Stockton Beach Coastal Processes Study

Final Report

Stage 1- Sediment Transport Analysis and Description of On-going Processes

Newcastle City Council

December 2006

Stockton Beach Coastal Processes Study

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1 EXECUTIVE SUMMARY

Newcastle City Council (NCC) in partnership with the NSW Government and the Newcastle Port Corporation engaged DHI Water & Environment (DHI) to undertake an analysis of the coastal erosion processes of Stockton Beach.

This is the final report in the Stockton Beach Coastal Processes Study, Stage I. It aims to provide a description of the on-going coastal processes as the background information to undertake the second and third stages that include the coastal management study and the Newcastle Coastline Management Plan Revision.

1.1 Background information

Stockton Beach is an extensive sand beach stretching from the northern side of the Hunter River at Newcastle to beyond the Council boundary. The beach as a recreational area is an important community asset for the Hunter Coast.

For many years the area has been prone to erosion therefore a number of studies have been carried out to assess the erosion problems. The most relevant studies are:

- Stockton Beach Coastal Engineering Advice – Dept. Public Works (1985);
- Stockton Beach Coastline Hazard Study, Dept. Land and Water Conservation (1995);
- Newcastle Coastline Hazard Definition Study, WBM Pty Ltd (1998);
- Shifting Sands at Stockton Beach, Umwelt Pty Ltd (2002); and
- Newcastle Coastline Management Study Umwelt Pty Ltd (2003).

These studies analysed the coastal processes based mainly on historical data. In 1998 a coastal hazard study was carried out and as a result of that study, coastal hazard lines were defined based on historical information and initial modelling results. As a result of some of these investigations the Mitchell Street rock seawall and later on a geotextile bag wall protecting the surf club were constructed. Both of these structures have limited the extent of recent erosion events and thereby protected assets from erosion. The construction of these control structures within the active beach zone has, however, significantly impacted on beach amenity and is not considered to provide a sustainable solution to the erosion problem in the long term.

The most recent study carried out by Umwelt (2003) suggested that based on the recent investigations the erosion problem is progressively worsening with a significant volume of sand having been lost from the beach and the nearshore zone over the last century. The coastal erosion threat at Stockton is therefore considered severe and the hazard risk will need to be further assessed as part of additional studies into the stability of Stockton Beach.

In light of this information, The Hunter Coast and Estuary Management Committee (HCEMC), that includes Newcastle City Council, the Department of Natural Resources (DNR), Newcastle Port Corporation (NPC) and the Newcastle Community, recognised

the need for a study update of previous investigations to provide input for a new coastal management strategy for the area.

1.2 Objectives of the Study

This study should provide a detailed qualitative and quantitative analysis of the sediment transport along the beaches to assist in the development of the revised plan of management. The study should conduct a detailed analysis of the causal factors of erosion, with a view to identifying and testing realistic options for future management of the problem. The main objectives of this study are to:

- Develop a comprehensive understanding of the processes driving the sediment transport and shoreline changes at Stockton Beach;
- Verify the sediment budgets predictions using available data;
- Quantify the sediment budget for the southern end of Stockton Beach;
- Review current hazard projections at the southern end of Stockton Beach;
- Identify and assess management options for the management of coastal hazards and beach amenity in accordance with the Coastline Management Manual and the NSW coastal policy;
- Consult with the community in developing preferred management options; and
- Revise the Newcastle Coastline Management Plan 2003, to incorporate the preferred management option, or combination of options, for Stockton Beach.

1.3 Scope of work and study approach

As stated in the project tender documents, the scope of work involves seven main components in three project stages.

Stage 1 - Process Study

- Data collection and analysis;
- Numerical modelling of waves and currents and the resulting sediment transport;
- Prediction of ongoing beach response/trends; and
- Definition of hazard lines.

Stage 2 – Management Study

- Identification and assessment of management options; and
- Presentation of options to the Hunter Coast and Estuary Management Committee and the community;

Stage 3 – Newcastle Coastline Management Plan Revision

- Revision of the Newcastle Coastline Management Plan 2003.

1.4 Stage I – Main Findings

A detailed analysis of the sediment transport conditions at Stockton has been carried out to determine the on-going processes at Stockton Beach. The analysis has been undertaken at a range of time scales including short, medium and long-term. The main findings of Stage I can be summarized:

1.4.1 Short Term Processes

A detailed analysis of the short-term erosion at Stockton Beach has been undertaken based on the application of a dune erosion model and modelled wave conditions in the Stockton nearshore areas.

The model was applied to the most severe storm events observed in the Newcastle area since 1974. The short-term predictions show an increase in dune erosion risk from south to north for the most frequent events from the south east, with a maximum predicted retreat of 24.5m at the Council boundary. For conditions from the E and NE the dune erosion is still severe but occurring more evenly along the beach. The effect of repetition of events was also investigated for the storm events of May and June 1974, which were one of the most severe observed on the NSW coast.

The influence of nearshore deepening on the short term dune erosion was also investigated. The analysis shows that an increase of the dune retreat is predicted with a further deepening of the nearshore/offshore areas. Therefore if an event such as the May- June 1974 storm occurs under the present conditions it is likely that the dune erosion rates would increase between 15 to 35% when compared to the 1974 conditions. It has been estimated that if no corrective measures are taken the dune erosion risk will further increase 5% for a further deepening of the nearshore areas of 1m.

1.4.2 Medium and Long- Term Processes

The medium and long term net littoral drift for the period 1992-2004 was investigated by applying a detailed 2-dimensional modelling approach. The results were compared to historical data between 1866 and 2000. The predictions show that the net transport is a combination of south and northwards events where the first are more episodic and caused by more infrequent NE-E events, while the latter are due to the more frequent SE swell conditions. A large variability is observed in the littoral transport with periods of large northwards transport and other periods where the littoral transport changes direction becoming predominantly southwards.

Medium term effects

The results indicate that there is a medium-term, inter-annual or decadal fluctuation between 1994-1999 and this is of similar magnitude or larger to what is observed in the mean long term conditions. These conditions therefore have to be considered in the definition of the hazard lines and the coastal protection alternatives.

Long-term analysis

A detailed two dimensional analysis of the littoral transport processes was undertaken on the 1992-2004 wave data, and compared to historical data for the period between 1866 and 2000. The results show that the wave induced longshore transport is the most significant sediment transport mechanism.

The long-term sediment transport in the Stockton area can be described as follows:

1. Net northward sediment transport. The results indicate an estimated value of 55,000m^3/yr in the area north of the sewage treatment plant ponds for the period

1992-2004 and should be considered as an estimate due to the uncertainties in the wave climate conditions;

2. For the period 1866-2004 a net northward sediment transport between 20,000 and 30,000 m^3/year is predicted;
3. The difference between the 1992-2004 and 1866-2004 values can be attributed mainly to the variability of the wave conditions;
4. A complex two dimensional sediment transport mechanism has been predicted and is presented in Figure 1-1;
5. The 2-D results show that the port structures tend to redirect the sediment transport travelling from the south into the deep offshore areas therefore obstructing the bypassing mechanism at the Hunter River entrance;
6. The sediment transport process around the study area is highly two dimensional and can be described as:
    a. The most frequent south-easterly waves diffract around the Port breakwaters;
    b. A nodal or neutral point (area where the sediment transport changes direction) is predicted at the northern end of the Mitchell St seawall. Here the sediment transport splits into two directions, southwards and northwards. It is predicted that this is the major eroding stretch in Stockton Beach;
    c. An increasing northward sediment transport occurs north of the Mitchell St seawall. As a result this area north of the seawall is eroding;
    d. There is a variation in the wave set-up along the coastline with the smallest set-up in the sheltered area north of the northern breakwater that induces an anticlockwise eddy (moving southwards along the coast). This mechanism drives a local current along the shoreline towards the area immediately north of the northern breakwater that induces slight accretion in this area;
    e. The incoming east and north-easterly waves will refract producing uniform southward longshore transport, and sediment accumulation north of the northern breakwater;
7. At the northern end of the seawall the beach tends to retreat but the seawall imposes a limitation therefore inducing a local steepening of the beach profile;
8. The river is not a significant source of sediment in the Stockton area. Suspended sediment delivered to the sea during floods is too fine to be retained on the beach; coarser catchment sands and gravels are trapped further upstream of the estuary;
9. A northwards transport at Nobbys Head has been predicted. The estimated littoral transport at this location is 33,000m^3/yr based on the sediment properties at Stockton. Due to its exposure to the incoming waves the sediment sizes at Nobbys Head may be larger and rocky outcrops may be present, therefore the computed littoral transport may be overestimated. No sediment bypassing is predicted into the southern end of Stockton Beach;
10. Nobbys Head generates a deposition zone north of the southern breakwater. Part of the sediment is deposited in the navigation channel and part is transported into the southern areas of the river entrance (Horseshoe Beach). The latter of these sediment transport mechanisms is caused by the effect of the NNE waves which break along the shallow eastern margin of the navigation channel;
11. A wave focussing mechanism north of the Mitchell St seawall has been predicted. This process has already been identified by vessels sailing in or out of

the Port. The wave focussing mechanism is produced when waves propagate from the S and SE and shoal at the southeast of the Port entrance. The results indicate that the focussing area is located from the treatment ponds area to the southern end of Fern Bay. It is expected that this process tends to exacerbate erosion in this area;

12. The area north of Fern Bay is expected to be in equilibrium as the beach orientation reaches the equilibrium angle; and
13. Erosion at the tip of the northern breakwater was indicated in the study of Umwelt (2002) that was reported to propagate back towards Stockton Beach with a drainage pathway from the nearshore zone of Stockton Beach to the entrance channel. This study does not indicate this pattern however it is acknowledged that localised erosion or scouring around the tip of the breakwater is possible.

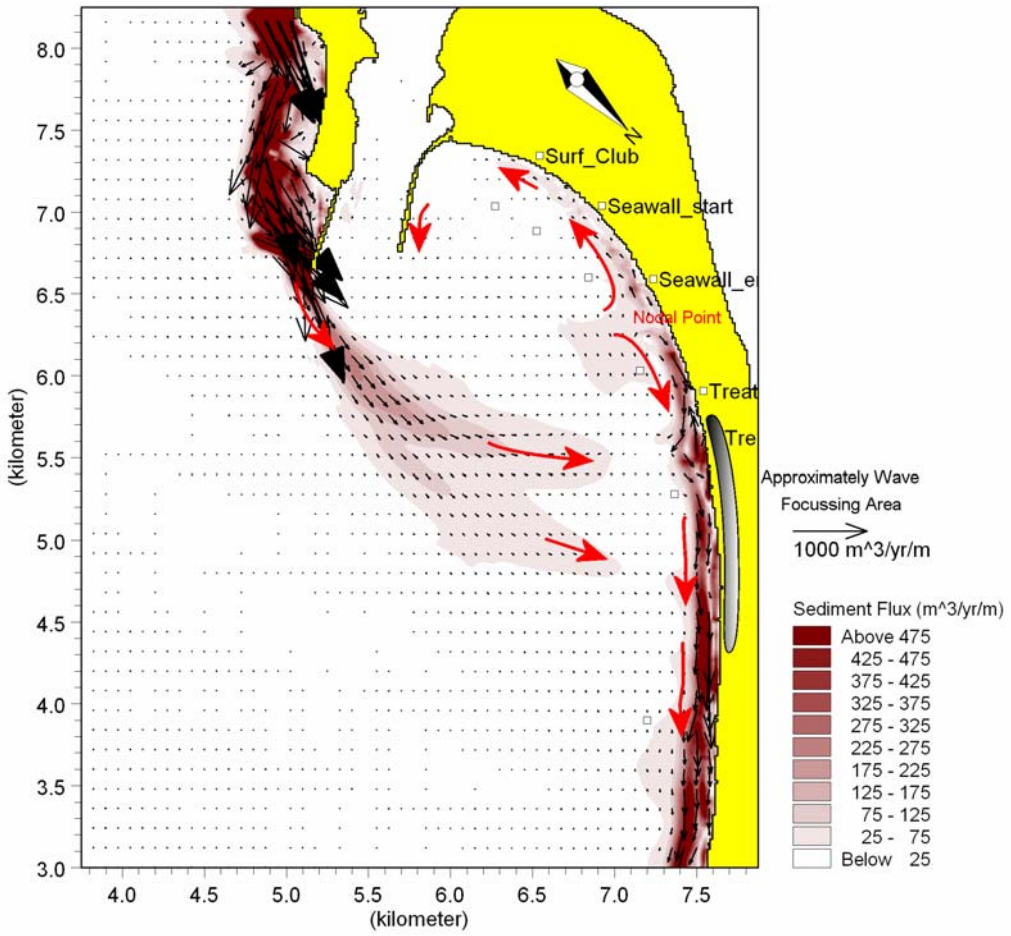


Figure 1-1 Predicted sediment transport mechanism at Stockton based on a typical yearly conditions from 1992-2005

1.5 Greenhouse Effects

The complexity of processes in the climate system means it is inappropriate to simply extrapolate past trends to forecast future conditions. To estimate future climate change, scientists have developed scenarios, which are not predictions but more of a "what if"

analysis. These scenarios provide an estimation of the likely sea-level rise. Based on Bruun's (1962) (1988) approximation the estimated retreat for 20 and 50 years is:

Table 1-1 Predicted beach retreat due or shoreline recession due to sea level rise for the next 20 and 50 years at Stockton (in metres).

Year	Low	Mid	High
2026	0.8	3.6	8.0
2056	3.2	14.0	25.6

2 INTRODUCTION

Newcastle City Council (NCC) in partnership with the NSW Government and the Newcastle Port Corporation engaged DHI Water & Environment (DHI) to undertake an analysis of the coastal erosion processes of Stockton Beach.

This is the final report in the Stockton Beach Coastal Process Study, Stage I. It aims to provide a description of the on-going coastal processes as vital background information in order to undertake the second and final stages of the study. These include the coastal management study and the Newcastle Coastline Management Plan Revision.

2.1 Background

Stockton Beach is an extensive sandy beach which extends from the northern side of the Hunter River at Newcastle to the Council boundary as presented in Figure 2-1. Due to its recreational value the beach is an important community asset for the Hunter Coast Region.

For many years the area has been prone to erosion and in response a number of studies have been carried out to assess these problems. These studies have mainly been based on historical data and are:

- Stockton Beach Coastal Engineering Advice - Public Works Dept, PWD (1985);
- Addendum to: Stockton Beach Coastal Engineering Advice - Public Works Dept, PWD (1987);
- Stockton Beach Coastline Hazard Study, Department of Land and Water Conservation, DLWC (1995);
- Newcastle Coastline Hazard Definition Study, WBM (1998);
- Shifting Sands at Stockton Beach , Umwelt (2002); and
- Newcastle Coastline Management Study Umwelt (2003).

In 1985 the Public Works Department, in response to a request from Newcastle City Council, undertook a coastal engineering assessment of Stockton Beach. This study comprised photogrammetric analyses to determine beach fluctuations, an assessment of coastal hazards and discussion of the scope of options that may be implemented to mitigate the problem. Based on the most recent information at that time it was found that a 500 metre section of Mitchell Street, north of Pembroke Street was at immediate threat from storm damage.

A recessional trend in the position of the dune escarpment was also observed in this area. Comparison of beach cross sections showed that there appeared to be little change in the beach profiles adjacent to Mitchell Street indicating that the long term change in beach sand reserves appeared to be small. .

Further north there has been a marked trend of erosion with an average long-term loss of $1.3m^3/m$ above mean sea level. Associated with this erosion, recession of the dune escarpment was observed, particularly at the northernmost profiles.

An addendum to this report was prepared in 1987, where additional information was examined by PWD (1987). The main conclusions of this report were that Stockton Beach is characterised by considerable short-term fluctuations in the sub-aerial area of the beach which are associated with storm events.

Figure 2-1 Overview of the Stockton and neighbour areas.

A long term assessment of the beach was also carried out which was based on a 27 year data set (1959-1986). It was concluded, that there has been a slight accretional trend between the northern breakwater and Hereford Street and a slight recessional trend between the northern section of Hereford St and the Stockton sewage treatment ponds. The results also suggested that there may be longer term erosion at two locations which also have a history of storm damage; these locations are:

- Between Stone Street and Griffith Avenue; and
- At the Stockton Sewage Treatment ponds, where an effluent pond once stood prior to the 1974 storms.

In 1989 the Mitchell Street seawall was constructed. In 1995 the Department of Land and Water Conservation carried out a Coastline Hazard Study to provide specialist technical advice on coastal processes and the shoreline behaviour of Stockton Beach. Further photogrammetric analysis of the area was undertaken to provide quantitative volumetric and historical shoreline movement information based on data from 1952, 1954, 1959, 1965, 1969, 1974, 1986, 1991 and 1994. Analysis of historical shoreline behaviour was also carried out based on newspaper reports and shoreline position indicators at either mean high water (MHW) or at the back of the beach erosion escarpment. The main conclusions of this study were summarised as:

- Realignment of Stockton Beach has occurred due to construction of the northern breakwater south of the Surf Life Saving Club (SLSC);
- The shoreline has fluctuated over a width of 80 to 130 metres (excluding the effects of realignment after breakwater construction);
- The most landward shoreline position was in 1952, also in 1946 and 1995 along Mitchell Street;
- The most seaward position of the beach was in 1913 immediately after completion of the northern breakwater;
- The most significant short term shoreline erosion was 70 to 100 metres between 1938 and 1946 north of the SLSC;
- The most significant shoreline recovery was about 50 metres between 1952 and 1965 which was located both north of the SLSC and up to 130 metres adjacent to the breakwater:
- No long-term recessional trend of the shoreline was evident. This conclusion was made as the shoreline position in 1893 (pre breakwater construction) was found to be generally consistent with the position in 1994 (excluding breakwater realignment effects).

In 1998, WBM (1998) undertook a coastal hazard study of beaches in the Newcastle region, which included Stockton Beach. The lines of coastal hazards were defined based on historical information and modelling estimations. Based on these investigations geotextile bag protections were constructed at both the surf club and at Pembroke St. These structures and the Mitchell St seawall have limited the extent of recent erosion events, but have had a significant impact on beach amenity. The implementation of such structures is therefore not considered to provide a sustainable long term solution to the erosion problem.

The erosion studies on Stockton Beach done by WBM (1998), PWD (1985) and DLWC (1995) were contradictory in their interpretation of historical shoreline changes and their

prognosis for future trends at Stockton Beach. DLWC (1995) presented the view that Stockton Beach experiences large fluctuations but no long term recessionary trends. In contrast, WBM (1998) assumed that present shoreline recession would prevail for 50 years, but at reduced rates after the initial 20 years. The PWD (1985) indicated a slight recessional trend between the northern section of Hereford St and the Stockton sewage treatment ponds and slight accretion trend between the northern breakwater and Hereford St.

Following these studies, Umwelt (2002) undertook an analysis of the coastal processes at Stockton based on historical analysis of the shoreline and nearshore areas as well as numerical modelling of initial beach erosion. This study suggested that the erosion is progressively worsening with a significant volume of sand having been lost form the beach and nearshore zone over the last century. Since the mid 1860's parts of the seabed adjacent to Stockton have been lowered by 4 to 7m resulting in an estimated 4 million m^3 of sand being lost from the beach and the adjoining sea bed since 1921. This equates to an average rate of erosion/sand loss of approximately 50,000 m^3/year for that period. Available hydrosurvey data indicates that between 1988 and 2000 the rate of erosion/sand loss from Stockton beach increased to approximately 170,000 m^3/year.

The Newcastle Coastline Management Study, Umwelt (2003), analysed the conditions at Stockton Beach. Based on the analysis and interpretation of the long term changes in seabed morphology and numerical beach erosion modelling of Umwelt, (2002), it was recognised that the conditions found were not consistent with those presented in the Newcastle Coastline Hazard Definition Study undertaken by WBM (1998). Furthermore it could be expected that the recent higher-than-average erosion rates were likely to increase at an accelerating rate under large wave activity and the threat to development at Stockton Beach would increase with time.

Consequently, it could be assumed that the hazard definition presented in WBM (1998) was likely to be a significant underestimate of what realistically may occur. Therefore in light of the previous investigations the hazard risk at Stockton Beach was likely to be exacerbated and would need to be further assessed as part of additional studies.

Based on the above information, The Hunter Coast and Estuary Management Committee (HCEMC) that includes Newcastle City Council, the Department of Natural Resources (DNR), Newcastle Port Corporation (NPC) and the Newcastle Community recognised the need for an updated study in order to provide input for a new coastal management strategy for the area.

2.2 Objectives of the Study

This study should provide a detailed qualitative and quantitative analysis of the sediment transport along the beaches to assist in the development of the revised plan of management. The study should conduct a detailed analysis of the causal factors of erosion, with a view to identifying and testing realistic options for future management of the problem. The main objectives of this study are to:

- Develop a comprehensive understanding of the processes driving the sediment transport and shoreline changes at Stockton Beach;
- Verify sediment budget predictions using available data;

- Quantify the sediment budget for the southern end of Stockton Beach;
- Review current hazard projections at the southern end of Stockton Beach;
- Identify and assess management options for the management of coastal hazards and beach amenity in accordance with the Coastline Management Manual and the NSW coastal policy;
- Consultation with the community in order to develop preferred management options; and
- Revise the Newcastle Coastline Management Plan 2003, to incorporate the preferred management option, or combination of options, for Stockton Beach.

2.3 Scope of work and study approach

As stated in the project tender documents, the scope of work involves six main components in three project stages.

Stage 1 - Process Study

- Data collection and analysis;
- Numerical modelling of waves and currents and the resulting sediment transport; and
- Prediction of ongoing beach response/trends;

Stage 2 – Management Study

- Identification and assessment of management options; and
- Presentation of options to the Hunter Coast and Estuary Management Committee and the community;

Stage3 – Newcastle Coastline Management Plan Revision

- Revision of the Newcastle Coastline Management Plan 2003.

The three stages are further described below.

2.3.1 Stage 1 - Process study. Determination of the on-going trends of the coastal processes at Stockton Beach.

The first stage focuses on the analysis of the coastal processes to determine the sediment transport conditions. This stage has been based on four major tasks, as follows:

- Data collection and analysis: This task is focused in the analysis of available data;
- Development and calibration of numerical model. During this task an advanced numerical model has been established to describe the sediment transport processes at Stockton Beach;
- Prediction of ongoing processes. The numerical model has been applied to predict the ongoing conditions and future trends at Stockton Beach; and
- Definition of coastal hazard lines.

The relationship between the tasks is presented in Figure 2-2 below and a more detailed description of these tasks is presented in the following sections of this report.

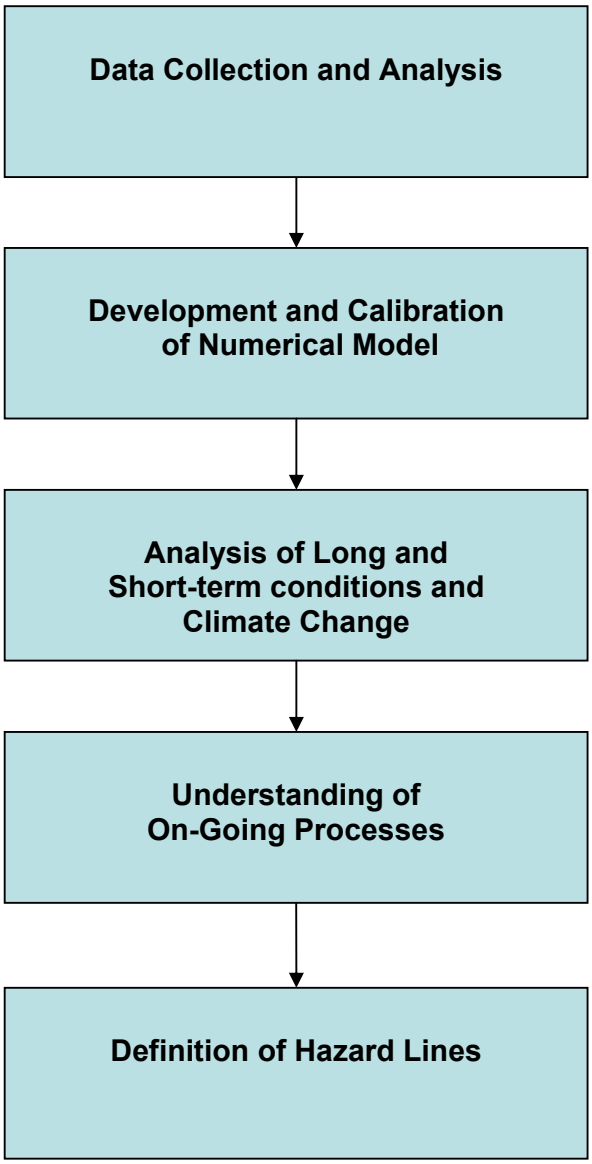


Figure 2-2 Overview of the methodology applied in Stage 1 of the study.

2.3.2 Stage 2 - Management study

Based on the findings of Stage 1 a number of mitigation measures will be proposed and on preliminary communication with the stakeholders whereby a shortlist of options will be defined to be evaluated in more detail. The outcomes of this stage of the project will be, in conjunction with community consultation, identification and assessment of the option or combination of options that provide the best value for management of coastal hazards along Stockton Beach.

2.3.3 Stage 3 - Newcastle Coastline Management Plan Revision

Once the preferred management option, or combination of options, has been identified and assessed, information relating to the proposed management will be

presented to the community in a follow-up consultation program. The information will then be recommended to the Hunter Coast and Estuary Management Committee for acceptance and subsequent incorporation into a revised Newcastle Coastline Management Plan for adoption by Council.

The revised Newcastle Coastline Management Plan will integrate the results of the Stockton Beach Coastal Processes Study, assessment of preferred options from the Management Study and other related amendments into a document that gives vision to Council and the broader community to assist in the definition of coastline management priorities.

3 DATA COLLECTION AND ANALYSIS

3.1 Motivation

The understanding of the coastal processes at Stockton Beach requires a detailed analysis of the available historical data. This section presents a review of water level, wave, currents and wind information collected for the study with the aim at establishing and calibrating the numerical model to investigate the sediment transport conditions of the area and the historical sediment transport processes.

3.2 Water Level, Wave and Current Measurements at Stockton Beach Nearshore Area

Newcastle City Council (NCC) and the Department of Infrastructure, Planning and Natural Resources (DIPNR) provided water level, wave and current measurements at Stockton Beach. The measurements were carried out by the Department of Commerce's Manly Hydraulics Laboratory (MHL) during the period 26 March 2001 -3 December 2001 at the location AMG E386877 N6358167 or MGA E386981 N6358356.4. Figure 3-1 shows the location of the measurement station in approximately 8 metres of water depth. The measurements were carried out with an Acoustic Doppler Current Profiler (ADCP) with 1 metre bins separation and the data was averaged in 20 minute intervals. The data was processed and depth integrated values were delivered.

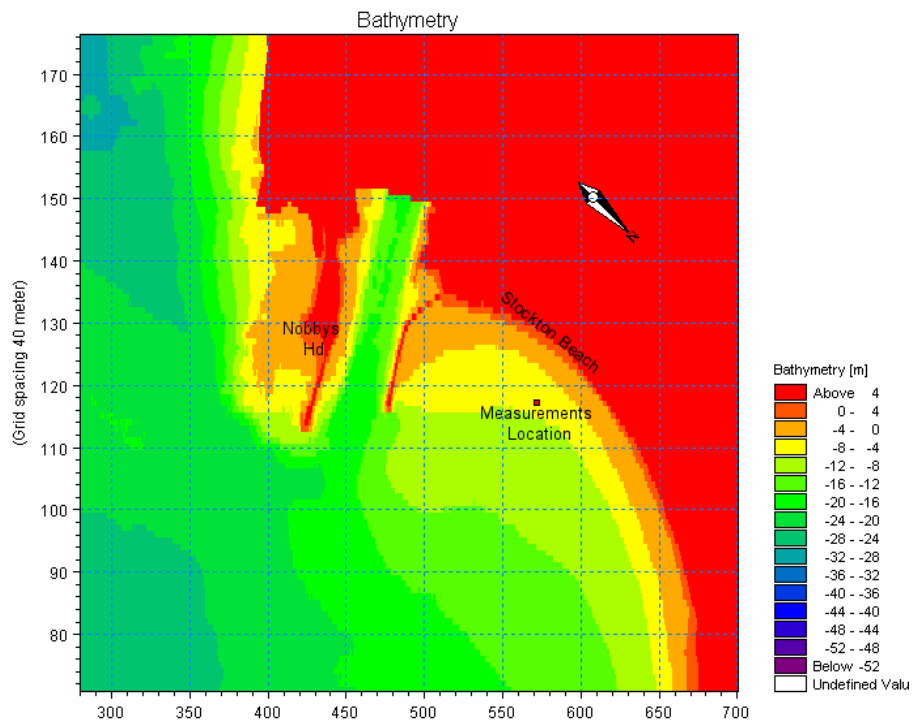


Figure 3-1 Location of the measurements carried out by MHL at Stockton Beach.

The measurements were undertaken in four deployment periods as follows:

Table 3-1 Details of the measurements undertaken by MHL at Stockton Beach

Deploy. Name	Measurement Period	File Name	Averaging Time/Comment
1-000	26/03/01 to 14/04/01	Stock1_000.wvs	20 min 1 hour
1-001	26/03/01 to 28/03/01	Stock1_001.wvs	20 min 1hour. Data is damaged
2-000	06/06/01 to 29/07/01	Stock2_000.wvs	20 min 3 hour
2-001	29/07/01 to 09/08/01	Stock2_001.wvs	20 min 3 hour
3-000	09/08/01 to 01/10/04	Stock3_000.wvs	20 min 3 hour
4-000	18/10/01 to 03/12/04	Stock4_000.wvs	20 min 3 hour

Each of Figure 3-2, Figure 3-3, Figure 3-4, Figure 3-5 and Figure 3-6 presents the following data for deployments 1-000, 2-000, 2-001, 3-000 and 4-000:

(a) water levels;
(b) significant wave heights (Hs);
(c) wave period (Tp);
(d) mean wave direction (MWD);
(e) current speed (Current Sp); and
(f) current direction (Current Dir)

Deployment 1-001 is not presented as the data was corrupted.

The measurements have been reviewed to determine the quality of the information, however no detailed evaluation of the data has been carried out as this will be performed at later stages of the project.

3.2.1 Water Levels

Water level measurements were compared to predicted tidal levels at the Newcastle Tidal Station. The tidal predictions were based on tidal harmonic constants obtained from the Australian National Tide Tables Handbook and the measured and predicted water levels are presented for all deployments except 1-001. It is observed that there is good agreement between measurements and predictions; however some differences are observed at deployment 2-000. These predictions can be explained by atmospheric effects that are not included in the tidal predictions.

3.2.2 Waves

The wave measurements were also compared to offshore wave rider measurements at the Sydney (directional) and Crowdy Head (non-directional) buoys. It should be noted that the offshore wave buoys have been inactive during different intervals and data is not available for some of the deployment period as shown in the figures.

Measured significant wave height (Hs), peak wave period (Tp) and mean wave direction (MWD) at the Stockton Beach and Sydney and Crowdy Head buoys are presented for each deployment period. Good correlations are observed for Hs and Tp. The measured

wave directions at Stockton are difficult to correlate to the offshore mean wave directions without the use of a numerical wave model, however by applying basic wave transformation principles it is possible to describe the transformation of offshore waves. Generally it can be confidently presumed that the prevailing waves from the SE are diffracted by the training wall at the Hunter River entrance and as they get closer to the beach, refracted by the variation of depth contours. Waves from the NE region will be mainly refracted and will tend to rotate to approach the beach almost perpendicularly. Due to these effects it should be expected that the mean wave direction at Stockton Beach be between 60 and 120 degrees and this is what it is observed in the measurements.

3.2.3 Current Measurements

Current speed and direction have been evaluated. The data shows weak observed currents mostly travelling southward. Periods of larger currents are observed during large wave events and it can be confidently presumed that these are mostly wave induced. This can be observed in the Deployment 2-000 carried out in August 2001 (Figure 3-3).

Large tidal variations are not observed in the measured current data as might be expected in a location close to a major river entrance. The lack of tidal variations may be due to the deployment location being remote from the area of influence of the flows through the river entrance

Deployment 1-000

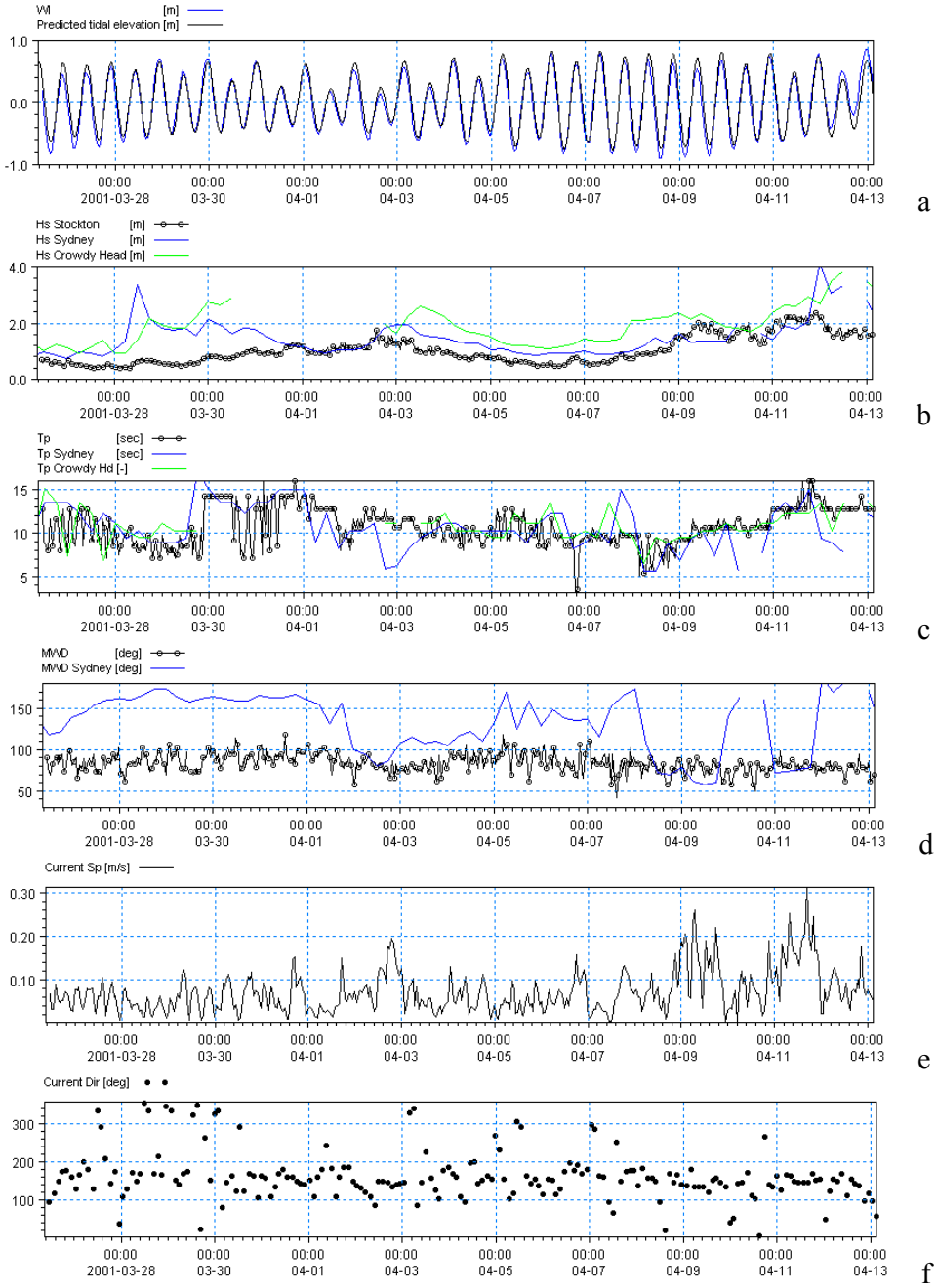


Figure 3-2 Water Level measured (black) and predicted (blue), significative wave height (Hs), peak period (Tp), mean wave direction (deg), current speed (Sp) and current direction (from top to bottom) Deployment 1-000.

Deployment 2-000

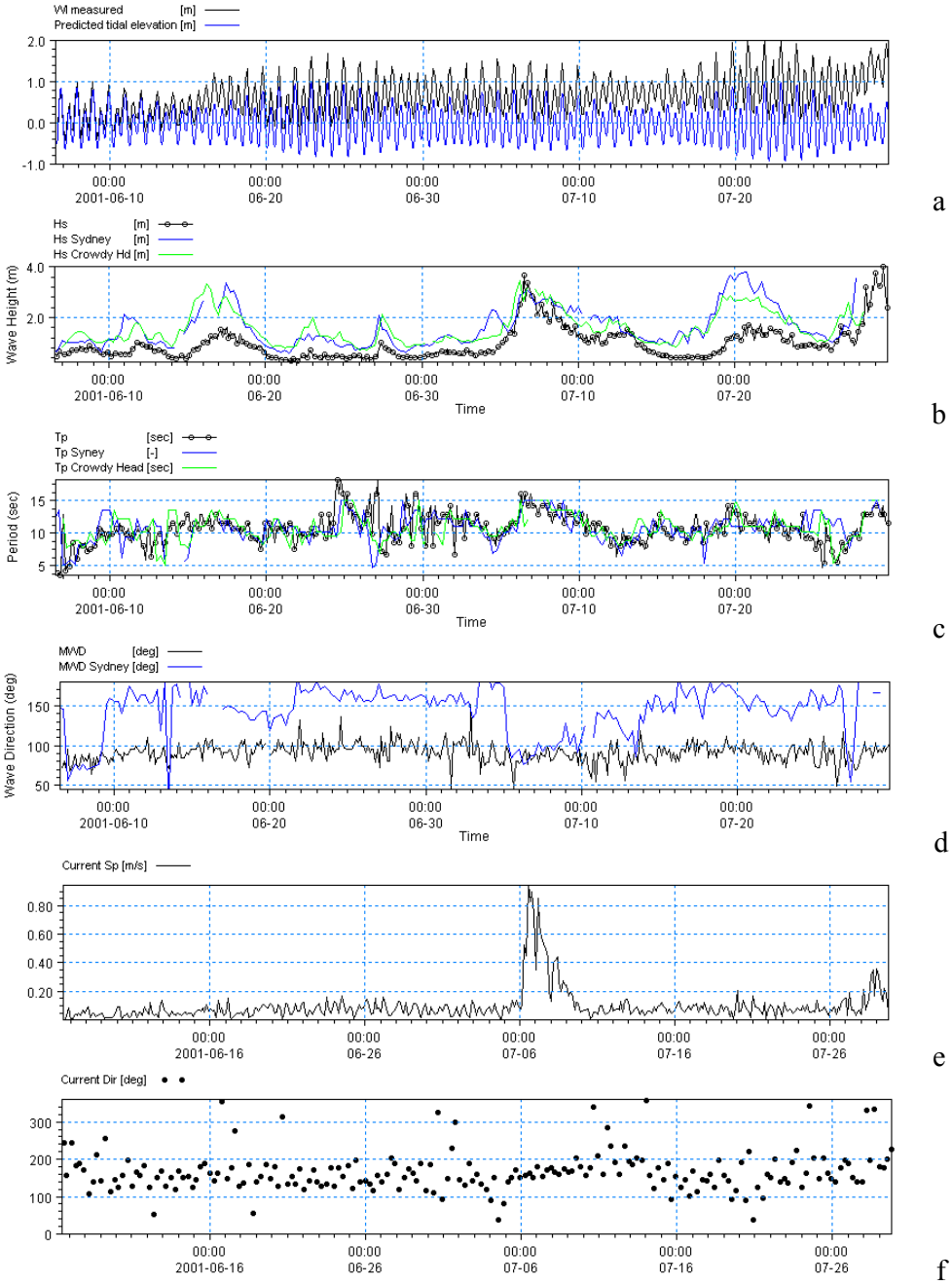


Figure 3-3 Water Level measured (black)and predicted (blue), significative wave height (Hs), peak period (Tp), mean wave direction (deg), current speed (Sp) and current direction (from top to bottom) Deployment 2-000, First period.

