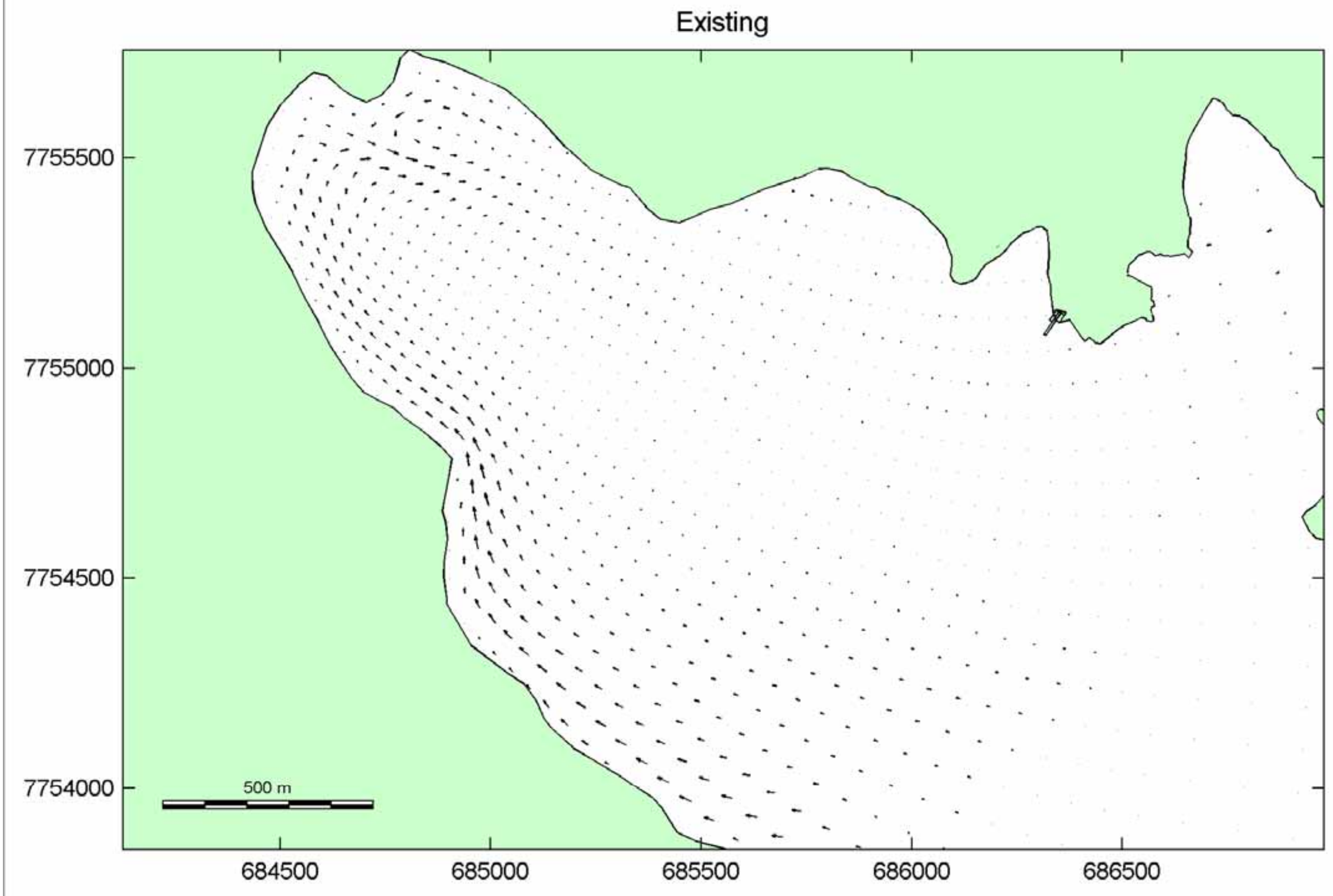


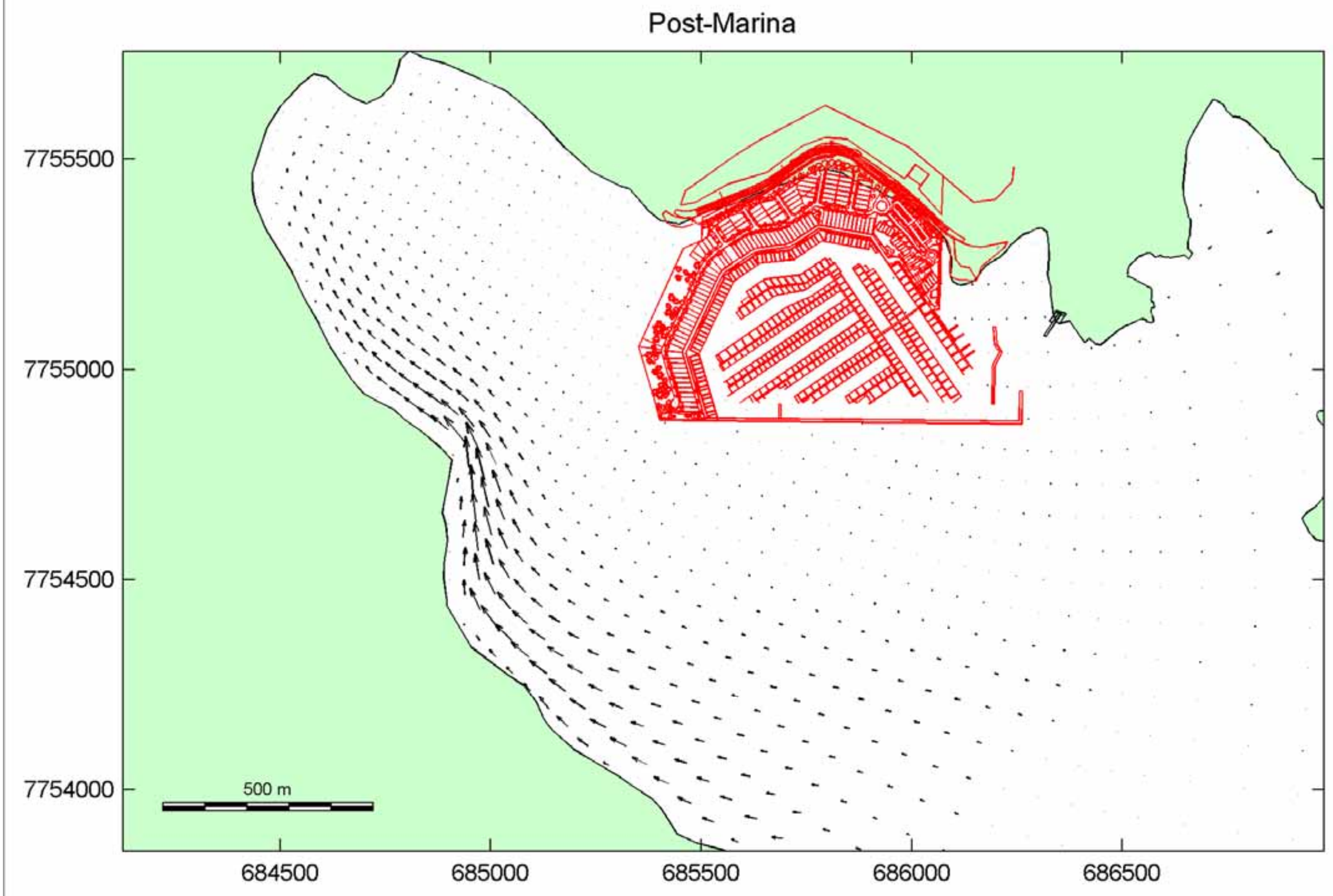
Shute Harbour Marina Development - Coastal Processes Issuse

CURRENTS - DEPTH-AVERAGED (SE Wind @ 7m/s)

POST-MARINA

Figure 4.15





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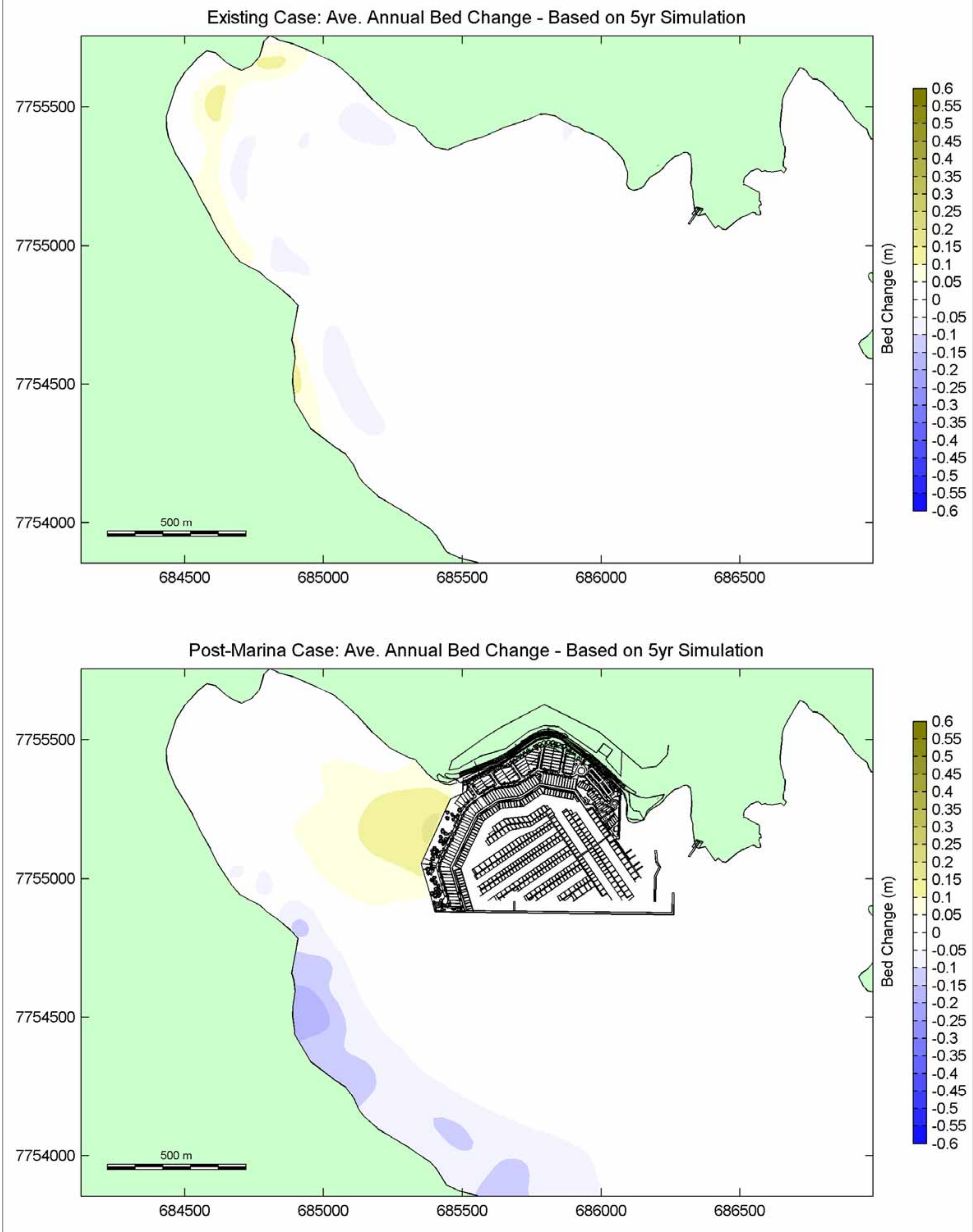
Shute Harbour Marina Development - Coastal Processes Issues

NET SEDIMENT TRANSPORT - SPRING TIDE, WIND 7m/s @ 146.25deg

EXISTING AND POST-MARINA LAYOUTS (Boat Ramp Included)