Deployment 2-001

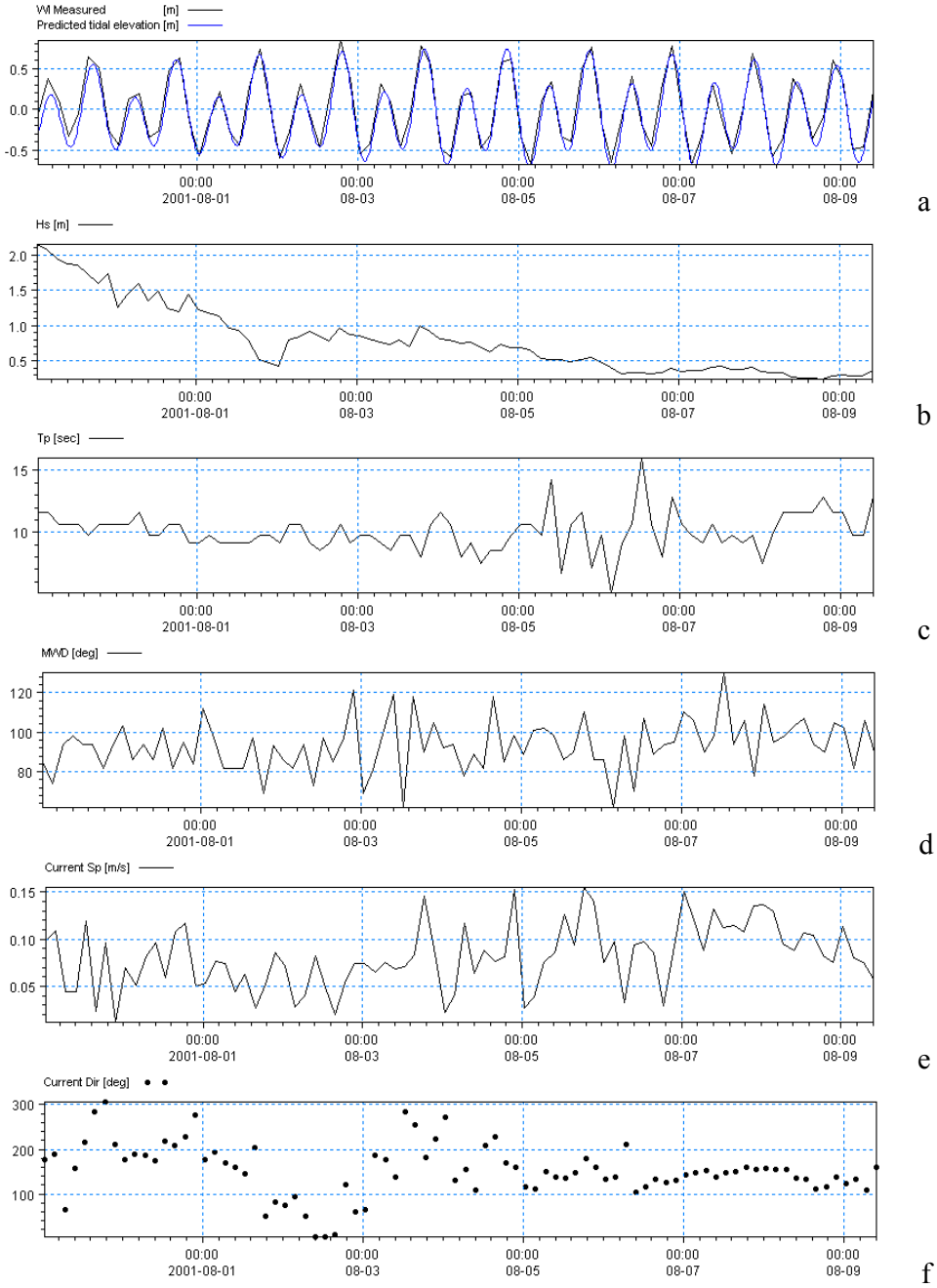


Figure 3-4 Water Level measured (black) and predicted (blue), significative wave height (Hs), peak period (Tp), mean wave direction (deg), current speed (Sp) and current direction (from top to bottom) Deployment 2--001, second period.

Deployment 3-000

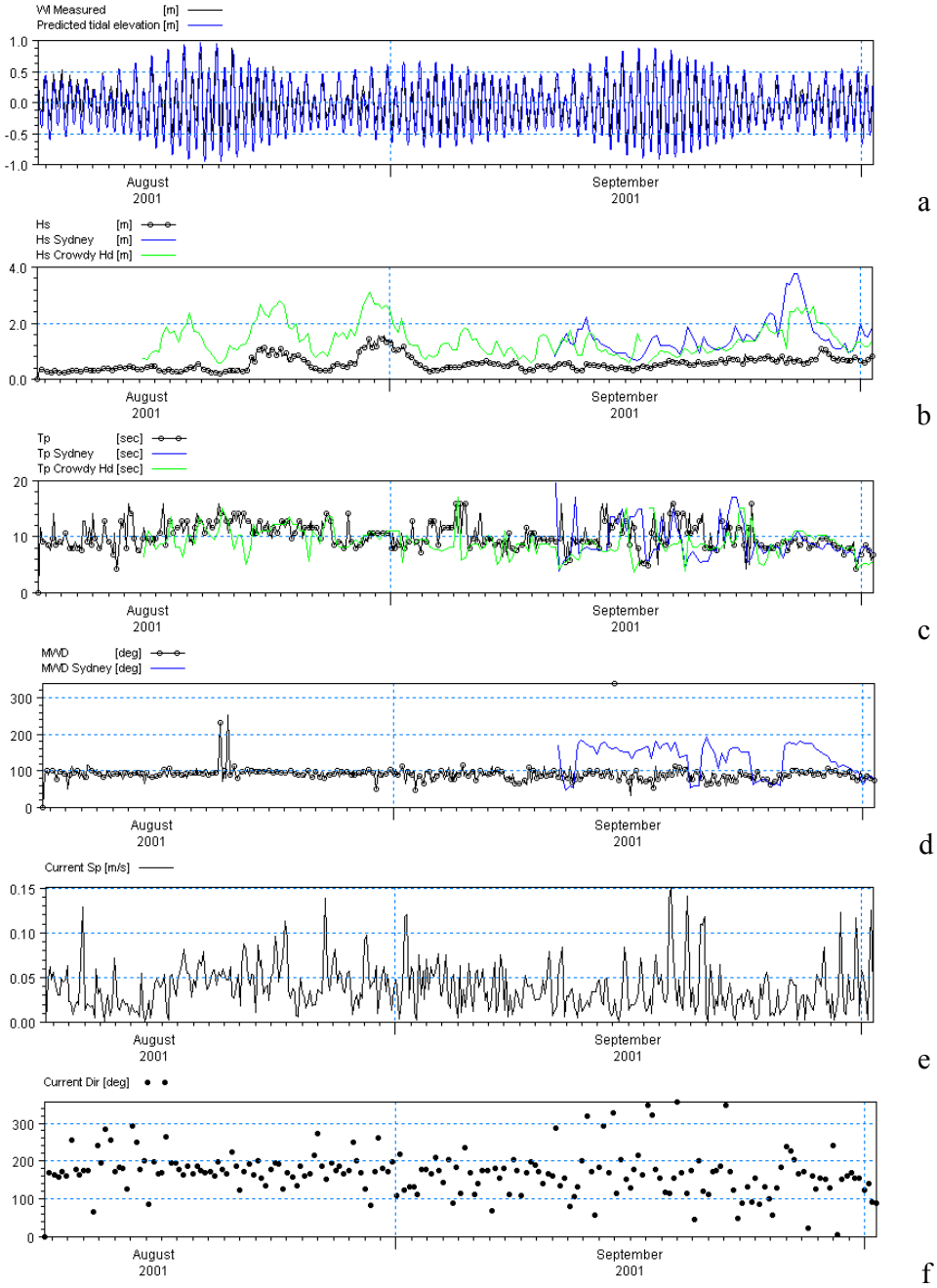


Figure 3-5 Water Level measured (black) and predicted (blue), significative wave height (Hs), peak period (Tp), mean wave direction (deg), current speed (Sp) and current direction (from top to bottom) Deployment 3-000.

Deployment 4-000

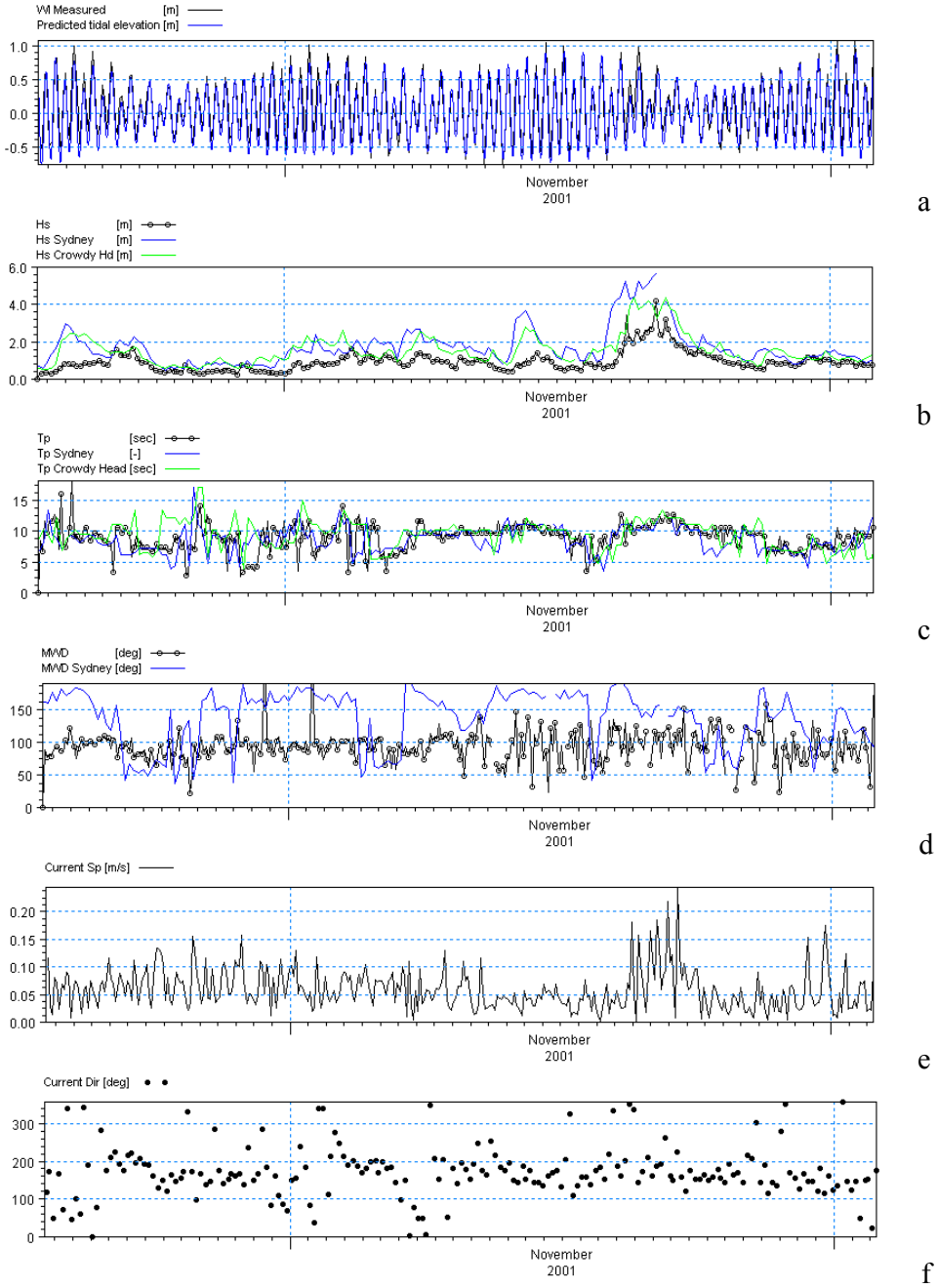


Figure 3-6 Water Level measured (black) and predicted (blue), significative wave height (Hs), peak period (Tp), mean wave direction (deg), current speed (Sp) and current direction (from top to bottom) Deployment 4-000.

Based on the analysis of this data no additional information of wave conditions and currents is necessary at the nearshore areas. The provided wave, water levels and current measurements allow a good overview of the conditions in the nearshore areas because they provide a good overview of the wave induced currents

3.3 Flow and velocity measurements at the Hunter River Entrance

The measured data at Stockton Beach does not describe the flow conditions at the entrance of the Hunter River. Flows from the river are likely to have a relevant effect on the morphological conditions and on the sediment bypassing mechanism across the river entrance. It was recommended to complement the available current measurements at Stockton Beach with ADCP measurements at the Hunter River entrance. The recommendation was to measure water levels and flow currents in a fixed location for a period of 2 weeks. It was also suggested that these measurements be supplemented by ADCP transects during two tidal cycles during spring and neap tidal conditions if possible. The measurement period was defined so that the measurements would be able to capture a tidal spring period. Based on the tidal condition at the Newcastle Port the measurement programme was established from 08-Dec-2004 till 21-Dec-2004 as shown in Figure 3-7. This period contains maximum tidal amplitude of 2 metres on Dec-14.

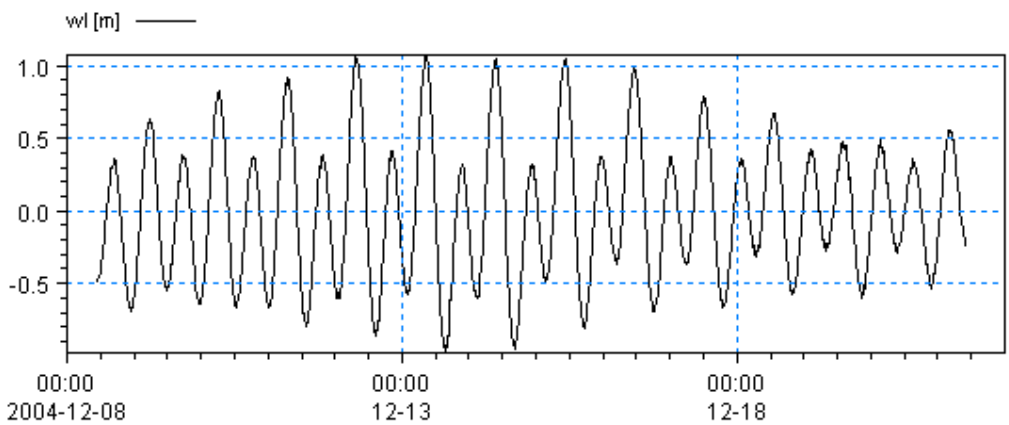


Figure 3-7 Water levels at the Hunter River Entrance during the measurement period.

Figure 3-8 illustrates the location of the bottom mounted ADCP and the two transects. Transect 1 was not initially defined but was included during the Dec-14 measurements due to large wave action at the river entrance. However as the wave conditions improved the measurements were completed in transect 2 (as originally planned). Transect 2 was also used during the neap measurements on Dec 21.

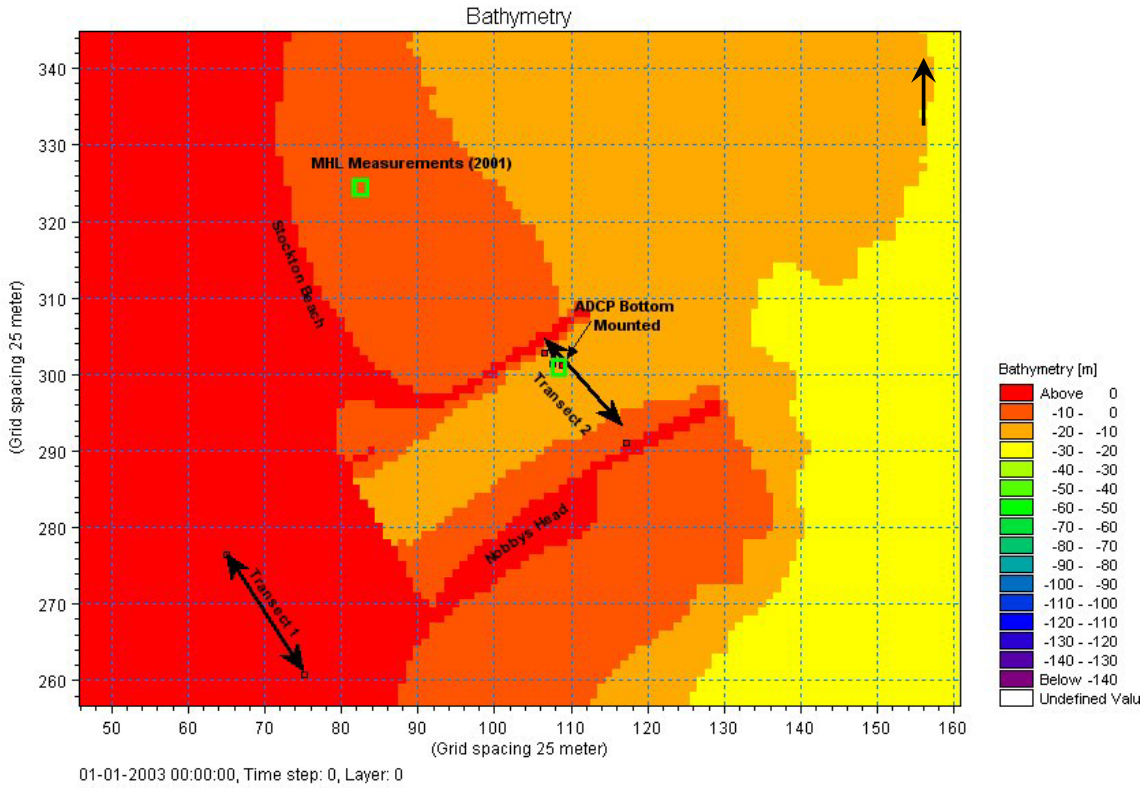


Figure 3-8 Location of the proposed ADCP bottom mounted and transects at the river entrance as well as the location of the nearshore measurements carried out by MHL off Stockton Beach.

The bottom mounted ADCP provides a detailed description of the flow velocities and direction at different distances from the seabed. As an example Figure 3-9 shows the measured current speed (middle) and direction (below) at 9.9 metres from the seabed. This figure also includes the measured mean pressure (above) that allows relation of current speed and direction to water levels. As it can be observed, current speeds exceeding 0.8 m/s are observed during Dec 14 when spring tidal levels occurred however the typical current speeds are approximately 0.4 m/s.

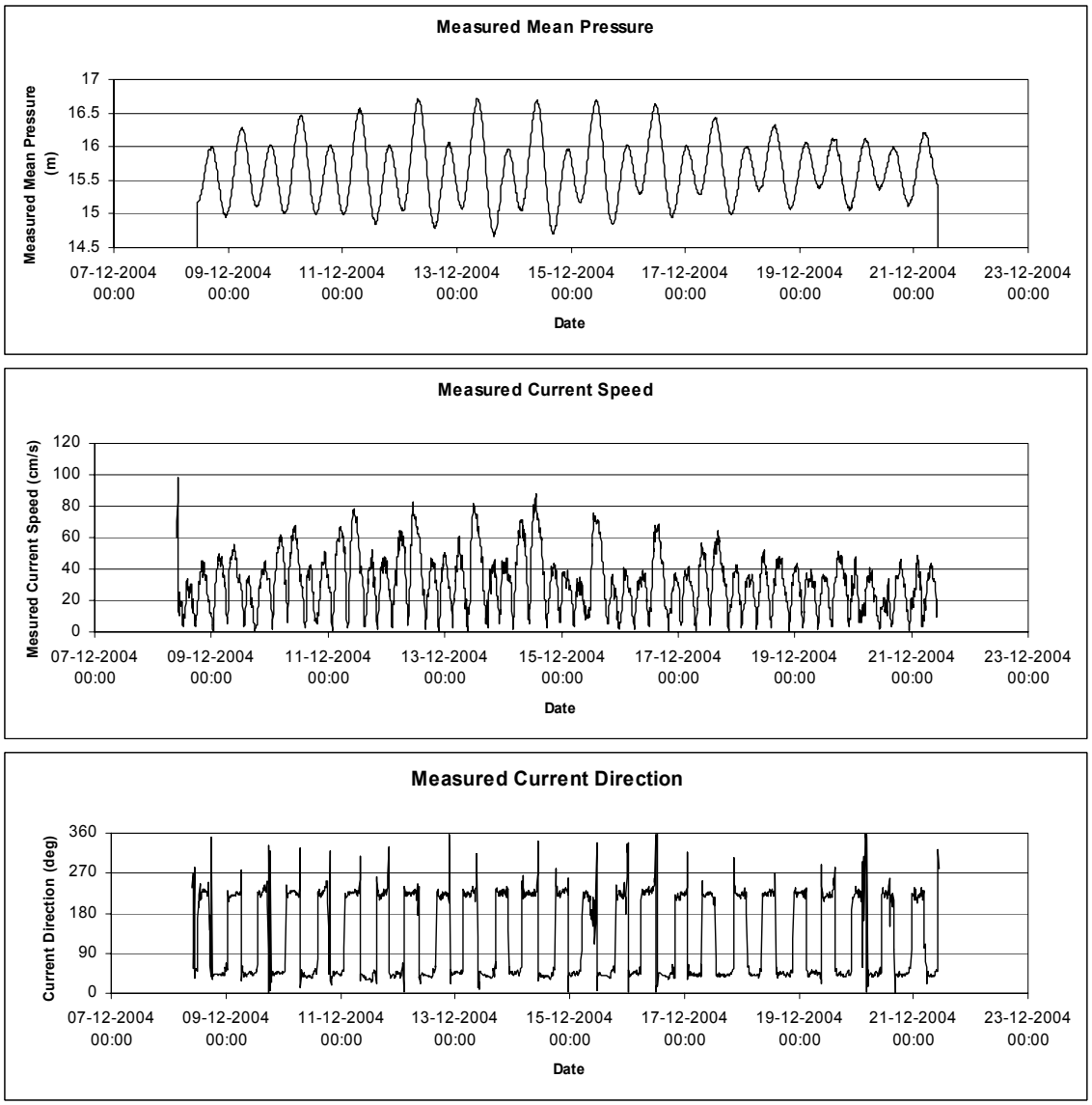


Figure 3-9 Measured mean pressure and measured speed and direction at 9.9 metres from the seabed

During this two week measurement period ADCP transects were carried out during spring tide and neap tide water levels the following days:

- Spring tide - 14 December 2005;
- Neap tide - 21 December 2005.

The ADCP transects were undertaken in a surveying boat with the ADCP instrument attached. The transects were carried out at approximately 15 minutes intervals and then integrated across the measured cross section to obtain the tidal flow in and out of the river. It should be noted that for the spring measurements the transects had to be carried out at the Pilot Station (transect line 1) due to large wave action at the river entrance. As

the conditions improved later in the day the measurements moved to the river entrance as originally proposed (transect line 2) as shown in Figure 3-10.

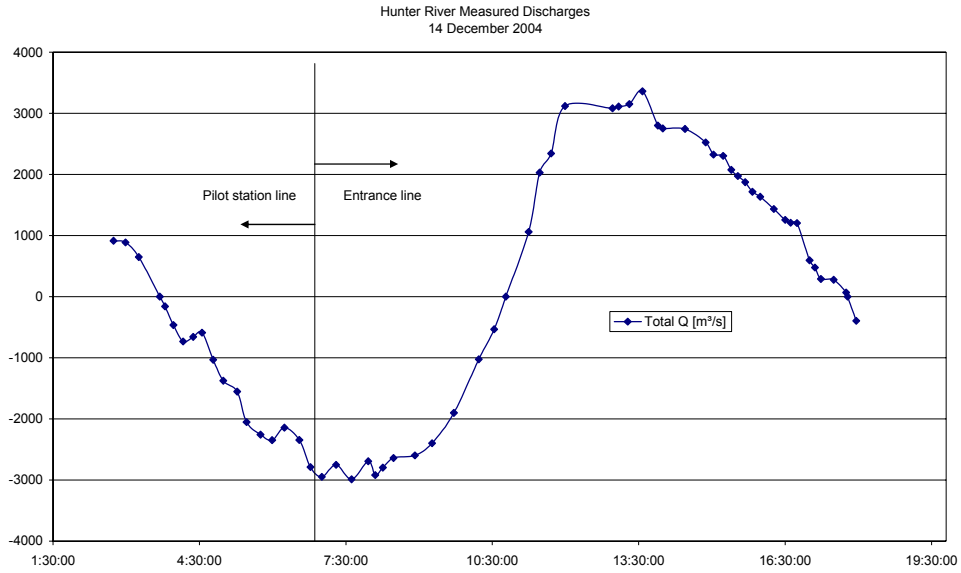


Figure 3-10 Computed discharges during spring tidal levels.

The computed discharges at the Hunter River entrance for the spring and neap periods are shown in Figure 3-9 and Figure 3-10. Maximum discharges of 3000 m^3/s were measured during spring period and 1500 m^3/s during neap period.

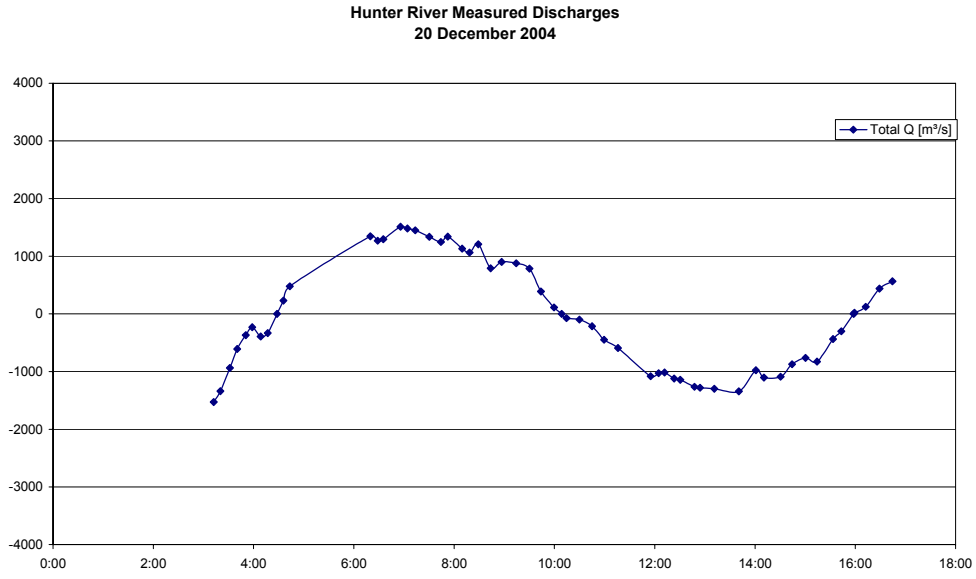


Figure 3-11 Computed discharges during neap tidal levels.

3.4 Wind Data

Wind data was sourced from the Bureau of Meteorology (BOM) for Nobbys Head (Newcastle), Williamtown and Norah Head meteorological stations for the period 1992-2004. A time series plot of the wind speed at the three locations is presented in Figure 3-12 that shows similar velocities at Norah Head and Nobbys Head; but this should be expected as these two stations are closely located facing the coast without major land influence. Some differences are observed however but these are related to the spatial variability of the wind conditions. The wind measurements at Williamtown show consistently lower speeds compared to the other stations. This can be related to the location of the station inland where the wind profile is affected by vegetation and the land features.

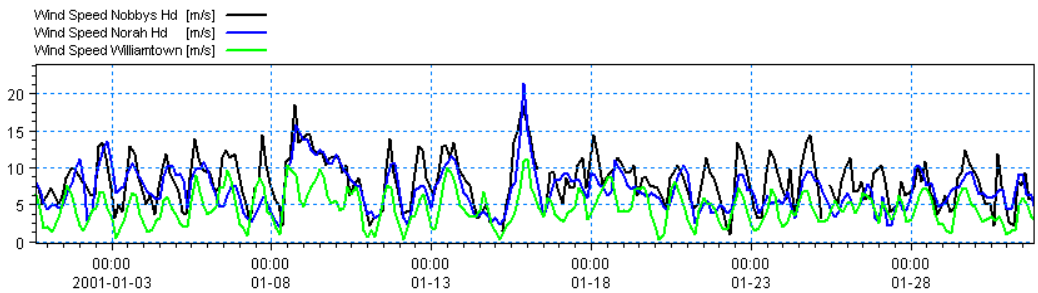


Figure 3-12 Wind speed measurement comparisons at Nobbys Head (Newcastle) (black), Norah Head (blue) and Williamtown (green)

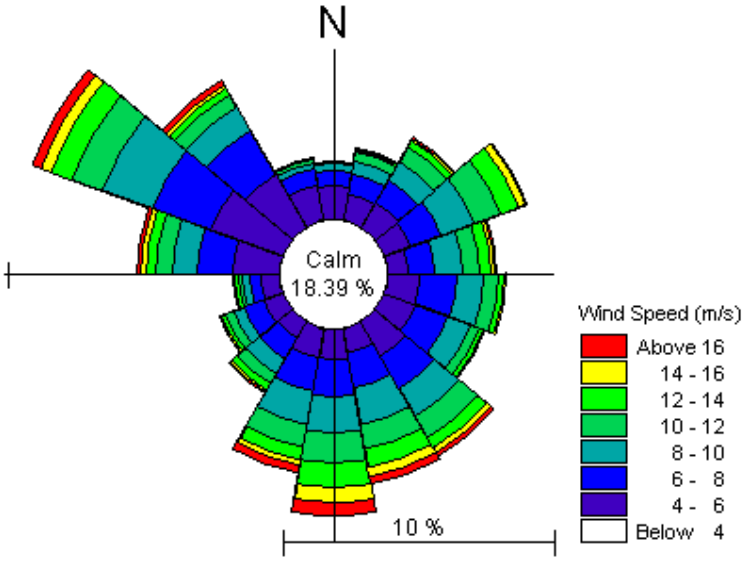


Figure 3-13 Wind Rose at Nobbys Hd (Newcastle) for the period 1992-2004

A wind rose at Nobbys Head for the period 1992-2004 is presented in Figure 3-13 and shows the frequency of the winds for different directions (reference 10%) and the speed of the wind in a colour scale. It can be observed that the most frequent and strongest

winds occur from the NW and SE-SW orientations. In this analysis wind speeds below 4 m/s are referred to as calm conditions and this period sum up to 18.39% of the measurements.

3.5 Offshore Wave climate

The determination of the sediment transport at Stockton beach requires the understanding of the offshore wave conditions in the area.

Stockton Beach is exposed to the incoming waves due to a free fetch from Antarctica in the South, a fetch in the order of 2000km across the Tasman Sea to New Zealand towards the East to Southeast and a free fetch from the Pacific Ocean (Figure 3-14). Offshore wave conditions can be severe, in particular in relation to storm and swell waves from the Southern Ocean and Tasman Sea from the southerly to easterly quadrants. During summer, tropical cyclones originating in the Coral Sea produce waves that may travel long distances before reaching the Stockton Beach area.

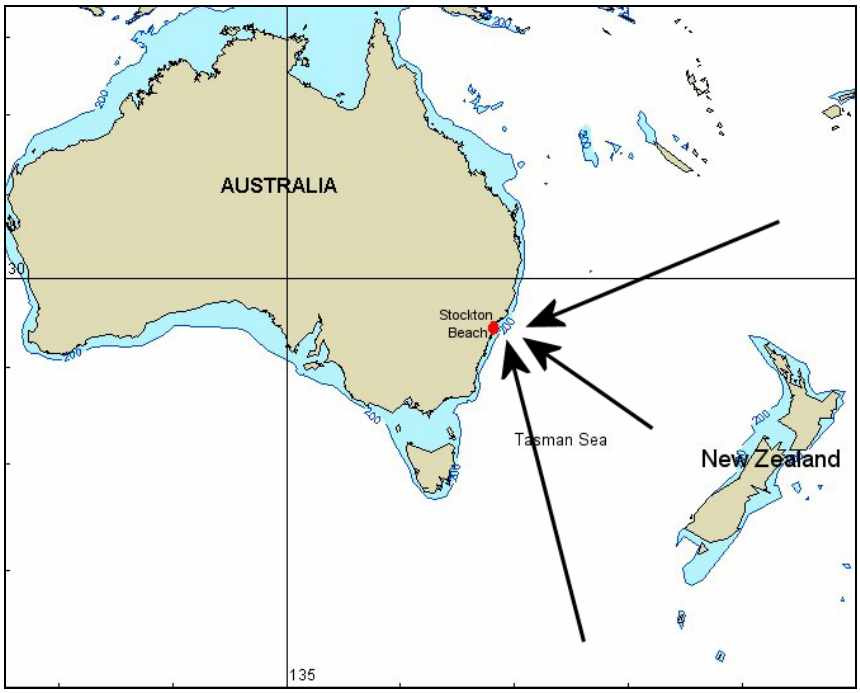


Figure 3-14 Overview of the Stockton Beach area and exposure to offshore waves.

3.5.1 Offshore Wave Data

Two sets of offshore wave data measurements were provided by the Department of Commerce, Manly Hydraulics Laboratory. The data was measured at two waverider buoys located close to the study area: These being:

- a directional waverider off Curl Curl Head in Sydney at location 354160 East and 6260930 North (MGA-56) in 85 m water depth; and

- a non-directional waverider located off Crowdy Head at 486720, 6478910 (MGA-56) in 79 m water depth. The location of both waverider buoys is presented in Figure 3-15, Figure 3-16 and Figure 3-17.

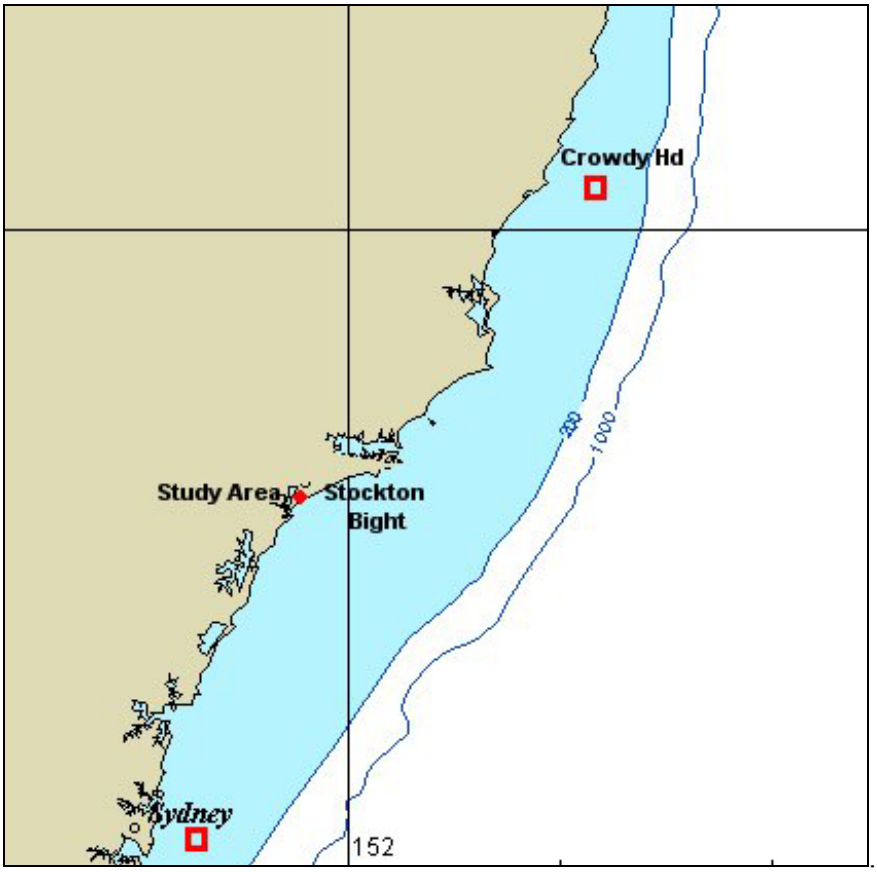


Figure 3-15 General overview of the offshore wave data location referred to the study area.

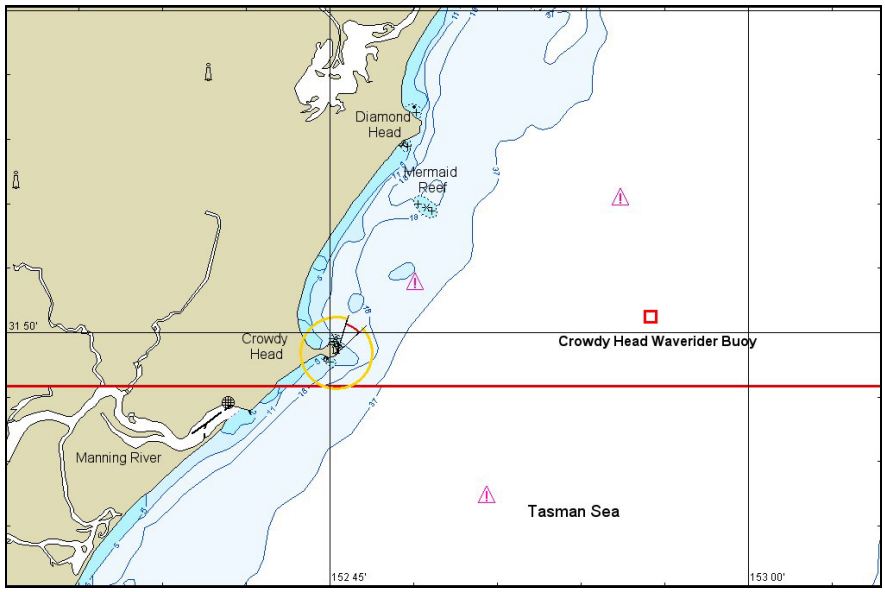


Figure 3-16 Location of the Crowdy Heads waverider buoy at 486720,6478910 (MGA-56)

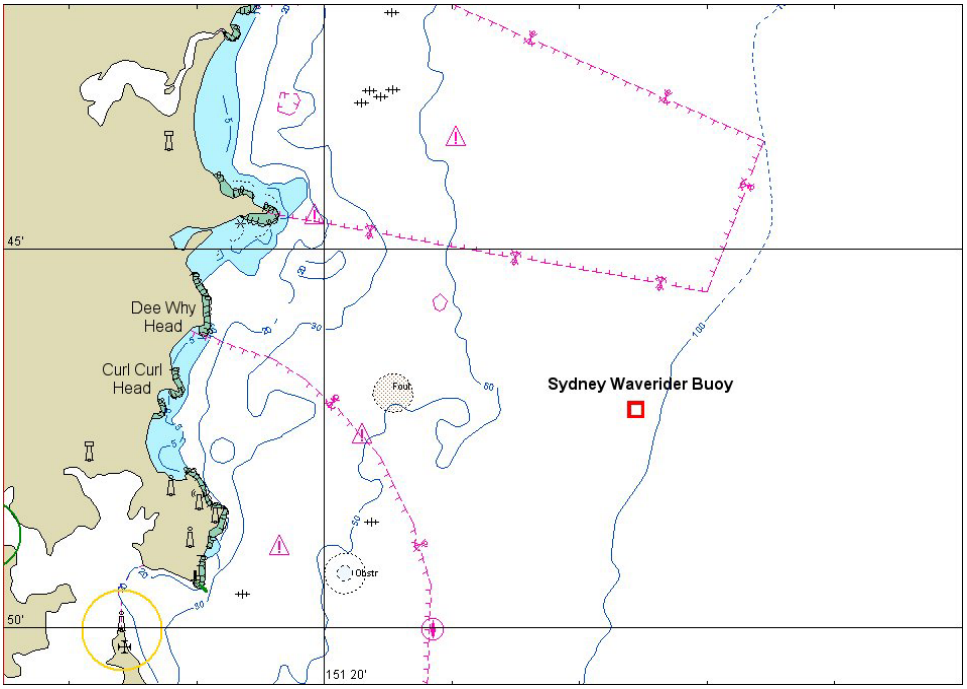


Figure 3-17 Location of the Sydney waverider buoy at 354260,6260930 (MGA-56)

The data coverage included the following variables:

- Non-directional recordings (Hs, Tp) from the Crowdy Head waverider buoy for the period 10/10/1985 to 31/07/2004;
- Directional recordings (Hs, Tp and wave direction) from the Sydney waverider buoy for the period 3/3/1992 to 1/8/2004

Even though these buoys are not located directly offshore of Stockton Beach, they provide a good representation of the offshore wave conditions of the region. This is particularly so in that the wave data collected offshore Sydney provides directional wave information which is essential for the analysis of sediment transport.