Figure 4.16



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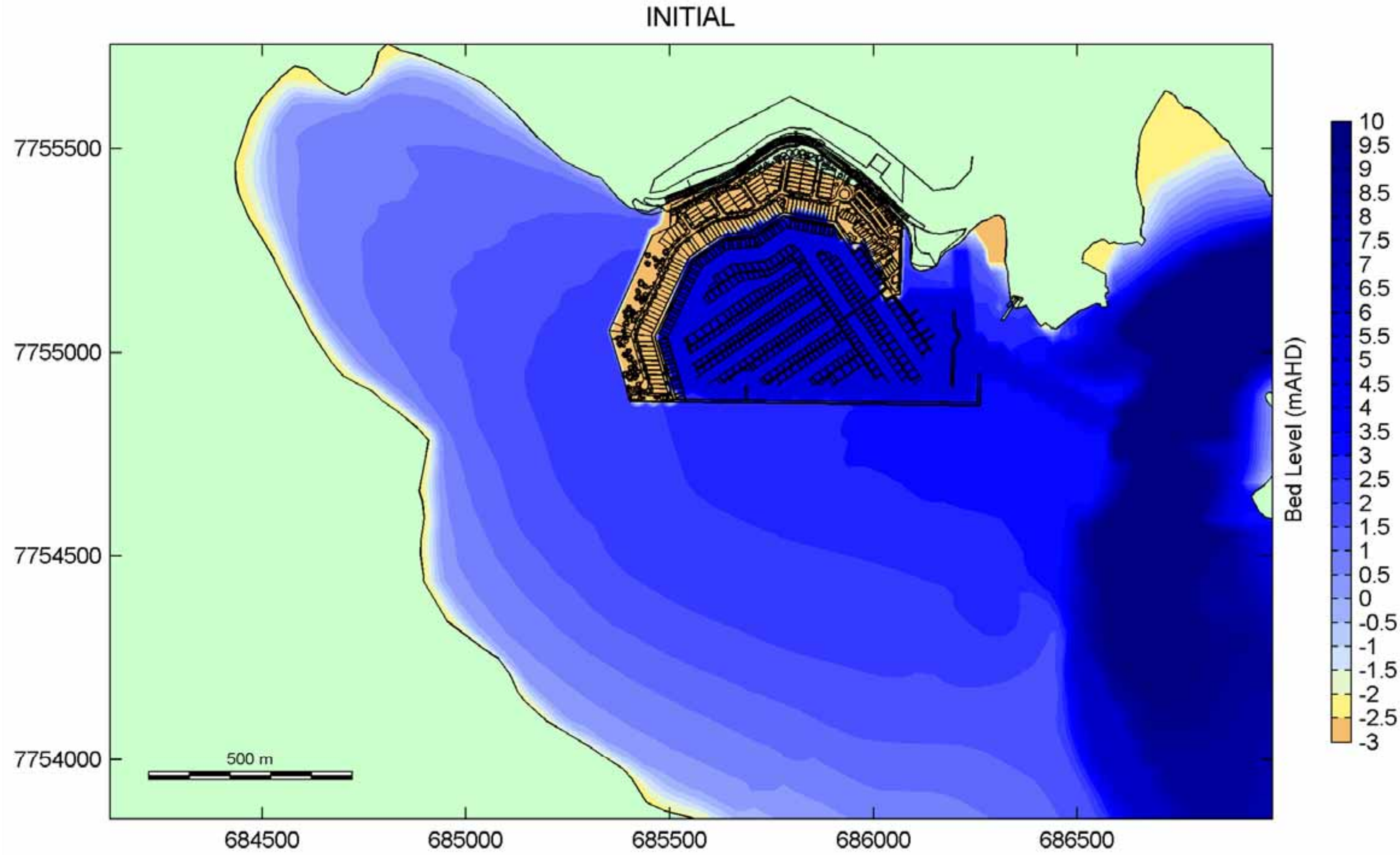
File: J:/CM/790048-Shute/Figures/R2344/Figure4.17.png

Shute Harbour Marina Development - Coastal Processes Issues

AVERAGE ANNUAL BED CHANGE

0 to 5 YEARS POST-CONSTRUCTION

Figure 4.17



INITIAL
7755500
7755000
7754500
7754000
500 m
684500
685000
685500
686000
686500
Bed Level (mAHD)
10
9.5
9
8.5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
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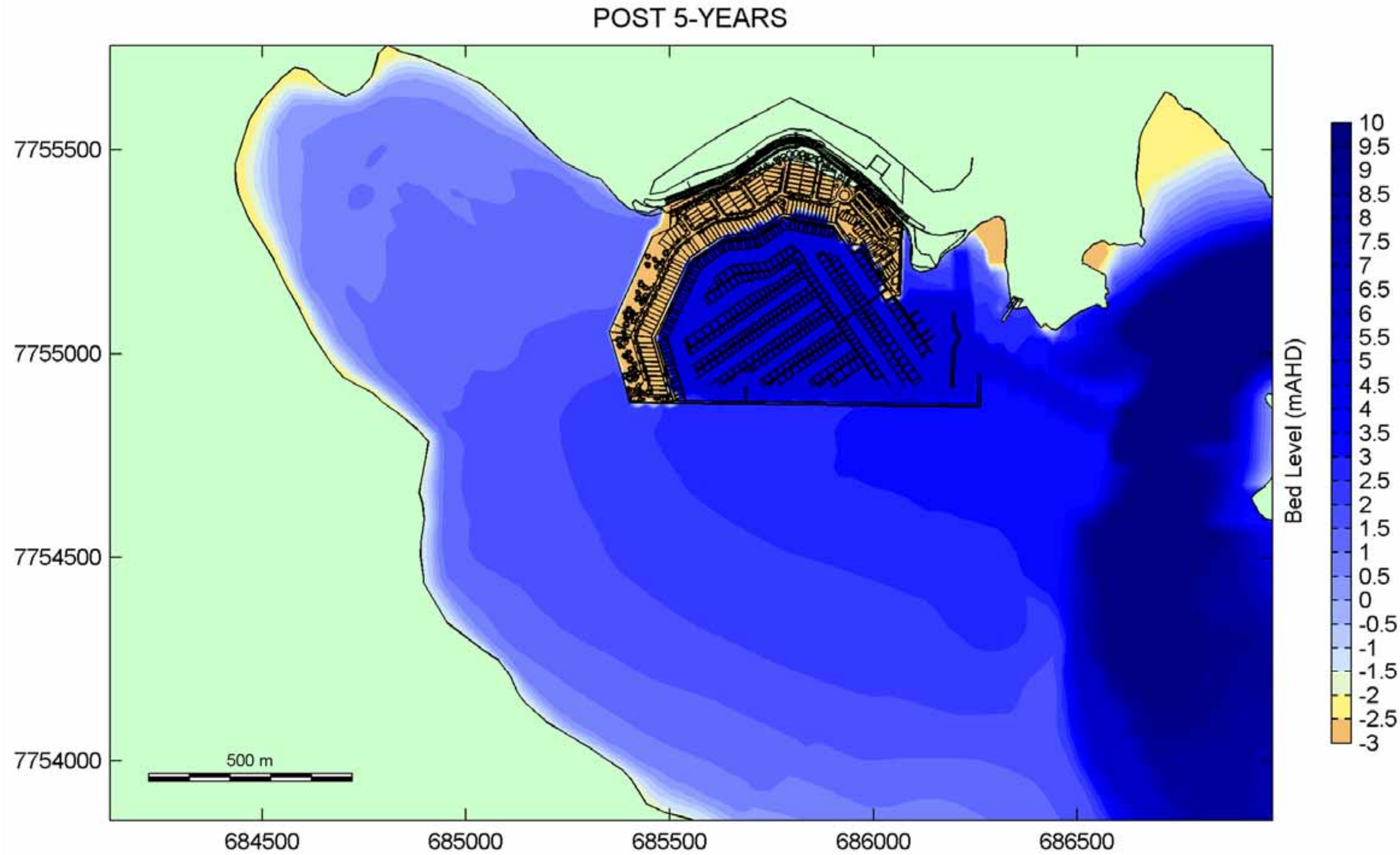
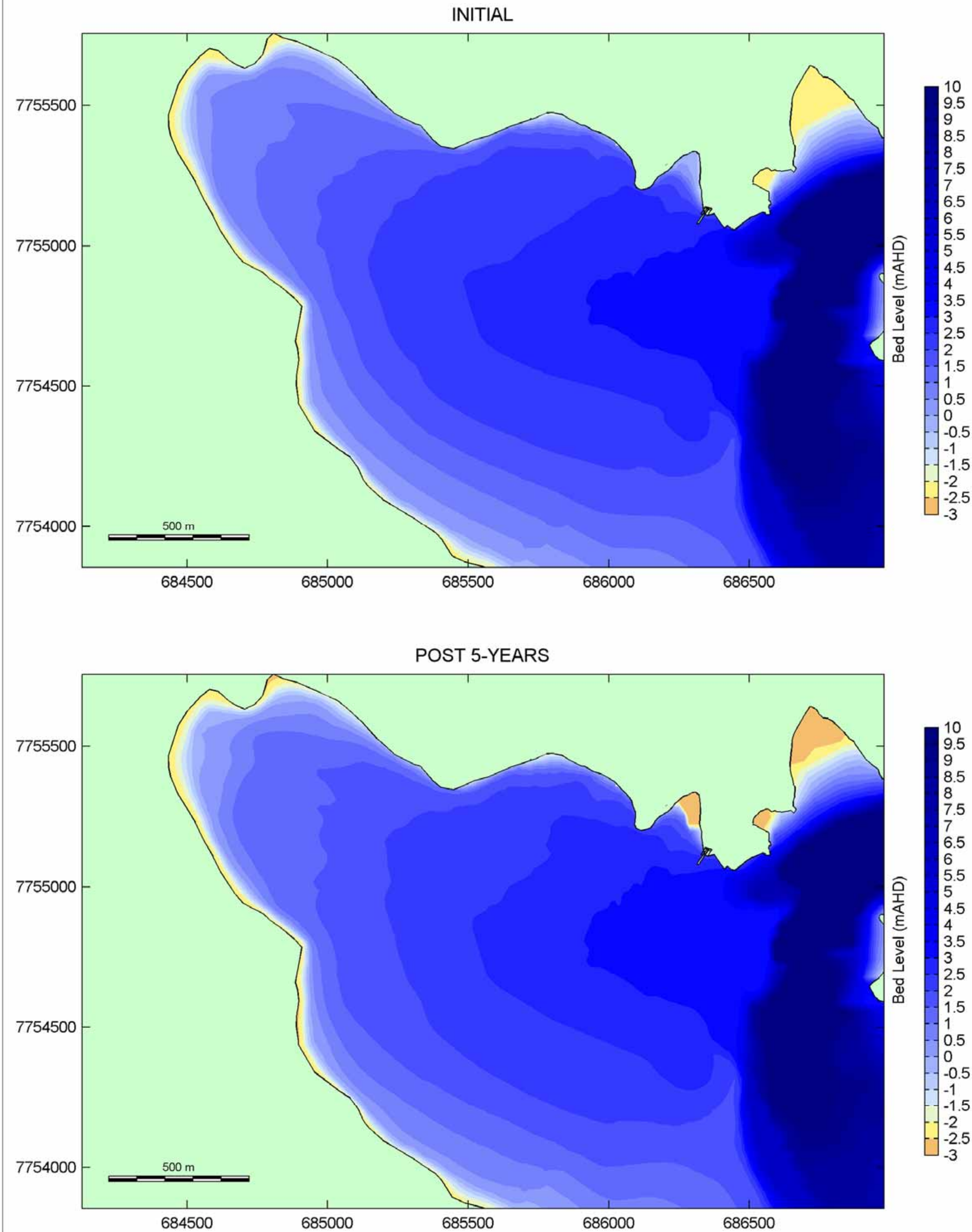
-1
-1.5
-2
-2.5
-3



POST 5-YEARS
7755500
7755000
7754500
7754000
500 m
684500
685000
685500
686000
686500
Bed Level (mAHD)
10
9.5
9
8.5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
-2.5
-3