Wave Scatter Plots

Scatter plots of significant wave height (H_{m0}), mean wave direction (MWD) and peak period (Tp) have been produced for the Sydney waverider buoy data and are presented in Figure 3-18 and Figure 3-19. These figures show that the most frequent waves propagate from between 30 to 200 deg and the largest waves occur between 135 and 190 deg. A single maximum wave height of 8.34 m from 151deg and period of 13 sec has been recorded. The bulk of the measured significant wave heights are in the interval between 0.5 to 4m travelling from 45-190 degrees with typical peak wave periods between 5 and 15 sec. These periods are exceeded for larger waves propagating between 135 and 190 deg.

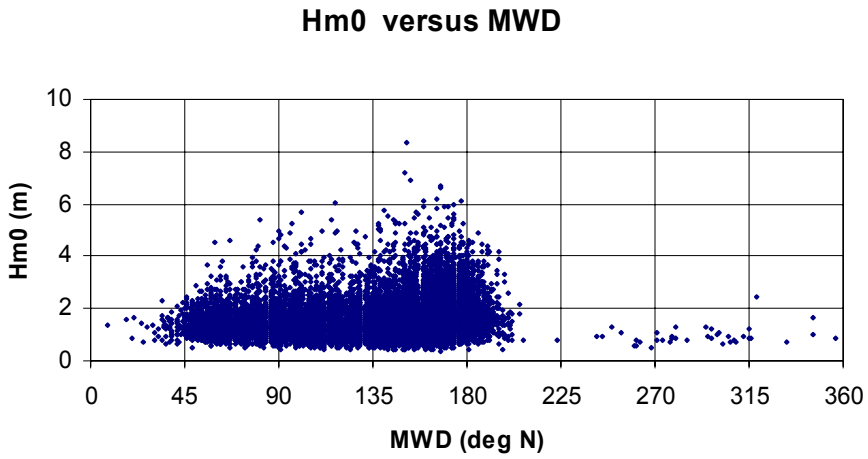


Figure 3-18 Scatter plots the significant wave height (Hm0) versus the mean wave direction

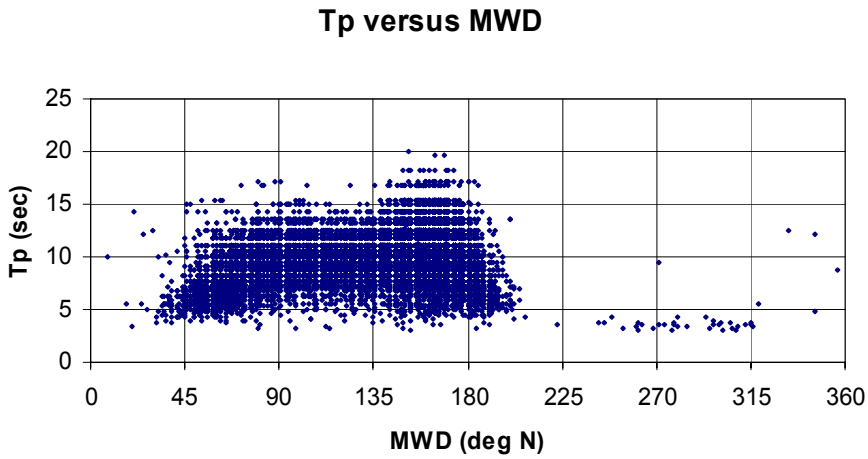


Figure 3-19 Scatter plots of the peak period (Tp) versus the mean wave direction

A scatter plot of the peak period versus significant wave heights is presented in Figure 3-20 for the Sydney instrument. This figure shows that there are two different wave groups being measured. These are waves where the wave period follow a relationship with the wave heights (wind waves) and waves that do not show a clear pattern between these two parameters (swell waves).

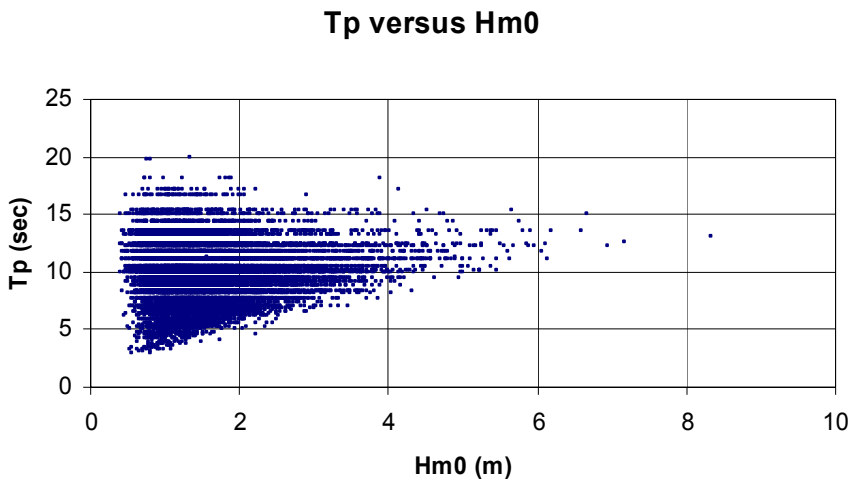


Figure 3-20 Scatter plots the peak period (Tp) versus significant wave height

3.5.2 Yearly wave Roses

In order to provide further detail of the wave conditions for the Sydney waverider buoy, wave roses have been produced for each individual year from 1992 till 2004 and are presented in Figure 3-21 Wave roses at the Sydney waverider buoy for 1992, 1993, 1994, 1995, 1996 and 1997.

These figures show a consistent pattern year to year with the majority of the wave energy propagating from the SE with a second predominant direction of NE to E. The SE waves originate from storm or swell waves which from the Southern Ocean and the Tasman Sea, predominately occur during the winter months. In contrast, the NE-E waves are generated by summer sea breeze systems and tropical cyclones in the Pacific Ocean which predominately occur during the summer months

Yearly wave condition variations are observed, as an example in 1999 a larger percentage of waves propagating from the NE-E direction are observed indicating a larger cyclonic activity during that year.

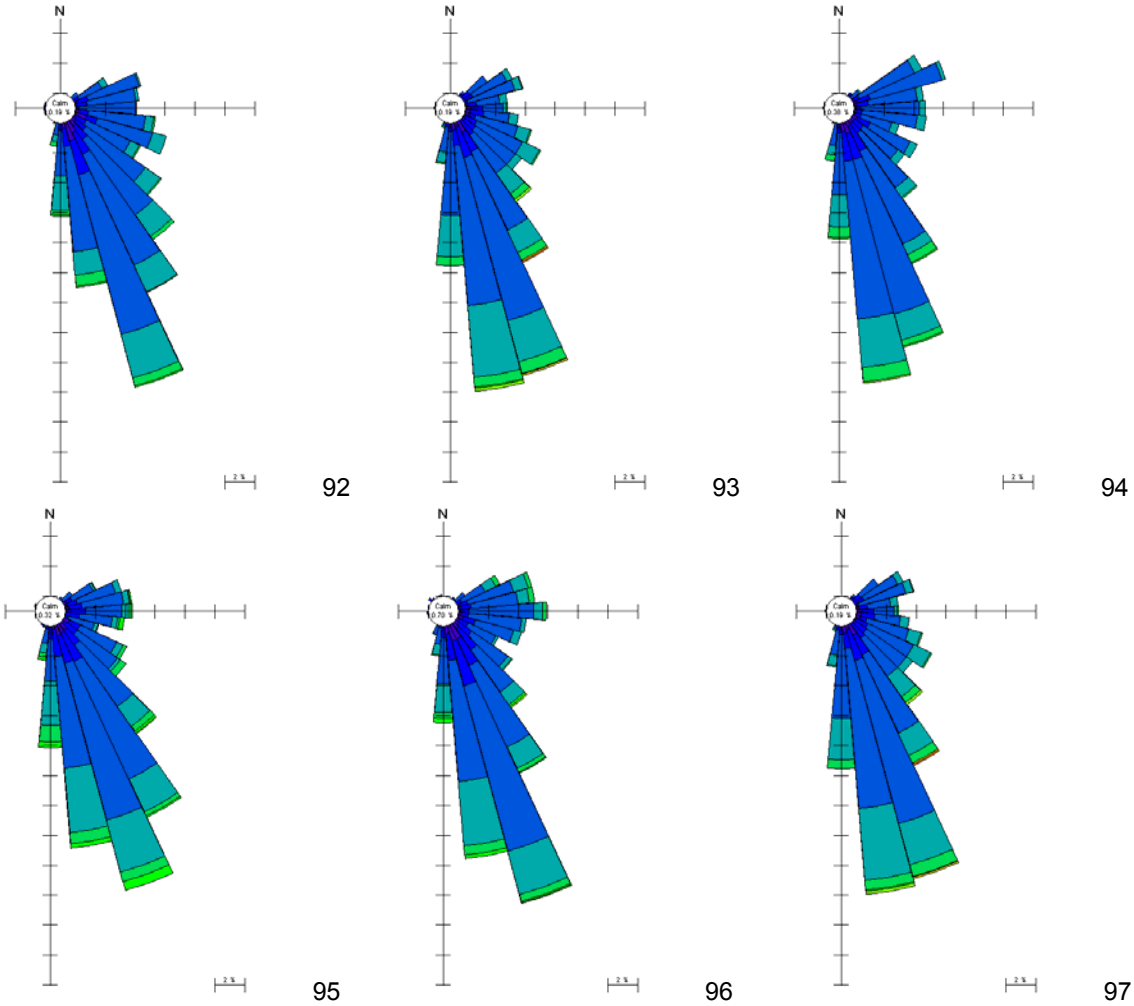
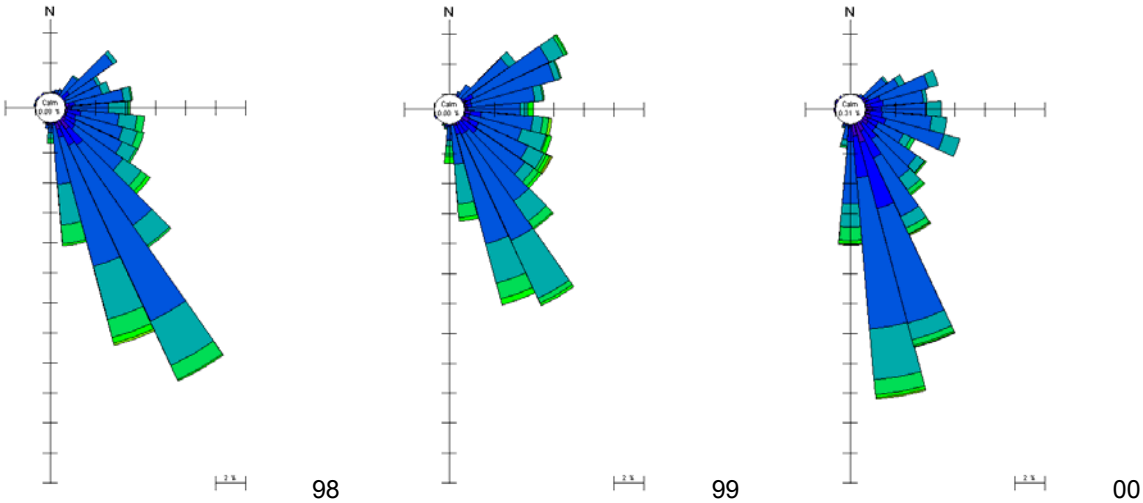


Figure 3-21 Wave roses at the Sydney waverider buoy for 1992, 1993, 1994, 1995, 1996 and 1997.



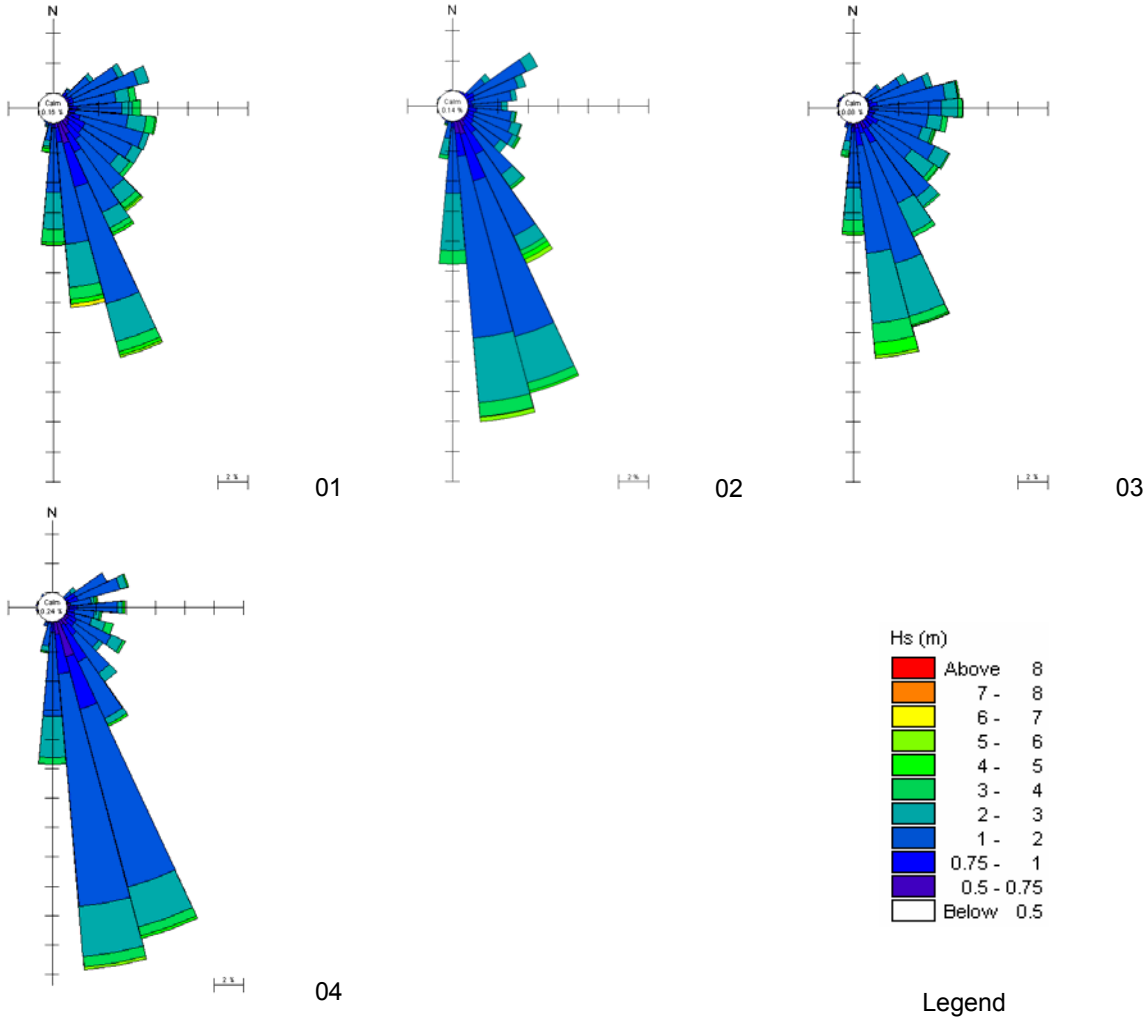
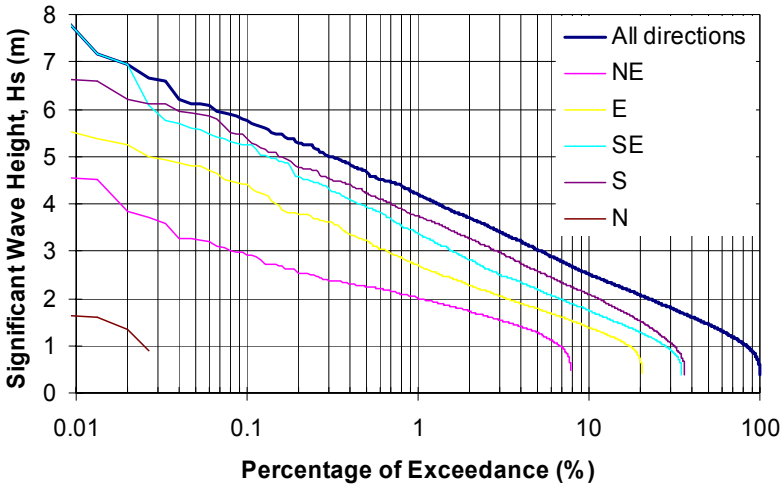


Figure 3-22 Wave roses at the Sydney waverider buoy for 1998, 1999, 2000, 2001, 2002, 2003, 2004 and legend.

Wave Exceedance Curves

Exceedance curves for the Sydney and Crowdy Head waverider buoy data have also been constructed and presented in Figure 3-23. They provide the percentage of time of the year that a wave height is exceeded. The figures demonstrates that the Crowdy Head and Sydney buoys exceedance curves show similar behaviour, with significant wave heights of 5.6 metres being exceeded 12 hours a year (0.137%). In the case of the Sydney data, exceedance curves have also been classified by direction with the most significant waves propagating from the S and SE.

Sydney Waverider Buoy



Crowdy Head Waverider Buoy

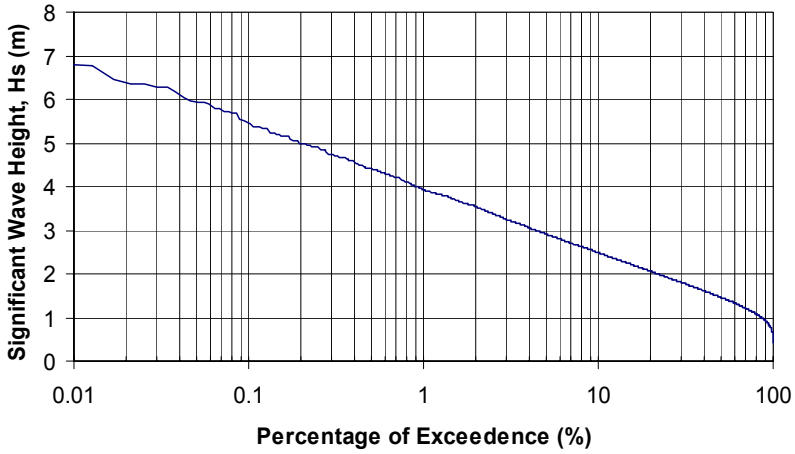


Figure 3-23 Wave height duration exceedance distribution, Sydney (above) Crowdy Head (below)

4 WAVE TRANSFORMATION

4.1 Motivation

The determination of the sediment transport processes at Stockton requires the transformation of the wave conditions from the offshore areas into the nearshore beaches. In order to incorporate the most relevant physical wave processes and simultaneously apply a computational sound approach, the transformation is carried out in two steps as follows:

- Regional wave modeling: to transform offshore waves into the Newcastle Area. These wave conditions will be applied as boundary conditions into a local wave model that will provide input for the computations of the sediment transport; and
- Local wave modeling: establishment and calibration of a local wave model of the Stockton area. The model domain has to be configured so as to include all relevant bathymetric features that may potentially influence the complex sediment transport processes in the area. The wave model has been defined to describe the local wave conditions of the Stockton and vicinity areas. This is required to accurately describe the wave conditions, especially diffraction, which is a necessary task as wave driven currents play a relevant role on the sediment transport mechanism around the Port entrance and Stockton Beach.

4.2 Regional Wave Modelling

The offshore wave data provided by the Department of Commerce, Manly Hydraulics Laboratory needs to be transformed into the nearshore areas of the study region. This is necessary so that it can be applied as boundary conditions for a local wave model that will provide input to the sediment transport calculations in the study area. This regional model can also provide wave information on the areas not affected by the Port breakwaters (where diffraction is not relevant). This information will allow determination of the sediment transport conditions that will provide important information for the detailed 2D morphological assessment of Stockton Beach to be carried out later on the study.

This wave transformation has been undertaken using DHI's two dimensional (2D) numerical wave transformation model MIKE21 SW (Spectral Wave Model) which propagates waves from deep water into nearshore areas. The model simulates the growth, decay and transformation of wind-generated waves and swell in offshore and nearshore areas.

MIKE 21 SW includes two different formulations:

- Fully spectral formulation; and
- Directional decoupled parametric formulation.

The fully spectral formulation is based on the wave action conservation equation, as described in Komen et al (1994) and Young (1999). The directional decoupled parametric formulation is based on a parameterization of the wave action conservation equation. The parameterization is made in the frequency domain by introducing the zeroth and first moment of the wave action spectrum. The basic conservation equations are formulated in either Cartesian co-ordinates for small-scale applications and polar spherical co-ordinates for large-scale applications.

The fully spectral model includes the following physical phenomena:

- Wave growth by action of wind;
- Non-linear wave-wave interaction;
- Dissipation due to white-capping
- Dissipation due to bottom friction;
- Dissipation due to depth-induced wave breaking;
- Refraction and shoaling due to depth variations;
- Wave-current interaction; and
- Effect of time-varying water depth.

The directional decoupled parametric formulation is based on a parameterization of the wave action conservation equation; following the Holthuijsen et al (1989) approach, the parameterization is made in the frequency domain by introducing the zeroth and the first order moment of the wave action spectrum as dependent variables.

The discretization of the governing equation in geographical and spectral space is performed using cell-cantered finite volume method. In the geographical domain, an unstructured mesh technique is used. The time integration is performed using a fractional step approach where a multi-sequence explicit method is applied for the propagation of wave action.

To obtain the most accurate description of the nearshore wave climate it has been chosen to transform the entire 12 years of offshore wave data into selected nearshore locations along Stockton Bight. It should be remarked that this model does not allow a description of wave diffraction around structures therefore the extraction locations have been defined in areas where diffraction is not negligible.

Due to the extension of the area and the numerical model requirements, the computational effort associated with modelling of each single wave event in the offshore wave data is relatively heavy. Correlations between the offshore and the nearshore wave conditions have been established through wave modelling of a number of selected offshore wave conditions, covering the whole range of wave conditions in the offshore data. Based on the results of these simulations, three dimensional transformation matrices covering all relevant combinations of wave heights, wave periods and wave directions between the offshore and the near shore waves have been established for selected locations along the study area. Time series of nearshore waves at each location are subsequently established by interpolating the transformation matrices. Due to the large range of computed offshore wave conditions the potential interpolation errors introduced in this procedure have been minimised.

4.2.1 Model Setup

MIKE 21 SW utilises a calculation mesh that requires a good representation of the coastal region around Stockton Beach. A model mesh was established on the basis of data provided by DNR and available sea charts for the area. The extension and direction of the applied model mesh is shown in Figure 4-1.

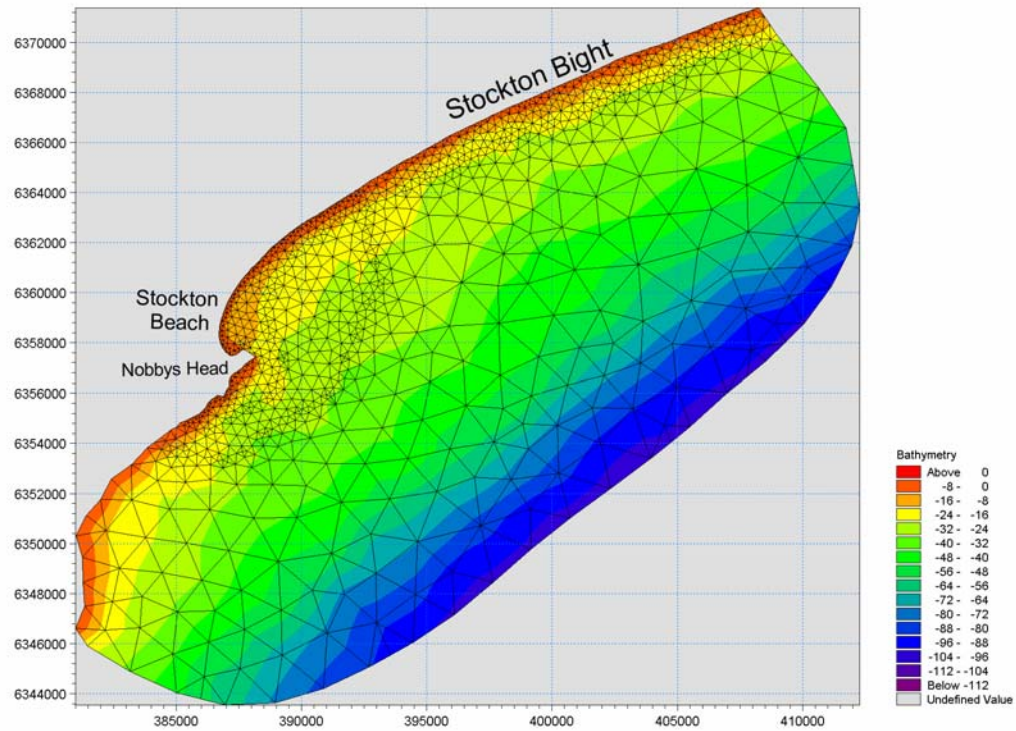


Figure 4-1 Coverage and orientation of the model mesh applied in the nearshore wave transformation modelling and the direction of the main offshore mean wave directions..

The simulations have been carried out with the following conditions:

- Decoupled parametric spectral formulation;
- Quasi stationary time formulation;
- Directional discretisation 10degrees;
- Bottom friction k_n=0.002m
- Wave breaking parameters in the Battjes & Janssen wave breaking model have been set to γ_1 =1.0, γ_2 =0.8 and α=1.0.

Both the bottom roughness and the wave breaking are only primarily important in the surf zone. Parameters related to the discretisation of the wave spectra in the model are maintained at well proven default values.

A simulation matrix was set up to cover the entire range of the offshore wave conditions. The three input parameters in the transformation matrix are the offshore mean wave direction (MWD), the significant wave height (H_{m0}) and the peak wave period T_p. All combinations of these three parameters listed in Table 4-1 were simulated, leading to a total of 2142 conditions.

Table 4-1 Simulated offshore wave parameters. All 2142 combinations were simulated

Mean Wave Direction MWD (°N)	Significant Wave Height, H_{m0} (m)	Peak Wave Period, Tp (sec)
30	0.2	4.8
40	0.5	7.2
50	1.0	9.6
60	1.5	12
70	2.0	14.4
80	2.5	16.8
90	3.0	19.2
100	3.5	
110	4.0	
120	4.5	
130	5.0	
140	5.5	
150	6.0	
160	6.5	
170	7.0	
180	7.5	
190	8.0	
200		

The nearshore wave simulations were carried out with a water level of 0.0m AHD that approximately corresponds to mean sea level (MSL) at Newcastle Port. This is considered to be appropriate to derive average as well as extreme wave conditions at water depths down to 23 and 17m AHD.

The fetch between the locations of the offshore wave data measurements is limited to a few kilometres. This short fetch means that there is limited opportunity for local wind waves to develop further between the location of the waverider buoys and the study site. This being the case the transformation simulations were carried out without the inclusion of wind action. A sensitivity test was carried out which showed that the effect of the wind in the model area is not significant.

Three-dimensional transformation matrices containing the correlations between the offshore waves and the waves at the different extraction points have been established. Based on these correlations, time series of nearshore wave data corresponding to the available offshore data have been produced. Examples of the correlations between the offshore and nearshore significant wave heights are shown in Figure 4-2 for different wave heights, mean wave direction and Tp=12s just offshore the Port entrance (at 23m depth approximately).

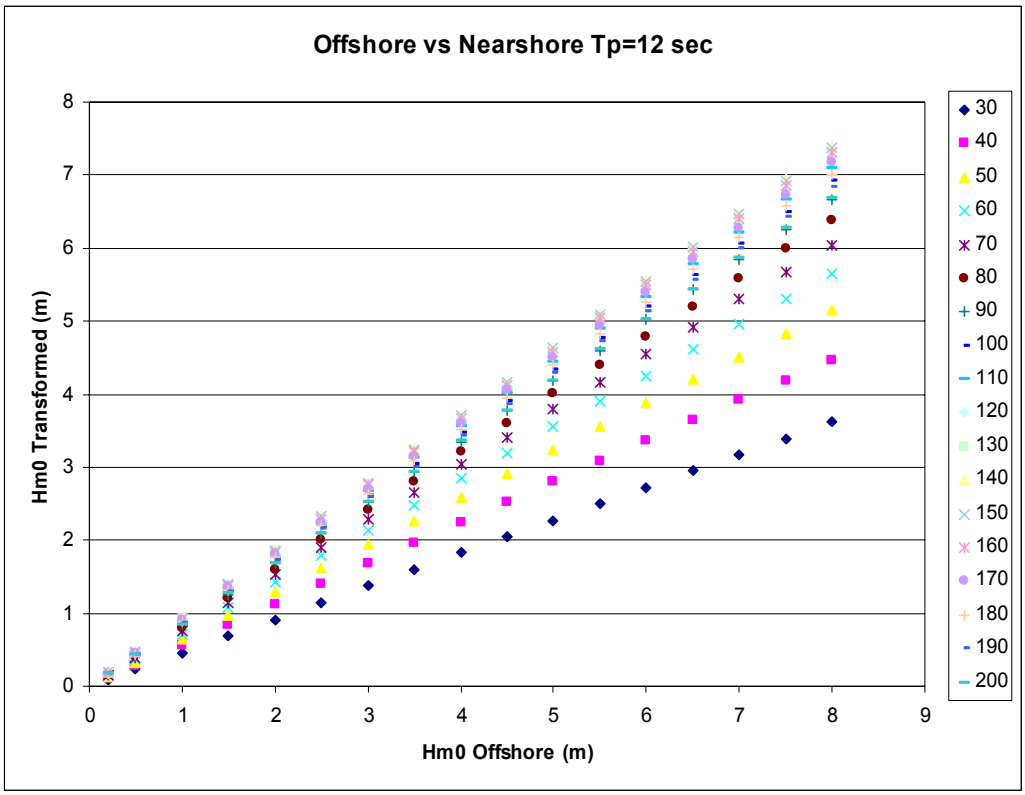


Figure 4-2 Example of correlations between offshore significant wave height and nearshore significant wave height just outside the Port breakwaters. Peak wave period Tp=12 sec.

With the relative linearity of the correlations, it is clear that there is minimal potential for interpolation error by using a correlation approach instead of modelling each event individually. Figure 4-2 shows that the highest waves at Stockton Beach are reached for offshore waves from south to south easterly directions.

4.2.2 Model Calibration

To provide further confidence on the numerical predictions a comparison between model results and measured data in the study area has been performed. This task required that wave data measurements were available at different locations within the study area either simultaneously to the offshore wave measurements or as long-term statistics. In this case the calibration was carried out by comparing model results against wave measurements at a wave measurement station presented in Table 4-2 and shown in Figure 4-3.

Table 4-2 Location of the wave measurement stations

Measurement Station	Coordinates (MGA-56)	
	Easting	Northing
NPC Inner waverider buoy	388634.5	6357482.6

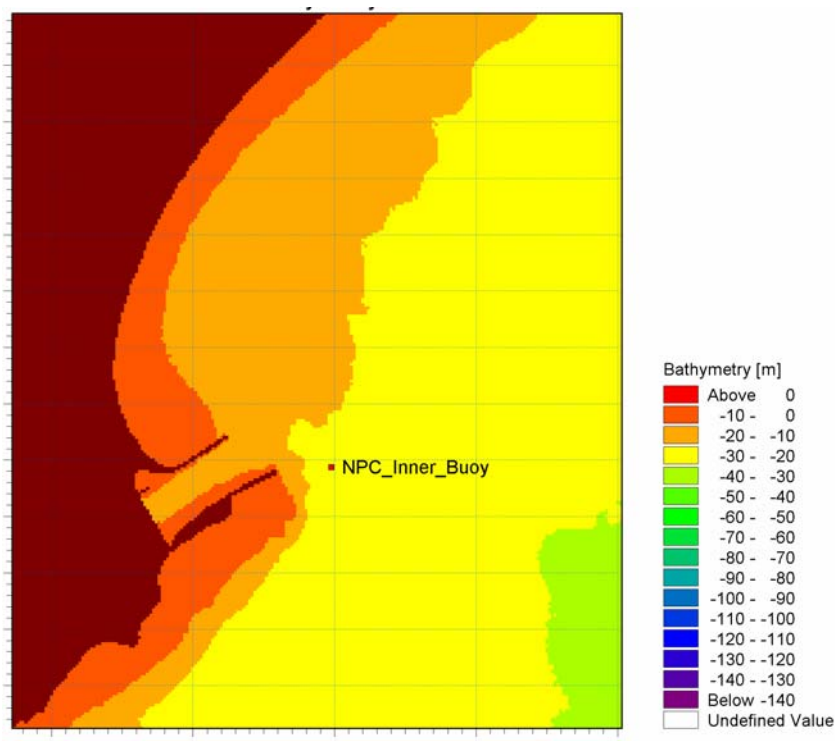


Figure 4-3 Location of wave buoyrider measurements

The model results have been compared to wave measurements at the Newcastle Port Corporation's inner wave rider measurement station. Figure 4-4, Figure 4-5 and Figure 4-6 show the model results compared to measurements for the period November 15 2004 till January 15 2005. The model results show good agreement with the measured data indicating an adequate performance of the model.

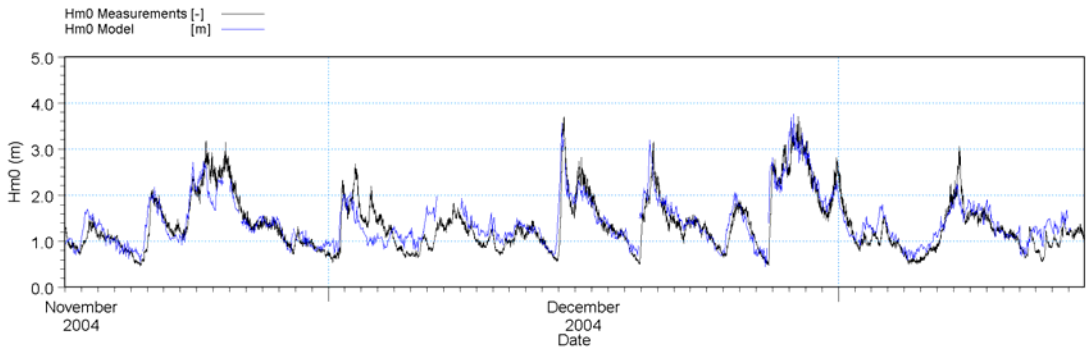


Figure 4-4 Predicted and measured significant wave heights (Hm0) at the Newcastle Port Inner waverider buoy.

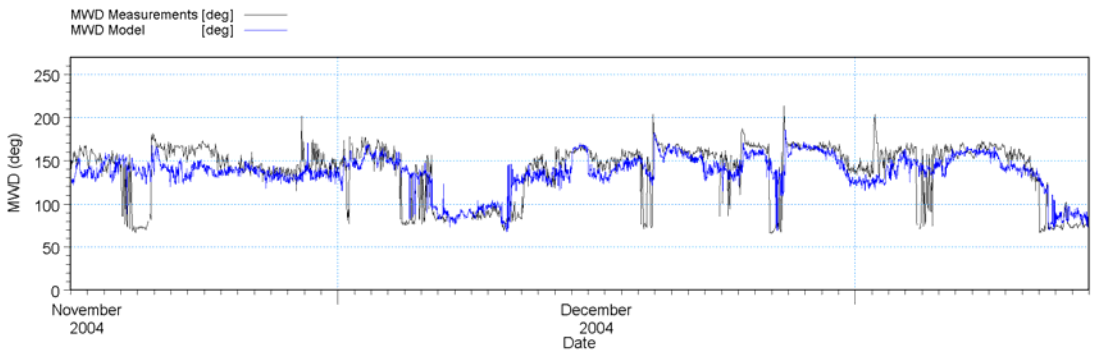


Figure 4-5 Predicted and measured wave direction (MWD) at the Newcastle Port Inner waverider buoy

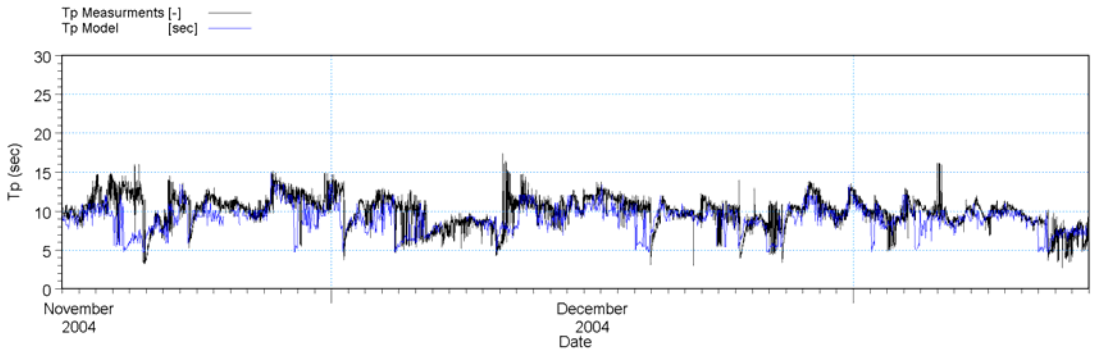


Figure 4-6 Predicted and measured wave peak periods (Tp) at the Newcastle Port Inner waverider buoy

The results show that the model is able to accurately predict the wave conditions in the area. This is expected as the wave propagation is mainly influenced by the varying depth irregularities causing shoaling, refraction, directional spreading and diffraction. All of these processes are included in MIKE 21 SW. Some differences are evident during some intervals and this is most likely caused by variations of the offshore wave conditions at Newcastle as opposed to the location of the Sydney buoy where the waves were measured. Variation is also due to the fact that the Sydney buoy data is based on statistical parameters (Hs, Tp, MWD). It should also be remarked that the wave spectrum contains information on waves propagating from preferential directions eg from both the NE and SE, and this is partially lost during the statistical analysis which may also contribute to the differences observed in the comparisons.

The results also show that the predicted mean wave directions closely follow measurements that are in the range of 100 to 120 degrees. This is expected due to the bathymetric shape of the area.

It should be remarked that wave heights behind the breakwaters may be underestimated because MIKE 21 SW does not include diffraction, however this is not an important issue in this case as the transformation areas are not influenced by structures and therefore diffraction is not relevant for the wave transformation procedure.