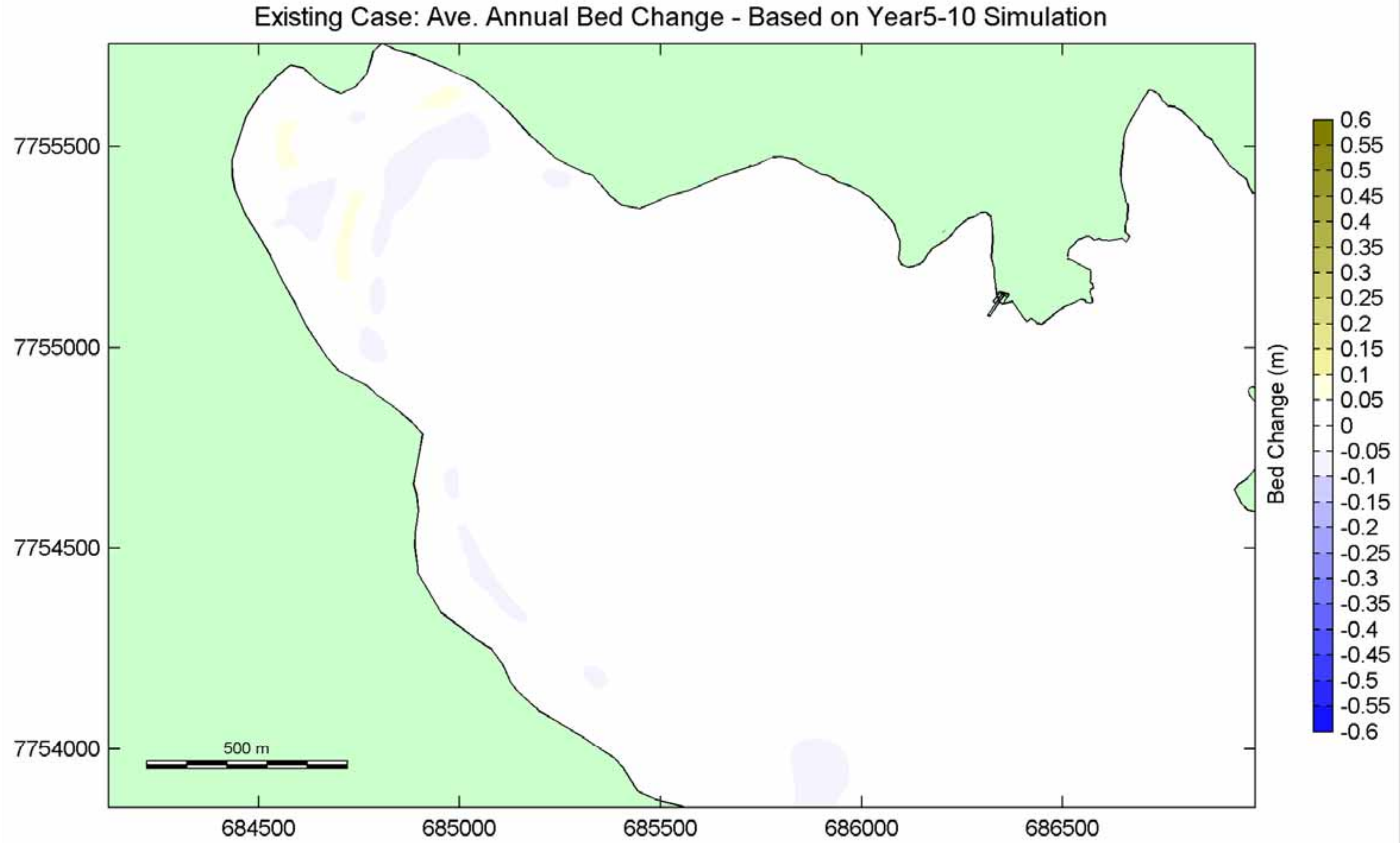
Figure 4.18

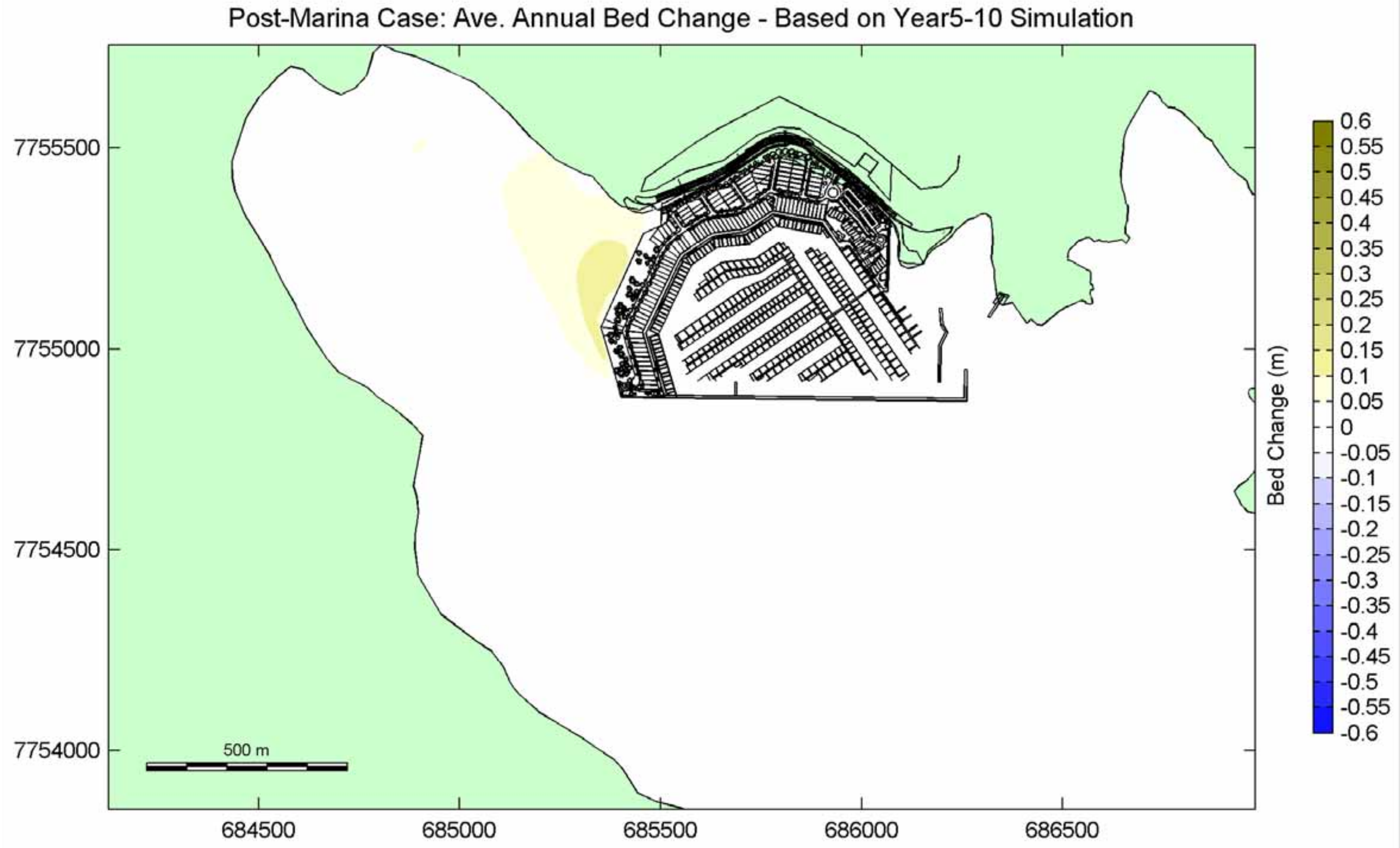


Shute Harbour Marina Development - Coastal Processes Issues

5-YEAR BATHYMETRY CHANGE - EXISTING

Figure 4.19

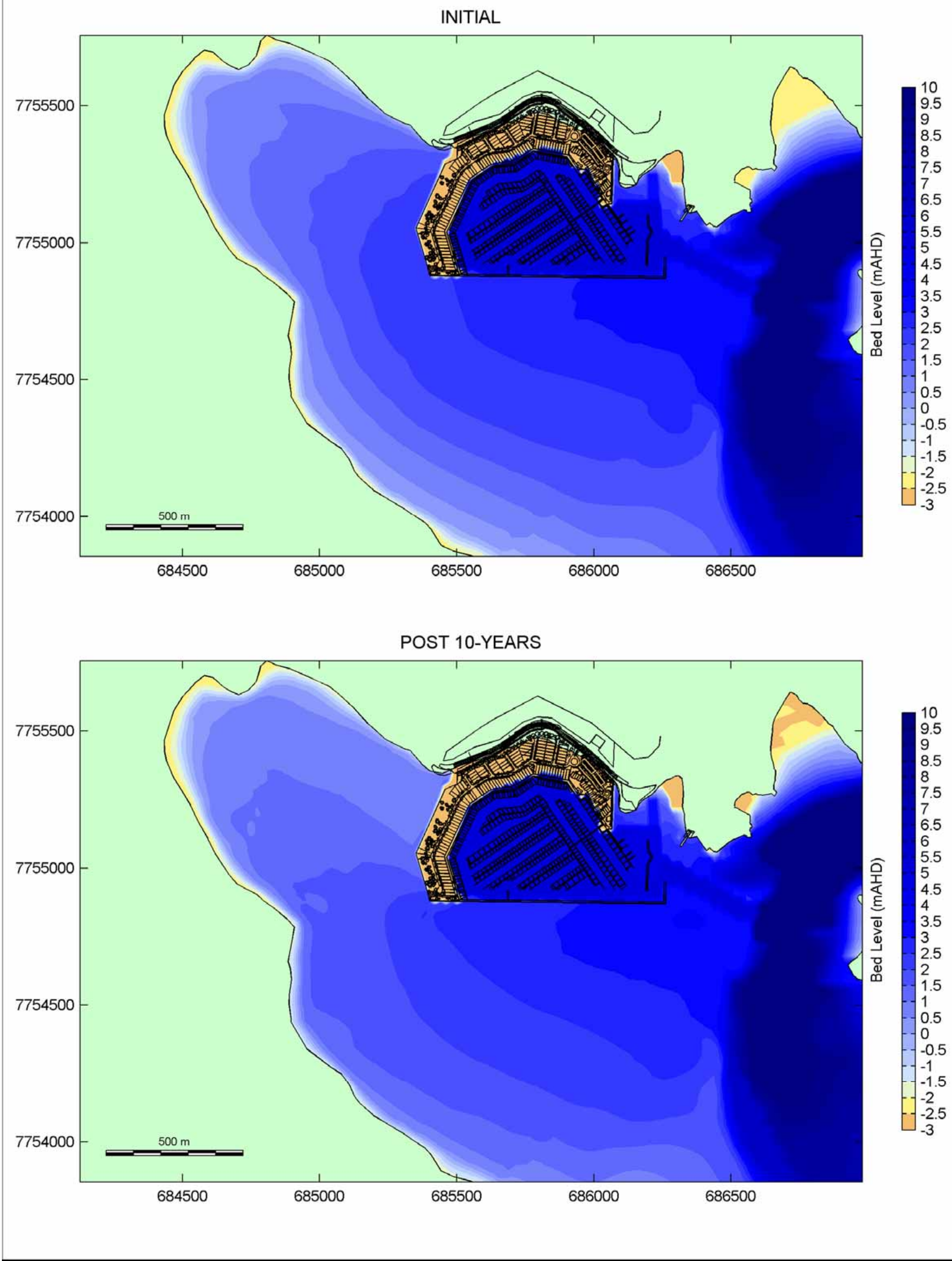




Shute Harbour Marina Development - Coastal Processes Issues

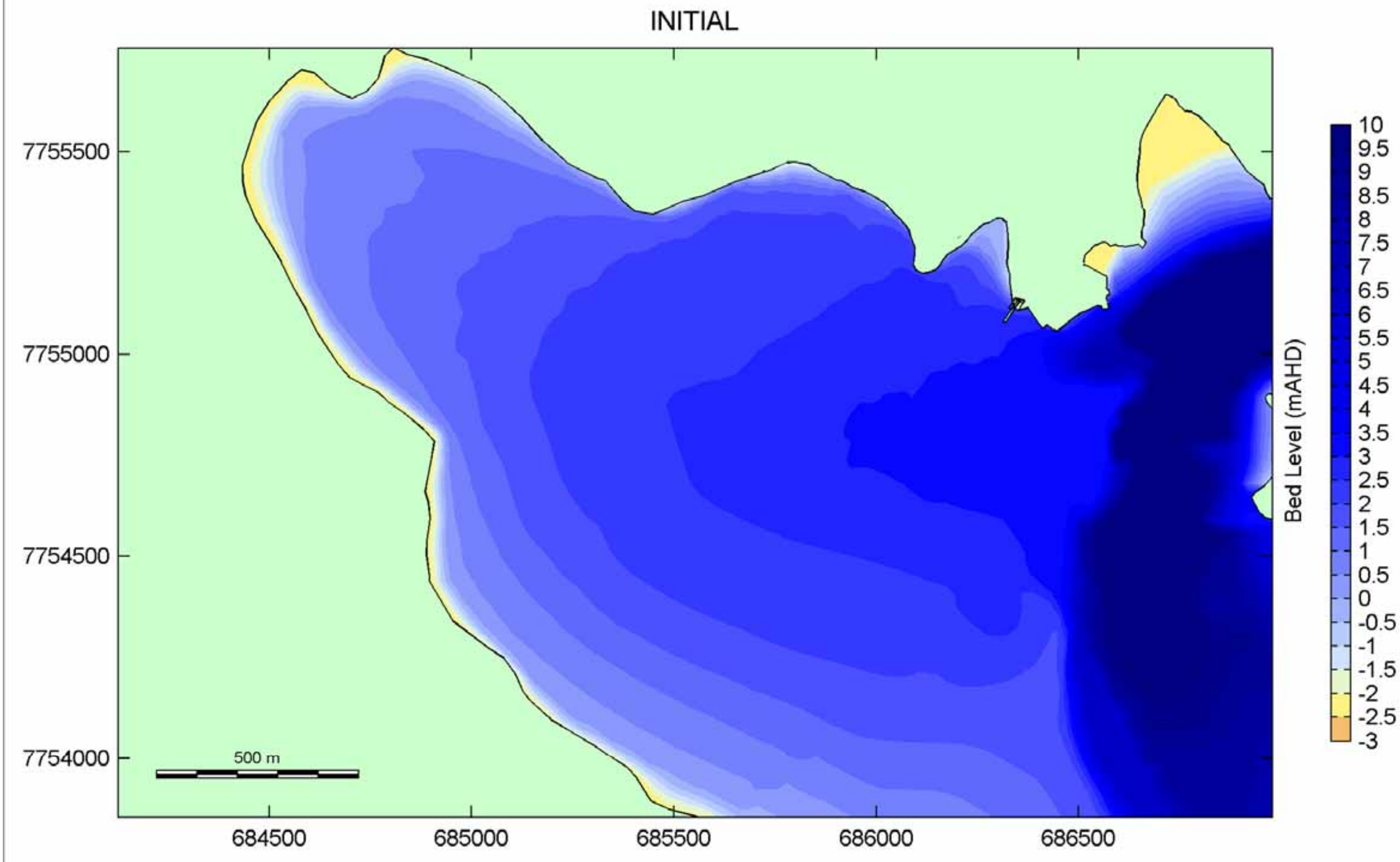
AVERAGE ANNUAL BED CHANGE
5 to 10 YEARS POST-CONSTRUCTION