Statistical Analysis of the Model Results

In order to quantify the model data against measurements, different statistical indices have been obtained to verify the accuracy of the model results. The following statistical parameters have been evaluated as follows:

$$\overline{m_e}(mean) = \frac{1}{N}\sum_1^n me_i$$

$$Bias = \overline{dif} = \frac{1}{N}\sum_1^n dif_i$$

$$RMS = \sqrt{\frac{1}{N}\sum_1^n dif^2{}_i}$$

$$\rho \quad (correlation) = \frac{\sum_1^n (me_i - \overline{me})(mo_i - \overline{mo})}{\sqrt{\sum_1^n (me_i - \overline{me})^2 \sum_1^n (mo_i - \overline{mo})^2}}$$

where:

me_i = Measured value
mo_i = Model value
$dif_i = mo_i - me_i$

The computed statistical values are presented in Table 4-3, below.

Table 4-3 Computed statistical values

Mean value (m)	bias (m)	RMS (m)	Bias Index (Bias/Mean)	Scatter Index (RMS/Mean)	Correlation/r^2
1.35	0.07	0.29	0.05	0.22	0.87/0.76

As observed the Bias Index, the Scatter Index, the correlation and the r^2 show good estimates.

Model Calibration Conclusions

The comparison of model results and wave measurements has been undertaken just off the Newcastle Port Corporation. The comparison shows good agreement, indicating that the model provides a good spatial and temporal representation of the wave conditions just offshore the Stockton Beach area. This is a key element in the study as it provides further confidence on the use of the numerical model which will be used for the wave data transformation. This information will be used to provide boundary conditions of the local wave model and the littoral transport budget analysis in the areas not influenced by the effect of coastal structures, especially around the Port entrance.

4.2.3 *Wave Model Results*

Regional Wave conditions

In order to illustrate the regional wave condition, examples of output from the wave models for all the simulated wave directions with offshore wave conditions of H_{m0}=4.0m, Tp=9.6s and MWD = 60 and 90 are presented in Figure 4-7 and Figure 4-8.

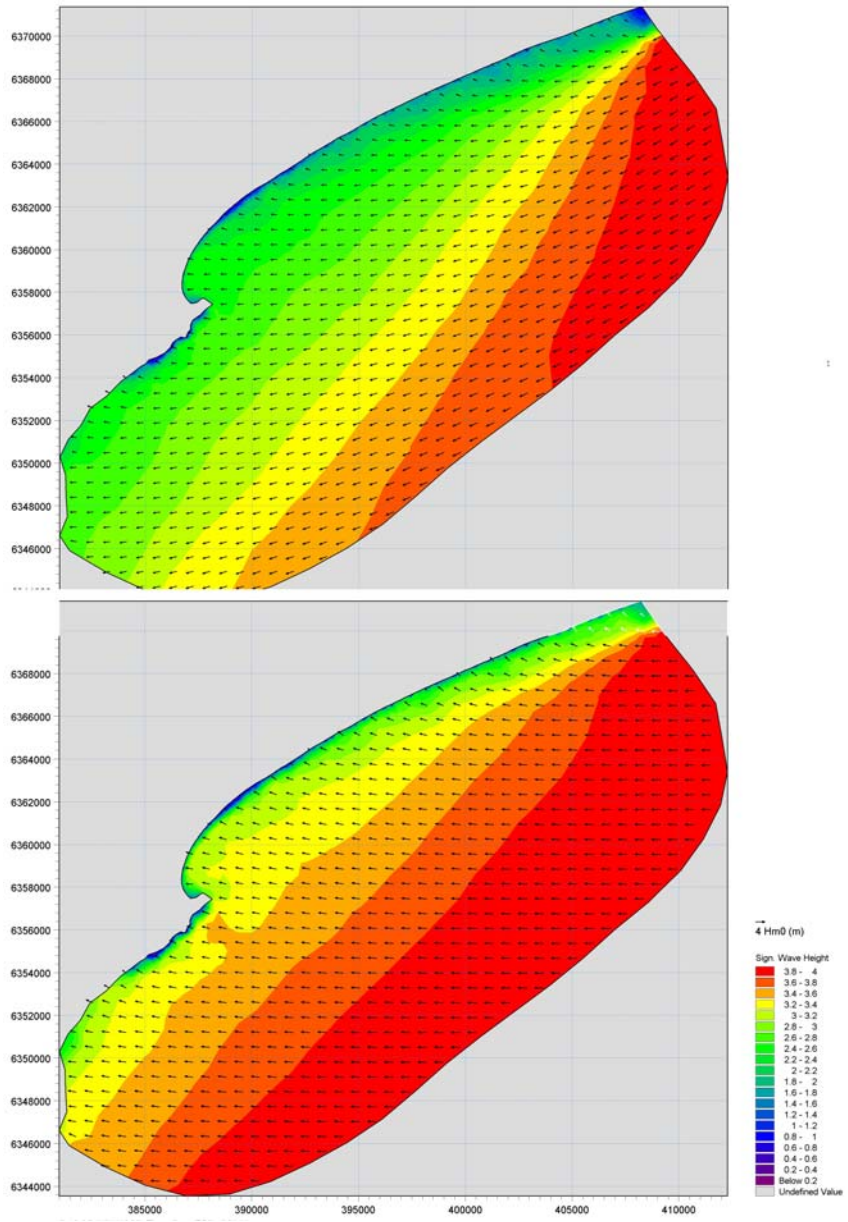


Figure 4-7 Output from nearshore wave simulation offshore H_{m0}=4m, and T_p=9.6s, and MWD = 60degN (above) and 90deg N.(below).

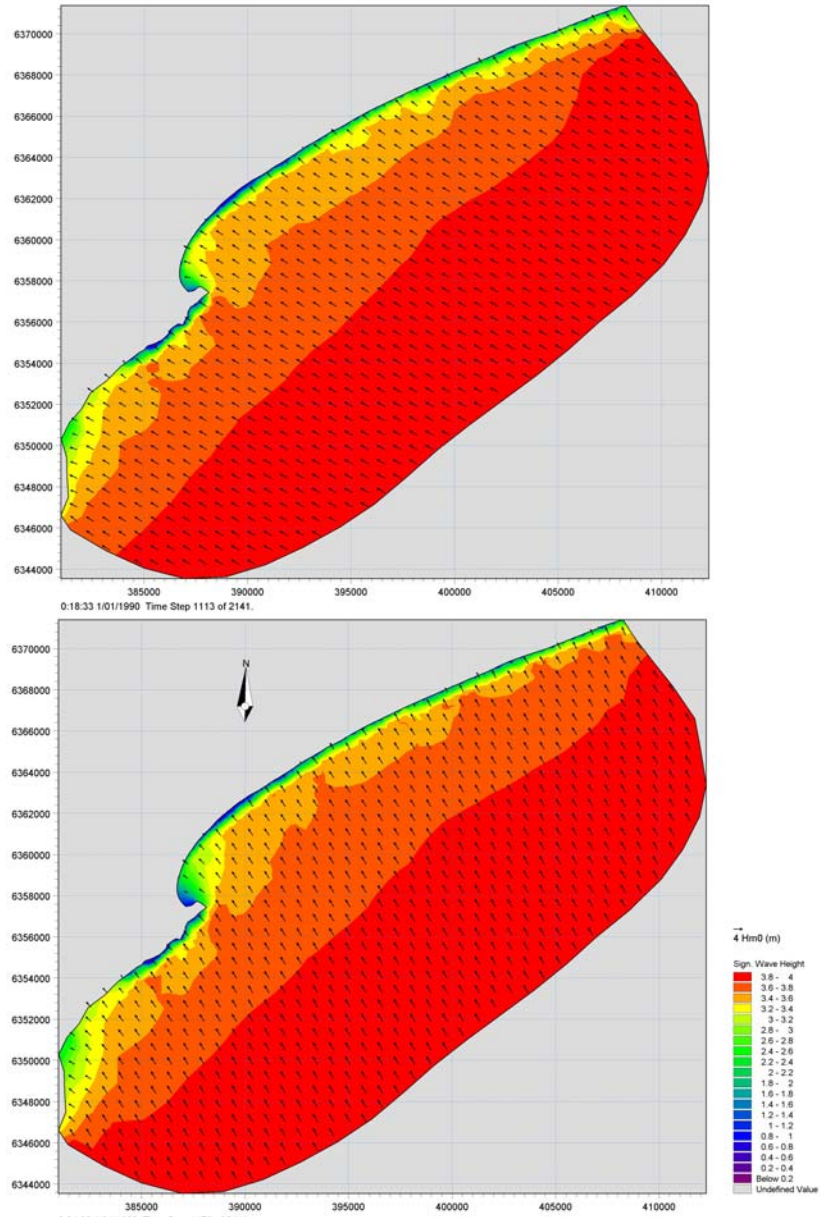


Figure 4-8 Output from nearshore wave simulation offshore H_{m0}=4m, and T_p=9.6s, and MWD = 120deg N (above) and 150deg N (below).

Although not including diffraction, the results show the sheltering effect of the Port of Newcastle breakwaters on Stockton Beach for waves travelling from the S and SE. As expected this effect tends to reduce in a northerly direction until near uniform conditions are reached along the Stockton Bight. This variation in the wave conditions tends to generate a lee area with significant spatial variation of the wave heights.

The data shows that the largest waves at the Stockton area occur when waves offshore propagate from the NE and are able to reach the Stockton nearshore area with minimum energy dissipation. Waves propagating from other directions tend to dissipate due to the

large angle between the incoming waves and the coastal contour orientation that tends to induce a reduction of the wave height due to refraction.

A wave focusing area is also observed southeast of the Port entrance. This is induced by a wave focussing effect of the shoal east of Nobbys Head. For the waves travelling from the E (90 deg), this effect tends to focus the wave energy on Nobbys Beach, whereas for waves travelling from the SE the waves tend to focus north of Nobbys Beach at the port entrance and the navigation channel.

It is also observed that wave conditions at Nobbys Beach are rather complex due to the non-uniformity of the bathymetric contours in this area that induces a complex wave pattern. It is expected that this characteristic will most likely have an effect on the generation of the littoral drift in this area and consequently the sediment bypassing of the Port structures.

Nearshore Wave Roses

For transformation of the offshore time series into the nearshore areas, wave parameters were extracted at water depth of approximately -17m AHD at different locations along the Stockton Bight. The locations varied from Stockton Beach to the north of Stockton Bight and also at the beaches south of the Port entrance. Figure 4-9 and Table 4-4 show the location of the extraction points. At this stage, data will not be extracted from point 1, but will from point 2 because this area may be affected by physical processes not represented by the wave model. The extraction of data in these points will be carried out during the establishment of the local wave model.

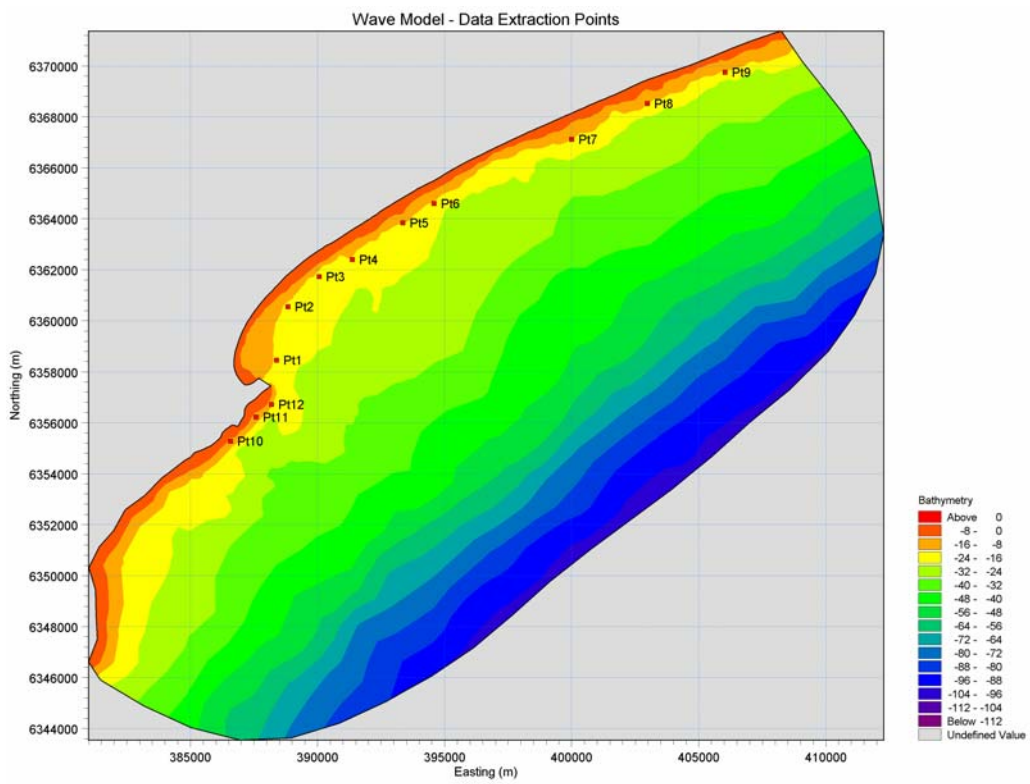


Figure 4-9 Location of the 12 nearshore extraction point along the Stockton Bight at -17m AHD.

Table 4-4 Geographical location of the 12 extraction points for nearshore wave conditions.

Extraction Point	Easting ; Northing (m) MGA -56
3	390064 ; 6361735
4	391372; 6362396
5	393353; 6363835
6	394581; 6364598
7	399995; 6367124
8	402979; 6368525
9	406033; 6369751
10	386571; 6355273
11	387587; 6356220
12	388187; 6356717

By interpolation in the established transformation matrices, time series for nearshore resulting wave conditions have been determined on the basis of the 12 years of offshore wave data. Figure 4-10 shows wave roses for the significant wave height (H_{m0}) at the extraction points along the Stockton Bight.

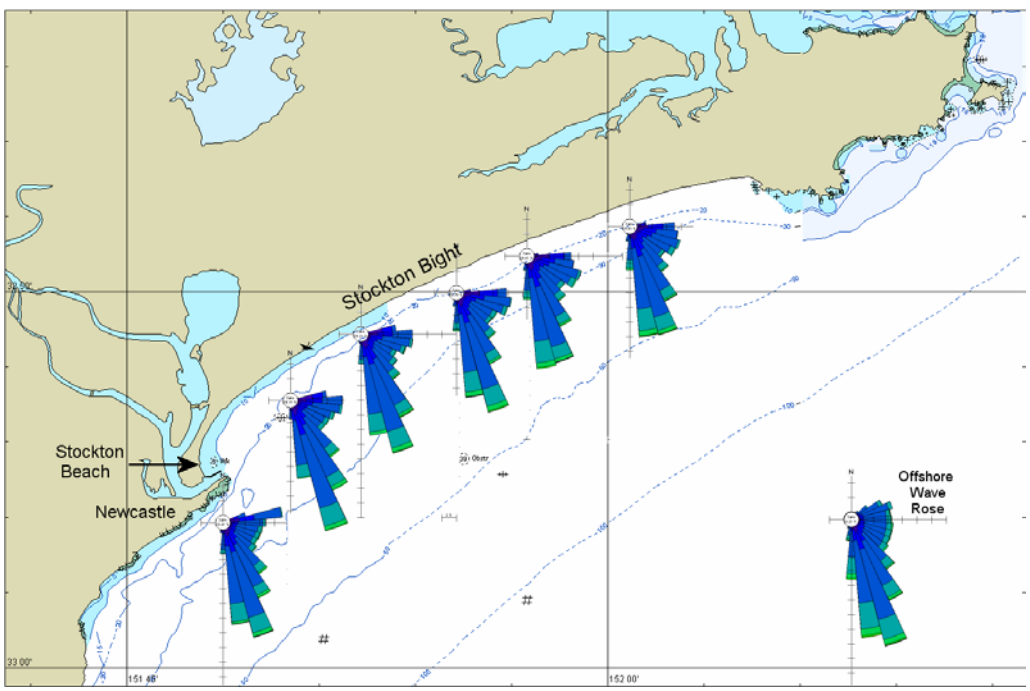
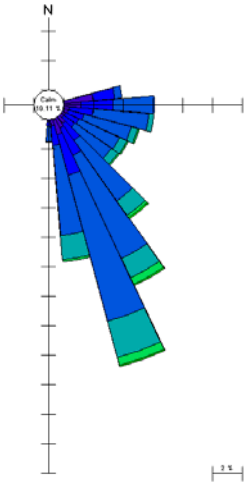
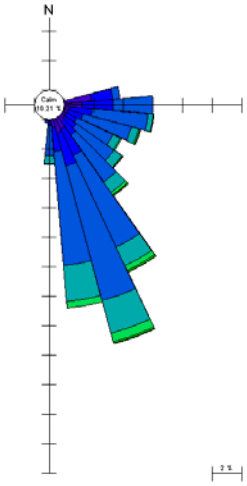


Figure 4-10 Overview of the offshore and nearshore wave roses at Stockton Beach and vicinity areas.

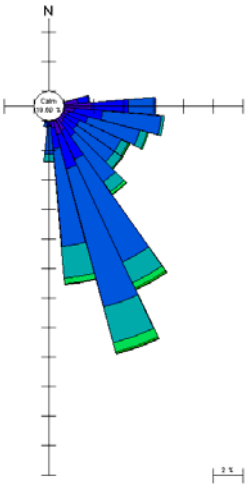
Wave roses for points 3 to 9 are also presented in detail in Figure 4-11.



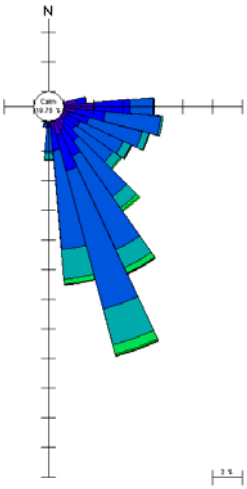
Pt3



Pt4



Pt5



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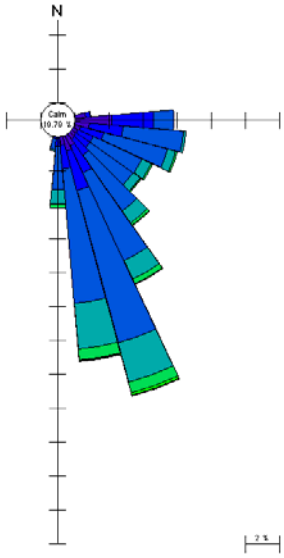
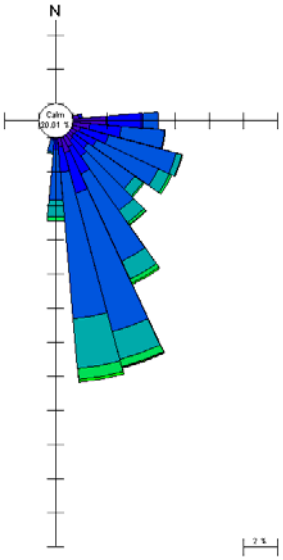


Pt7



Pt8

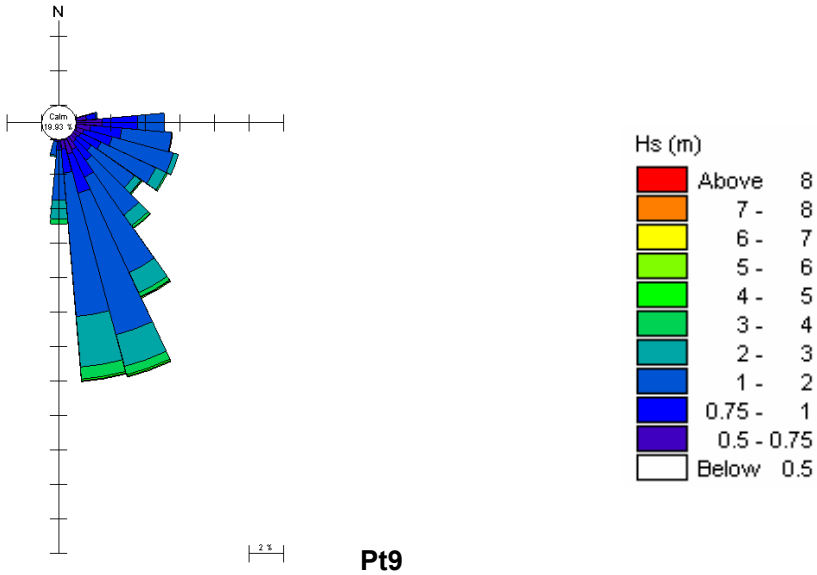


Figure 4-11 Wave roses at the extraction points along Stockton Bight ($H_{m0,res}$) and Nobbys Head at -17m AHD.

4.3 Local Wave Modelling

In order to describe the local wave conditions a local numerical wave model of the Stockton Beach area has been established. The model domain covers Stockton Beach, the Newcastle Port area, the area south of Nobbys Head and the southern end of the Stockton Bight at the vicinity of Stockton Beach.

The model area extends 12 km from SW to NE and 7.9 km from NW to SE. This model extent has been defined to describe the coastal tidal flow patterns, flow interactions with discharges from the Hunter River and the wave driven currents. The model domain has been configured so as to include all relevant bathymetric features that may potentially influence the complex sediment transport processes in the area. The maximum water depth observed in this area is -33m AHD. An overview of the study area and model domain is presented in Figure 4-12.

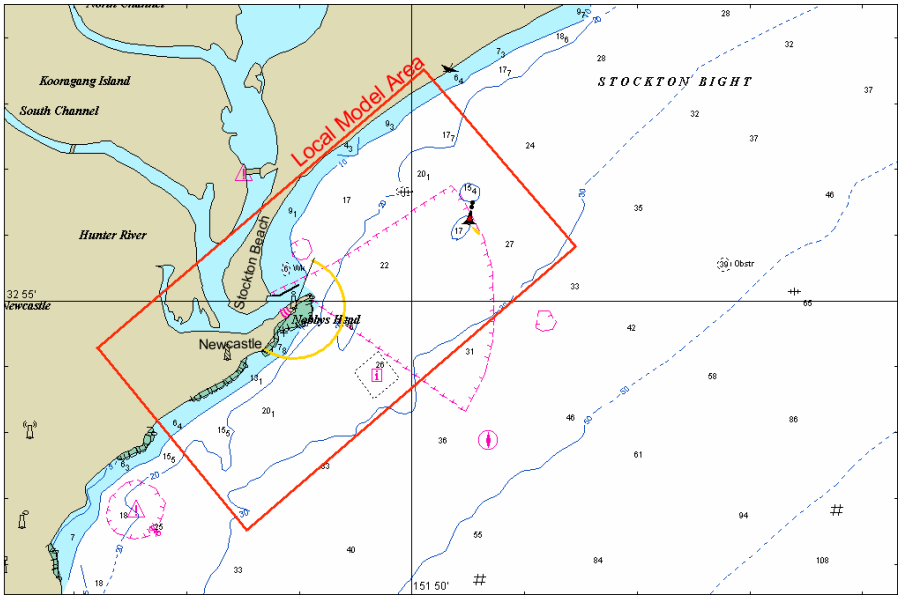


Figure 4-12 Overview of the extension of the local model area.

4.3.1 Wave Model Selection

The definition of the wave model that provides the best representation of the wave conditions in the study requires that the main physical processes involved in the wave propagation are properly represented. Processes such as shoaling, directional and frequency dispersion, refraction, diffraction, reflection, wind-wave interaction, wave breaking, wave-current interaction, etc may need to be described if they have an important effect on the wave pattern.

Depending on the physical processes and the required computational speed, different numerical models are applied, as follows:

- Boussinesq type: this model is based on the solution of the Boussinesq equations and is able to predict most of the wave physical processes;
- Spectral models: this model is mostly applied to determine wave conditions in offshore and nearshore areas; and
- Parabolic mild approximations (PMS): This model is an efficient tool for the determination of wave fields in coastal areas where diffraction and other effects are relevant.

Briefly, the main characteristics of each model are presented in Table 4-5, below.

Table 4-5 Description of different model types and wave related processes included in the model

Process\Model Type	Boussinesq	Spectral	PMS
Directional spreading	Yes	Yes	Yes
Frequency spreading	Yes	Yes	Yes
Shoaling	Yes	Yes	Yes
Refraction	Yes	Yes	Yes
Diffraction	Yes	No	Partly
Reflection and back scatter	Yes	No	No
Bottom friction	Yes	Yes	Yes
Breaking	Yes	Yes	Yes
Wind generation	No	Yes	No
Wave-current interaction	Yes	Yes	No

As it can be observed the Boussinesq model describes most physical processes, however the application of this model is not feasible for the analysis of long-term sediment transport conditions due the run-time requirements of the model. For this reason the model selection is usually restricted to the Spectral and PMS models. In order to justify the selection a comparison based on the application of the Boussinesq, Spectral and PMS model in the Stockton area has been carried out. The simulations were specified for Hm0 = 1m, Tp= 12 sec and MWD= 135 degrees. The model specifications are presented in Table 4-6 below:

Table 4-6 Model specifications

Model Spec\Model Type	Boussinesq	Spectral	PMS
Dt (sec)	0.15	N/A	N/A
Dx (m)	5	5	1.25
Dy (m)	5	5	5

It should be remarked that the models were applied without wave breaking.

Figure 4-13 shows the results of the simulations for the three models for waves only with an additional simulation of the Boussinesq model including wave and currents interactions.

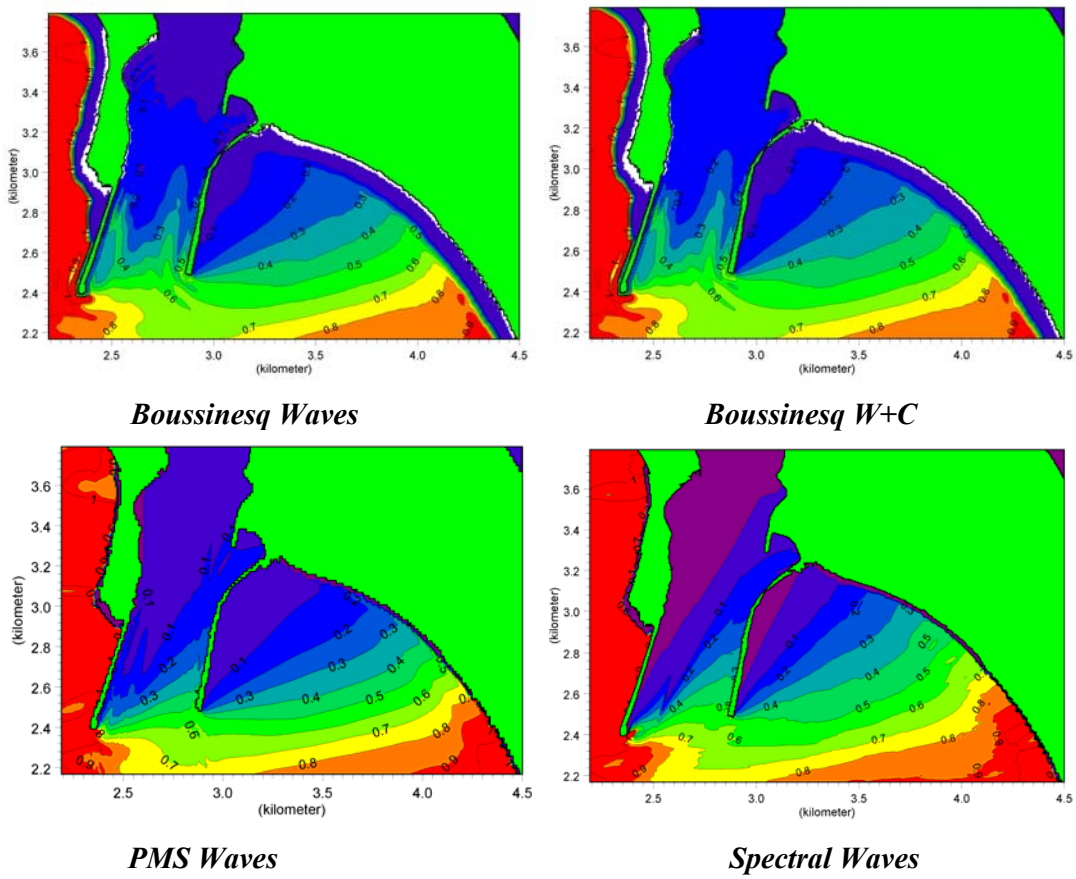


Figure 4-13 Overview of the model results Hm0 (m) Top left Boussinesq model waves only, top right Boussinesq model waves and currents, bottom left PMS model waves only and bottom right Spectral model waves only.

As it can be observed in Figure 4-13, the model results show a very similar wave pattern. Some differences are observed behind the northern breakwater, and this is mainly due to the capacity of the Boussinesq and PMS models of diffracting waves around the structures. This, in turn, tends to reduce the amount of incoming wave energy approaching the northern areas. In order to provide a better method of comparison, wave heights along two extraction lines, along the entrance channel and behind the northern breakwater have been extracted. The location of these extraction lines is shown in Figure 4-14.

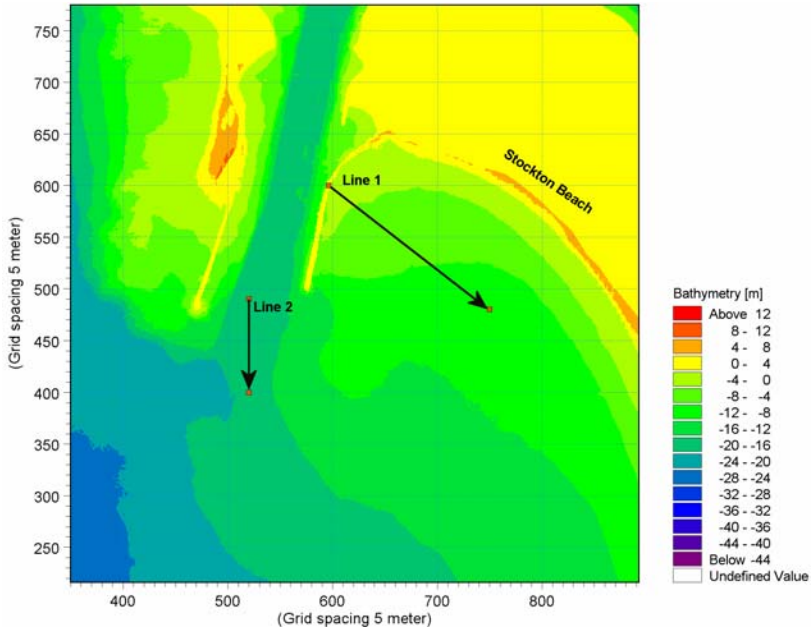
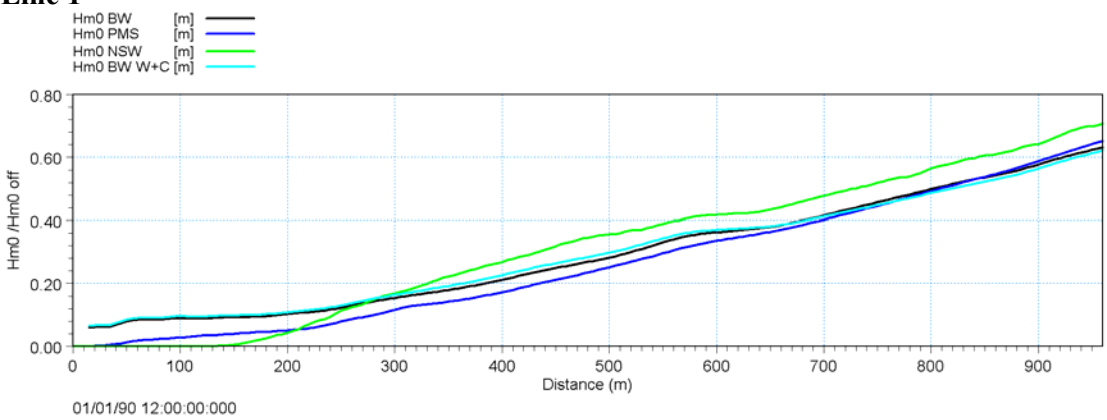


Figure 4-14 Overview of the location of the extraction lines.

Line 1



Line 2

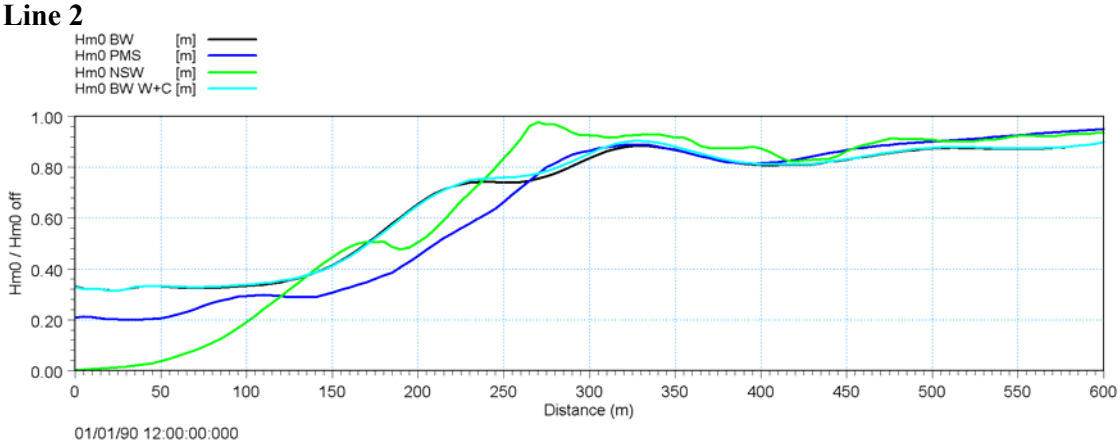


Figure 4-15 Wave height extractions for the different conditions Boussinesq currents only (black), PMS waves only (blue) and Spectral waves only (green) Boussinesq waves and currents (light blue).

Figure 4-15 shows the wave heights extracted along lines 1 and 2. Due to the accuracy of the Boussinesq model, the wave only simulation, this will be used as the base case (black line) for the comparison. The PMS and spectral wave only model results are presented in blue and green lines while the Boussinesq model results for combined waves and currents are shown in the light blue line.

The comparisons show that the NSW model largely underestimates the wave heights behind the Port breakwaters, where the influence of diffraction is significant. The PMS model, on the other hand, performs more adequately with only a slight under estimation of the wave heights being observed. This should be expected as the PMS model is able to partially simulate diffraction. As analysis move away from the structure the PMS model predictions become very similar to the Boussinesq model results, whereas the Spectral model results tend to overestimate the wave heights. This is due to the spectral model not being able to diffract the waves around the breakwaters, therefore a larger amount of the incoming wave energy is able to reach the areas north of the breakwaters.

The comparisons also show that the effect of the wave current interaction is nearly negligible along the two lines. Based on these results, the following conclusions can be drawn:

- The PMS model results show good agreement compared to the Boussinesq model behind the Port breakwaters. A slight under prediction is observed and this should be expected as the PMS model is able to simulate diffraction partially;
- The spectral model under predicts wave heights behind the breakwater, but over predicts the wave heights as analysis is moved away from the breakwater; and
- The effect of the wave current interaction is nearly insignificant. The analysis has been carried out for a discharge of 2800 m^3/s, typical at spring tidal periods at the Port. It should be remarked that discharges during mean and neap tidal conditions will be smaller therefore the effect of the wave-current interactions will be even less significant.

Based on this analysis and the computational requirements of this study, it is concluded that the PMS model provides the best representation of the wave conditions at the Stockton area therefore was applied to carry out the wave computation in the local modelling area.

4.3.2 PMS Model Setup

MIKE 21 PMS is based on a parabolic approximation to the elliptic mild-slope equation. This equation governs the refraction, shoaling, diffraction and reflection of linear water waves propagating on gently sloping bathymetry. The parabolic approximation is obtained by assuming a principal wave direction (x-direction), neglecting diffraction along this direction and neglecting backscatter. In addition, improvements to the resulting equation, Kirby (1986), allow the use of the parabolic approximation for waves propagating at large angles to the assumed principal direction.

An additional feature of MIKE 21 PMS is the ability to simulate directional and frequency spreading of the propagating waves by use of linear superposition.

MIKE 21 PMS can be applied to any water depth on a gently sloping bathymetry, and it is capable of reproducing phenomena, such as shoaling, refraction, dissipation due to bed friction and wave breaking, forward scattering and partial diffraction.

MIKE 21 PMS utilises a bathymetry calculation that requires a good representation of the coastal region around Stockton Beach. A model mesh was established from the data provided by DNR, the NPC and sea charts of the area. The extension and orientation is shown in Figure 4-16.

The model setup has been established as follows:

- Quasi-stationary formulation;
- Boundary conditions obtained from the regional wave model described in Report 2;
- Grid spacing dx=7.5m and dy=15m;
- Symmetrical lateral boundary conditions;
- Varying surface elevation (following tidal conditions at Newcastle);
- PMS coefficients Minimax model, aperture 60 degrees;
- Dissipative interface, coefficient 0.25;
- Bottom dissipation constant Nikuradse n=0.002;
- Wave breaking included γ_1=1, γ_2=0.8 and α=1

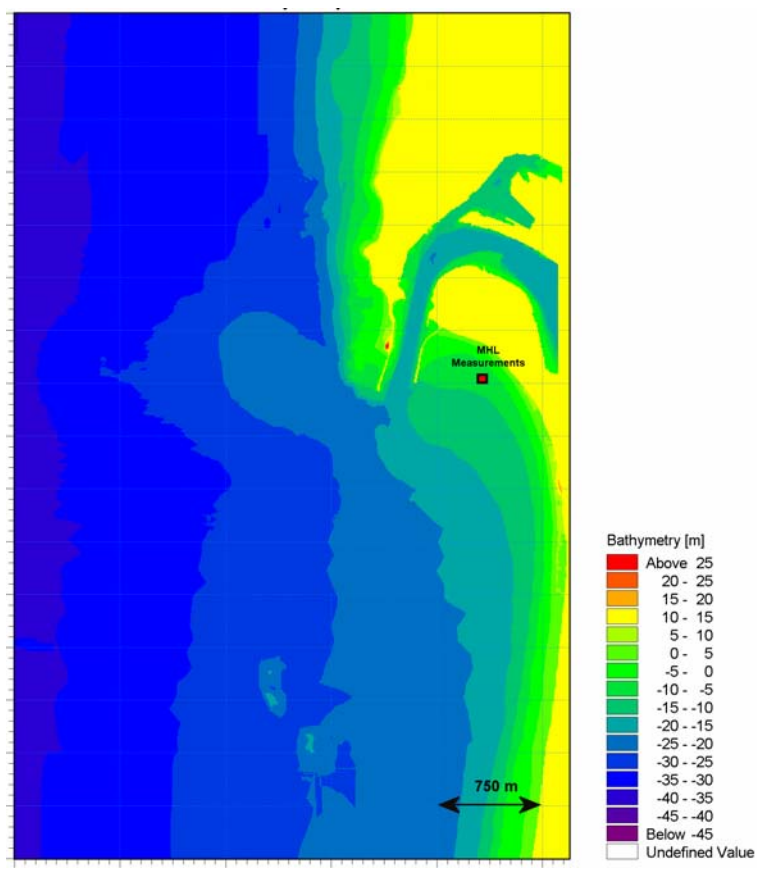


Figure 4-16 Overview of the Parabolic Mild Slope Wave Model bathymetry

4.3.3 Model Comparisons

The PMS model results have been compared to the wave measurements by MHL in the area. Two comparisons were carried out, the first one in June 2001 and the second in July 2001. The model comparisons are shown in Figure 4-17 and Figure 4-18.

As it can be observed the performance of the model is very good. Both wave heights and mean wave directions are predicted accurately, however some differences are observed, which are expected as the boundary conditions are provided by the Sydney waverider buoy which is located 150 km south of the study area. As previously mentioned in the regional wave modelling comparisons, these differences could also be explained due to the bi-peak of the wave spectrum. Waves propagating from the SE and NE, are not fully captured by the offshore wave statistical values obtained from the Sydney waverider buoy. Some of the most significant differences are observed during the storm event on July 6 and 7, where the wave height predictions are smaller than the measurements.