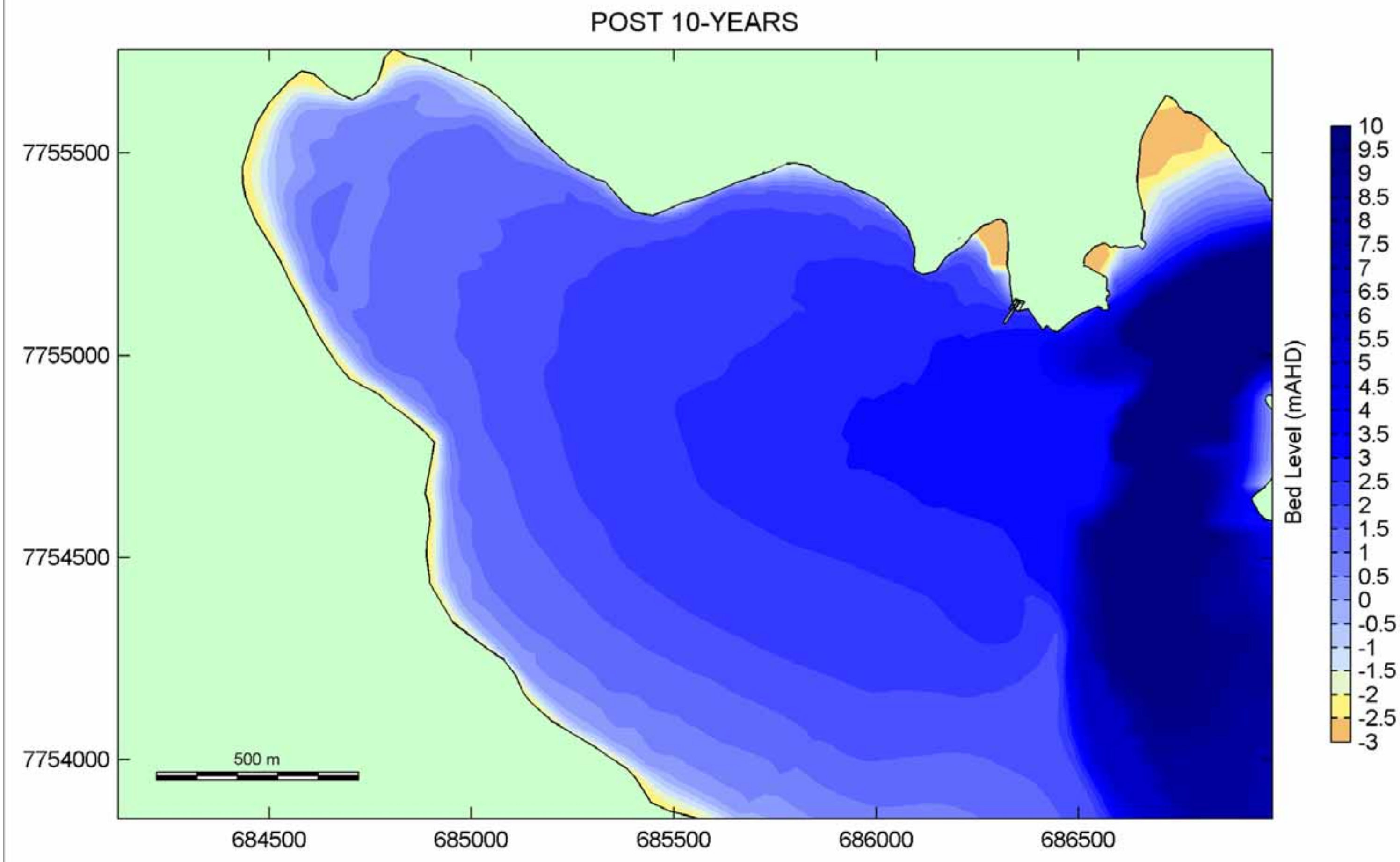
Figure 4.20



10-YEAR BATHYMETRY CHANGE - POST-MARINA

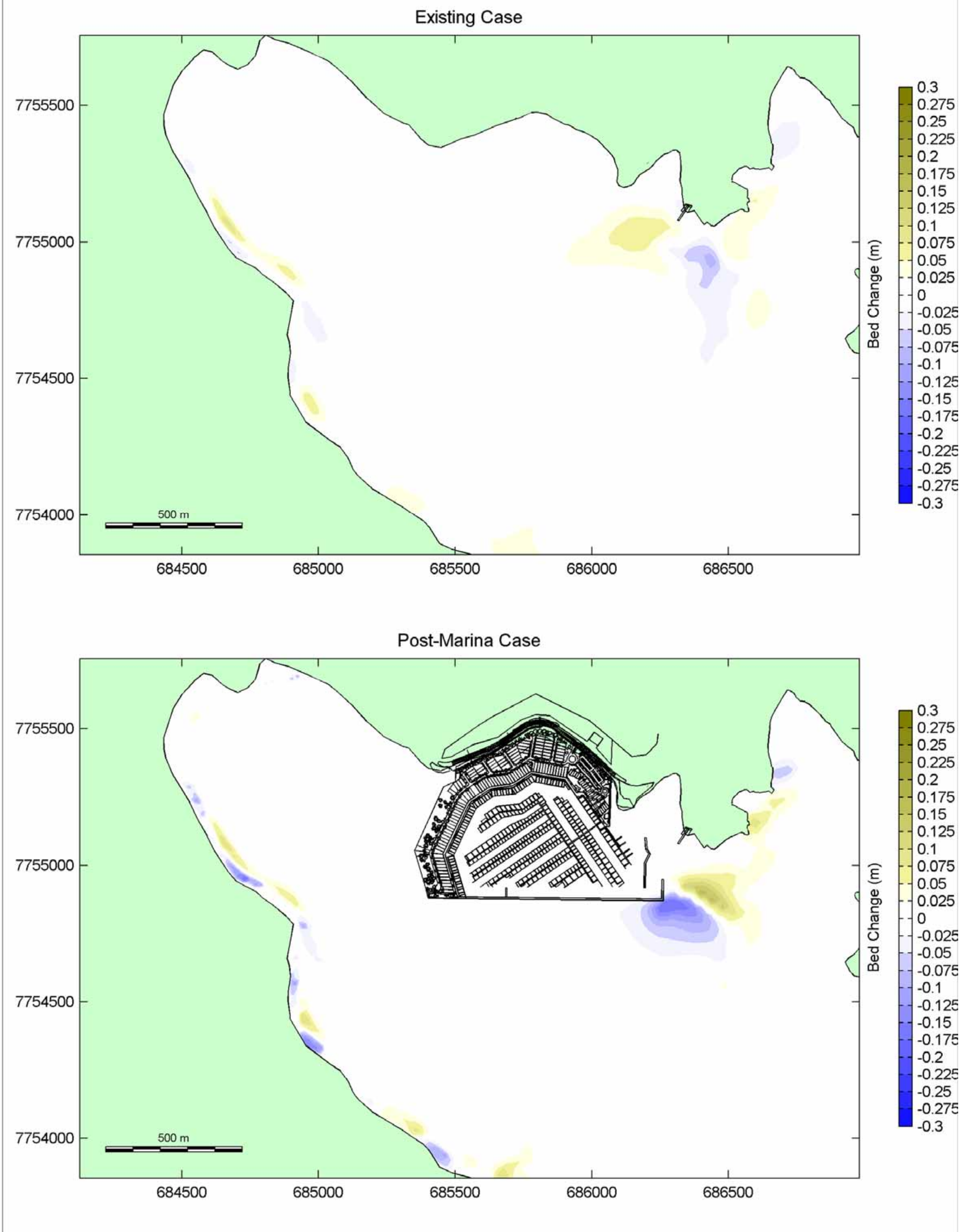
Figure 4.21



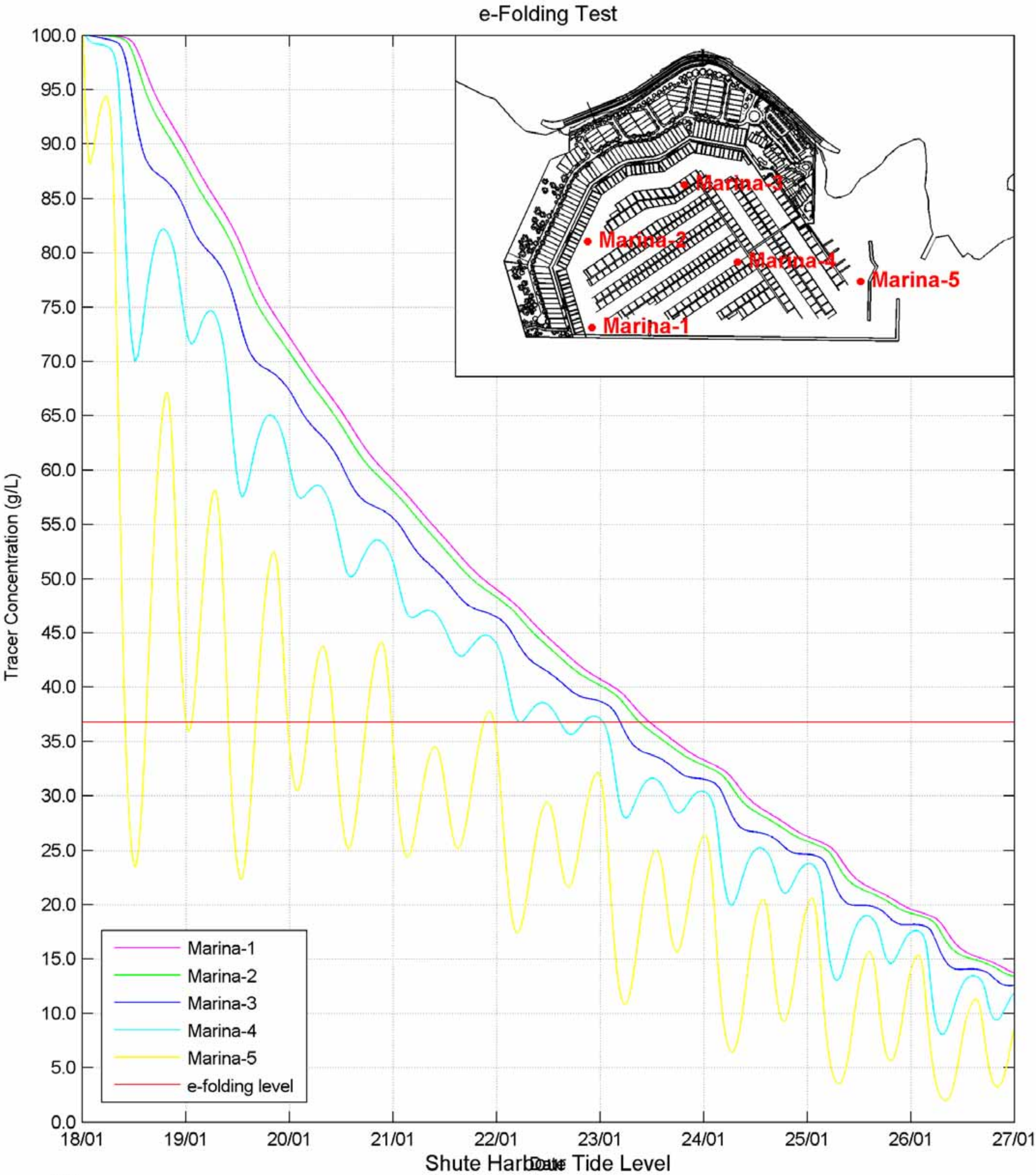


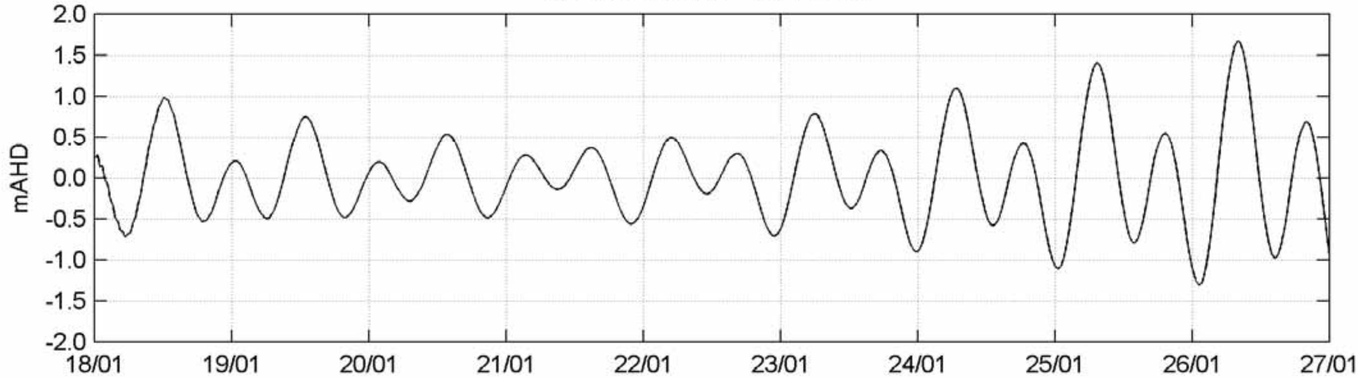
10-YEAR BATHYMETRY CHANGE - EXISTING

Figure 4.22



Shute Harbour Marina Development - Coastal Processes Issues
BED CHANGE DURING CYCLONE CONDITIONS
TROPICAL CYCLONE CELESTE HINDCAST
Figure 4.23





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Shute Harbour Marina Development - Coastal Processes Issues

e-FOLDING TIME

NEAP TIDE SCENARIO (18/1/06 - 27/1/06)

Figure 5.1

APPENDIX A

Masterplan (Revised, November, 2007)

APPENDIX B

Glossary of Terms

GLOSSARY*

Advective Transport	The transport of dissolved material by water movement.
Australian Height Datum (AHD)	A common national plane of level corresponding approximately to mean sea level.
Amenity	Those features of an estuary/beach that foster its use for various purposes, eg. Clear water and sandy beaches make beach-side recreation attractive.
ARI	Average Recurrence Interval
Bed Load	That portion of the total sediment load that flowing water moves along the bed by the rolling or saltating of sediment particles.
Calibration	The process by which the results of a computer model are brought to agreement with observed data.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
CD	Chart Datum, common datum for navigation charts - 0.92m below AHD in the Sydney coastal region. Typically Lowest Astronomical Tide.
Discharge	The rate of flow of water measured in terms of volume per unit time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is flowing.
Dispersive Transport	The transport of dissolved matter through the estuary by vertical, lateral and longitudinal mixing associated with velocity shear.
Diurnal	A daily variation, as in day and night.
Ebb Tide	The outgoing tidal movement of water within an estuary.
Eddies	Large, approximately circular, swirling movements of water, often metres or tens of metres across. Eddies are caused by shear between the flow and a boundary or by flow separation from a boundary.
EIS	Environmental Impact Statement
Estuarine Processes	Those processes that affect the physical, chemical and biological behaviour of an estuary, eg. predation, water movement, sediment movement, water quality, etc.

Estuary	An enclosed or semi-enclosed body of water having an open or intermittently open connection to coastal waters and in which water levels vary in a periodic fashion in response to ocean tides.
Flocculate	The coalescence, through physical and chemical processes, of individual suspended particles into larger particles ('flocs').
Flood Tide	The incoming tidal movement of water within an estuary.
Fluvial	Relating to non-tidal flows.
Fluvial Processes	The erosive and transport processes that deliver terrestrial sediment to creeks, rivers, estuaries and coastal waters.
Fluvial Sediments	Land-based sediments carried to estuarine waters by rivers.
Foreshore	The area of shore between low and high tide marks and land adjacent thereto.
Fortnightly Tides	The variation in tide levels caused by the monthly variation of Spring and Neap Tides.
Geomorphology	The study of the origin, characteristics and development of land forms.
H_s (Significant Wave Height)	H_s may be defined as the average of the highest 1/3 of wave heights in a wave record ($H_{1/3}$), or from the zeroth spectral moment (H_{mo}), though there is a difference of about 5 to 8%.
Hydraulic Regime	The variation of estuarine discharges in response to seasonal freshwater inflows and tides.
Intertidal	Pertaining to those areas of land covered by water at high tide, but exposed at low tide, eg. intertidal habitat.
Isohaline	A line connecting those parts of a water mass having the same salinity, that is, a contour of equal salinity levels.
Littoral Zone	An area of the coastline in which sediment movement by wave, current and wind action is prevalent.
Littoral Drift Processes	Wave, current and wind processes that facilitate the transport of water and sediments along a shoreline.
Mangroves	An intertidal plant community dominated by trees.
Marine Sediments	Sediments in sea and estuarine areas that have a marine origin.

Mathematical/ Computer Models	The mathematical representation of the physical processes involved in runoff, stream flow and estuarine/sea flows. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with wave and current processes.
MHL	Manly Hydraulics Laboratory
MSL	Mean Sea Level
Neap Tides	Tides with the smallest range in a monthly cycle. Neap tides occur when the sun and moon lie at right angles relative to the earth (the gravitational effects of the moon and sun act in opposition on the ocean).
NSW	New South Wales
NTU	Nephelometric Turbidity Units
Numerical Model	A mathematical representation of a physical, chemical or biological process of interest. Computers are often required to solve the underlying equations.
Phase Lag	Difference in time of the occurrence between high (or low water) and maximum flood (or ebb) velocity at some point in an estuary or sea area.
Salinity	The total mass of dissolved salts per unit mass of water. Seawater has a salinity of about 35g/kg or 35 parts per thousand.
Siltation	The movement of sediment particles along the bed of a water body in a series of 'hops' or 'jumps'. Turbulent fluctuations near the bed lift sediment particles off the bed and into the flow where they are carried a short distance before falling back to the bed.
Sediment Load	The quantity of sediment moved past a particular cross-section in a specified time by estuarine flow.
Semi-diurnal	A twice-daily variation, eg. two high waters per day.
Shear Strength	The capacity of the bed sediments to resist shear stresses caused by flowing water without the movement of bed sediments. The shear strength of the bed depends upon bed material, degree of compaction, armouring,
Shear Stress	The stress exerted on the bed of an estuary by flowing water. The faster the velocity of flow the greater the shear stress.
Shoals	Shallow areas in an estuary created by the deposition and build-up of sediments.

Slack Water	The period of still water before the flood tide begins to ebb (high water slack) or the ebb tide begins to flood (low water slack).
Spring Tides	Tides with the greatest range in a monthly cycle, which occur when the sun, moon and earth are in alignment (the gravitational effects of the moon and sun act in concert on the ocean)
SS	Suspended Solids
Storm Surge	The increase in coastal water levels caused by the barometric and wind set-up effects of storms. Barometric set-up refers to the increase in coastal water levels associated with the lower atmospheric pressures characteristic of storms. Wind set-up refers to the increase in coastal water levels caused by an onshore wind driving water shorewards and piling it up against the coast.
Suspended Sediment Load	That portion of the total sediment load held in suspension by turbulent velocity fluctuations and transported by flowing water.
Tidal Amplification	The increase in the tidal range at upstream locations caused by the tidal resonance of the estuarine water body, or by a narrowing of the estuary channel.
Tidal Exchange	The proportion of the tidal prism that is flushed away and replaced with 'fresh' coastal water each tide cycle.
Tidal Excursion	The distance travelled by a water particle from low water slack to high water slack and vice versa.
Tidal Lag	The delay between the state of the tide at the estuary mouth (eg. high water slack) and the same state of tide at an upstream location.
Tidal Limit	The most upstream location where a tidal rise and fall of water levels is discernible. The location of the tidal limit changes with freshwater inflows and tidal range.
Tidal Planes	A series of water levels that define standard tides, eg. 'Mean High Water Spring' (MHWS) refers to the average high water level of Spring Tides.
Tidal Prism	The total volume of water moving past a fixed point in an estuary during each flood tide or ebb tide.
Tidal Propagation	The movement of the tidal wave into and out of an estuary.
Tidal Range	The difference between successive high water and low water levels. Tidal range is maximum during Spring Tides and minimum during Neap Tides.

Tidally Varying Models	Numerical models that predict estuarine behaviour within a tidal cycle, that is, the temporal resolution is of the order of minutes or hours.
Tides	The regular rise and fall in sea level in response to the gravitational attraction of the Sun, Moon and Earth.
Tributary	Catchment, stream or river which flows into a larger river, lake or water body
Training Walls	Walls constructed at the entrances of estuaries to improve navigability by providing a persistently open entrance.
Turbidity	A measure of the ability of water to absorb light.
T_z (Zero Crossing Period)	The average period of waves in a train of waves observed at a location.
Velocity Shear	The differential movement of neighbouring parcels of water brought about by frictional resistance within the flow, or at a boundary. Velocity shear causes dispersive mixing, the greater the shear (velocity gradient), the greater the mixing.
Wind Shear	The stress exerted on the water's surface by wind blowing over the water. Wind shear causes the water to pile up against downwind shores and generates secondary currents.

* A number of definitions have been derived from the Estuary Management Manual (1992).

APPENDIX C

Physical Processes

APPENDIX C - PHYSICAL PROCESSES

1. General

The purpose of this section is to describe the physical processes that are important to the overall physiography of Shute Harbour and ongoing changes. These processes are: -

- Waves
- Currents
- Water Levels
- Winds
- Sediment Transport

A glossary of terms is presented in Appendix B.

2. Wave Processes

Ocean waves may have energy in two distinct frequency bands. These are principally related to the generation and propagation of ocean swell and local sea. Large waves generated by a storm are generally categorised as sea because wind energy is still being transferred to the ocean. Within Shute Harbour swell is typically blocked by the complex offshore island and bathymetric features and therefore only local sea is of concern.

Ocean waves are irregular in height and period and so it is necessary to describe wave conditions using a range of statistical parameters. In this study the following have been used: -

- H_{mo} – significant wave height (H_s) based on where m_o is the zeroth moment of the wave energy spectrum (rather than the time domain $H_{1/3}$ parameter).

- H_{max} – maximum wave height in a specified time period

- T_p – wave energy spectral peak period, that is, the wave period related to the highest ordinate in the wave energy spectrum

- T_z – average zero crossing period based on upward zero crossings of the still water line. An alternative definition is based on the zeroth and second spectral moments.

Wave heights defined by zero upcrossings of the still water line fulfil the Rayleigh Distribution in deep water and thereby provide a basis for estimating other wave height parameters from H_s. In shallow water, that is, near landfall locations, significant wave height defined from the wave spectrum, H_{mo}, is normally larger (typically 5% to 8%) than $H_{1/3}$ defined from a time series analysis.

Ocean waves also have a dominant direction of wave propagation and directional spread about that direction that can be defined by a Gaussian or generalised cosine ($\cos^n$) distribution (amongst others), and a wave grouping tendency. Directional spread is reduced by refraction as waves propagate into the shallow, nearshore regions and the wave crests become more parallel with each other and the seabed contours. Although neither of these characteristics is addressed explicitly in this study, directional spreading was included in the numerical wave modelling work. Directional spreading causes the sea surface to have a more short-crested wave structure in deep water, becoming more long-crested near the shoreline.

Waves propagating into shallow water may undergo changes caused by refraction, shoaling, bed friction, wave breaking and, to some extent, diffraction.

Wave refraction is caused by differential wave propagation speeds. That part of the shoreward propagating wave that is in the more shallow water has a lower speed than those parts in deeper water. When waves approach a coastline obliquely these differences cause the wave fronts to turn and become more coast parallel. There are changes in wave heights associated with this directional change. On irregular seabeds wave refraction becomes a very complex process. This process also causes shoreward propagating waves to develop a more long-crested appearance.

Waves propagating shoreward develop reduced speeds in shallow water. In order to maintain constancy of wave energy flux (ignoring energy dissipation processes) their heights must increase. This phenomenon is termed shoaling and leads to a significant increase in wave height near the shoreline.

A turbulent boundary layer forms above the seabed with associated wave energy losses that are manifested as a continual reduction in wave height in the direction of wave propagation - leaving aside further wind input, refraction, shoaling and wave breaking. The rate of energy dissipation increases with greater wave height and more shallow water.

Wave breaking occurs in shallow water when the wave crest speed becomes greater than the wave phase speed. For irregular waves this breaking occurs in different depths so that there is a breaker zone rather than a breaker line. Seabed slope, wave period and water depth are important parameters affecting the wave breaking phenomenon. As a consequence of this energy dissipation, wave set-up (a rise in still water level caused by wave breaking), develops shoreward from the breaker zone in order to maintain conservation of momentum flux. This rise in water level increases non-linearly in the shoreward direction and allows larger waves to propagate shoreward before breaking. Field measurements have shown that the slope of the water surface is normally concave upward. Wave set-up at the shoreline can be in the order of 15% of the equivalent deep-water significant wave height. Less set-up occurs in estuarine entrances, but the momentum flux remains the same. Wave set-up is smaller where waves approach a beach obliquely, but then a long shore current can be developed. Wave grouping and the consequent surf beats also cause fluctuations in the still water level.

In a random wave field each wave may be considered to have a period different from its predecessors and successors and the distribution of wave energy is often described by a wave energy spectrum. In fact, the whole wave train structure changes continuously and individual waves appear and disappear until quite shallow water is reached and dispersive processes are reduced. In developed sea states, that is swell, the Bretschneider modified Pierson-Moskowitz spectral form has generally been found to provide a realistic wave energy description. For developing sea states the JONSWAP spectral form, which is generally more 'peaky', has been found to provide a better spectral description.