June 2001

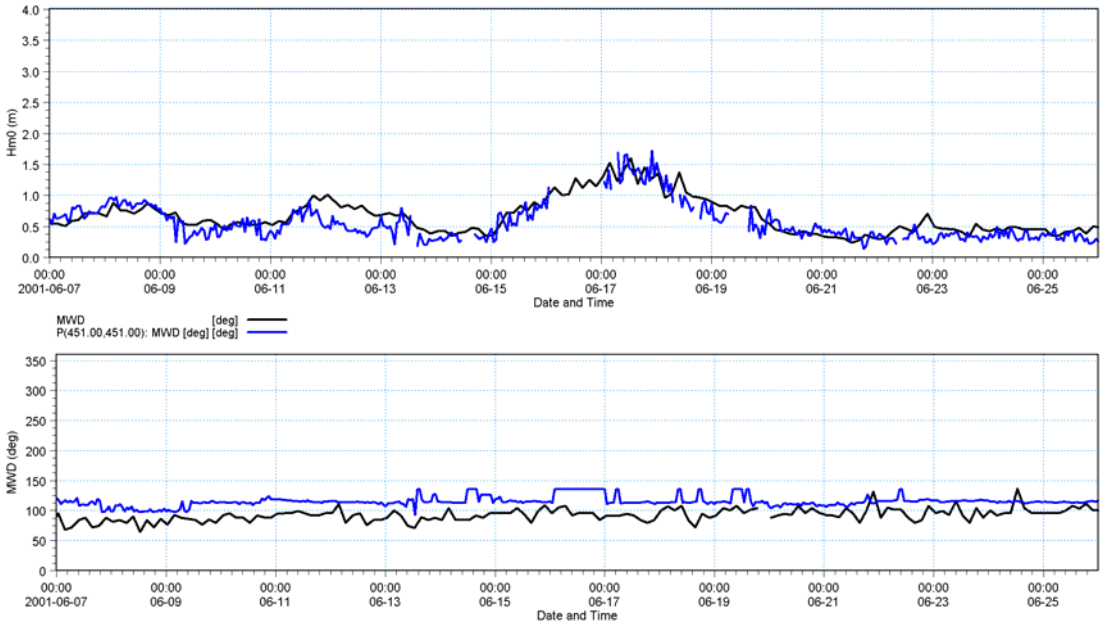


Figure 4-17 Wave predictions compared to wave measurements at the MHL location at 8m depth (AHD)

July 2001

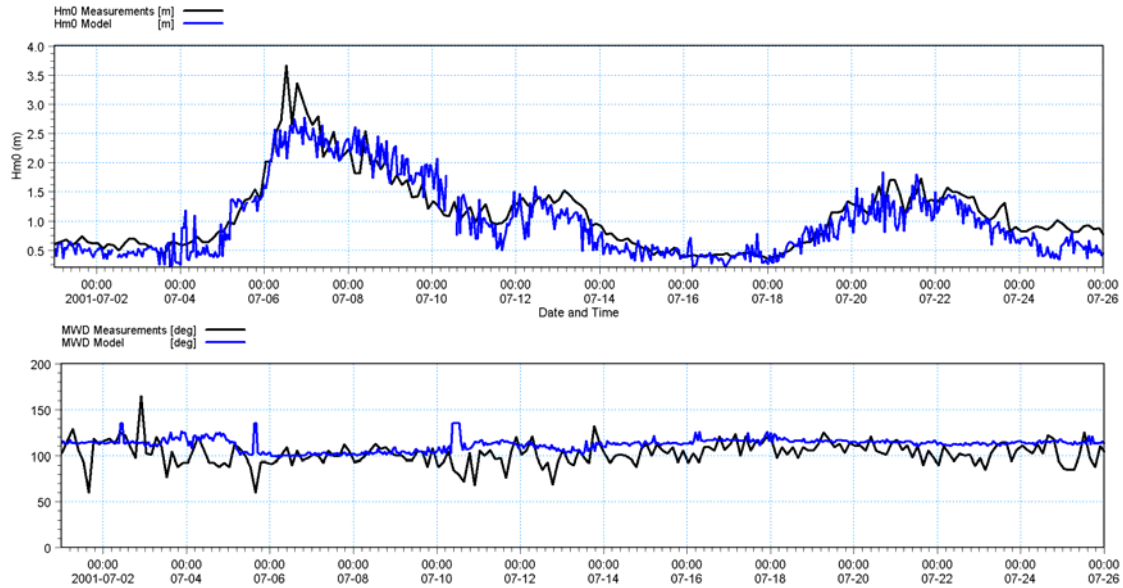


Figure 4-18 Wave predictions compared to wave measurements at the MHL location at 8m depth (AHD)

The results show also that the predicted mean wave directions closely follow the measured mean wave direction in the range of 100 to 120 degrees. This is expected due to the bathymetric characteristic of the area, which tends to diffract and refract the incoming waves.

Statistical Analysis of the Local Model Results

In order to quantify the model results against the measurements, different statistical indices have been obtained to verify the accuracy of the model.

The following statistical parameters have been evaluated as follows:

$$\overline{m_e}(mean) = \frac{1}{N}\sum_1^n me_i$$

$$Bias = \overline{dif} = \frac{1}{N}\sum_1^n dif_i$$

$$RMS = \sqrt{\frac{1}{N}\sum_1^n dif^2{}_i}$$

$$\rho \quad (correlation) = \frac{\sum_1^n (me_i - \overline{me})(mo_i - \overline{mo})}{\sqrt{\sum_1^n (me_i - \overline{me})^2 \sum_1^n (mo_i - \overline{mo})^2}}$$

where:
me_i = Measured value
mo_i = Model value
$dif_i = mo_i - me_i$

The computed statistical values are presented in Table 4-7, below.

Table 4-7 Statistical values of the model wave heights compared to measurements

Period	Mean (m)	Bias (m)	RMS (m)	Bias Index (Bias/Mean)	Scatter (RMS/Mean)	Correlation/r^2
Jun 2001	0.65	-0.08	0.18	-0.12	0.27	0.86/0.74
Jul 2001	1.13	-0.14	0.32	-0.13	0.28	0.89/0.81

Correlation coefficients of 0.86 and 0.89 and r^2 values of 0.74 and 0.81 have been obtained for the two comparison periods. These values show very good statistical agreement between the model results and the measured data.

5 ESTABLISHMENT OF A 2D HYDRODYNAMIC MODEL

5.1 Motivation

The determination of the sediment transport processes at Stockton requires the description of the hydrodynamic (flow) conditions. This section describes the implementation and calibration of a 2D hydrodynamic model of the Stockton Area.

5.2 Introduction

A 2D numerical model of the Stockton Beach area has been established. The model domain covers Stockton Beach, the Newcastle Port area, the area south of Nobbys Head and the southern end of the Stockton Bight at the vicinity of Stockton Beach.

The hydrodynamic model has been developed to describe the coastal tidal flow patterns, flow interactions with discharges from the Hunter River and wave driven currents in the study area

The model area extends 12 km from SW to NE and 7.9 km from NW to SE. This model extent has been defined to describe the coastal tidal flow patterns, flow interactions with discharges from the Hunter River and the wave driven currents. The model domain has been configured so as to include all relevant bathymetric features that may potentially influence the complex sediment transport processes in the area. The maximum water depth observed in this area is -33m AHD. An overview of the study area and model domain is presented in Figure 5-1.

5.3 Bathymetry

No single bathymetric data set was available that covered the required study area. This being the case, the bathymetry of the study domain has been developed using a range of suitable data sources:

- 2002 Bathymetric data of the Stockton area, nearshore and offshore (data provided by DIPNR, blue points)
- 2004 bathymetric data at the Newcastle Port entrance (data provided by NPC, blue points);
- 2001 Beach profiles (provided by DIPNR, blue points); and
- Sea chart data to complement area where measurements or little information was available.

A number of bathymetries with different grid resolutions have been developed and tested for calibration purposes in order to ensure that numerical model is not only able to describe the tidal flows but also wave driven currents. Model grid sizes tested included:

- Dx=Dy=10m grid;
- Dx=Dy=12m grid;
- Dx=Dy=15m grid; and

- Dx=Dy= 20m grid.

All grids have been established with an orientation of 225 degrees from the y-direction axis of the grid relative to North. Figure 5-1 illustrates the 2D bathymetry at a 15m resolution grid that has been produced with the collated data.

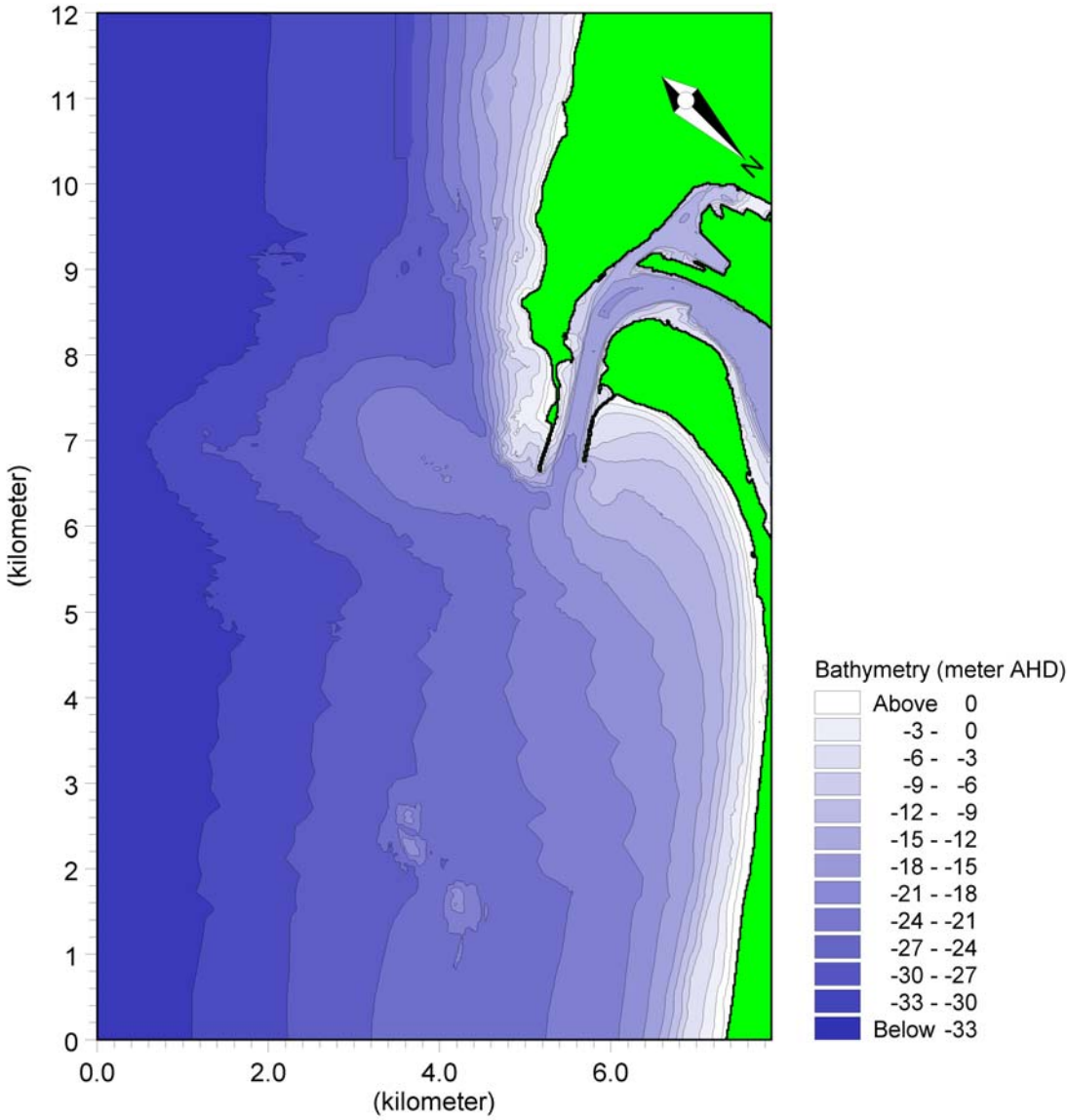


Figure 5-1 General overview of the 2D bathymetry of the study area.

Figure 5-1 demonstrates that the nearshore coastal areas and the main bathymetric features offshore are well described. These features include the shoal located south east of the port entrance and the rocky formation of Stockton Bight commonly known as the Pinnacles. The Port entrance area is also well defined. This is an important requirement if the complex flow exchange occurring between the ocean, port and river are to be correctly represented.

5.4 Boundary Conditions

The establishment of the 2D numerical model requires the definition of the consistent and representative hydrodynamic conditions at the open model boundaries. The numerical model contains four open boundary conditions listed below and illustrated in Figure 5-2 as follows:

- Boundary 1: connects the Hunter River to Newcastle Port. This boundary is located at the right side of the bathymetry within 6 to 8.5 km;
- Boundary 2: top open area of the model;
- Boundary 3: offshore open area; and
- Boundary 4: bottom open area of the model.

Boundaries 2, 3 and 4 are open ocean conditions whereas boundary 1 is a connection between the Hunter River and the Port area. Representative and consistent boundary conditions describing the flow conditions at all boundary locations are required if the model is to correctly predict flows in the study area.

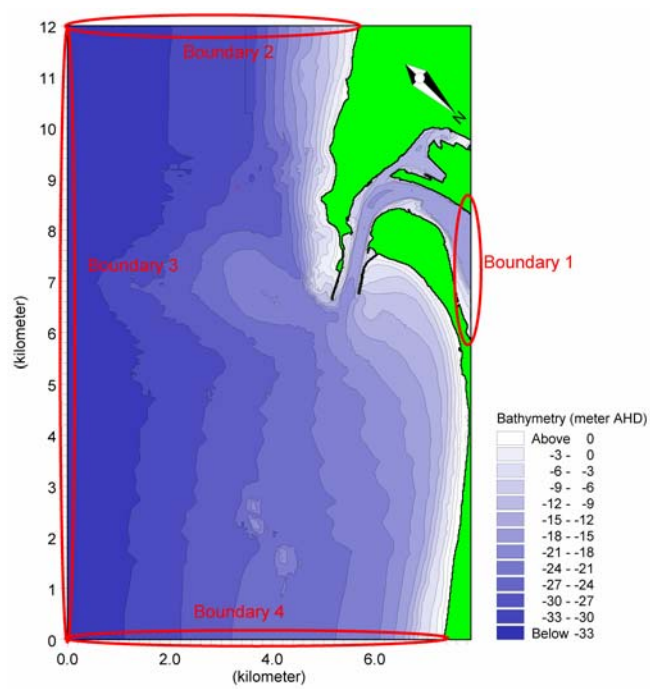


Figure 5-2 Description of the 2D model boundaries.

5.4.1 Ocean Boundary Conditions

Analysis of flow conditions in the Stockton Beach area have been undertaken as part of the data analysis. These results are presented in Status Report 1 and show that tidal currents are not significant in the nearshore area of the Stockton region. The small spatial variations in ocean water level that are required to define these ocean boundaries using water level conditions can be obtained from tidal predictions produced from tidal constituents such as those obtained from the Australian Tidal Tables. An example of

the tidal constituents used to determine the ocean boundary conditions in the model is presented in Table 5-1.

Table 5-1 Harmonic Constants amplitude (m) and phase (degrees) for Newcastle

Component	M2	S2	K1	O1	Sa	Ssa	Mm	Msf	Mf	S1	Q1
Amp (m)	0.502	0.125	0.162	0.091	0.041	0.022	0.003	0.002	0.005	0.005	0.021
Ph (deg)	238.4	262.8	117.4	78.4	43.9	135.4	127.9	33.4	208.8	31.4	54.1

Component	P1	N2	NU2	K2	L2	2N2	MU2	T2	M4	MS4	2MS6
Amp (m)	0.047	0.109	0.02	0.037	0.015	0.02	0.021	0.007	0.005	0.002	0.003
Ph (deg)	110.7	225	224.8	251.7	246.1	199.7	215	281.9	136.3	187.5	128.4

These harmonic constants can be used to predict a tidal water level time series. An example of predicted tidal levels for Newcastle Port in December 2004 is presented in Figure 5-3.

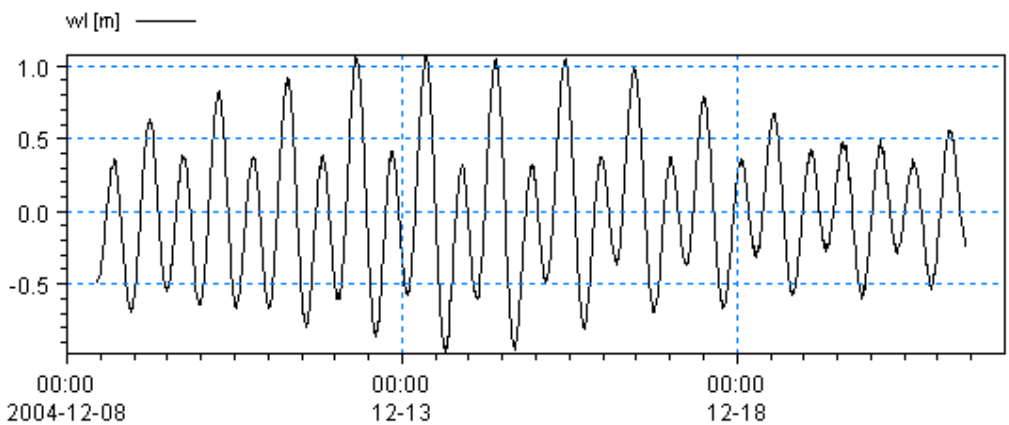


Figure 5-3 Predicted water levels at Newcastle base on tidal harmonics for a period in December 2004.

5.4.2 River Boundary Condition

The water volume exchange across the port entrance, known as the tidal prism, is one of the key factors influencing sediment bypass at the port entrance and the sediment transport conditions at Stockton.

The tidal prism is dependant on the amplitude of the tide and the storage capacity of the river tidal flats and is also influenced by the river discharge. Modelling these flow behaviours with a 2D numerical model requires a definition of the river that represents all tidally influenced river reaches. This allows the model to account for the full tidal storage capacity and the conveyance of the system.

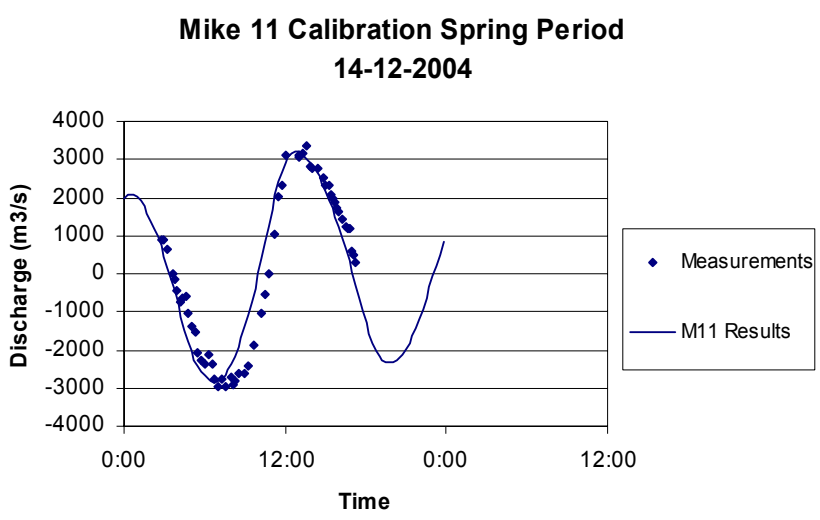
Development of a full 2D model including the river is possible, but it requires that the model includes the full river system at a grid resolution that enables the definitions of all tidal channels. In the case of the Hunter River, which is tidal to Bolwarra, this is obviously a very computational demanding exercise. It is difficult to justify a full a detailed 2D model of the river on the basis of the additional information that it would provide, in the coastal areas where the sediment transport analysis is to be carried out.

To overcome this computational issue, a hybrid approach has been applied that includes the application of the 1-dimensional hydrodynamic model MIKE 11. The application of this model will determine the flow conditions of the wider Hunter River Estuary, providing boundary conditions for the 2D numerical model. The Lower Hunter River MIKE 11 model developed by the Public Works Department in 1994 for the Lower Hunter River Flood Study (Green Rocks to Newcastle) was applied and compared to 2004 conditions for this purpose.

The flow conditions in the lower reaches of the Hunter River are strongly affected by tidal conditions. As part of the Lower Hunter River Flood Study, the MIKE-11 model was calibrated tidally for a period of low river flow in November 1990.

For the purposes of this study, the previously calibrated MIKE-11 model was used to simulate tidal conditions for the period in December 2004 for which ADCP flow measurements at the river mouth were available. The model predicted flows for the December period and were compared to the ADCP transect measurements in order to verify the validity of the model for the 2004 river conditions. Computed and measured discharges were compared and significant variations were observed, indicating that the Lower Hunter channel has undergone significant changes during the period between 1994-2004 that are not included in the model which has a fixed bathymetry derived from survey measurements between 1984 and 1990.
In order to improve the predictive accuracy of the model, the model bed level was modified in the lower river reaches to account for these river bed level changes, Figure 5-4 shows the computed and measured discharges from the adjusted model for spring and neap tidal periods. The figure shows that the adjusted model has achieved results with good agreement to the measurements. Slight differences are observed during the spring period during flood tide conditions; however these are not significant and will not play a significant role on the sediment transport calculations.

It should be noted that the MIKE 11 model could be more rigorously and better improved to match tidal behaviour with an updated river channel bathymetry, especially at the lower end of the river, where dredging works have been carried out in the interim period.



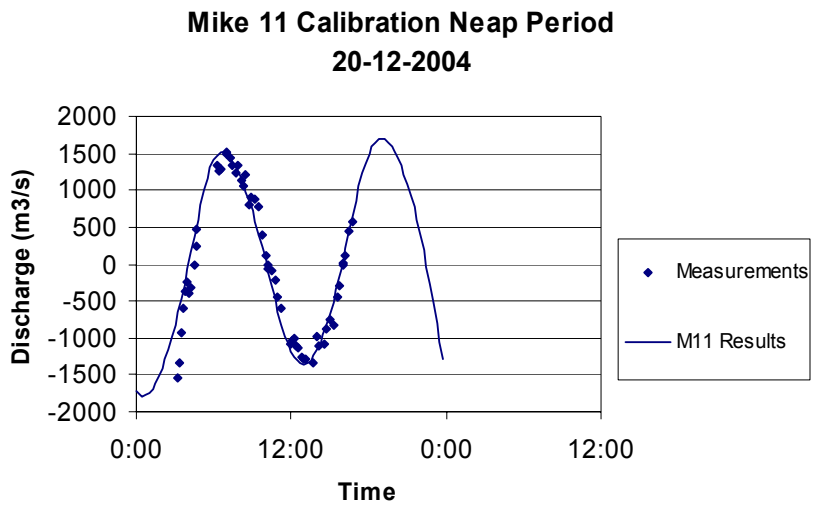


Figure 5-4 Mike 11 Model results compared to measurements at spring and neap tidal periods.

5.5 General Model Specifications

The development of a 2D hydrodynamic model requires the definition of a number of model parameters. Many of the model parameters are defined based on previous coastal modelling experience, however in many cases it is necessary to re-assess parameters to account for the specific flow behaviour observed in a local area. The process of parameter selection is aided by the availability of data measurements when they are available. The following model parameters are relevant in this instance:

- Model Roughness coefficient: the roughness in the model has been defined as constant equal to Manning number. A range 20-40 $m^{1/3}/s$ is recommended, in our simulations a constant value of 40 $m^{1/3}/s$ was specified;
- Eddy viscosity: the eddy viscosity can be defined as constant or a time varying-function of the local gradients in the velocity field (Smagorinsky concept);
- Flooding and drying: has been defined 0.1 and 0.2m.

5.6 Model Calibration

The 2D hydrodynamic model has been calibrated against data measurements at the Hunter River entrance. An adequate model calibration provides additional confidence in the hydrodynamic model results which is essential for the sediment transport calculations.

The calibration period has been defined as being from 08-12-2004 till 21-12-2004. This period provides a good representation of the flow conditions at the river entrance as it includes both spring and neap tidal conditions. A water level series for the period is illustrated in Figure 5-5. Note that the amplitude during the spring period is the maximum occurring for the year.

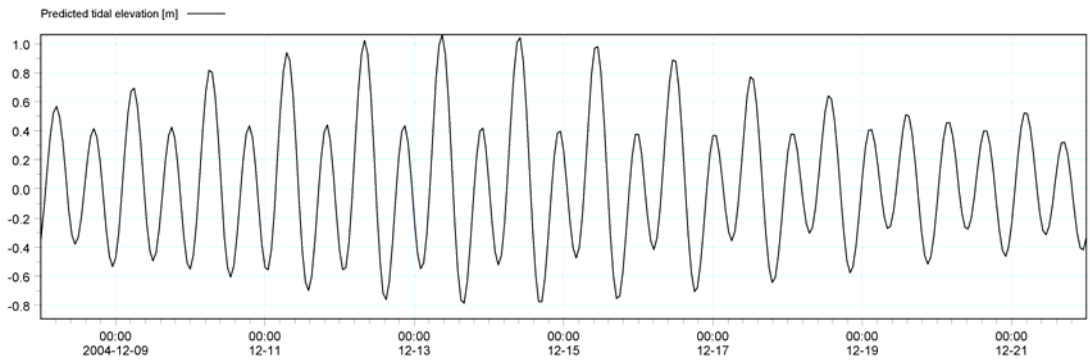


Figure 5-5 Water levels during the measurement period.

A bottom mounted ADCP was placed at the river entrance for the period and discharge transects measured from a vessel mounted ADCP were computed during spring and neap periods. The location of the discharge transects and the bottom mounted ADCP measurements are shown in Figure 5-6.

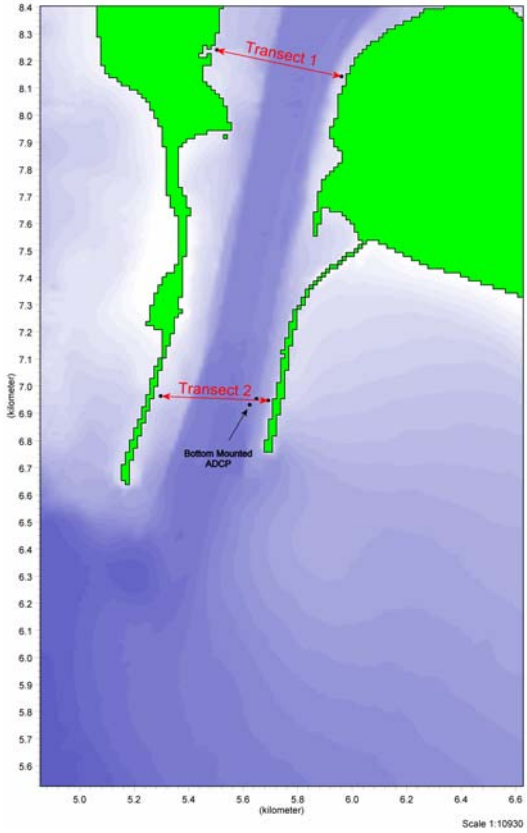


Figure 5-6 Location of the bottom mounted ADCP and the ADCP transects.

A comparison of the predicted water levels for models with the different grid resolutions of 20m, 15m, 12m and 10m grid have been extracted and compared as illustrated in Figure 5-7. The model predictions show good agreement with the data and the results of the different grid models are very close.

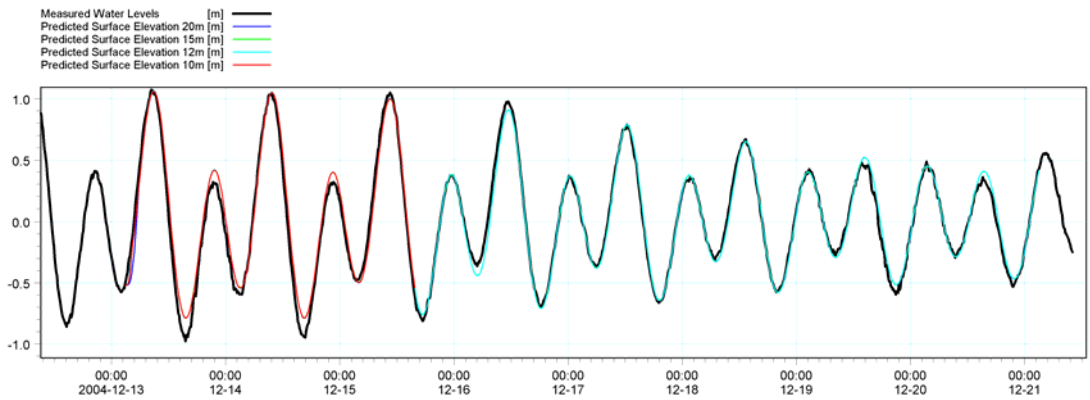


Figure 5-7 Predicted and computed water levels comparisons for the different models.

The model results were also compared to measured speed and current direction directions at the bottom mounted ADCP location and are presented in Figure 5-8. As Figure 5-8 shows there is a good agreement between measurements and model results. It should be noted that the location of the bottom mounted ADCP is in an area with large velocity gradients next to an eddy close to the northern wall. This eddy tends to modify its position therefore some of the differences in Figure 5-8 may be produced by small errors in the location of the extraction point that can generate some variation in the model predictions.

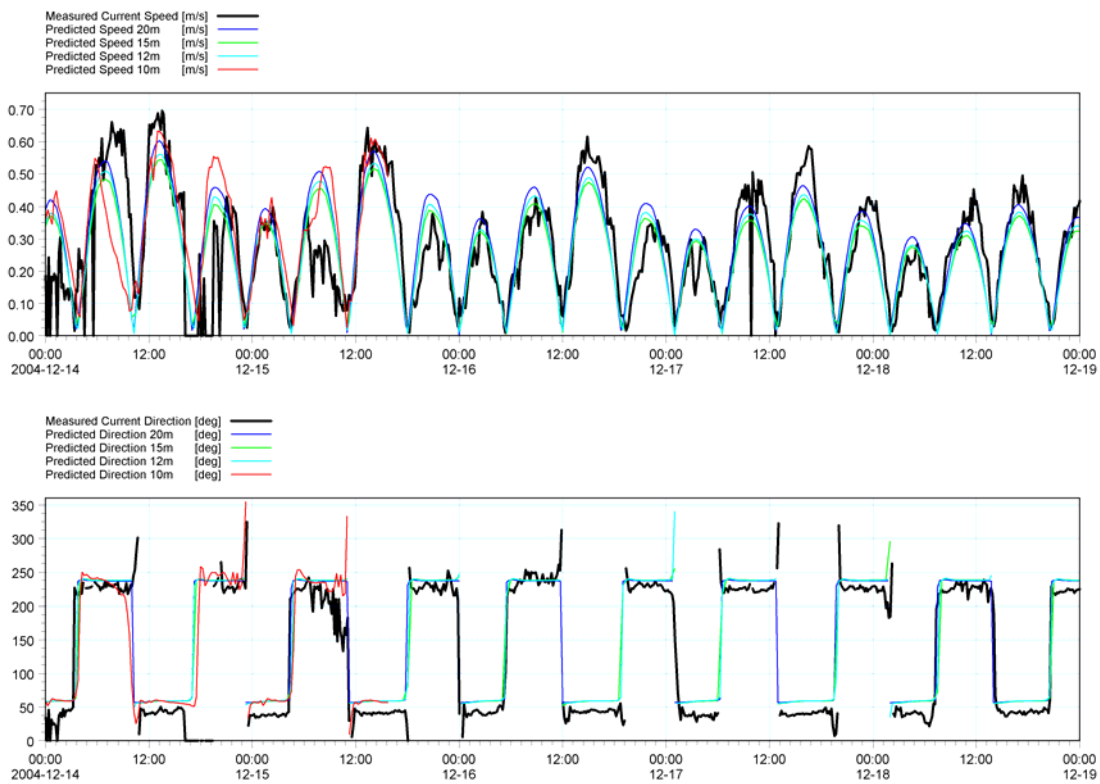
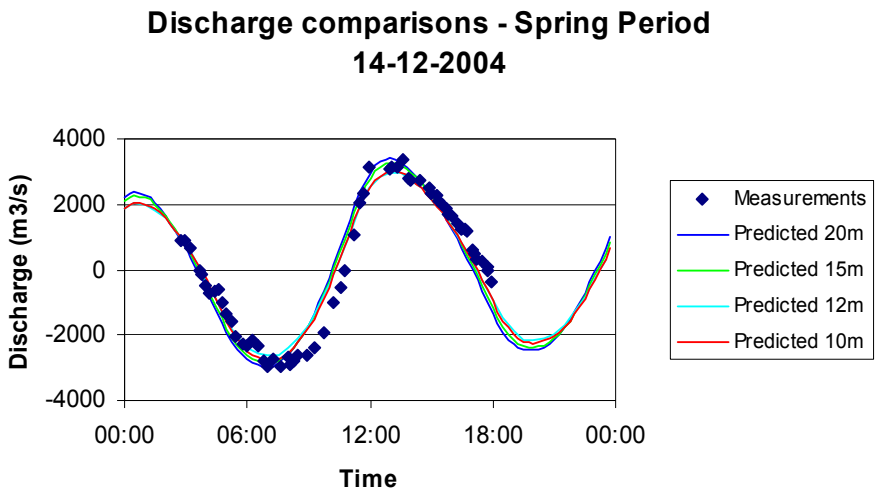


Figure 5-8 Speed (above) and current direction comparisons

Finally the computed and measured discharges at the river entrance were compared as shown in Figure 5-9. As it can be observed, the numerical model predictions closely

follow the field measurements. No major changes are observed for increasing grid resolutions. A slight variation occurs during spring conditions for the smaller resolutions (10 and 12m grid spacing), that is produced by the increased vorticity that this grid size is able to predict. This is especially so right at the river entrance where the flow generates a recirculation pattern behind the breakwaters.



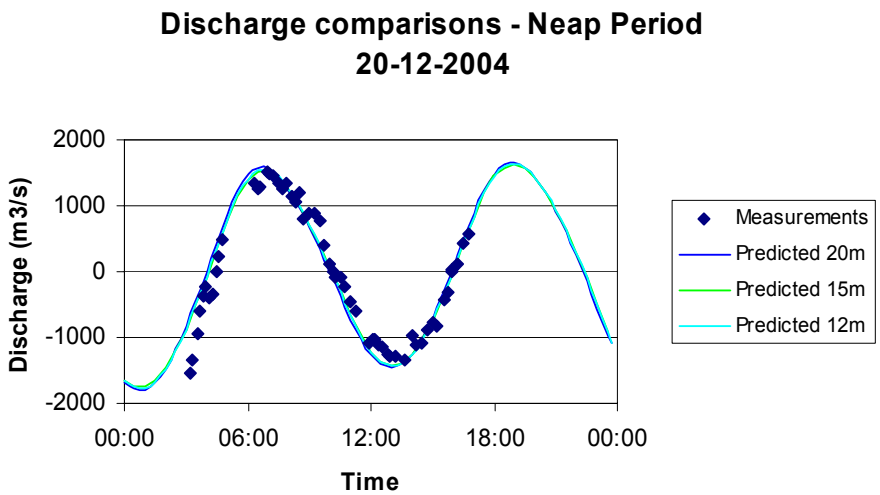


Figure 5-9 Discharge comparisons during Spring (above) and Neap (below) conditions

The calibration shows that the model is able to predict the main flow features and behaviours occurring at the channel entrance. Velocities, discharges and water levels show very good agreement.

A comparison has also been carried out for the measured and predicted velocities along the ADCP transects as shown in Figure 5-10 for flood and ebb conditions. The comparisons show a very good agreement along the port entrance. As it can be observed the model also predicts the eddy at the northern side of the breakwater during flood tide conditions. The yellow arrows in Figure 5-10 indicate measured velocities and the red computed values.

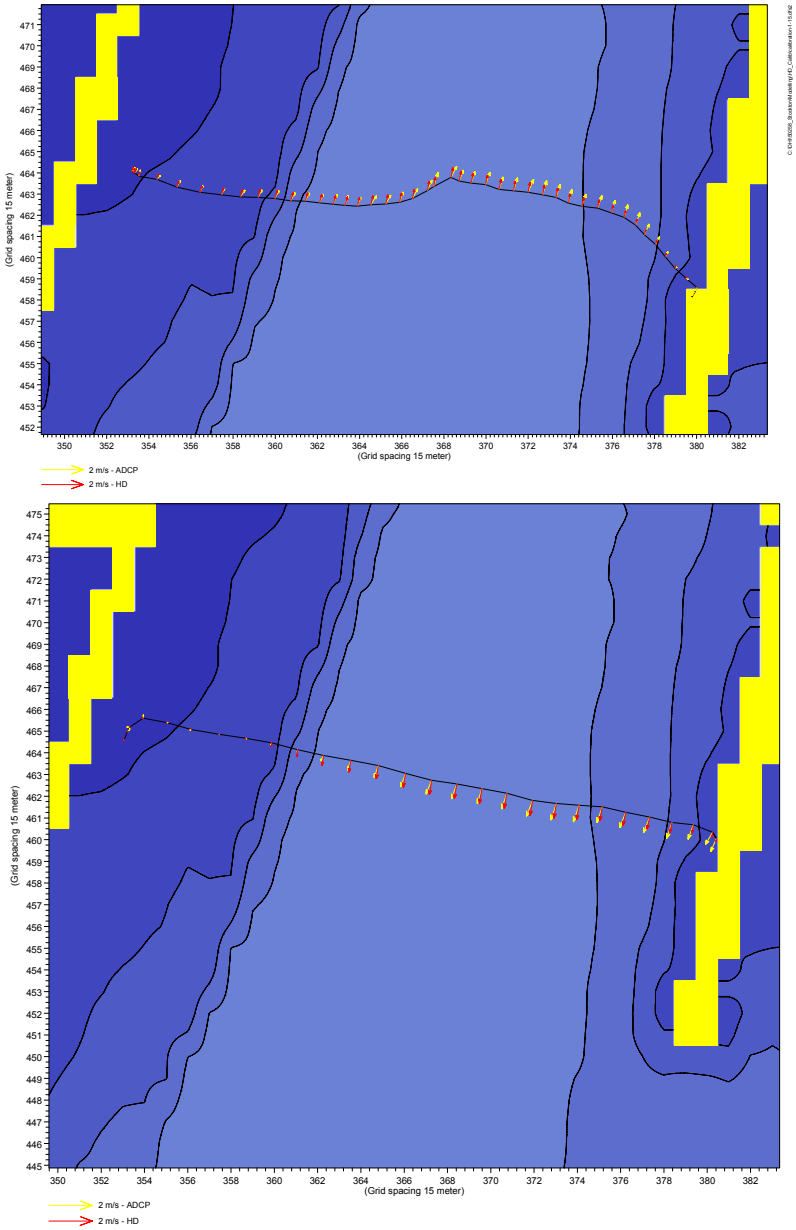
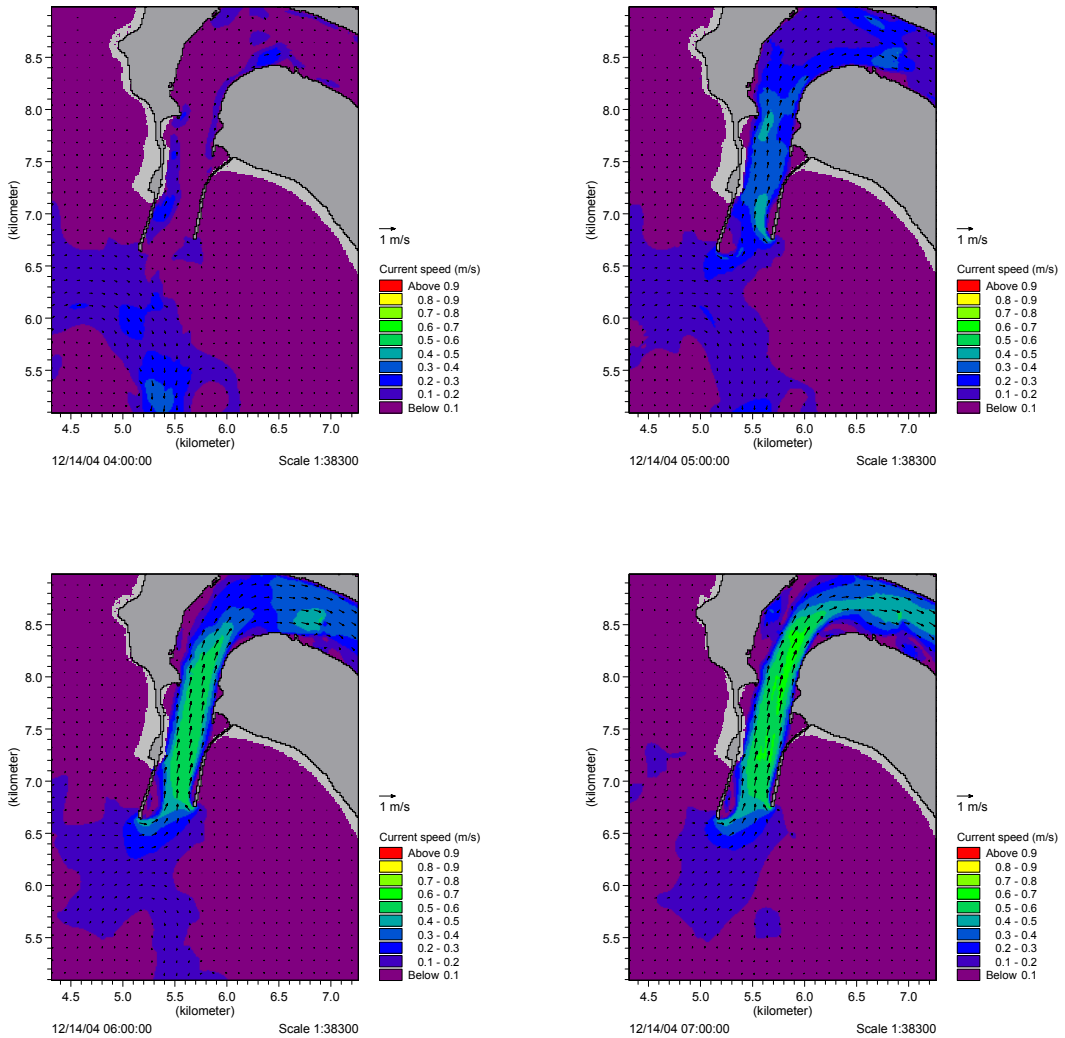


Figure 5-10 ADCP Comparisons computed and measured depth average velocities at the port entrance for flood (above) and ebb (below) conditions.