For structural design in the marine environment it is necessary to define the H_{max} parameter related to storms having average recurrence intervals (ARI) of R years. However, the expected H_{max}, relative to H_s in statistically stationary wave conditions, increases as storm/sea state duration increases. Based on the Rayleigh Distribution the usual relationship is: -

$$H_{max} = H_s \sqrt{0.5 LnN_z}$$

where is the number of waves occurring during the time period being considered, where individual waves are defined by.

ln is the natural logarithm

This relationship has been found to overestimate H_{max} by about 10% in severe ocean storms. In shallow water the relationship is not fulfilled. In very shallow water H_{max} is replaced by the breaking wave height, H_b.

Waves propagating through an area affected by a current field are caused to turn in the direction of the current. The extent of this direction change depends on wave celerity, current speed and relative directions. Wave height is also changed. Opposing currents cause wavelengths to shorten and wave heights to increase and may lead to wave breaking. When the opposing current speed is greater than one quarter of the phase speed, the waves are blocked. Conversely, a following current reduces wave heights and extends wavelengths. This point has relevance to the potential propagation of storm waves into the lagoon entrance near peak flood outflow and following break-out of the berm.

3. Currents

Currents within the study site are caused by a range of phenomena, including: -

- Astronomical Tides
- Winds
- Coastal Trapped Waves and Other Tasman Sea Processes
- Nearshore Wave Processes
- Density Flows

The astronomical tides are caused by the relative motions of the Earth, Moon and Sun, see Section 4.4. The regular rise and fall of the tide level in the sea causes a periodic inflow (flood tide) and outflow (ebb tide) of oceanic water to the Bay and mixed oceanic and bay/river water from the Bay to the sea. A consequence of this process is the generation of tidal currents. The volume of sea water that enters the Bay or leaves the Bay on flood and ebb tides, respectively, is termed the tidal prism; which parameter varies due to the inequality between tidal ranges. The tidal prism is affected by changes in inter-tidal areas, such reclamations, but not by dredged areas below low tide, such as navigation channels and trenches.

Wind forcing is applied to the water surface as interfacial shear, the drag coefficient and consequent drag force varying with wind speed. Momentum from the wind is gradually transferred down through the water column by vorticity, the maximum depth of this effect being termed the Ekman depth. At the surface, wind caused currents are in the direction of the wind, but in the southern hemisphere they gradually turn to the left of the wind direction until they flow in the opposite direction at the Ekman depth. Shute Bay is too shallow for this condition to develop fully and wind driven currents are affected by the seabed boundary layer. Wind driven currents diminish with depth. Because wind forcing is applied at the water surface, the relative effect is greater in shallow water where there is less water column volume per unit plan area. Therefore wind driven currents are greater in more shallow areas. Maximum surface current speed is in the order of 1% to 3% of the wind speed, depending on water depth. Where water is piled up against a coastline by wind forcing, a reverse flow develops near the seabed.

Density currents may be caused by freshwater inflows, for example, when the Georges River is in flood. The freshwater is more buoyant and tends to spread across the Bay surface until mixing with the ambient seawater occurs; however due to the relatively small catchment size versus the shape and volume of Shute Bay, density currents are not likely to occur.

Coastal Trapped Waves (CTW) are long period wave phenomena that propagate northward along the continental shelf, Freeland et al, 1986. Their origin is not fully understood, but they are believed to originate from the passage of successive high and low pressure meteorological systems across southern Australia. These systems have inter-arrival times varying from 3 to 7 days, typically, and these are the periods of the observed CTW. These waves are irregular and cause approximate coast parallel currents and variations in water levels. They are trapped on the continental shelf by refraction and the Coriolis force. CTW are known to occur on the continental shelf of Queensland and may affect observed water levels in the Shute Harbour region.

The propagation of ocean waves (swell) into the nearshore region leads to wave breaking and energy dissipation. Where waves propagate obliquely to the shoreline this process leads to the generation of a long shore current in the surf zone, and to some extent seaward of that line. These currents are of some importance to shoreline processes in the Bay generally. Wave breaking and subsequent wave run-up are discussed further in Section 3.4.

4. Water Levels

Water level variations in the Bay and at the coastline result from one or more of the following natural causes:-

- Eustatic and Tectonic Changes
- Tides
- Wind Set-up and the Inverse Barometer Effect
- Wave Set-up
- Wave Run-up
- Fresh Water Flow
- Tsunamis
- Greenhouse Effect
- Global Changes in Meteorological Conditions

Eustatic sea level changes are long term world wide changes in sea level relative to the land mass and are generally caused by changes to the polar ice caps. No rapid changes are believed to be occurring at present and this aspect has not been addressed. Nevertheless, a minimum rise of 1mm per annum is now generally accepted. Tectonic changes are caused by movement of the Earth's crust; they may be vertical and/or horizontal

Tides are caused by the relative motions of the Earth, Moon and Sun and their gravitational attractions. While the vertical tidal fluctuations are generated as a result of these forces, the distribution of land masses, bathymetric variation and the Coriolis force determine the local tidal characteristics.

Wind setup and the inverse barometer effect are caused by regional meteorological conditions. When the wind blows over an open body of water, drag forces develop between the air and the water surface. These drag forces are proportional to the square of the wind speed. The result is that a wind drift current is generated. This current may transport water towards the coast upon which it piles up causing wind set-up. Wind set-up is inversely proportional to depth.

In addition, the drop in atmospheric pressure, which accompanies severe meteorological events, causes water to flow from high pressure areas on the periphery of the meteorological formation to the low pressure area. This is called the 'inverse barometer effect' and results in water level increases up to 1cm for each hecta-Pascal (hPa) drop in central pressure below the average sea level atmospheric pressure in the area for the particular time of year, typically about 1010 hPa. The actual increase depends on the speed of the meteorological system and 1cm is only achieved if it is moving slowly. The phenomenon causes daily variations from predicted tide levels up to 0.05m. The combined result of wind set-up and the inverse barometer effect is called storm surge.

Wave run-up is the vertical distance between the maximum height a wave runs up the beach or a coastal structure and the still water level, comprising tide plus storm surge. Additionally, run-up level varies with surf-beat, which arises from wave grouping effects.

Tsunamis are caused by sudden crustal movements of the earth and are commonly, but incorrectly, called 'tidal waves'. They are very infrequent and unlikely to occur during a storm and so have not been included in this study. Nevertheless, in the context of events having recurrence intervals in the order of 100 years, one should keep this point in mind.

Global meteorological and oceanographic changes, such as the El Nino Southern Oscillation phenomenon in the eastern southern Pacific Ocean, and continental shelf waves, cause medium term variations in mean sea level. The former phenomenon may persist for a year or more. The causes are not properly understood, but analyses of long term data from Australian tide gauges indicate that annual mean sea level may vary up to 0.1m from the long term trend, whilst mean sea level may vary by more than 0.2m over the time scale of weeks as a result of coastal trapped wave activity.

Many scientists believe that global warming of the Earth's atmosphere will lead to a rise in mean sea level. Predictions of global sea level rise due to the Greenhouse effect vary considerably. It is impossible to state conclusively by how much the sea may rise, and no policy yet exists regarding the appropriate provision that should be made in the design of new coastal developments.

Based on a number of global greenhouse models, a guide to future ocean level rises is presented in Table C1.

	Sea Level Rise (m) to Year 2100		
Greenhouse Scenario	**Min**	**Max**	**Central**
IP92a	0.11	0.77	0.44
SRES	0.09	0.88	0.48

Table C.10-2: Predicted Greenhouse Related Mean Sea Level Rises (IPCC, 2001)

Other recent investigations undertaken by CSIRO (1998) advise a mean sea level rise of 0.2m over the 50-years period from 1998 for the Queensland coastline. Investigation of the Australia State of the Environment Report 2001 web-site advises a mean sea level rise of 0.09m to 0.88m by 2100. Thus there is considerable uncertainty in this parameter estimate. An increase in storm tide level of 0.3m has been adopted by CLT for this investigation and included in wave force calculations to allow for potential climate change effects over a 50-years planning period.

Tides in Shute Bay are semi-diurnal, that is, there are two high and two low tides each day, normally. On rare occasions there may be only one high or low tide because the lunar tidal constituents have a period of about 25 hours. There may also be a significant diurnal difference, that is, a significant difference between successive high tides and successive low tides.

Tidal Plane	Water Level	
	m LAT	**m AHD**
Highest Astronomical Tide (LAT)	4.3	2.4
Mean High Water Springs (MHWS)	3.3	1.4
Mean High Water Neaps (MHWN)	2.5	0.6
Mean Sea Level (MSL)	1.9	0
Mean Low Water Neaps (MLWN)	1.2	-0.7
Mean Low Water Springs (MLWS)	0.5	-1.4
Lower Astronomical Tide (LAT)	0.0	-1.9

Table C.10-3 Tidal Planes for Shute Harbour

Table C.3 presents extreme water levels for typical Average Recurrence Intervals (ARI), derived from Storm tide modelling of the Whitsunday Coast (SEA1 GHD, 2003). These levels exclude wave setup and potential effects due to climate change and relate to locations seaward of the breaker zone.

Average Recurrence Interval (years)	Water Level	
	m LAT	m AHD
20	4.4	2.5
50	4.7	2.8
100	4.9	3.0

Table C.10-4: Extreme Water Levels at Shute Harbour
(Excluding Potential Climate Change Effects)

5. Winds

Wind affects both the wave and current climates in Shute Bay and Rooper Inlet. Wind data for this study was obtained from Hamilton Island, the nearest available reliable site.

6. Sediment Transport - Coastal Stability

The nearshore and shoreline regions of Shute Bay are formed from marine silty-sands and rocky headlands, with some muddy areas in the more sheltered regions such as north-west Shute Bay.

Sediment transport is caused by the water particle motions of waves and currents that lead to a shear stress on the seabed sediment particles. In some parts of the Shute Bay region waves and currents cause combined shear stresses. Generally, sediment motion commences when the seabed shear stress exceeds a threshold value, which depends on particle size and density. Sediment may be transported as bed load or suspended load. Bed load transport is affected as a series of siltations or hops. Suspended sediment transport occurs when the turbulent mixing of the flow counteracts the fall velocity of the finer sediment particles that disperse upward from the seabed.

Where a seabed is disturbed, for example, by dredging, and where the threshold condition for sediment movement is exceeded, wave and current caused sediment transport may act to restore the pre-condition of the seabed. In terms of the proposed works, any changes to the seabed or shoreline would tend to reverse, where the works are exposed to these sediment transport processes.

At shoreline locations sediment transport may be alongshore and/or onshore/offshore. Where waves break obliquely to the shoreline, a longshore current may cause longshore transport. Offshore transport normally occurs during a storm, with a longer term onshore transport following storm abatement. However, post-storm onshore transport may not occur in very low wave energy regions, which characterised by a flat inter-tidal area with a steep drop-off near the low tide line.

Waterways that enter Shute Bay may transport fine silt particles from the catchments to the Bay. These fine particles eventually settle in the most sheltered regions of the Bay, or leave the Bay to sea.

APPENDIX D

Calculation of Design Wave Forces – Vertical Wall

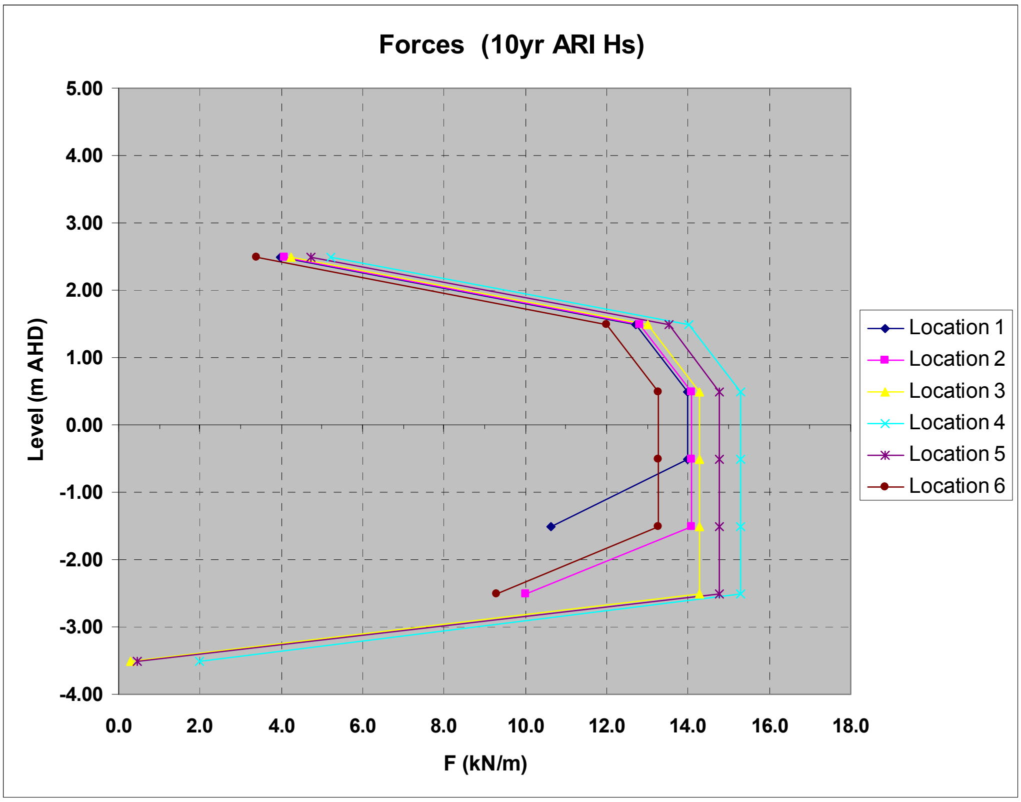
D – I) 10 years ARI

Table D-I-1: Maximum wave loads on vertical wall 1m blocks for 10-years ARI wave conditions, under wave crest and wave trough – wall crest at 5 m AHD.