5.7 Model Results

The flow conditions at the Hunter River entrance are complex due to the large velocities along the channel and also due to the vortex formations generated by the breakwaters. In order to illustrate the flow conditions during a tidal cycle the flow conditions are presented during spring tidal conditions in four one hour intervals on December 14, 2004. Figure 5-11, Figure 5-12 and Figure 5-13 illustrate the flow conditions, where the colour scale represents the flow speed and the arrows the intensity and direction of the flow.

The figures show initial slack conditions at 04:00 followed by incoming flow until 10:00. Observed flow conditions exceed 0.7m/s velocity. The flow conveys uniformly into the channel, however two eddies are observed on either side of the entrance channel due to the contraction of the flow. Slack conditions are observed again at 10:00 when the flow changes direction and the ebb flow starts. Figure 5-13 shows the flow conditions at ebb tide when the river discharges into the ocean. Larger velocities are observed during ebb events than during flood events. No major velocity features occur at Stockton Beach, however a number of eddies are observed generating a complex flow pattern that extends nearly two kilometres from the port entrance. This complex pattern will have an effect on the sediment transport conditions in the area, especially during storm conditions when wave action induced currents will introduce further complexity on the flow and the sediment transport mechanism in the Stockton area.



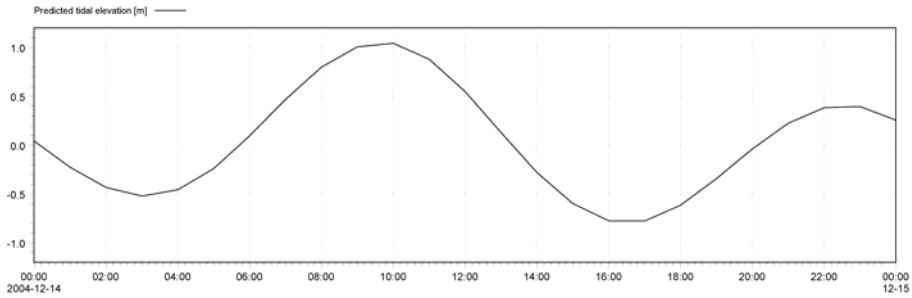
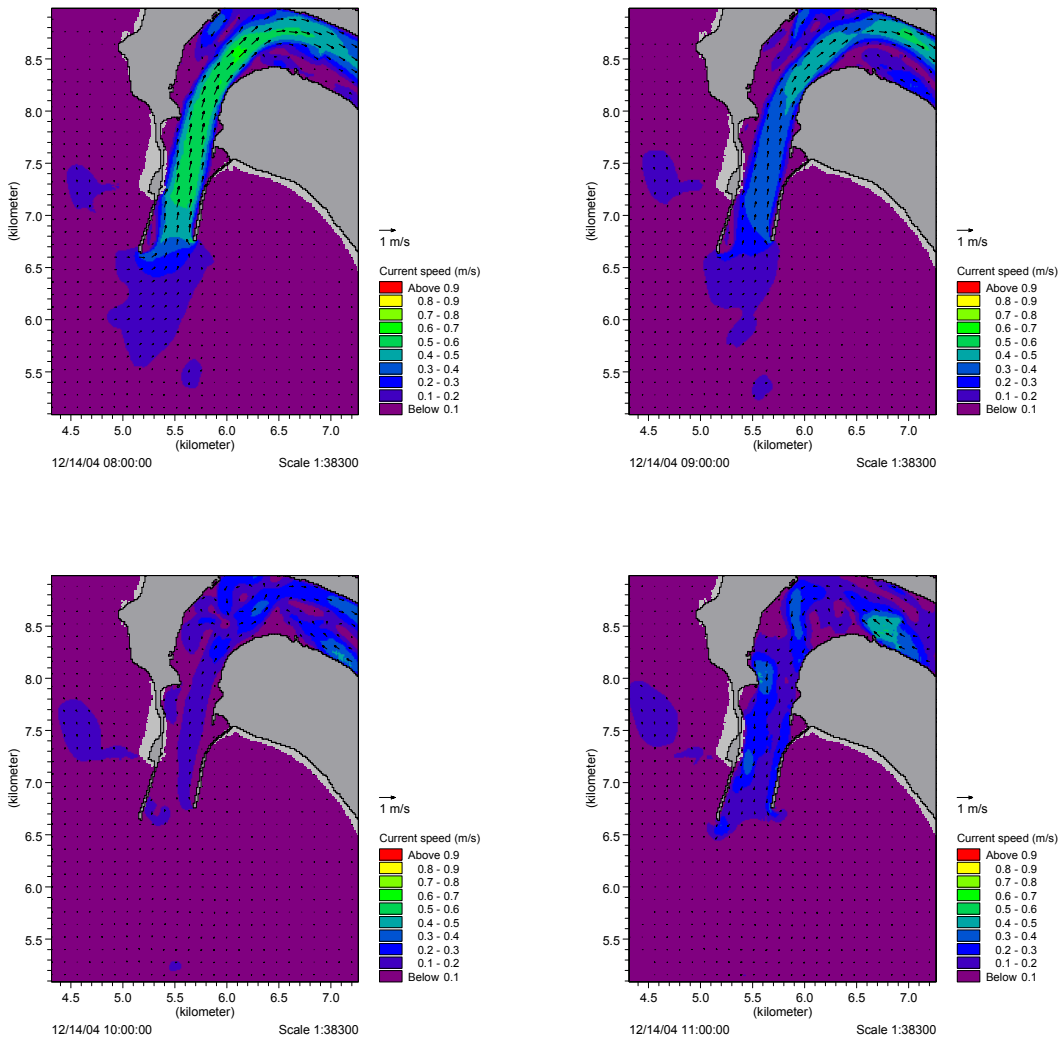


Figure 5-11 Computed flow pattern at the river entrance on Dec-14-2004 at 04:00, 05:00,06:00 and 07:00. . Below tidal levels at Newcastle Port.



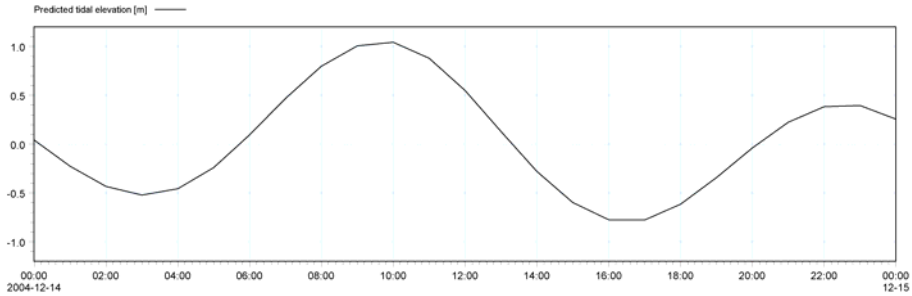
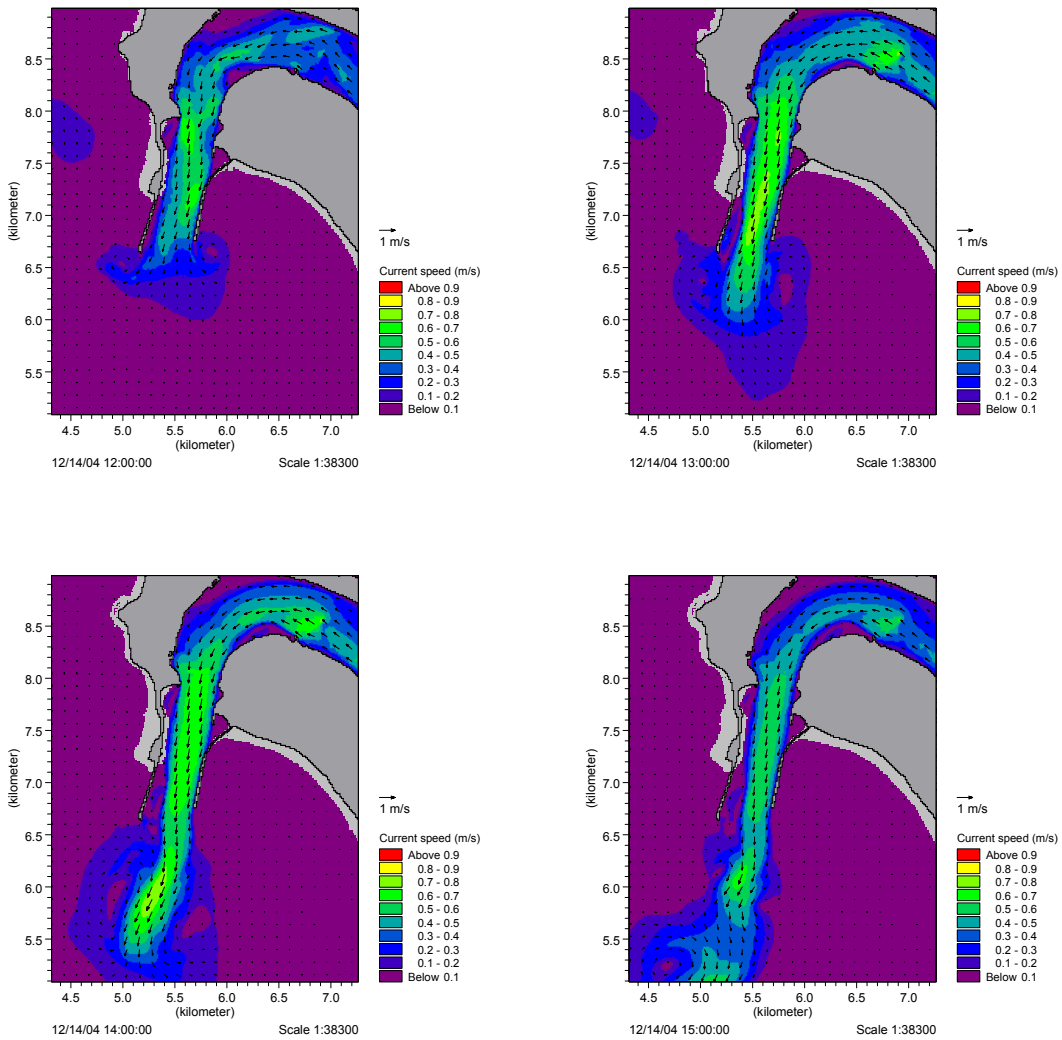


Figure 5-12 Computed flow pattern at the river entrance on Dec-14-2004 at 08:00, 09:00, 10:00 and 11:00. . Below tidal levels at Newcastle Port.



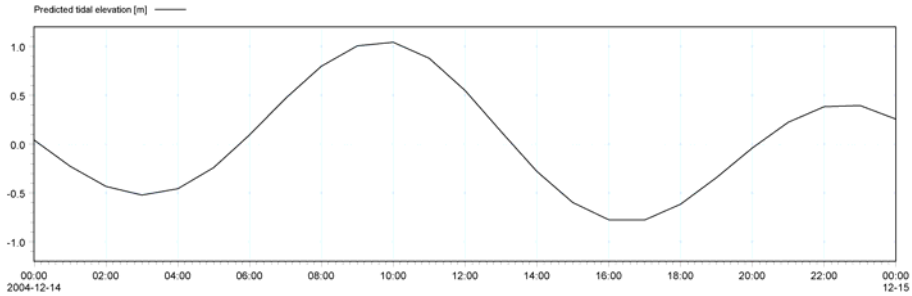


Figure 5-13 Computed flow pattern at the river entrance on Dec-14-2004 at 12:00, 13:00, 14:00 and 15:00. Below tidal levels at Newcastle Port.

5.8 Wave driven current comparisons

In addition to the tidal calibration, a comparison of the model results and measurements was carried out at the MHL measurement station. In this comparison tidal and wave driven currents are included and compared to the measurements. Current speeds were extracted from the model at the measurement station and compared to the measurements. Figure 5-14 shows the model results compared to the measurements during the storm of July 6 and 7, 2001. This period has been chosen because it corresponds with significant wave-driven currents, which are nearly negligible during most of the measurement periods, as this area is partially protected by the Port entrance from the prevailing SE waves.

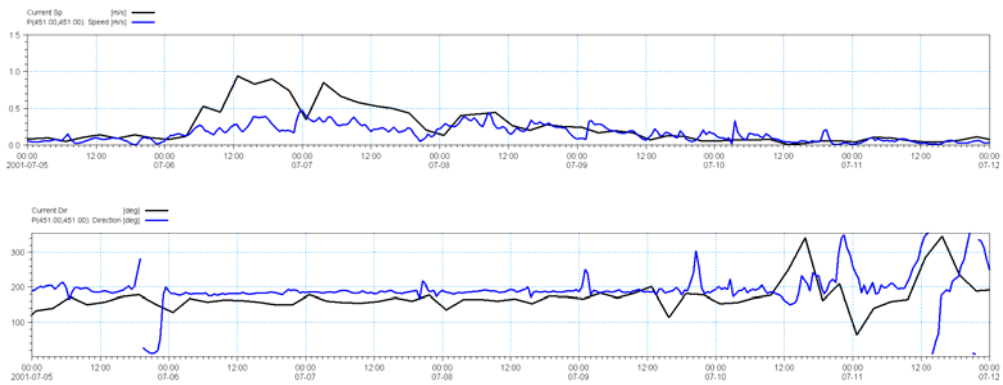


Figure 5-14 Comparisons computed and measured depth average velocities and current direction at the MHL measurement station on July 2001.(Black measurements, blue predictions)

The comparisons show good agreement between the measured and predicted velocities during mild conditions; however the currents are under predicted during the peak of the storm. The differences could be associated to an under estimation of the wave conditions as presented in Figure 4-18, which is the main forcing mechanism of the wave driven current. It is not possible to provide a detailed explanation of these differences, but it should be noted that the wave conditions have been transformed from the Sydney waverider buoy and spatial variations may be play a role in the observed differences.

The measured and predicted current directions are also compared and a good match observed. A slight mismatch is observed but this may be associated to the specification of the geographical north. It should be noted that the angle between the magnetic and true north in this area is approximately 12 degrees and no correction has been included in this case.

The wave driven currents are expected to provide the forcing mechanism that induces the long and medium term sediment transport effects at Stockton. It should be remarked that even though all data sources have been carefully analysed, they may contain some discrepancies because they have been provided by different organizations, which may apply different procedures and methods in their analysis.

6 HISTORICAL COASTLINE ANALYSIS

6.1 Motivation

The description of the coastal processes at Stockton Beach requires the analysis of the beach profiles, nearshore and offshore bathymetric data (depth distribution, where depth is the distance between seabed and mean sea level) and any type of coastal and marine information that allows the understanding of the sediment transport processes and coastal morphology of the study area.

6.2 Definition of coastal processes

The shoreline is not static and undergoes natural movements under the action of natural forces such as waves, winds and currents. If these movements are analysed, it is possible to observe fluctuations which occur over a range of time scales. These processes are usually divided into:

- **Short-term**: Occur during storms, when sand is transported offshore to form bars and part of the dune may be eroded. During periods of calmer weather, the milder wave climate slowly moves sand back onshore, re-establishing the beach and foredune. If the longshore transport is in balance, these processes do not involve any net loss or gain of sediment from the beach. In the event that the longshore transport is not in balance, part of the sediment will return to the beach and the remaining will be transported away due to longshore drift. Short term events are associated with time scales of hours to weeks;
- **Medium-term** It is recognized that a medium-term process or beach rotation has significant consequences on beach stability. Short et al. (1995, 2000) suggested that beach rotation could be the result of variations in the wave climate associated with shifts in the monthly averaged Troup Southern Oscillation Index (SOI). Further work by Ranasinghe (2004) using time series analysis and image processing techniques showed a link between beach width fluctuations, SOI, and incident wave conditions. The analysis was carried out in Narrabeen and Palm Beaches, north of Sydney. Medium-term events are usually associated with trends of several months; and
- **Long-term**: If the longshore transport is not in equilibrium and there is a net loss of sediment, there will be a landward or seaward movement of the beach. Sediment losses induce a landward movement (recession or erosion) of the beach, whereas a sediment gain induces a seaward movement or accretion. Long term events occur over time periods of several years, usually longer than 20 years.

When analysing historical data it is necessary to determine each of these processes. This however may be a complex task as frequent data collection is required to determine the different components. A general description of the historical coastal processes at Stockton Beach is presented.

6.3 Previous coastal historical analysis

A number of historical coastal analyses have been carried out for Stockton Beach. The most relevant are:

1. Stockton Beach Coastal Engineering Advice - Public Works Dept Dec (1985) ;
2. Stockton Beach Coastline Hazard Study, Land and Water Conservation (1995);
3. Newcastle Coastline Hazard Definition Study, WBM Pty Ltd (1998);
4. Shifting Sands at Stockton Beach (2002), Umwelt Pty Ltd; and
5. Newcastle Coastline Management Study (2003).

6.3.1 1985 and 1995 Studies

These studies analysed the historical shoreline behaviour at Stockton Beach, including both short and long term effects. This analysis was based on beach profiles surveys, photogrammetric information, newspaper reports, hydrographic charts, metocean conditions and anecdotic information. The main conclusions of these are:

- Realignment of Stockton Beach due to the construction of the northern breakwater;
- Shoreline/High Water Mark fluctuations over a width of 80 to 130 metres (excluding realignment);
- The most landward shoreline positions of the beach were observed in 1952, 1946 and 1995 along Mitchell Street;
- The most seaward shoreline position was in 1913 immediately after completion of the northern breakwater;
- The most significant short term shoreline erosion was 70 to 100 metres between 1938 and 1946 north of the Surf Life Saving Club;
- The most significant recovery was about 50m between 1952 and 1965 north of the SLSC and up to 130 metres adjacent to the breakwater between these dates;
- No-long term recessional trend was found to be evident from their analysis; and.
- Several short term events were identified (these events and new identified events have been included in Appendix B).

A detailed historical review of short term events was prepared in the 1995 Report, which was summarised by Moratti (1997). In this report storms events, being periods with significant wave heights exceeding 3 metres, were examined along with peak ocean levels during the period of 1994-1997.

The classification criterion is based on the wave heights, as proposed by the Public Works Department, as follows:

- X- Extreme storm: Hs $\geq$ 6m;
- A- Severe storm: 5m $\leq$ Hs < 6m;
- B- Moderate storm: 3m$\leq$ Hs < 5m Hs

Moratti (1997) also determined volumetric dune and beach berm variations from 1994 till 1997. The computed variations are presented in Figure 6-1.

The volumetric analysis showed that on average, 130 m^3/m (130 m^3 per m of the beach) of erosion has occurred south of the seawall during the period between June 1994 to May 1997, and the majority of this erosion occurred during the 1994 and early 1995 storms. The major storm of 1997 resulted in only small additional overall volume changes south of the seawall.

North of the seawall an average erosion of 150 to 200 m^3/m has been determined. A slightly different pattern is evident here in that the initial erosion of about 150 m^3/m continued until 1995. Recovery occurred through to early 1997 and the May 1997 storm resulted in further substantial erosion of 90 m^3/m attributing the net erosion since June 1994 to 200 m^3/m. These volumes quoted are the volumes above MSL.

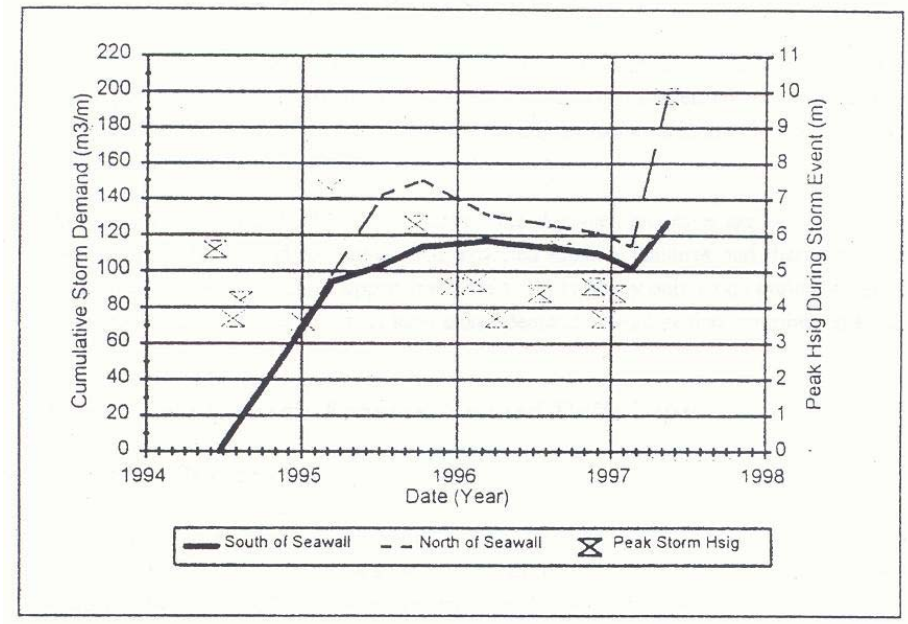


Figure 6-1 Sub-aerial beach fluctuations/storm Hsig June 1994-May 1997. From Moratti (1997).

6.3.2 Newcastle Coastline Hazard Definition Study - WBM (1998)

This study carried out a review of the available data for long-term events. An assessment of the long-term conditions was carried out based on

- Beach profile analysis; and
- Hydrographic survey analysis.

The beach profiles provided a good representation of the period between 1952-1997. Two compartments were considered defined as being south of the Mitchell Street seawall and north of the Mitchell Street seawall.

The analysis showed that for the area south of the seawall, periods of accretion and erosion were observed but with little net change reported in the 1995 study.

For the northern part of the study area, the beach profiles illustrated a different pattern to the one described in the 1995 report. In this case progressive erosion since the mid

1960's with some periods of strong erosion, and beach recovery superimposed on that progressive trend was observed.
In addition to this information, a hydrographic survey analysis was carried out to extend the timeframe where possible. Historical hydrographic survey plans dating to 1866 were examined. The high water mark was interpreted and the following conclusions were drawn:

- The change in alignment of the shoreline in response to the harbour breakwaters (northern breakwater constructed between 1898 and 1912) are evident by comparing 1893 and 1913 positions
- Large scale fluctuations in the order of 80 to 100m and up to 130m adjacent to the breakwater (excluding the above realignments effects) with periods of both erosion and subsequent recovery; and
- Little overall long term net change south of the Mitchell Street seawall (excluding breakwater realignment effects).

The analysis also showed that for the area north of the Mitchell Street seawall, if the 1866 and 1913 HWM positions are taken into account, it could be considered that there is no evidence of any long term recession. However, within this 130 years time frame, there is evidence that from the early 1900's there is a progressive trend of long term recession, with periods of erosion and subsequent recovery superimposed on that trend. It is also evident that the recovery is generally not as great as the erosion which is indicative of a long term loss situation.

WBM also produced a difference plot between the 1957 and 1995 surveys at Stockton. The analysis showed that erosion is generally present on the offshore bar area where longshore transport is the highest. The comparison shows trends similar to that shown by the beach surveys, with little change south of the seawall and increasing erosion north of the seawall.

The difference plot also shows accretion adjacent to the seaward end of the northern breakwater and the edge of what may be an extensive area of accretion offshore and to the north of the breakwater. The difference plot is presented in Figure 6-2.

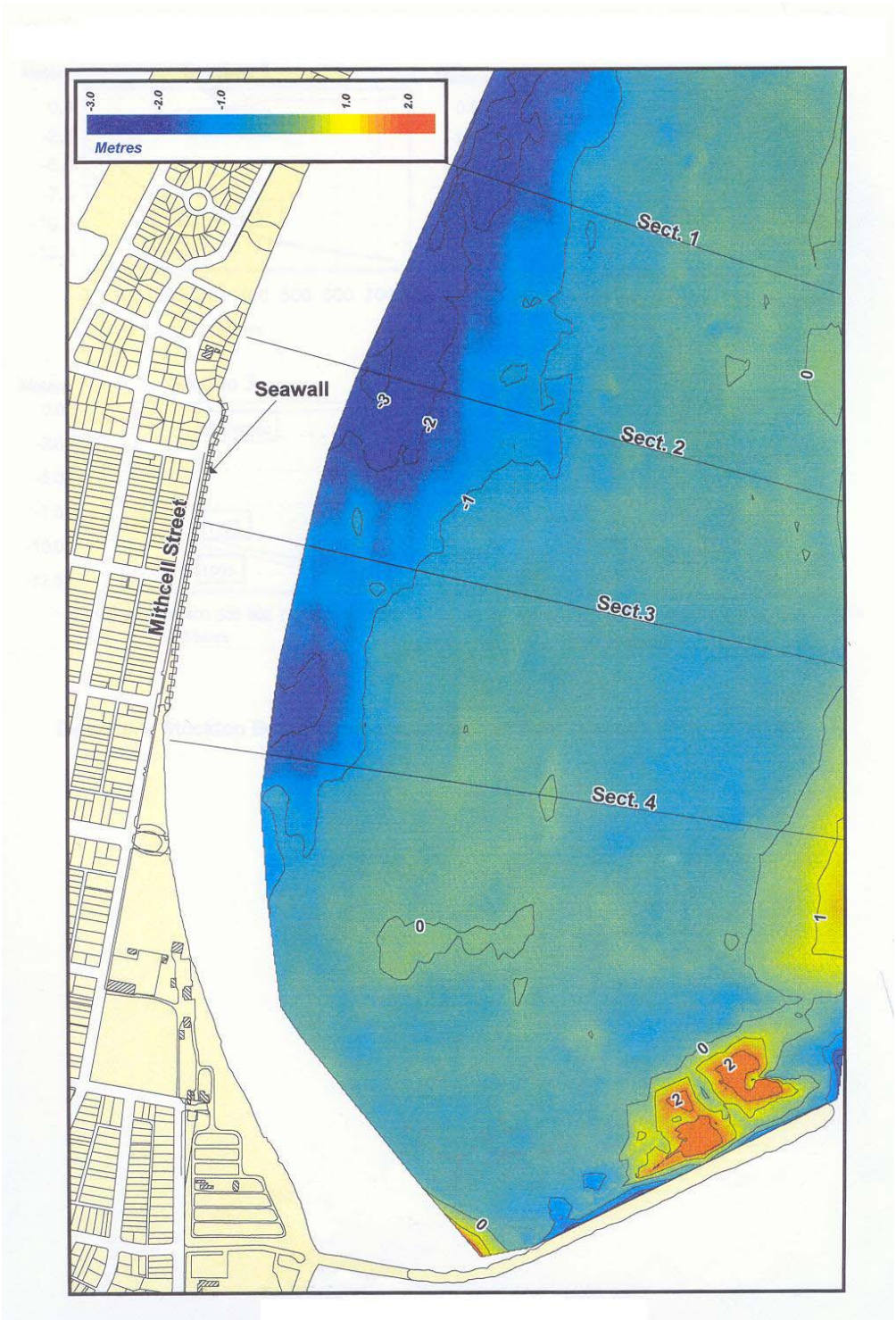


Figure 6-2 Stockton Beach Hydrographic Survey Data Difference Plot 1957-1995. WBM (1998).

As part of this study WBM (1998) constructed an immediate erosion hazard zone for storm scour. The width of this zone varied from 0 metres at the northern breakwater to 5 metres at the southern end of the seawall. North of the seawall it was set to 10 metres. South of the seawall to the Surf Club, the seaward boundary of this zone was defined by a continuous smooth line located 5 metres landward of the 1952/99 erosion

escarpments, increasing to 15 metres at the breakwater. North of the seawall, it varied from 5 metres at the seawall to 15 metres at the sewerage ponds.

WBM (1998) presented a prognosis for ongoing erosion at Stockton Beach, which included an allowance for recession, associated with a sea level rise due to Greenhouse effects. Accordingly, WBM (1998) constructed a 2050 erosion hazard zone with width that varied from 0 metres at the breakwater to 20 metres at the southern end of the seawall and which was set to 55 metres north of the seawall. South of the seawall to the Surf Club, the seaward boundary of this zone was defined by a continuous smooth line located 15 metres landward of the 1952/99 erosion escarpments, increasing to 25 metres at the breakwater. North of the seawall, it varied from 15 metres at the seawall to 25 metres at the sewage ponds.

WBM (1998) recognised that the erosion rates over recent years had been much higher than average, nevertheless it advised that there was reasonable change and that the present general trend of erosion may reduce or reverse at some future time.

The future hazard assessment made in WBM (1998) was based on the assumptions that:

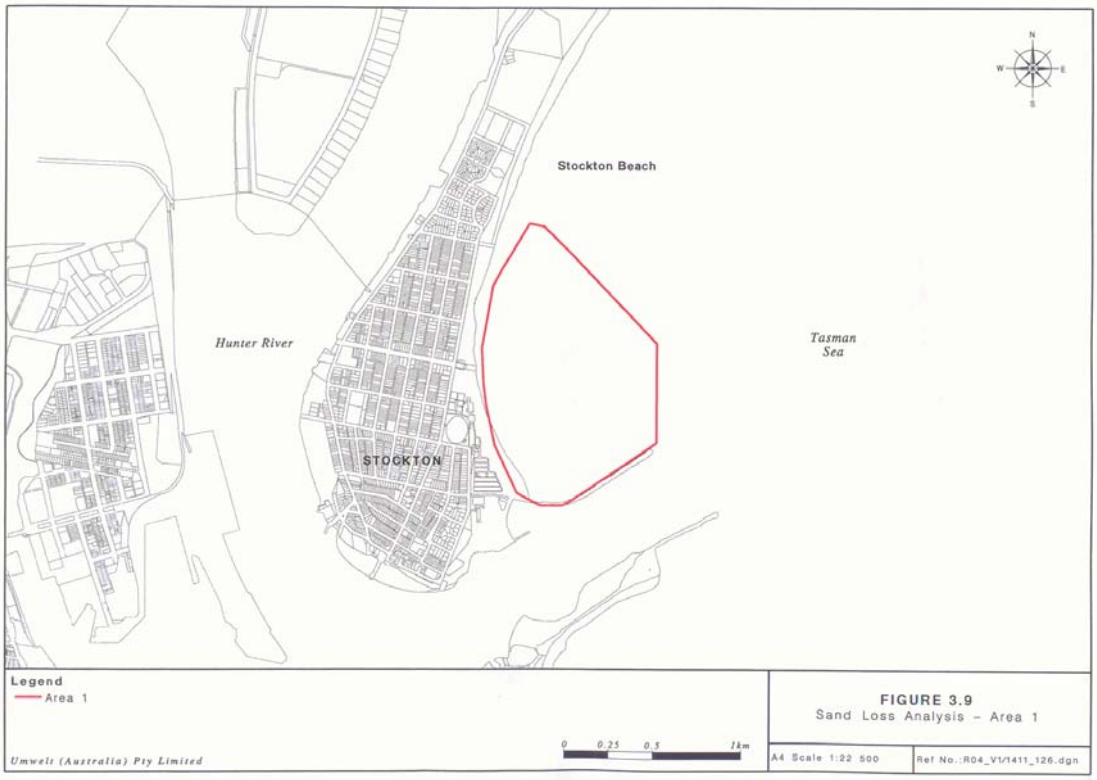
- The adopted trend of shoreline retreat will continue unchanged over the next 20 years;
- Shoreline retreat will then progress at a rate of 50% for the following 30 years;
- A transition zone exists south of the Mitchell Street seawall with the shoreline retreat rates reducing both along the beach (towards south) and in time as outlined above.

6.3.3 Shifting sands at Stockton Beach –Umwelt Pty Ltd - SMEC Pty Ltd (2002)

Umwelt (2002) further analysed long term variations using recent bathymetric and historic hydrosurvey information. The analysis was undertaken in an area just north of the northern breakwater (Area 1), and later extended to include a larger area further north (Area 2). The extension of these two areas is presented in Figure 6-3.

In this study the volumetric sand changes within Area 1 were calculated and the results are presented in Table 6-1. The analysis showed an average sand loss rate for the period 1816 to 2000 of 41,500m^3/y, for 1866 to 2000 26,667m^3/y and finally between 1921 and 2000 32,200 m^3/y. There is uncertainty with the data of the period before 1866 due to the unknown accuracy of the 1816 data.

The results show that there is significant variability in the data with predominant erosion however there are periods of accretion in the area. Most periods showed similar sand volume variations, with exception of the period between 1988 and 1995 where the annual average rate of loss increased to 245,000 m^3/year.



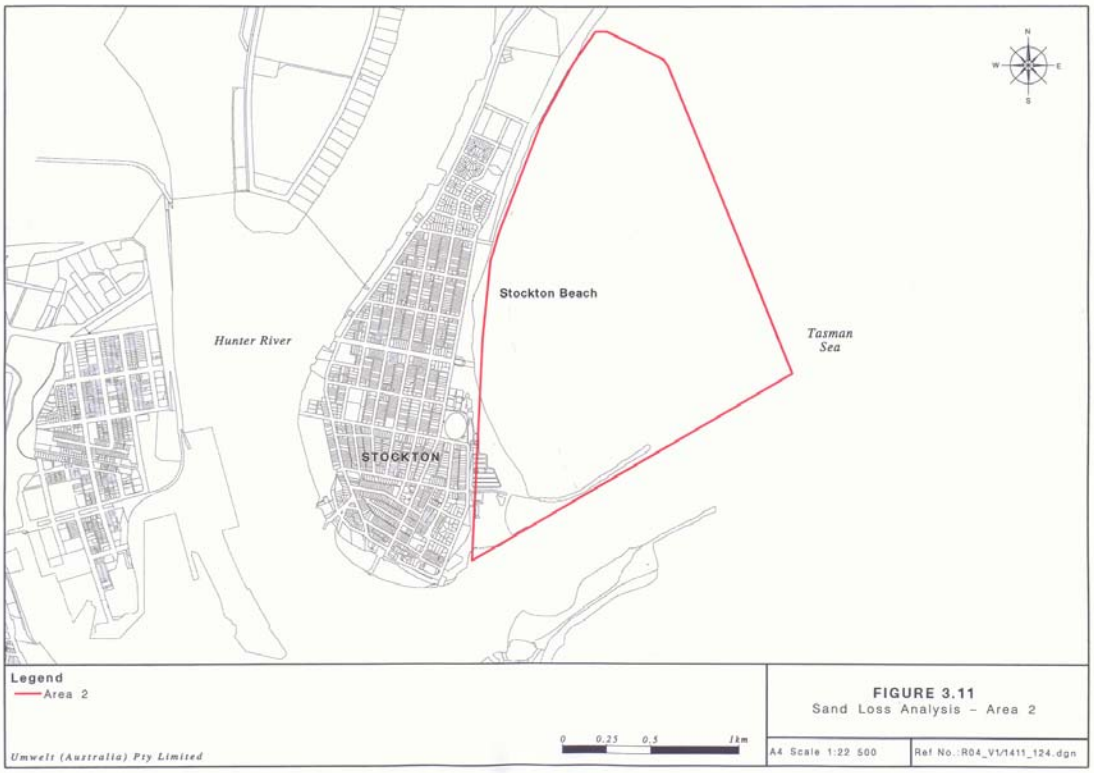


Figure 6-3 Overview of the sand volume analysis areas extension undertaken by Umwelt, (2002) Above Area 1, Below Area 2.

Table 6-1 Estimated changes in sand volume within Area 1 off Stockton Beach North of the Northern Breakwater (1816-2000) (positive values erosion, negative values erosion)

Period	Rate of change per year (m^3)	Average rate of change per year (m)
1816-1866	89,785	0.06
1866-1899	19,826	0.0135
1899-1909	25,869	0.017
1909-1913	-11,248	-0.008
1913-1921	-21,362	-0.015
1921-1926	10,905	0.007
1926-1950	28,899	0.02
1950-1957	-11,742	-0.008
1957-1988	10,130	0.007
1988-1995	244,862	0.167
1995-2000	52,565	0.036
Average (1816-2000)	**41,500**	**0.03**
Average (1866-2000)	**26,667**	**0.018**
Average (1921-2000)	**32,200**	**0.022**

Sand volume variations were also computed in Area 2 during the period 1921 – 2000. The analysis showed an average sand loss of 67,010 m^3/year. The results showed also a large rate of sand loss of 370,000 m^3/year during the period 1988 to 2000, as shown in Table 6-2.

Table 6-2 Estimated changes in sand volume within Area 2 off Stockton Beach North of the Northern Breakwater (1926-2000)

Period	Rate of change per year (m^3)	Average rate of change per year (m)
1921-1926	-34,348	-0.014
1926-1950	24,213	0.01
1950-1957	14,999	0.006
1957-1988	33,521	0.013
1988-2000	373,514	0.147
Average 1921-2000	**67,010**	**0.026**

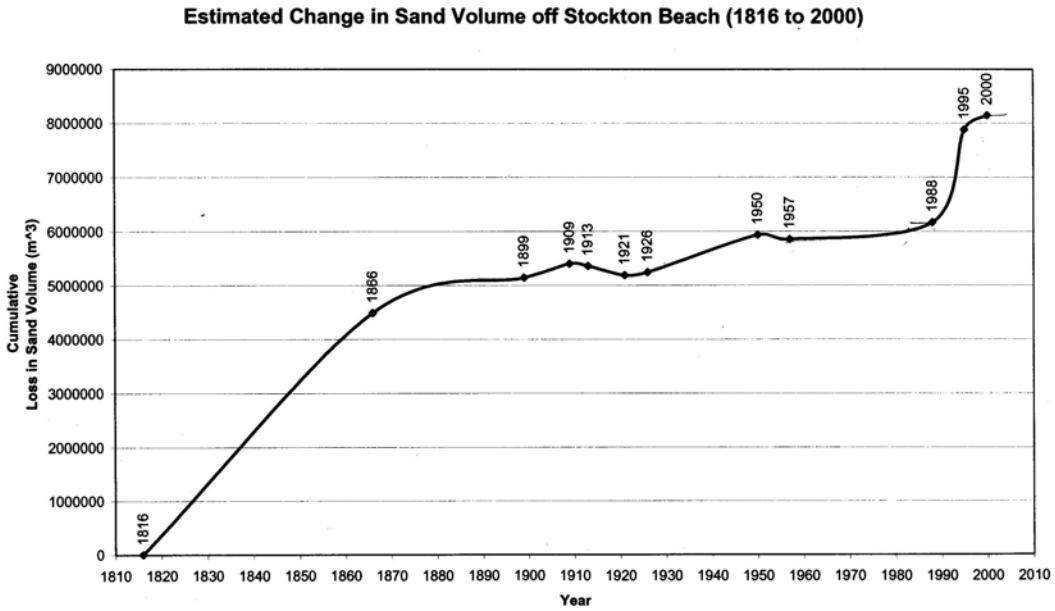


Figure 6-4 Sand volume changes in Area 1 (1816-2000) from Umwelt (2002)

6.3.4 Newcastle Coastline Management Study - Umwelt Pty Ltd (2003)

A coastline management plan was undertaken by Umwelt (2003) to identify options relevant to the environmental planning and management of the Newcastle coastal area. The study was the outcome of an integration of environmental, social, cultural, and economic factors which affect the Newcastle coastline and presents a range of strategic options for integrated and sustainable coastal management. The study evaluated a number of issues such as coastal landscape, coastal ecology, amenity and community use, social and economic setting, coastal processes and hazards, management issues and management options.

This study found that from the analysis and interpretation of the long term changes in the seabed morphology and numerical beach erosion modelling, Umwelt and SMEC (2002) are not consistent with what it was presented in the Newcastle Coastline Hazard Definition Study undertaken by WBM (2000). It could be expected that the recent higher-than-average erosion rates are likely to increase under large wave activity and the threat to development at Stockton Beach will increase with time. The level of risk, however, had yet to be quantified. Nevertheless, it could be assumed that the hazard definition presented in WBM (2000) is likely to be a significant underestimate of what, realistically, may occur.

Based on the review of the hazard risks at Stockton Beach this report concluded that in the light of recent investigations the hazard risk at Stockton is likely to be exacerbated. As a result, the hazard risk would need to be further assessed as part of additional studies into the stability of Stockton Beach.

6.4 Extended Coastal Analysis

Further to the analysis carried out in the previous studies the coastal shoreline analysis has been extended to include the latest available data.

6.4.1 Long and medium term

The analysis carried out in the previous studies was extended to include the latest data to evaluate the extended shoreline movement and the historical trends. A general view of profile data was imported into a GIS system generated for 1952 – 2001. A general overview of the location of the areas for both the Stockton and Nobbys Head areas is presented in Figure 6-5 and Figure 6-9. The information has been organised in areas as follows:

- Area A78: covers the Stockton Beach stretch, it has been subdivided into subareas A, B and C from south to north;
- Area A77: extends from area A78 up to the location of the Sygna wreck. It has been subdivided into subareas 3, 4, 1 and 2, from south to north; and
- Area B39: covers Nobbys Head. It is subdivided into three subareas 7, 8 and 9 (from south to north).

Since the characteristics of the northern (Stockton) and southern areas (Nobbys Head) are significantly different they will be analysed individually.

Northern Area (Stockton Beach and Fern Bay)

The extension of the area is 9500m from the northern breakwater to the end of subarea A77-2. A detail of the area and subareas as well as the beach profiles is presented in Table 6-3.

An overview of the available profiles for each year at the different areas is presented in Table 6-3. As it can be observed, beach profiles at Stockton and Fern Bay (north of Stockton) have been frequently processed since 1952. Some gaps are observed and they occur differently for Areas 78 and 77.

Table 6-3 Overview of the available beach profiles at Stockton

Year / Section	A78	B78	C78	A77-3	A77-4	A77-1	A77-2
27/8/1952	Yes	yes	yes	no	no	no	No
22/7/1954	Yes	yes	no	yes	yes	yes	Yes
11/1959	Yes	yes	yes	no	no	no	No
26/0/1965	Yes	yes	yes	yes	yes	yes	Yes
7/1969	Yes	yes	yes	no	no	no	No
1972	No	no	no	no	no	yes	Yes
19/6/1974	Yes	yes	yes	yes	yes	no	No
1975	No	no	no	no	no	yes	Yes
1977	No	no	no	no	no	yes	Yes
1983	No	no	no	yes	yes	yes	Yes
22/5/1986	Yes	yes	yes	no	no	no	No
1990	No	no	no	no	no	yes	Yes
11/8/1991	Yes	yes	yes	no	no	no	No
21/6/1994	Yes	Yes	yes	yes	yes	yes	Yes
1/12/1996	Yes	Yes	yes	no	no	no	No
26/9/1997	No	No	yes	no	no	no	no
19/9/1999	Yes	Yes	yes	no	no	yes	Yes
14/9/2001	Yes	Yes	yes	yes	yes	no	No

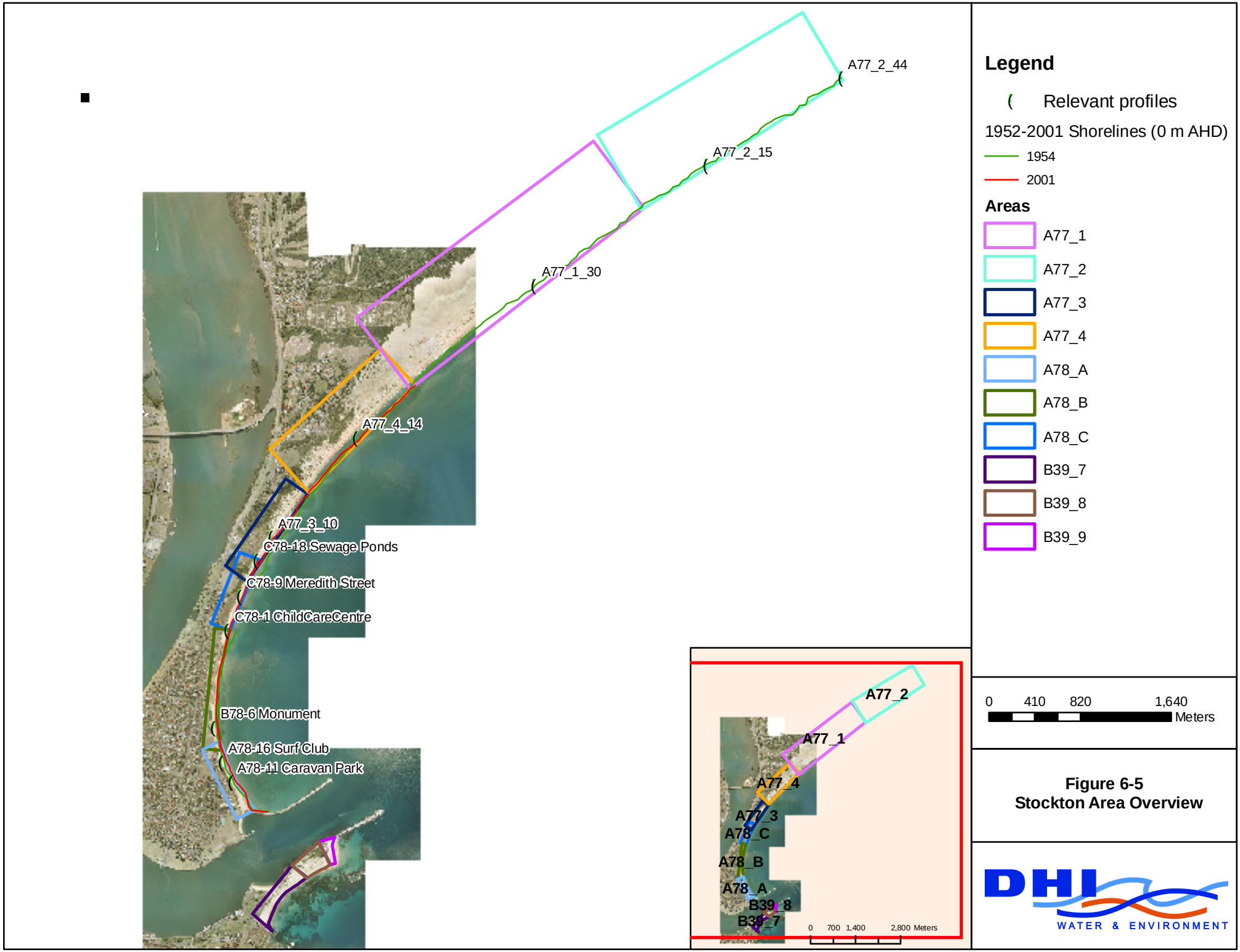


Figure 6-5
Stockton Area Overview

Beach profiles (land surveys) at Stockton Beach (A78), Fern Bay (A77) and the southern beaches (B39) have been provided by DNR. Figure 6-5 shows an overview of the location of the areas A78 and A77 and their corresponding subdivisions.

As an example of the beach profile movement Figure 6-6 shows the beach profiles at Area 78-B, profile 28. The profiles have been surveyed to AHD (very close to mean sea level). This information provides a good overview of the historical advance/retreat of the beach profiles, but only above the mean sea level line (0m elevation).

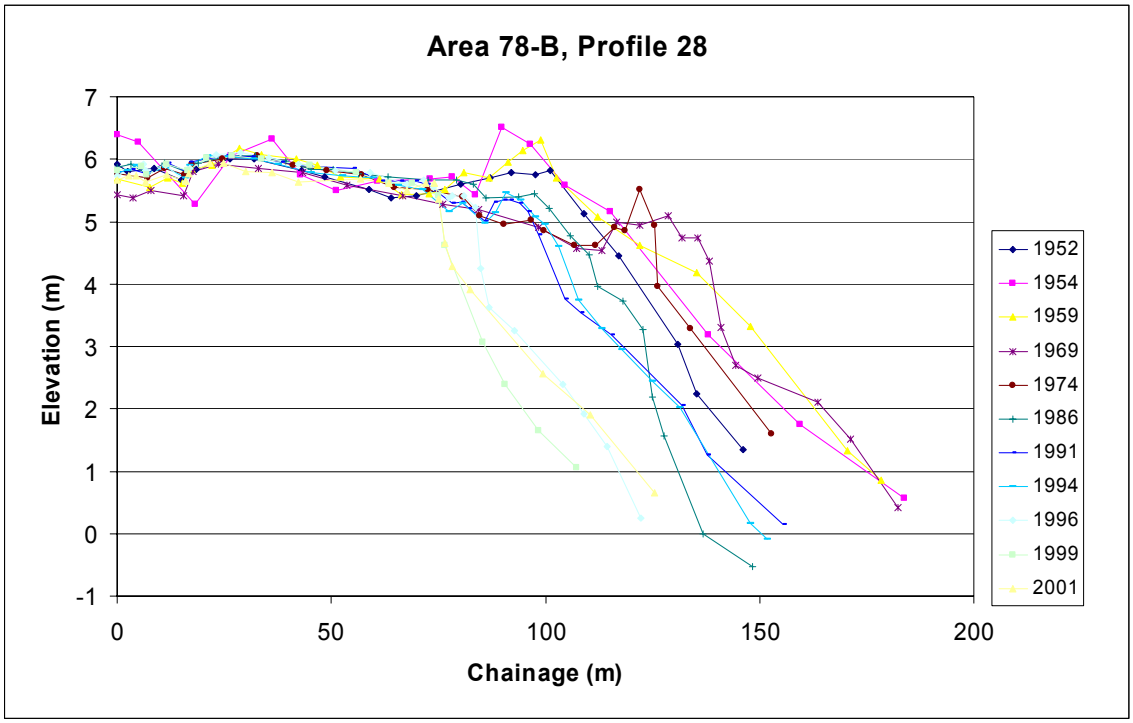
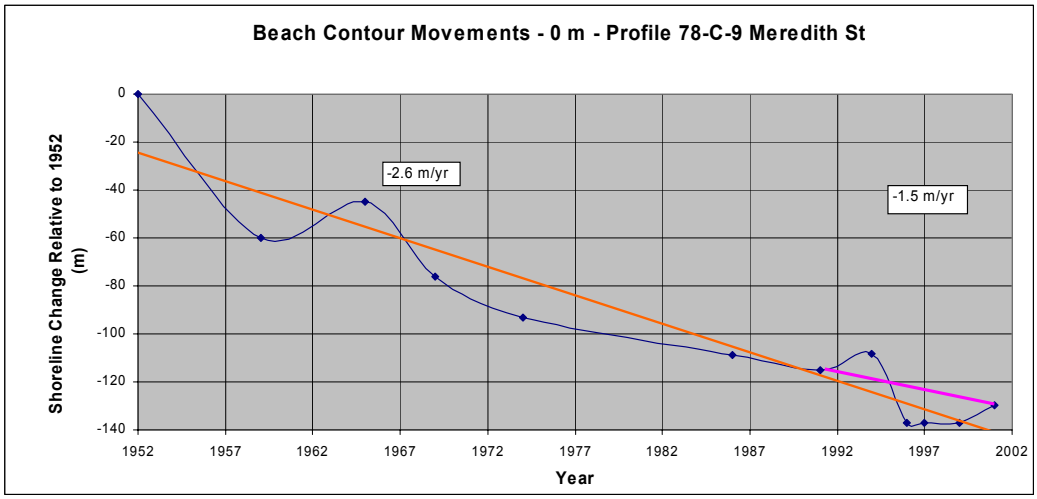
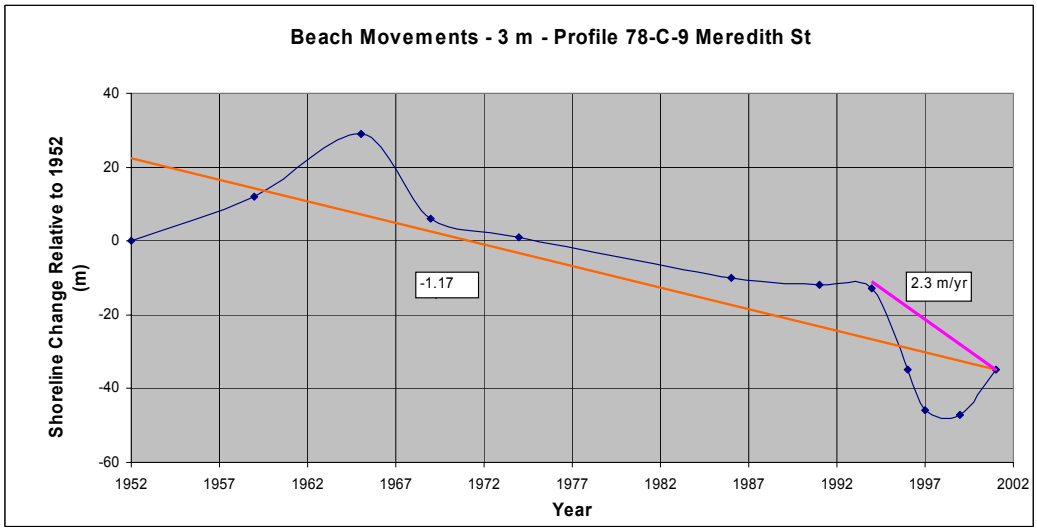


Figure 6-6 Beach profile advance/retreat at Area 78-B, profile 28

The most representative profiles have been imported to produce 0m, 3m and 4m AHD surface level contours for each individual year as an indicator of shoreline and dune change for these years. These three lines have been analysed to provide a representative view of beach movement. The shoreline movement at two beach profiles is presented at the Surf Club and Meredith St and is presented in Figure 6-7 and Figure 6-8. These two locations have been selected to provide a representation of the two most relevant areas, namely south and north of the seawall.





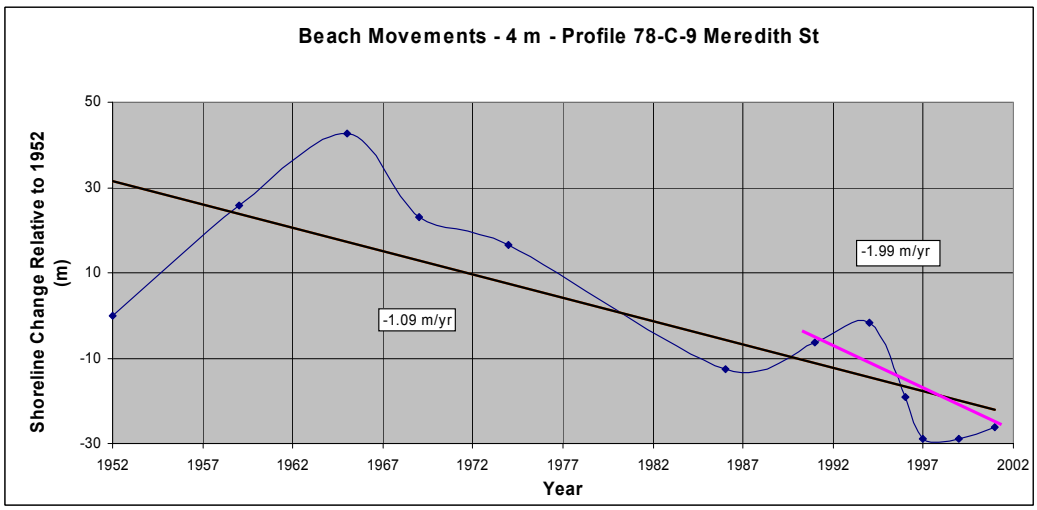
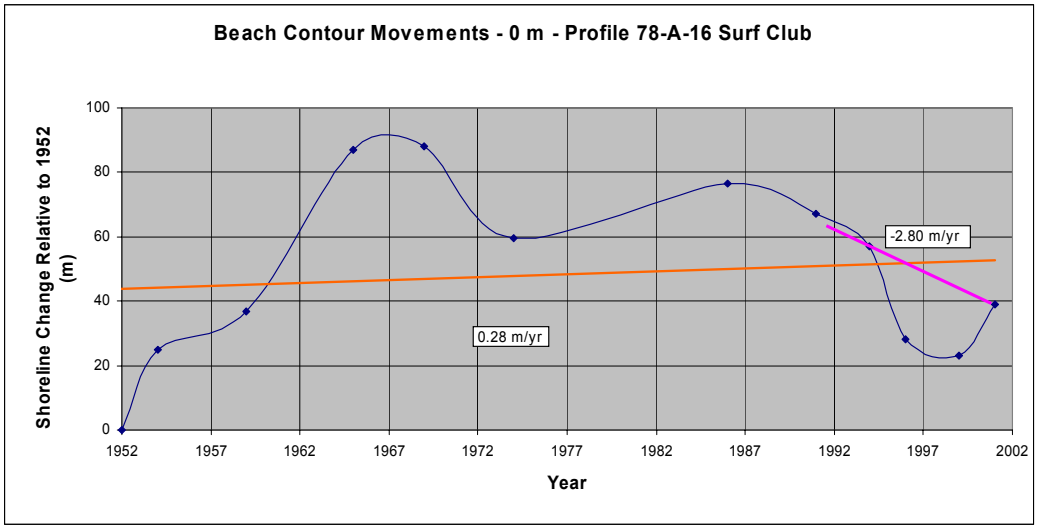
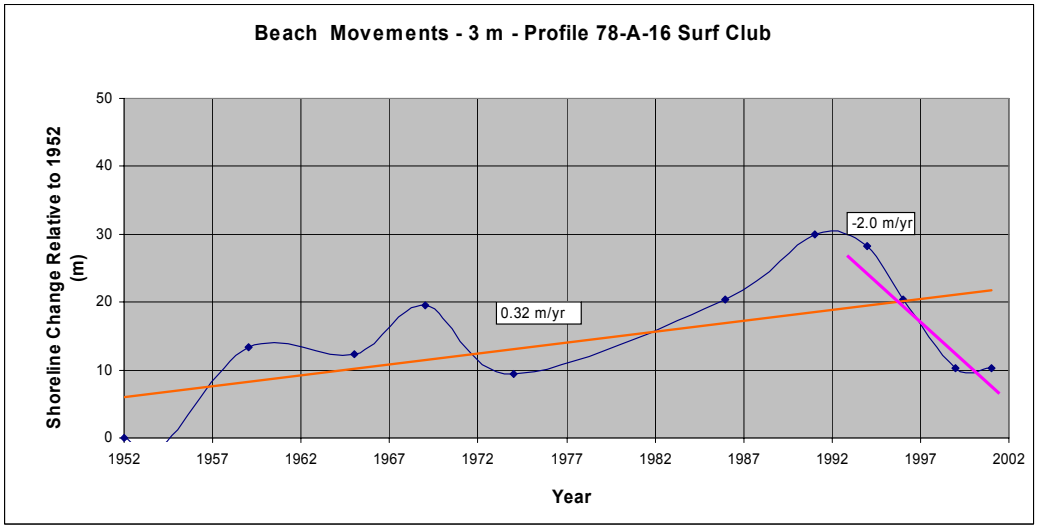


Figure 6-7 Overview of the historical coastline advance/retreat at Meredith St (just north of the Child Care Centre)





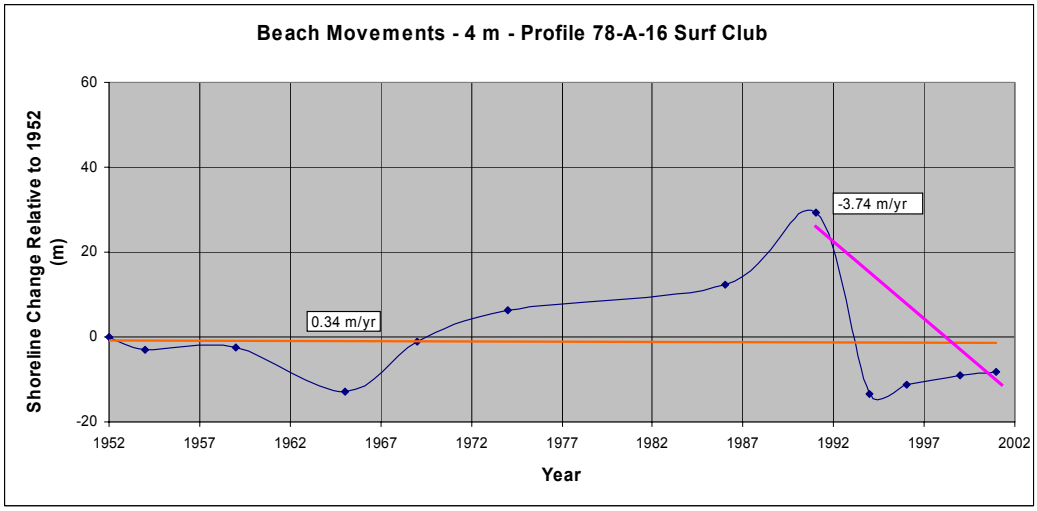


Figure 6-8 Overview of the historical coastal advance/retreat at the Stockton Surf Club

It can also be observed from the analysis that these two stretches show different patterns. The results of the shoreline analysis indicate a long term recession north of the seawall (Meredith St) while the stretch between the northern breakwater and the southern end of the seawall shows long term stability, even a slight accretion.

If the temporal shoreline variation is analysed during this period the following features can be observed:

Stockton Surf Club (south of seawall)

- The beach seems to be stable or slightly accreting;
- The most eroded condition of the beach face was in 1952 and 2001;
- Significant recovery of the beach occurred between 1954 and 1959.
- The most prograded condition of the beach berm was in 1991;
- The beach face receded 15 to 30 metres between 1991 and 2001;
- A maximum berm fluctuation of 40m is observed;
- Significant erosion has been observed from 1991 till 1995;
- Finally, a recovery period is observed between 1997 and 2001. It is unclear if some short-term or medium-term events have been captured during these surveys.

Meredith St (north of seawall)

- A long term erosion rate of 1m/year is observed for the period 1952 to 2001;
- The most eroded condition of the beach face was in 1997;
- The beach face receded 15 to 30 metres between 1994 and 1995;
- Significant erosion has been erosion has been observed from 1995 till 1997. The erosion rate during this period increases to 3m/year approximately;
- Finally, a recovery period is observed between 1997 and 2001.

One of the main constraints when investigating coastline movement is the separation of short and long-term events. For example, if a beach profile is surveyed after a storm (short-term event) the measurements will provide a wrong estimation of the long-term processes. Usually during a storm the swash zone is eroded and some of the material is transported offshore (the beach profile becomes flatter). After the storm, during mild wave conditions the sediment is transported back into the landwards area of the profile. This will occur in an area where the longshore transport variation is zero, however if there is a deficit in the sediment budget the beach profile will move landwards after the storm but will not reach its initial position.

An overview of beach stability and the rate of erosion and accretion has been computed along the Stockton area for the period 1992-2001 and the overall period, the results are presented in Table 6-4. As it can be observed the southern area has remained stable whereas the area north of the Mitchell St seawall shows a steady erosion rate. The information is consistent with previous studies, described in this section. It should be noted that the rate of erosion is larger during the period 1992-2001 than that observed during the period 1952-2001.

Table 6-4 Computed shoreline rates (positive sedimentation, negative erosion)

Area	Profile	Profile Description	Rate 1992-2001 (m/yr)	Overall Rate (m/yr)
A78_A	11	Caravan Park	-2.8	0.0
	16	Surf Club	-2.8	0.28
A78_B	6	Monument	-3.1	0.22
A78_C	1	Child Care Centre	-2.1	-1.34
	9	Meredith Street	-2.0	-1.09
	18	Sewage Ponds	-0.64	-1.09

Southern Area (Nobbys Head)

The extension of this area is 850m. Detail of the area and subareas is presented in Figure 6-9. Beach profiles have not been processed as frequently as at the northern areas and only four years of data have been made available. An overview of the available profiles for each year at the different areas is presented in Table 6-5.

Table 6-5 Overview of the available beach profiles at Nobbys Head

Year \ Section	B39-7	B39-8	B39-9
1954	yes	yes	yes
1974	yes	yes	yes
1996	yes	yes	yes
2001	yes	yes	yes

These profiles have been imported to produce 0, 3 and 4m AHD surface level contours for each individual year as an indicator of shoreline movement during the analysis period. The shoreline advance/retreat at three beach profiles is presented at the southern, central and northern end of Nobbys Beach. The location of these areas is presented in Figure 6-9.

Legend

- Relevant profiles

Profiles_B39

- B39_7 Profile 4
- B39_8 Profile 6
- B39_9 Profile 4

Area

- B39_7
- B39_8
- B39_9

0 95 190 380 Meters

Figure 6-9:
Nobby Head Overview

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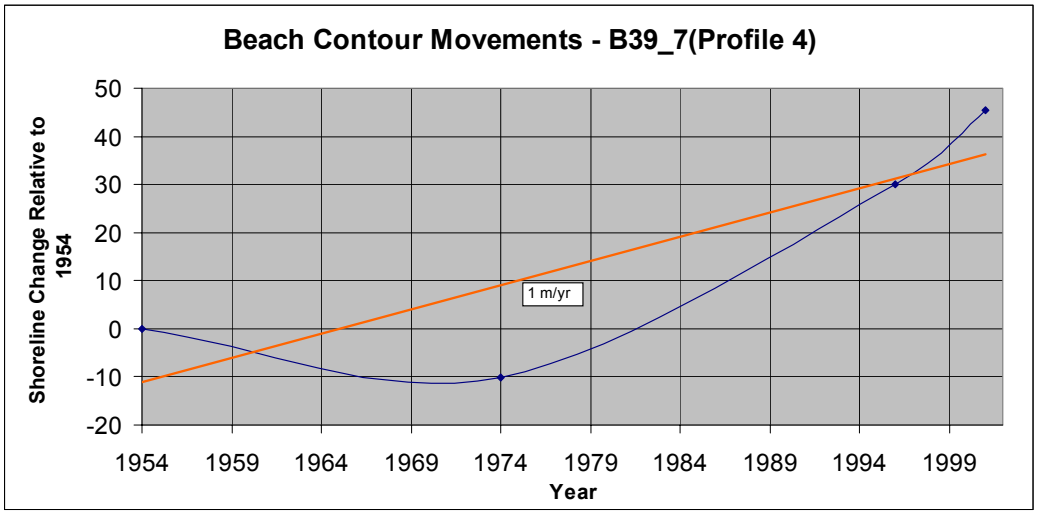
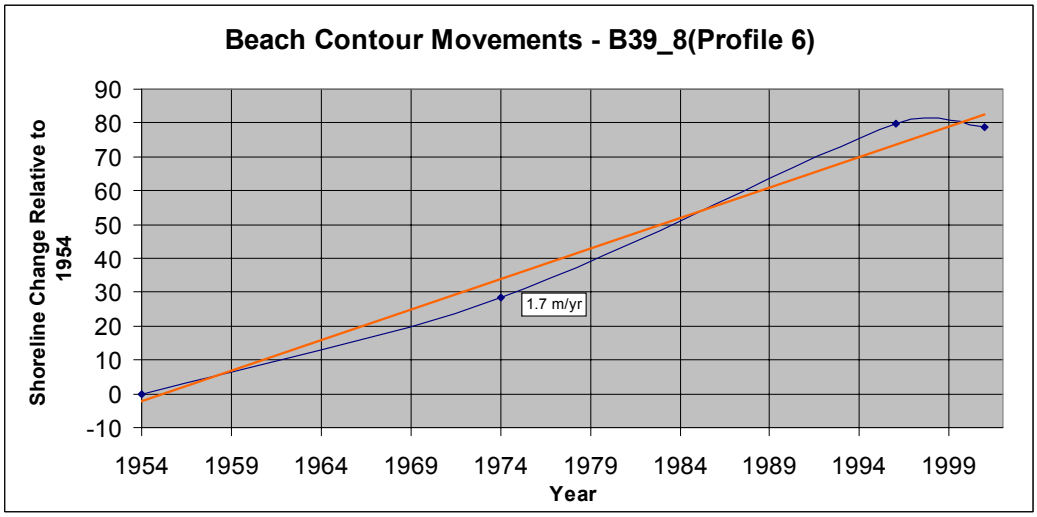
As observed from the analysis, the results show systematic accretion for the three areas. The largest is in the central section being 80 metres from 1954, and having an estimated rate of 1.7 metres/year. The northern profile has accreted 45m or 1m/year whereas the southern area has accreted only 13m or 0.36m/yr.

Table 6-6 Computed shoreline rates (positive sedimentation, negative erosion)

Area	Profile	Profile Description	Overall Rate (m/yr)
B39_7	4	Southern area	0.36
B39_8	6	Central area	1.7
B39_9	4	Northern area	1.0

These results indicate that this is an area of accumulation or otherwise stated the incoming amount of sand reaching these areas exceeds the amount that is being transported away. Based on this information it is not possible to determine the sources, however based on historical information there is evidence that the littoral transport from the south tends to deposit the material here. The sediment transport processes here are rather complex due to the rocky areas and the reef that largely influences the waves and the flow conditions and consequently the sediment transport.





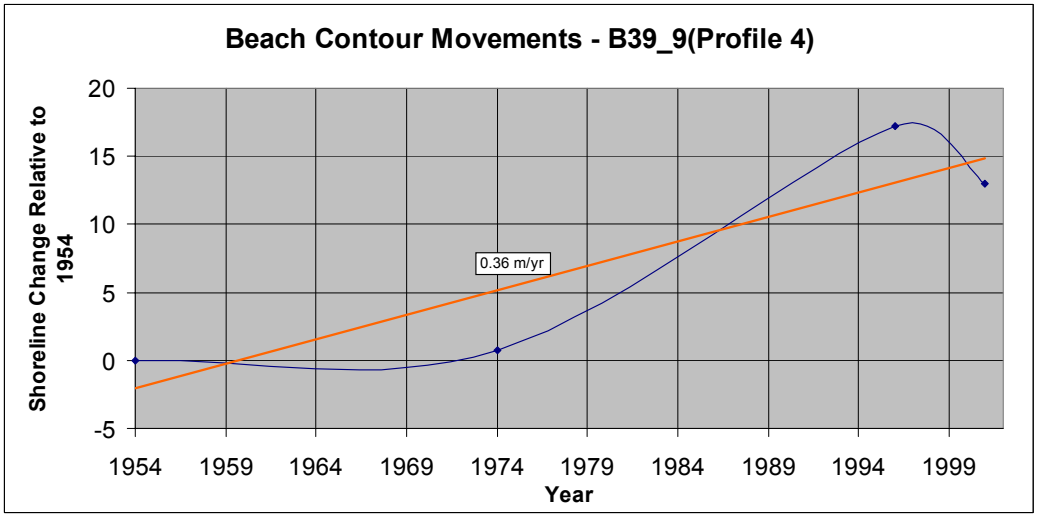


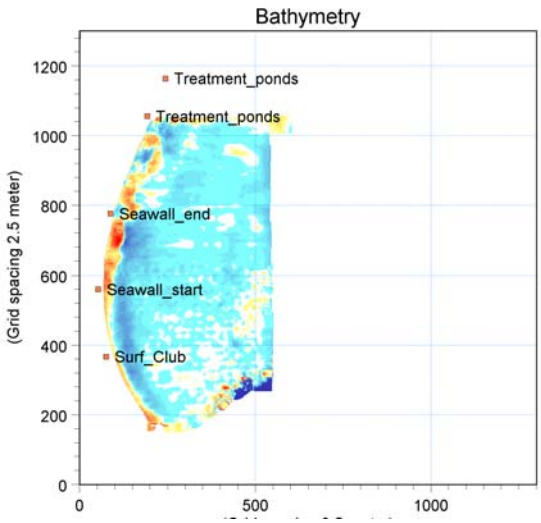
Figure 6-10 Shoreline movement at Nobbys Head at three locations

Historical analysis of bathymetric information of Stockton can provide more detailed information regarding the nearshore areas. Unlike the shoreline analysis, this information provides an estimation of the morphological irregularities below the mean sea level line.

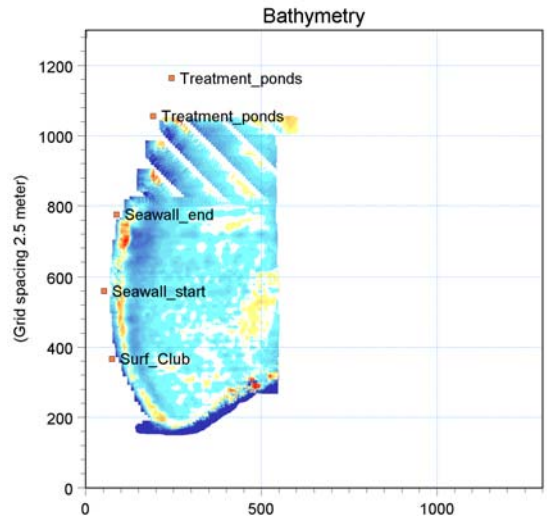
Based on bathymetric surveys provided by DNR, three digital terrain models were created from hydrographic surveys of the Stockton area for the periods 1995, 2000 and 2002. The purpose of this is to determine the spatial morphological changes during these periods. The computed seabed changes are presented in Figure 6-11. In this figure the seabed variation is presented as red to indicate accretion, blue for erosion and white to indicate no changes. The results show a general erosive pattern in the area north of the seawall and accretion in the southern end of the beach.

The seabed changes in the nearshore areas for the period 2000-2002 have been analysed indicating that most of the changes occur in the surf zone. The areas outside the surf zone show smaller variation during this period of analysis. A sedimentation area is observed at the tip of the southern breakwater. This is in agreement with information provided by the Port of Newcastle and also with details of dredging work that was carried out in 2005. This information indicates that some material may be transported from Nobbys Beach reaching the tip of the breakwater and bypassing it. This would indicate a sediment bypassing mechanism is starting to occur at the river entrance. These findings show a similar behaviour to what was observed in the difference plot for the period 1957-1995 carried out by WBM as shown in Figure 6-2.

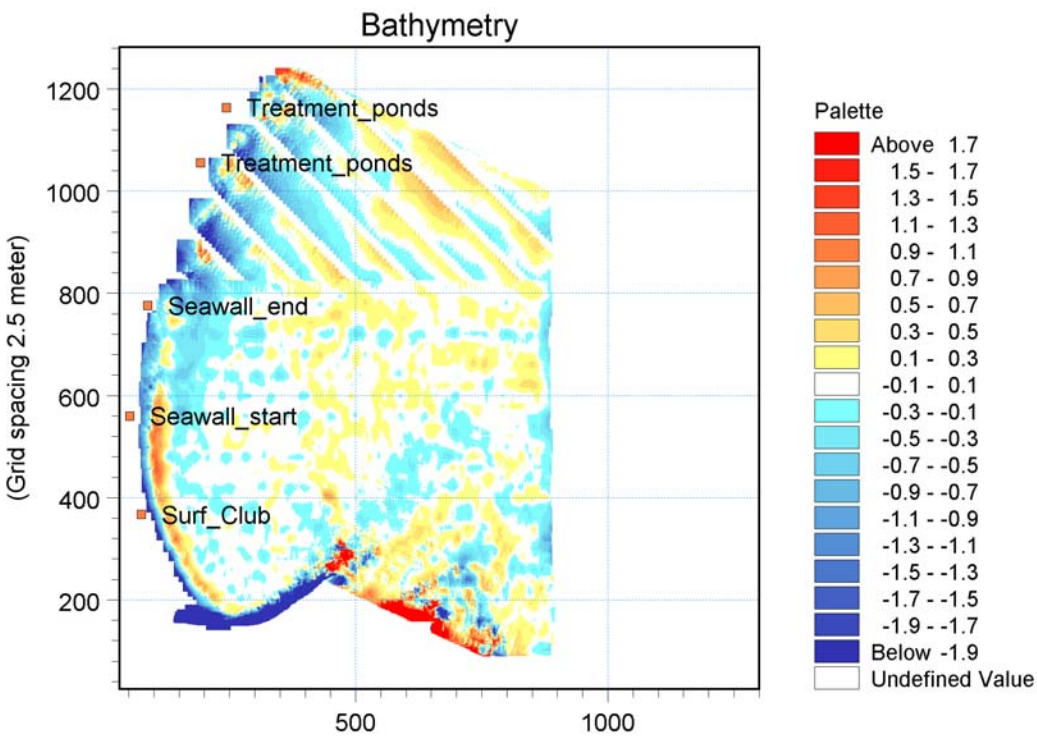
A sedimentation area is observed north of the northern breakwater for the period of analysis. Significant accretion is observed in the northern areas and this may indicate that some of the material that is being transported north from Nobbys Head may be able to reach the navigation channel. However, this is not clear and it may be associated with other processes.