CREST	FORCE (kN/m) – 10-years ARI					
Force Level (m AHD)	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
3.50						
2.50	4.0	4.1	4.3	5.2	4.7	3.4
1.50	12.7	12.8	13.0	14.0	13.5	12.0
0.50	14.0	14.1	14.3	15.3	14.8	13.3
-0.50	14.0	14.1	14.3	15.3	14.8	13.3
-1.50	10.6	14.1	14.3	15.3	14.8	13.3
-2.50		10.0	14.3	15.3	14.8	9.3
-3.50			0.3	2.0	0.4	

TROUGH	FORCE (kN/m) – 10-years ARI					
Force Level (m AHD)	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
3.50						
2.50						
1.50	-1.3	-1.3	-1.3	-1.3	-1.3	-1.3
0.50	-9.9	-9.9	-9.9	-10.0	-10.0	-9.7
-0.50	-13.3	-13.4	-13.6	-14.5	-14.1	-12.6
-1.50	-10.1	-13.4	-13.6	-14.5	-14.1	-12.6
-2.50		-9.5	-13.6	-14.5	-14.1	-8.8
-3.50			-0.3	-1.9	-0.4	

Figure D-I-1: Forces on sheet-piled wall (5 m AHD) for the 10-years ARI significant wave height.



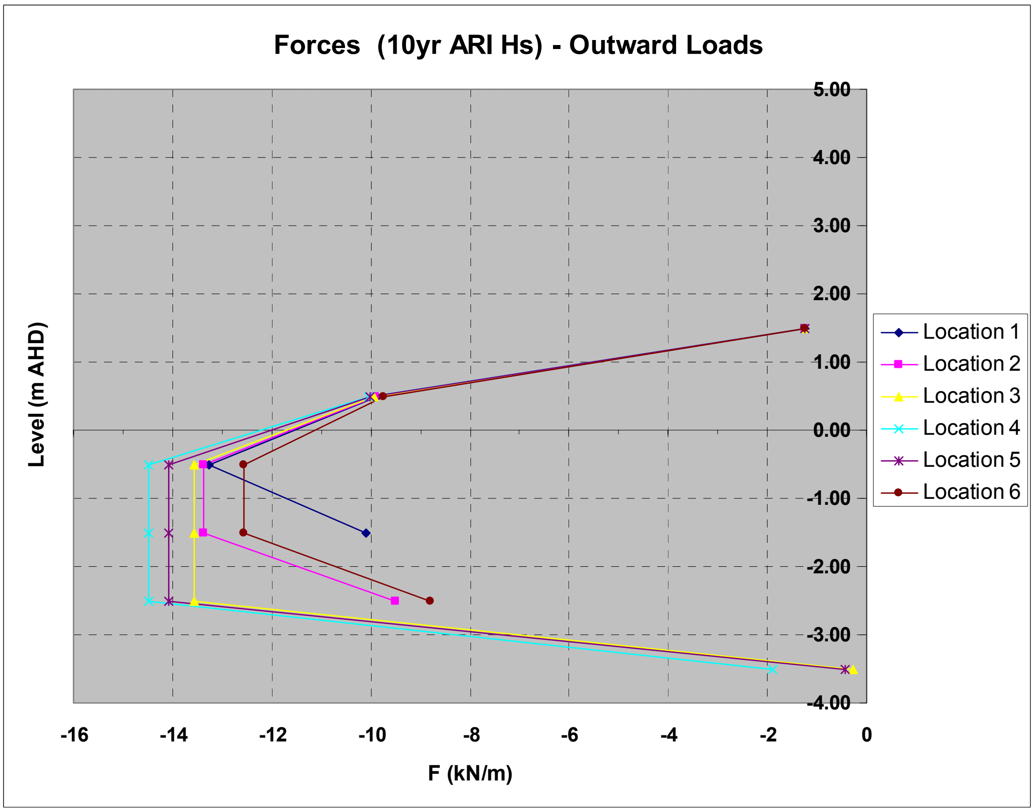


Table D-I-2: Wave Load Forces, Moment and Moment Arm height for 10-years ARI significant wave conditions – wall crest at 5m AHD.

Wave Loads - Crest (10 years ARI)

	Depth (m MSL)	Total Depth (m)	Structure Height (m)	H_s 10yrs ARI (m)	$H_{1/100}$ (m)	Q (m^3/s)	Y (m)	F (kN/m)	M (kN.m/m)	Moment Arm Z (m above seabed)
Location 1	1.76	3.26	6.76	0.83	1.39	0.00	0.00	35.19	70.22	2.00
Location 2	2.71	4.21	7.71	0.84	1.40	0.00	0.00	44.00	108.86	2.47
Location 3	3.02	4.52	8.02	0.85	1.42	0.00	0.00	47.54	125.07	2.63
Location 4	3.13	4.63	8.13	0.91	1.52	0.00	0.00	52.45	142.27	2.71
Location 5	3.03	4.53	8.03	0.88	1.47	0.00	0.00	49.55	131.26	2.65
Location 6	2.70	4.20	7.70	0.79	1.32	0.00	0.00	41.07	100.38	2.44

Wave Loads - Trough (10 years ARI)

	Depth (m MSL)	Total Depth (m)	Structure Height (m)	H_s 10yrs ARI (m)	$H_{1/100}$ (m)	H_r (m)	F (kN/m)	M (kN.m/m)	Moment Arm Z (m above seabed)
Location 1	1.76	3.26	6.76	0.83	1.39	1.25	-21.97	-29.16	1.33
Location 2	2.71	4.21	7.71	0.84	1.40	1.26	-30.18	-54.15	1.79
Location 3	3.02	4.52	8.02	0.85	1.42	1.28	-33.23	-64.52	1.94
Location 4	3.13	4.63	8.13	0.91	1.52	1.37	-36.04	-71.30	1.98
Location 5	3.03	4.53	8.03	0.88	1.47	1.32	-34.33	-66.42	1.94
Location 6	2.70	4.20	7.70	0.79	1.32	1.19	-28.61	-51.69	1.81

H_r = reflected wave height

D – II) 100 years ARI

Table D-II-1 Transmitted significant wave height (H_t) into the marina for 100-yrs ARI significant wave height

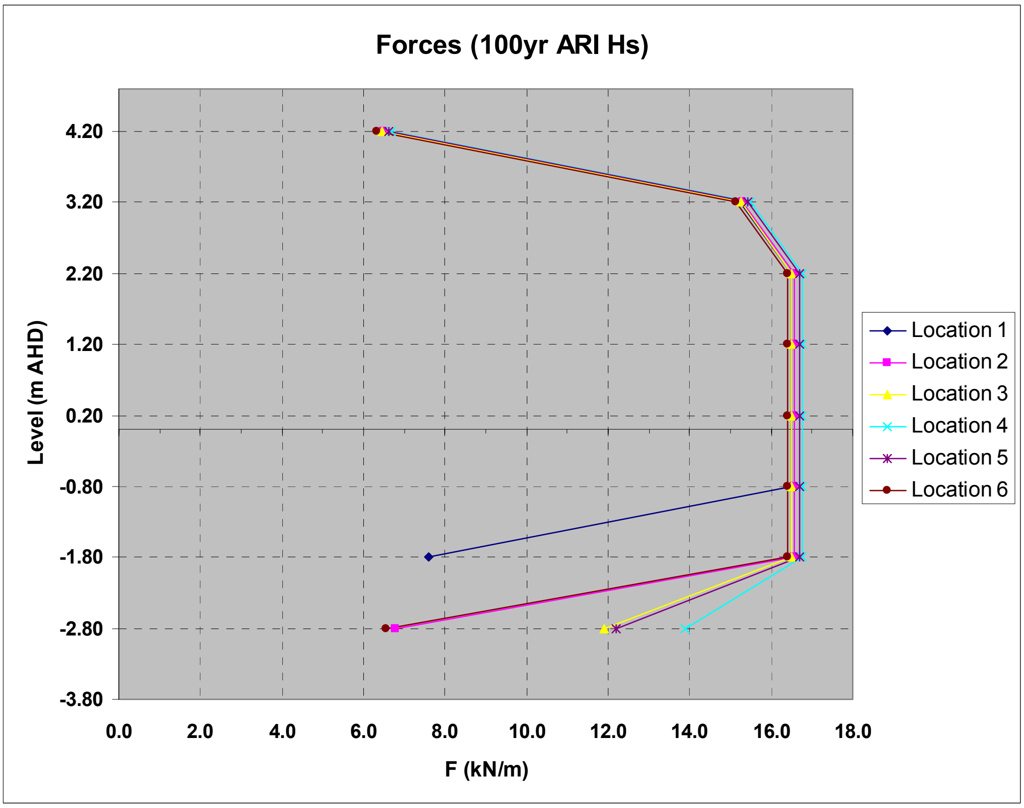
	Location	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
	Hs 100-yrs ARI (m)	1.65	1.68	1.66	1.78	1.75	1.6
	Structure Crest Level (m AHD)						
Ht (m)	5.0	0.02	0.03	0.02	0.07	0.06	0.00
	4.7	0.12	0.13	0.12	0.17	0.15	0.10
	4.5	0.18	0.19	0.19	0.23	0.22	0.16
	4.0	0.35	0.36	0.35	0.39	0.38	0.33
	3.5	0.51	0.52	0.51	0.56	0.55	0.49

Table D-II-2: Maximum wave loads on vertical wall 1m blocks for 100-years ARI wave conditions, under wave crest and wave trough – wall crest at 4.7m AHD.

CREST	FORCE (kN/m) – 100-years ARI					
Force Level (m AHD)	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
4.20	6.4	6.5	6.5	6.7	6.6	6.3
3.20	15.2	15.3	15.3	15.5	15.4	15.1
2.20	16.5	16.6	16.5	16.8	16.7	16.4
1.20	16.5	16.6	16.5	16.8	16.7	16.4
0.20	16.5	16.6	16.5	16.8	16.7	16.4
-0.80	16.5	16.6	16.5	16.8	16.7	16.4
-1.80	7.6	16.6	16.5	16.8	16.7	16.4
-2.80		6.8	11.9	13.9	12.2	6.6

TROUGH	FORCE (kN/m) – 100-years ARI					
Force Level (m AHD)	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
4.20						
3.20	-1.3	-1.3	-1.3	-1.3	-1.3	-1.3
2.20	-10.1	-10.1	-10.1	-10.1	-10.1	-10.1
1.20	-20.1	-20.1	-20.1	-20.1	-20.1	-20.1
0.20	-25.6	-26.0	-25.7	-27.3	-26.9	-24.8
-0.80	-25.6	-26.0	-25.7	-27.6	-27.1	-24.8
-1.80	-11.8	-26.0	-25.7	-27.6	-27.1	-24.8
-2.80		-10.7	-18.5	-22.9	-19.8	-9.9

Figure D-II-1: Forces on sheet-piled wall (4.7m AHD) for the 100-years ARI significant wave height.



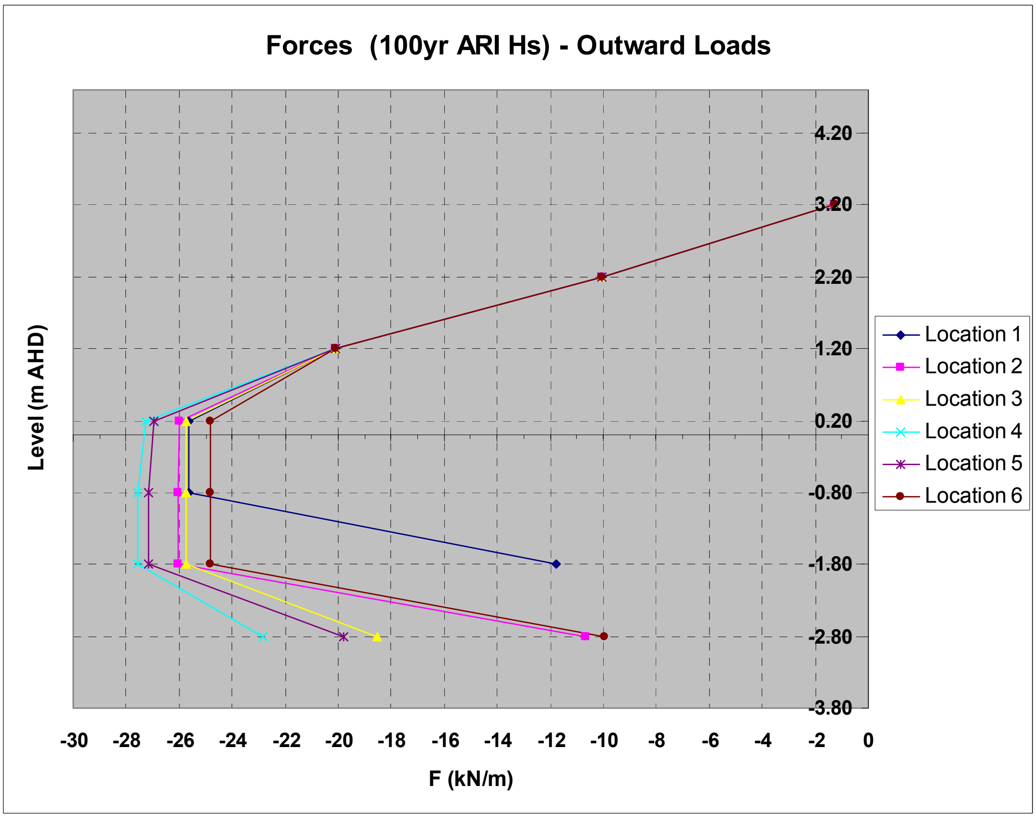


Table D-II-3: Wave Load Forces, Moment and Moment Arm height for 100-years ARI significant wave conditions – wall crest at 4.7m AHD.

Wave Loads - Crest (100 years ARI)

	Depth (m MSL)	Total Depth (m)	Structure Height (m)	H_s 100yrs ARI (m)	$H_{1/100}$ (m)	Q (m^3/s)	Y (m)	F (kN/m)	M (kN.m/m)	Moment Arm Z (m above seabed)
Location 1	1.76	4.96	6.46	1.65	2.76	0.17	0.14	60.64	176	2.91
Location 2	2.71	5.91	7.41	1.68	2.81	0.18	0.15	70.90	240	3.38
Location 3	3.02	6.22	7.72	1.66	2.77	0.17	0.14	73.94	261	3.53
Location 4	3.13	6.33	7.83	1.78	2.97	0.21	0.17	76.31	274	3.59
Location 5	3.03	6.23	7.73	1.75	2.92	0.20	0.16	74.93	265	3.54
Location 6	2.70	5.90	7.40	1.60	2.67	0.15	0.13	70.02	236	3.37

Wave Loads - Trough (100 years ARI)

	Depth (m MSL)	Total Depth (m)	Structure Height (m)	H_s 100yrs ARI (m)	$H_{1/100}$ (m)	H_r (m)	F (kN/m)	M (kN.m/m)	Moment Arm Z (m above seabed)
Location 1	1.76	4.96	6.46	1.65	2.76	2.34	-60.15	-115.25	1.92
Location 2	2.71	5.91	7.41	1.68	2.81	2.38	-76.52	-181.24	2.37
Location 3	3.02	6.22	7.72	1.66	2.77	2.36	-80.96	-204.49	2.53
Location 4	3.13	6.33	7.83	1.78	2.97	2.51	-87.00	-221.29	2.54
Location 5	3.03	6.23	7.73	1.75	2.92	2.47	-84.35	-211.03	2.50
Location 6	2.70	5.90	7.40	1.60	2.67	2.28	-73.76	-176.12	2.39

H_r = reflected wave height

D – III) 200-years ARI

Table D-III-1: Transmitted significant wave height (H_t) into the marina for 200-yrs ARI significant wave height

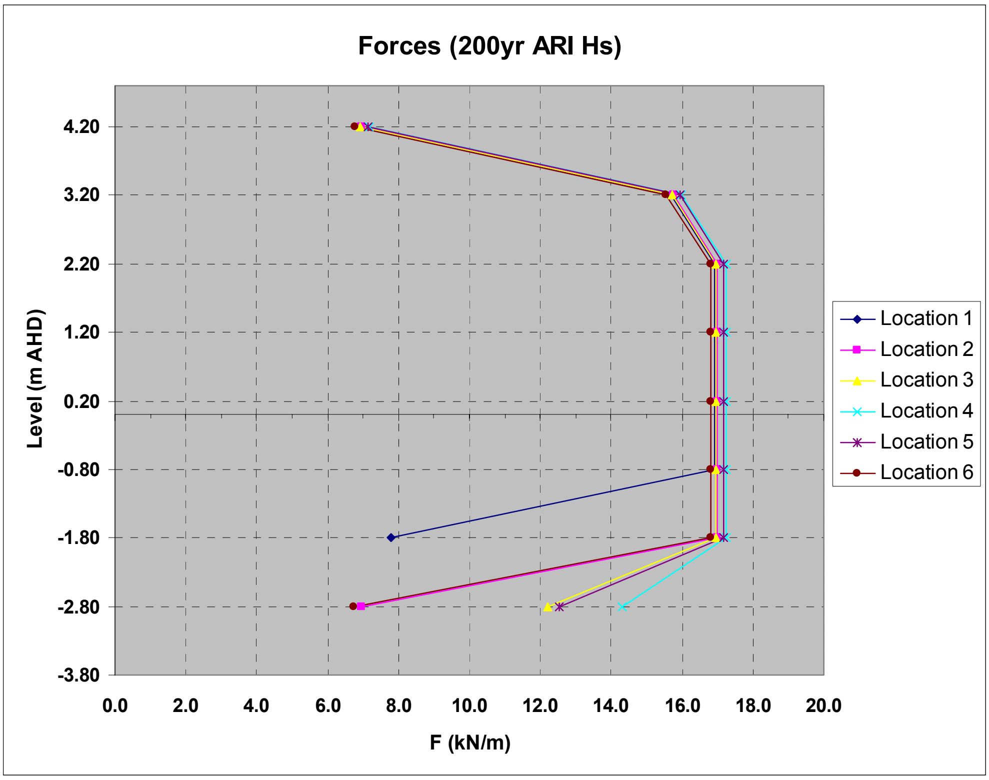
	Location	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
	Hs 200-yrs ARI (m)	**1.86**	**1.9**	**1.88**	**2**	**1.98**	**1.81**
	Structure Crest Level (m AHD)						
Ht (m)	5.0	0.10	0.11	0.10	0.15	0.14	0.08
	4.7	0.20	0.21	0.20	0.25	0.24	0.18
	4.5	0.26	0.28	0.27	0.31	0.31	0.24
	4.0	0.42	0.44	0.43	0.48	0.47	0.41
	3.5	0.59	0.60	0.60	0.64	0.63	0.57

Table D-III-2: Maximum wave loads on vertical wall 1m blocks for 200-years ARI wave conditions, under wave crest and wave trough – wall crest at 4.7m AHD.

CREST	FORCE (kN/m) – 200-years ARI					
Force Level (m AHD)	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
4.20	6.9	7.0	6.9	7.2	7.1	6.8
3.20	15.7	15.7	15.7	16.0	15.9	15.6
2.20	16.9	17.0	17.0	17.2	17.2	16.8
1.20	16.9	17.0	17.0	17.2	17.2	16.8
0.20	16.9	17.0	17.0	17.2	17.2	16.8
-0.80	16.9	17.0	17.0	17.2	17.2	16.8
-1.80	7.8	17.0	17.0	17.2	17.2	16.8
-2.80		7.0	12.2	14.3	12.5	6.7

TROUGH	FORCE (kN/m) – 200-years ARI					
Force Level (m AHD)	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
4.20						
3.20	-1.3	-1.3	-1.3	-1.3	-1.3	-1.3
2.20	-10.1	-10.1	-10.1	-10.1	-10.1	-10.1
1.20	-20.1	-20.1	-20.1	-20.1	-20.1	-20.1
0.20	-28.2	-28.5	-28.3	-29.2	-29.1	-27.6
-0.80	-28.9	-29.4	-29.1	-30.9	-30.6	-28.1
-1.80	-13.3	-29.4	-29.1	-30.9	-30.6	-28.1
-2.80		-12.0	-20.9	-25.6	-22.3	-11.2

Figure D-III-1: Forces on sheet-piled wall (4.7m AHD) for the 200-years ARI significant wave height.



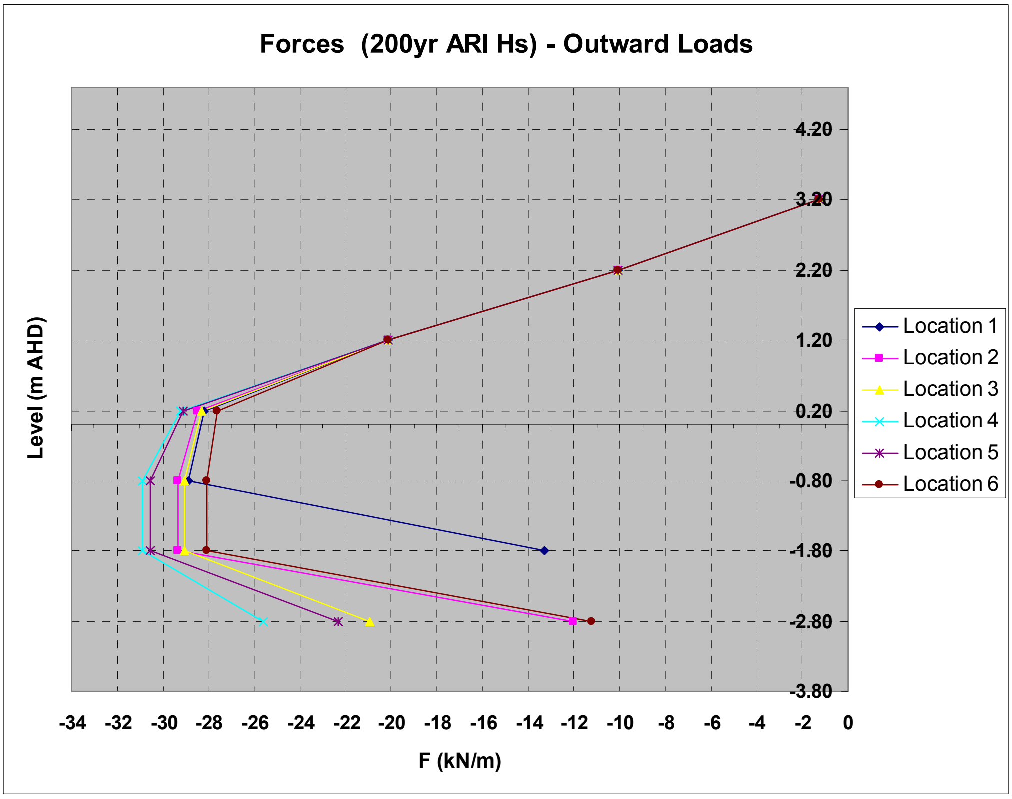


Table D-III-3: Wave Load Forces, Moment and Moment Arm height for 200-years ARI significant wave conditions – wall crest at 4.7m AHD.

Wave Loads - Crest (200 years ARI)

	Depth (m MSL)	Total Depth (m)	Structure Height (m)	H_s 200yrs ARI (m)	$H_{1/100}$ (m)	Q (m^3/s)	Y (m)	F (kN/m)	M (kN.m/m)	Moment Arm Z (m above seabed)
Location 1	1.76	4.96	6.46	1.86	3.11	0.25	0.18	62.42	182	2.92
Location 2	2.71	5.91	7.41	1.90	3.17	0.26	0.19	73.03	247	3.39
Location 3	3.02	6.22	7.72	1.88	3.14	0.25	0.19	76.19	270	3.54
Location 4	3.13	6.33	7.83	2.00	3.34	0.31	0.21	78.69	283	3.60
Location 5	3.03	6.23	7.73	1.98	3.31	0.30	0.21	77.40	275	3.55
Location 6	2.70	5.90	7.40	1.81	3.02	0.22	0.17	72.01	243	3.38

Wave Loads - Trough (200 years ARI)

	Depth (m MSL)	Total Depth (m)	Structure Height (m)	H_s 200yrs ARI (m)	$H_{1/100}$ (m)	H_r (m)	F (kN/m)	M (kN.m/m)	Moment Arm Z (m above seabed)
Location 1	1.76	4.96	6.46	1.86	3.11	2.62	-64.76	-120.43	1.86
Location 2	2.71	5.91	7.41	1.90	3.17	2.67	-83.18	-191.74	2.31
Location 3	3.02	6.22	7.72	1.88	3.14	2.65	-88.34	-217.38	2.46
Location 4	3.13	6.33	7.83	2.00	3.34	2.80	-94.24	-233.67	2.48
Location 5	3.03	6.23	7.73	1.98	3.31	2.78	-91.66	-223.41	2.44
Location 6	2.70	5.90	7.40	1.81	3.02	2.55	-80.46	-187.01	2.32

H_r = reflected wave height