Differences 2002 – 1995



Differences 2000-1995



Differences 2002 – 2000

Figure 6-11 Estimated sand volume variation in the Stockton Area between 1995, 2000 and 2002.

6.4.2 Historical short-term events Analysis

Short-term dune erosion is associated with large wave conditions that combined with high water levels attack the dune front. It is therefore necessary to collect reliable wave conditions and water levels during the storm events and measure beach profiles before and after the event.

As presented previously, one of the most detailed historical reviews of short-term events was prepared by the PWD in July 1995. At a later date, Moratti (1997) identified a number of events and during this study the review was further extended to include storm events till June 2004 and also the 1974 storms. The analysis was based on wave data from the Sydney waverider buoy and water levels at the Newcastle Pilot Station and Fort Denison in Sydney.

The1974 storms were included because they are considered as one of the most erosive events ever recorded on the coast of New South Wales. These events were reconstructed based Foster, D. N., et al. (2001) and Lord, D. and M. Kulmar (2000) because the directional waverider buoy at Sydney was not deployed at that time. A summary of all the events, including date, peak ocean levels, maximum wave height and mean wave direction is attached in Appendix B. The most severe storm events have been extracted and are presented in Table 6-7.

Table 6-7 Overview of extreme storm events from May 1974 till June 2004.

Storm Date	Peak Ocean Level (AHD)	Hs (m)	MWD (deg true N)	Duration (hours)
May 1974	1.47	9.2	155*	75
June 1974	1.09	6.7	170*	217.5
Sept 1995	0.86	6.3	165	48
May 1997	1.21	9.9	151	78
July 1999	1.18	6.1	117	66
June-July 2000	1.26	6.1	177	54
July 2001	0.93	6.7	167	36
Jun-Jul 2002	0.81	6.0	174	48

*Directions were estimated based on personal discussions with Mark Kulmar, Dept. of Commerce, Manly Hydraulics Laboratory.

From observations, severe storms with offshore waves larger than 6m occur annually or bi-annually. The May and June 1974 events are particularly severe due to the combination of large waves, high ocean levels, the duration and the fact that the June event occurred immediately after the May storm. As a result the waves during the June storm were able to reach areas that had previously been eroded only a few weeks before. This is only if protection works were not put in place after the first storm.

Another storm that was found relevant to this focus on is the event of July 1999, where waves propagating mostly from E-ESE were able to penetrate directly into the Stockton area.

Anecdotal information is also available, for example a report from the Newcastle Morning Herald indicates that during the storms of June 1945 a beach profile cut of 50 feet (15m) was observed into the embankment at Dalby Oval. The newspaper reported that “When the tide was at peak level each large wave swept away a foot of sand at a time". The paper reported heavy seas pounding the area and a large shelter shed, a dressing shed, park seats and tree guards being carried into the ocean. Debris was reported to have been swept 12m into the playing area. The northern end of Mitchell Street was severely damaged and a revetment was constructed to prevent water washing away a portion of Mitchell Street and flooding Pembroke Street.

Historical records also show that during the 1974 events extensive damage was caused to the Stockton sewerage effluent ponds which resulted in the ultimate abandonment of the fourth pond (In 1974 there were for ponds, today there are only three).

A number of photos taken on June 16, 1999 along the Stockton shoreline were provided by NCC and are presented in Figure 6-12. The pictures depict erosion at the Surf Club, where the sandbags have been exposed due to the action of the waves. Erosion increases as focus is shifted north. It is evident that the road of the monument collapsed and the seawall is fully exposed with severe erosion behind it. Although the photos do not provide an exact value of the dune foot retreat, they do indicate that it is in the order of 5 to 10 metres. A large run-up is observed in most areas, allowing waves to easily reach the foot of the back of the foreshore, subsequently eroding these areas. Another important point to note is that the severity of the erosion increases at the proximity of the seawall and next to the northern breakwater. These effects can be attributed to the localised scouring induced by the presence of the seawall and the breakwater.

Surf Club

Monument area (facing south)

Monument area (facing north)

From the Monument area (facing north)

Seawall (facing south)

Seawall (facing South)

Figure 6-12 Pictures of July 1999 at different locations along Stockton Beach that show the severity of the storm

6.5 Historical Dredging Activities at Newcastle Harbour

Dredging activities play a relevant role in the understanding of coastal processes as they are part of the historical sediment budget analysis. For this reason, this section has been included in the historical coastline analysis.

Based on information provided by the Port of Newcastle (personal communication), dredging has been carried out in the Port since 1859. Early dredging records are incomplete, but later records are usually more specific regarding dredged areas and other details. Dredging can be split into a number of different time periods which are briefly outlined below.

6.5.1 Dredging 1859-1961

Dredging commenced in Newcastle Harbour in 1859 when dredges began removing sand, silt, loose boulders and soft surface rock. In 1893 rock breaking commenced and continued sporadically until 1921 when a systematic dredging program was initiated. During this program rock and other material on the northern side of the entrance channel was removed to provide a depth of approximately 7.9m at Low Water Ordinary Spring Tide (LWOST). Drilling and blasting operations were abandoned in 1931 in favour of an alternate approach which worked in the entrance channel until 1941. In 1955 a new rock breaker was commissioned for further dredging.

6.5.2 Dredging 1962 -1966

Dredging works commenced in June 1962. During this period approximately 450,000 m^3 of rock and 620,000 m^3 of other material were removed. During this period maintenance dredging also commenced in order to provide 11 metres depth in some areas of the Harbour. A total of approximately 16M "barge tons" of material (both maintenance and improvement works) was removed between 1962 and 1966.

According to evidence provided by Prof. Ron Boyd, some of the material dredged during this period was released at Stockton Beach through a pipeline. This may be related with the shoreline accretion observed during this period in the shoreline analysis at Stockton Beach.

6.5.3 Dredging 1967 - 1976

During this period improvement and maintenance dredging was carried out using the NSW Department of Public Works dredging plant and the equipment of private companies. Dredging works included the Steelworks Channel, the swinging basin off Dempsey Point, the Horseshoe and Entrance approaches (to -12.8m)

6.5.4 Contract 76/2

A capital dredging campaign involved the removal of approximately 11million m^3 of material of which 2 million can be described as hard material. This contract was carried between 1976 and 1983; however there is no information available on the location of the dredging work.

6.5.5 Recent dredging at the Port Entrance

In 2005, maintenance dredging was carried out at the Harbour entrance by the dredger Brisbane. According to information provided by the Port of Newcastle the dredged volume was estimated to be 153,000 m^3 and this is the largest volume of material that has been dredged at this location since the channel deepening.

It is expected that during the extension of the southern breakwater, most of the littoral transport accumulated in the southern end of the breakwater building up a wide and stable beach at Nobbys Head. This dredging works indicated that Nobbys Beach has reached equilibrium conditions to allow for the sediment to be transported north into the navigation channel.

7 COASTAL PROCESSES- MEDIUM TO LONG-TERM PROCESSES

7.1 Motivation

Stockton Beach is experiencing an ongoing loss of sand. While the shoreline appears relatively stable near the breakwater, the areas north of the Mitchell St seawall show progressive erosion, with some periods of strong erosion and beach recovery superimposed on that progressive trend. The most recent studies by WBM (1998) and Umwelt (2002) have shown that there is a progressive erosion problem with a significant volume of sand having been lost from the beach and the nearshore zone over the last century and the coastal erosion threat at Stockton is therefore considered severe.

This section describes the analysis of the medium to long-term coastal processes at Stockton based on a detailed numerical investigation to provide an understanding of the on-going sediment transport processes at Stockton Beach.

7.2 General Considerations

The analysis of the medium and long-term morphological trends at Stockton Beach requires an understanding of the littoral transport to provide a description of the sediment transport processes. This sediment transport is produced by the relevant hydrodynamic conditions of the study area. The most relevant forcing mechanisms are currents (tidal, wave-driven, or any other) and wave action, however there are other elements that may also play a significant role on the sediment transport processes, these can be briefly described as:

- Water-levels;
- Shape and depth of the seabed;
- Sediment characteristics; and
- Sources or sinks of sediment, such as rivers, eroding coasts, etc.

Littoral transport is the term commonly applied to describe the transport of non-cohesive sediments (usually sand) in the littoral zone. The littoral transport is also called the longshore transport or littoral drift. Sediment transport occurs when the waves approach the shoreline obliquely and undergo a number of processes such as refraction shoaling and diffraction, tending to become steeper and higher until they eventually break. During these processes much of the sediment is brought into suspension and then carried along by the wave driven longshore currents. The larger sediment particles remain in the seabed and they roll and move. These two transport mechanisms are usually described as suspended transport and bed load, the sum of these two is the littoral drift.

The magnitude of the littoral drift depends on a number of variables, the most relevant are:

- Wave height;
- Grain size; and

- The wave incidence angle.

7.3 Modelling approach

The determination of the littoral drift is a very relevant element to describe the sediment transport analysis as it allows determining the yearly littoral transport conditions and historical trends of the study area. This requires a thorough understanding of the dominant physical processes. For example, the Stockton area can be divided into two sections, where processes are clearly different:

- The southern stretch, just north of the northern breakwater, is an area affected by the Port breakwater, with variations on the wave heights approaching the beach and therefore complex 2-dimensional wave driven flows;
- The northern stretch is more uniform and the waves approaching the beach shoal, refract and finally break generating a nearly uniform wave field with well defined longshore currents, which vary only due to the orientation of the beach or the incoming waves. In this area 2-D effects due to the breakwaters are not significant.

The description of the sediment transport in these two areas can be carried out by applying a fully dynamic 2 dimensional sediment transport model, however this is a computationally demanding process to investigate for long-term processes.

Another approach is the application of a 1-line type model that allows undertaking long-term simulations and yet providing sound simulation times. One of the main assumptions of this type of model is that the area of analysis has to be quasi-uniform where the beach shoreline and profiles changes occur gradually. Due to this fact they cannot be applied in the southern stretch of the study area because 2-dimensional effects occur and the littoral drift predictions would be incorrect.

In order to provide a full 2-dimensional description of the sediment transport processes at sound computation times, a hybrid approach was applied. This can be described as follows:

Baseline littoral transport analysis

1. A 1-Line model (LITPACK) is applied in an area not influenced by the Port breakwaters (in this case it was chosen north of Point 2 as shown in Figure 7-1). This model will provide a good description of medium and long-term processes (for the 12 years of data); and
2. An analysis of the LITPACK littoral transport results is carried out to identify the most relevant wave events, or those conditions that largely contribute in the sediment transport; and
3. Based on 2, the results of the previous section a number of events will be chosen as the representative of the longshore transport conditions in the area.

Two-dimensional analysis

1. Once the relevant conditions have been determined a 2-Dimensional model MIKE 21 will be applied (8 conditions approximately). These conditions are

weighted and the computed 2-D net longshore transport at Point 2 has to be equal to the littoral net drift computed with the LITPACK model in the baseline study;

2. Based on these results it will be possible to provide a representative estimation of the sediment transport in areas where 2-D effects are significant; and
3. Based on the 2-D simulations a sediment budget will be undertaken to determine the erosion-accretion rates for a number of coastal cells within the study area. This information will allow a description of the on-going processes and the expected future trends.

A detailed description of the baseline littoral transport analysis and the two-dimensional analysis is presented in the following sections.

7.4 Baseline Littoral Transport Analysis

To carry out the 1-D analysis the littoral transport model of the littoral drift model LITDRIFT of the LITPACK suite has been applied. LITDRIFT is a comprehensive deterministic numerical model that computes the longshore currents, the littoral drift and the sediment budget.

LITDRIFT consists of two major components, as follows:

- Hydrodynamic model;
- Sediment Transport Processes Model (STP).

The hydrodynamic model includes the description of the propagation and breaking of waves as well as a description of the driving forces due to the radiation stress variations and the momentum balance of the cross-shore and longshore sediment transport. This model is applied on complex coastal profiles that include features such as profile bars and variation on the sediment characteristics along the profile. Waves can be treated as regular or irregular and can be included as uni or multi-directional.

The hydrodynamic model provides input to the sediment transport components of LITDRIFT. In this way it is possible to compute the sediment transport conditions along the beach profile, including both the wave conditions and the wave driven currents. Other processes can also be included, as additional currents, wind effects, etc.

The sediment transport module, STP, computes the sediment transport for combined wave and current action. In combined waves and currents the turbulent interaction in the near bed boundary layer is of importance for the bed shear stresses as well as for the turbulent processes in the water depth.

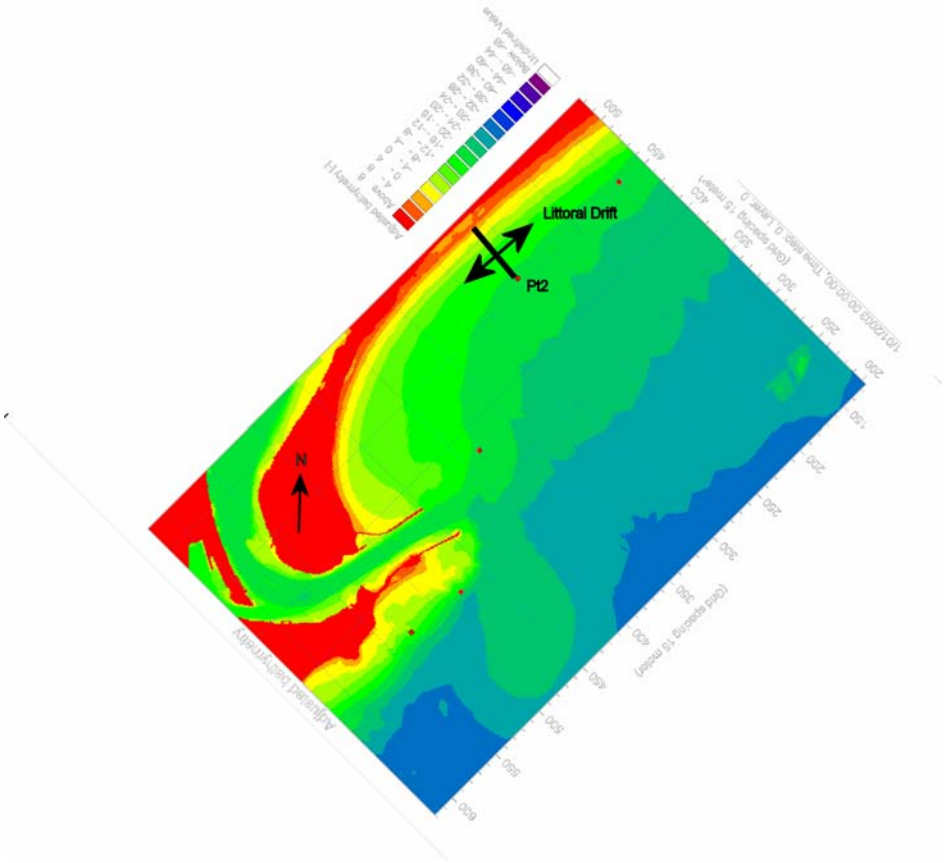


Figure 7-1 Location of the profile location where the LITDRIFT predictions have been carried out.

7.4.1 Input Data

The data required to evaluate the littoral transport includes three main components, namely:

- Beach profile;
- Sediment properties; and
- Wave conditions.

Beach Profile and Sediment Properties

LITDRIFT requires the definition of a beach profile of the area to be investigated. Most of the available bathymetric information in Stockton included bathymetric survey which extended approximately -3m AHD offshore and beach profiles covering a narrow stretch from the dune top to -3m AHD.

As an input the LITDRIFT model requires a beach profile that extends from offshore to the top of the dune line and since most information did not cover the whole beach profile, it was necessary to combine the bathymetric survey of Nov-2002 and the beach profile data provided by DNR from 1999. This profile is located north of the Mitchell St seawall. The latter were extracted from photogrammetric analysis of coastline images. This approach can be applied as the longshore transport processes occur below the mean sea level line which in this case is described by the 2002 profile therefore it is

considered that the profile allows a good representation of the sediment transport conditions in the area. The beach profile is presented in Figure 7-2.

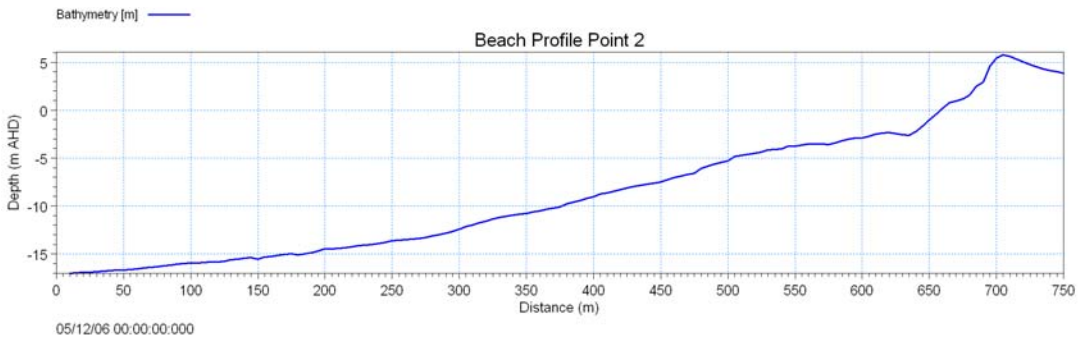


Figure 7-2 Beach profile applied for the littoral transport calculations.

Sediment properties of the area were obtained from the report produced by Roy and Crawford (1980). This report presents a detailed analysis of the sediment characteristics in the Newcastle Bight. Sediment information on the beach and nearshore area is the most relevant in this study therefore we have focussed on this information. According to this report a uniform mean grain size of 0.25mm is observed at the southern end of the embayment. A constant grain size of 0.25 mm has been defined in the model.

Wave Conditions

A relevant input into the numerical model is the wave climate at the toe of the beach profile that will be applied for the computations. To carry out this task wave conditions were transformed from the Sydney waverider buoy offshore Curl Curl Beach in Sydney into the Stockton Area applying the regional wave model MIKE 21 SW. A detailed description of the wave transformation from the Sydney waverider into the Stockton area has bee presented in section 5.

The transformed and the offshore wave conditions are presented in Figure 7-3 as well as the orientation of the beach at the location. As the waves propagate onshore they tend to refract and rotate to become nearly perpendicular to the coastline.

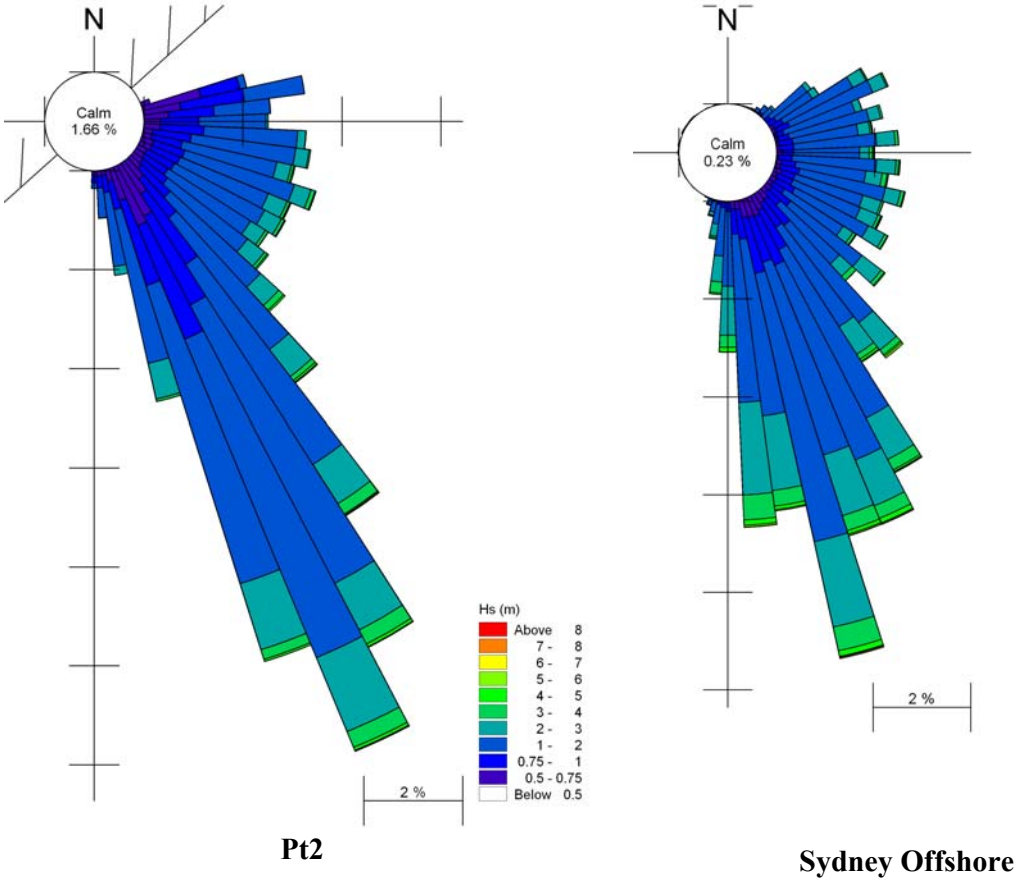


Figure 7-3 Transformed wave conditions at the toe of beach profile 2 (left) and offshore wave conditions (right).

7.4.2 Predicted Littoral Transport

The LITDRIFT model has been applied for the period March 1992 – August 2004 at Point 2. The sediment transport calculations have been carried out for a coastline orientation of 138 degrees (This angle is measured clockwise from the N to a line perpendicular to the beach).

The littoral transport predictions (accumulated in time) at Point 2 are presented in Figure 7-4. As indicated with the arrows, upward variations indicate southward transport whereas downward variations indicate northward transport. The results show that at this point there is a net northward drift, however there is a large yearly variation. The results also show that the southward transport is more episodic, being caused by more infrequent events, while the northward transport is due to more frequent conditions.

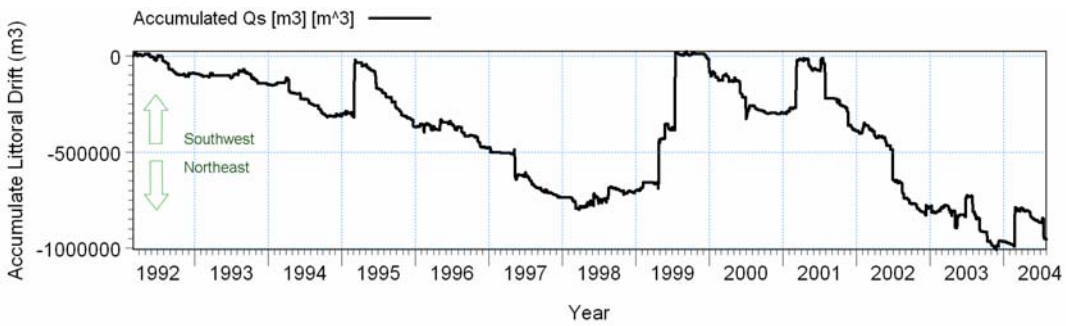


Figure 7-4 Accumulated littoral drift at Point 2.

The long-term average net littoral drift for these 12 years can be estimated as the total accumulated transport divided by the number of years, which indicates an average predicted littoral drift. At point 2 this value is measured as 95,000m^3/year for this period of analysis. Since this is a rather short period of analysis the results have been compared to the estimated historical sand volumes carried out by Umwelt (2002) (presented in section 6).

The historical volumetric analysis indicates sand volume changes of 52,500 m^3/year for the period 1995-2000 and 32,000m^3/yr between 1921 and 2000 in Area 1. Based on the historical data, the predicted littoral drift for the period 1992-2004 seems to be larger than that observed from the historical volumetric analysis. This is, however in good agreement as there are many uncertainties associated with the simulations and the data itself. For example, the wave conditions on the east coast of Australia are often bimodal. This means that they are generated in two different locations, usually in the Tasman Sea and also in the Coral Sea due to cyclones and low pressure systems. The largest components from the SE and NE often occur simultaneously, however this information is not available as the complete wave spectra records are not saved and only statistical values such as significant wave height, period and direction are maintained. Another uncertainty is related to the southward transport as this occurs infrequently during strong events and if there had been a slight alteration in the number of events the results would have changed substantially. Based on this analysis a littoral transport ranging from 50,000 to 60,000 m^3/yr is considered to be more adequate in represent the long-term effects for the period 1992-2004.

The littoral sediment budget for a coastal profile is the sum of the littoral transport contributions which is caused by all possible combinations of wave heights and directions. For example, in the Stockton area the coastline has a varying orientation, but for the purpose of simplification the general direction can be considered to be southwest –northeast.

The south to southeast wave components will yield a northeast littoral drift where as the north to southeast components will yield a southwest littoral drift. The sum of the northeast components is called the northeast littoral drift and the sum of the north to southeast contributions is referred to as the southwest littoral drift. The difference between the northeast and the southwest components is called the net littoral drift, and represents the most significant direction of littoral drift. The sum of absolute values of the two littoral drift components is called the gross littoral drift.

In order to provide a quantification of the annual littoral transport variations the net and gross littoral transport at Point 2 have been computed yearly during the period 1992 - 2004. As it can be observed, the variability of the littoral transport is very large. Mostly the sediment transport is northward (negative values) however there are periods where large variations and even changes in the transport direction have been observed as in 1998 and 1999. The gross transport also shows significant variations; an average value of 800,000m^3/year is observed, however there are a few years where wave activity is larger than normal producing larger values as in 1999, 2001 and 2003. The gross transport is also a relevant variable in the definition of mitigation alternatives.

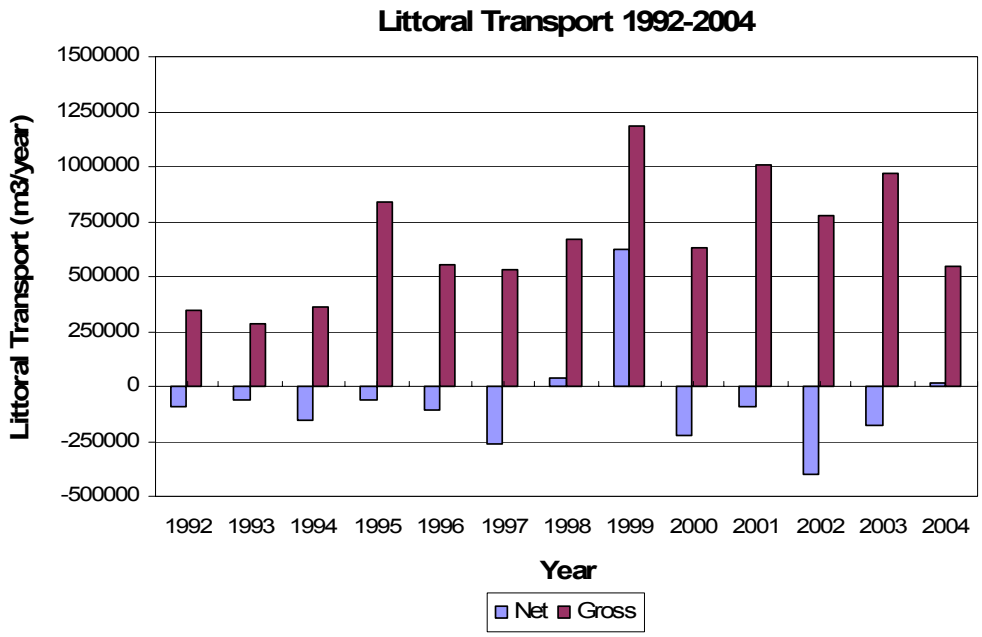


Figure 7-5 Computed net and gross littoral transport at Point 2.

A sediment transport rose at Point 2 has also been produced and is presented in Figure 7-6. This shows the wave conditions (wave height and direction) that produce the littoral transport. The length of each interval indicates the contribution of each of the waves, and as it can be observed the most significant transport is produced by the SSE wave components. This should be expected because they are the most frequent and also the most significant

The Stockton area is relatively protected it should therefore be expected that the littoral transport is largely reduced behind the breakwater; however 2-dimensional effects are relevant and create a complex sediment transport pattern. As mentioned before LITPACK is not able to handle such phenomenon therefore the analysis will be undertaken applying a 2-dimensional modelling approach.

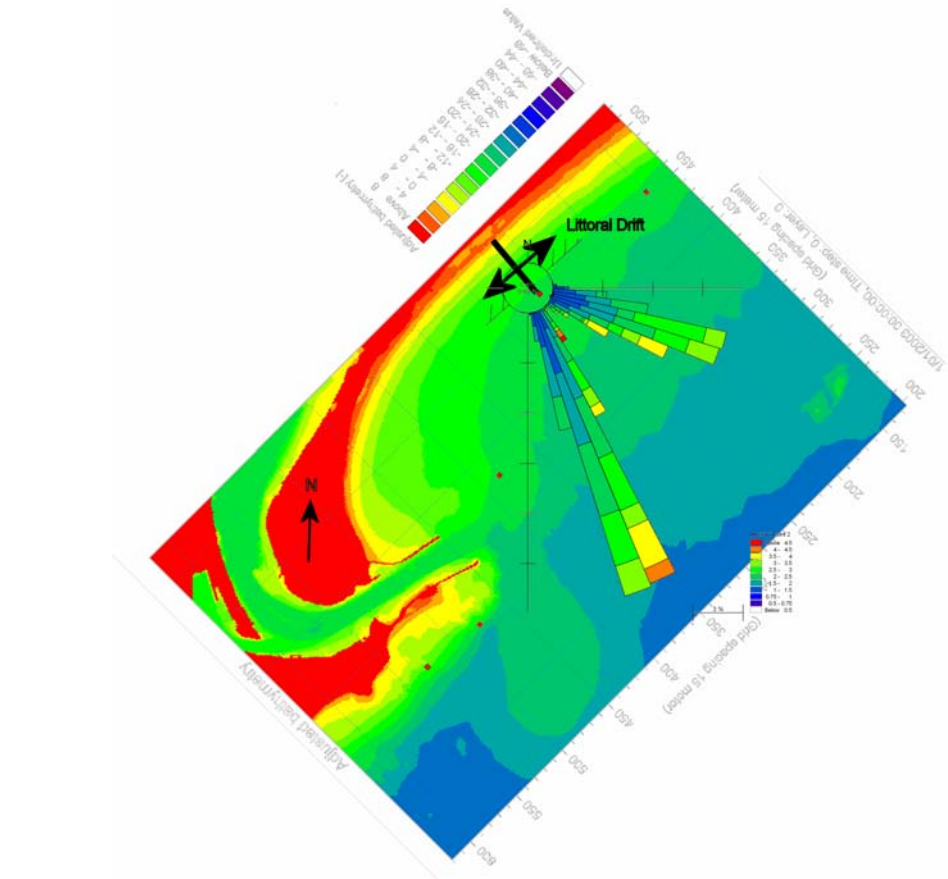


Figure 7-6 Sediment transport rose at Point 2.

An issue that is extremely relevant in the analysis of sediment transport conditions at Stockton is the large variability of the littoral transport as a response to the varying wave climate. For example, there are periods when a clear reversal of the littoral sediment drift is evident such as in March 1995, July 1999 and March 2001. There are also periods when large northward transport are observed such as in 1995, 2000, 2001, 2002, etc, which are usually defined as medium-term effects.

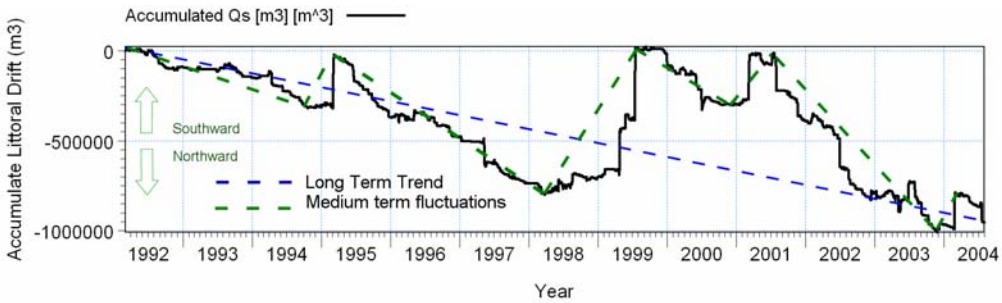


Figure 7-7 Littoral transport predictions for the 1992-2004 period. The blue dashed line shows the long term effects whereas the dashed green line shows the medium term events.

The effect of medium-term processes was investigated by Short et al (1995) and Ranasinghe (2004). These studies analysed the shoreline fluctuations, rotation and orientation at Narrabeen and Palm Beach in Sydney. Ranasinghe's analysis showed that during El Niño phases the northern end of these two embayed beaches rotated clockwise

whereas the opposite was observed during La Niña events. Further to this analysis Ranasinghe investigated the correlation between beach rotation, the SOI and the wave climate. Daily mean wave parameters were averaged over each month to obtain time series of monthly average wave parameters to perform a lagged cross-correlation between wave parameters and the monthly SOI time series. The results indicate that the SOI is significantly correlated with wave height and wave direction. A negative correlation between wave direction was observed which indicates that wave direction would become more southerly with decreasing SOI and more easterly with increasing SOI. A positive correlation between wave heights and SOI was also observed.

Goodwind (2005) investigated the inter- and multidecadal variability in the SE coast of Australia and their impact on coastal systems. This was achieved by hindcasting monthly mid-shelf mean wave direction (MWD) for southeastern Australia, based on the monthly trans-Tasman mean sea level pressure (MSLP) difference between Yamba (NSW) and the north island of New Zealand over a period of 124 years. In this analysis he identified that the MWD varies with strong annual cycles, coupled to mean, spectral-peak wave period. Accordingly, months and years where a more southerly MWD occurs are accompanied by an increase in the spectral wave period. The most significant multi-decadal fluctuation in the time series is from 1894 to 1914, when the Tasman Sea surface temperatures were 1 to 1.5 C cooler and monthly and annual wave direction was up to a few degrees more southerly.

In this study the SOI-longshore transport relationship was analysed by comparing the monthly longshore transport (excluding the long term component) to the SOI index. The results are presented in Figure 7-8. As it can be observed there is a poor correlation and it is not possible to conclude why, but it could be related to local effects, or other factors which are not included in the analysis. An alternate possibility is that Goodwin's approach based on the mean sea level pressure difference could provide a better local estimate of the conditions than the SOI. This is certainly an area of research that should be investigated in more detail as it has implications in the definition of the shoreline fluctuations.

The analysis shows however that there is a medium-term variability pattern which is similar or larger to what is observed in the mean long term conditions. These conditions therefore have to be considered in the definition of the hazard lines as well as in the definition of suitable alternatives. The present analysis does not provide enough information to define the shoreline fluctuations during medium-term events. Therefore an approach based on the historical shoreline fluctuations has to be followed. This discussion will later be addressed in the section that refers to the definition of the hazards lines.

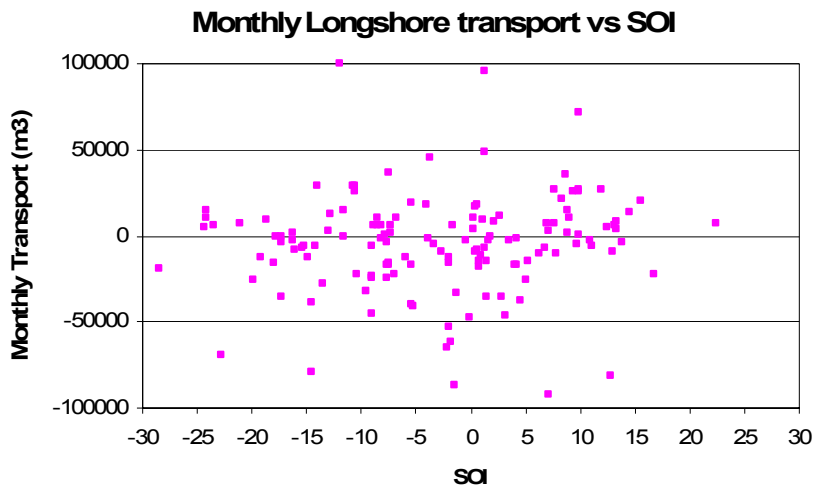


Figure 7-8 Monthly longshore transport excluding the long term component vs the Southern Oscillation Index, as it can be observed there is not clear relationship.

7.4.3 Determination of the representative wave events for the 2D analysis

The selection of an appropriate set of wave conditions that are representative of the annual littoral transport is not a straightforward task. Littoral transport varies non-linearly with wave height, period and direction and it is linearly proportional to the frequency of occurrence of the wave conditions. The conditions that represent the medium to long term littoral transport usually do not correspond to the largest waves, due to their reduced frequency or in other words they occur during short periods of time. The offshore wave direction is not a consistent parameter either, as the dominant wave direction may be different for the southern end of the beach than for the northern. This is due to the effect of the structure and the changing beach orientation along the study area.

A method for selecting the dominant wave events for sediment transport modelling is outlined below:

1. Calculate contributions to the longshore component from the longshore sediment transport calculated using LITDRIFT at Point 2. Secondly the littoral transport results should be grouped in ten degree sectors based on offshore wave height and direction. For example 60 to 70 degrees, 70 to 80, 80 to 90, 90 to 100, 100 to 110 etc. The littoral transport contribution for each wave height and direction is referred to as Pls_{jk} , where j indicates the wave height (H_j) and k the wave direction (MWD_k);
2. Determine a number of representative areas based on the sediment transport analysis. In this case 8 areas have been selected. A theoretical description of one of these areas is shown in Table 7-1;

Table 7-1 Overview of the procedure to determine the relationship between Pls and Hs and the Hrep.

MWD\H	H_1	H_2	H_3	H_4	H_5
MWD_1	Pls_{11}	Pls_{21}	***Pls_{31}***	***Pls_{41}***	***Pls_{51}***
MWD_2	Pls_{21}	Pls_{22}	***Pls_{32}***	***Pls_{42}***	***Pls_{52}***
MWD_3	Pls_{31}	Pls_{32}	***Pls_{33}***	***Pls_{43}***	***Pls_{53}***
MWD_4	Pls_{41}	Pls_{42}	Pls_{43}	Pls_{44}	Pls_{45}

3. Determine a value of H_s. for each of the areas selected in 2; this is taken as the representative H_s, or H_{rep}. Mathematically, it can be expresses as:

$$H_{rep} = \frac{\int H * P_{ls} * dH}{\int P_{ls} * dH} \cong \frac{\Delta H \sum H_i * P_{lsi}}{\Delta H \sum P_{lsi}} = \frac{\sum H_i * P_{lsi}}{\sum P_{lsi}} \qquad \text{Eq (7-3)}$$

   Select the closest wave height event to the representative wave condition calculated above. For example in Table 7-1, H4 is the representative wave height for the area from MWD1 to MWD3 and from H3 to H5 (shown in gray). If the representative wave height consists of events with many wave periods and directions within the group, the most dominant contributor for that wave height, should be chosen as the representative event;
4. The direction of the wave conditions is located in the centre of the interval; and
5. The determination of the equivalent frequency of occurrence, to give a representative P_{ls} of the group is based on the following approach:

$$f_{rep} = \frac{\sum P_{lsi} \, f_i}{P_{ls}} \qquad \text{Eq (7-4)}$$

6. Finally, the computed longshore transport in the 2D model at Point 2 with the integration method has to match that obtained from the baseline study (LITPACK). This provides the link between the baseline study (LITPACK) and the 2-D approach.

It should be noted that this analysis has been based on the littoral transport results at Point 2. The waves that propagate nearly perpendicular to the beach will not produce a great amount of transport, whereas in other areas these waves would induce significant amounts. Therefore the weighting coefficient will not accurately describe the conditions if only the result computed at Point 2 is applied. In order to overcome this issue the determination of wave conditions was carried out based on the LITPACK results at Point 2 and at a location south of Point 2 (Point 1b). Based on the information at these two points it was possible to define "overall" representative wave conditions. The selected wave conditions and the associated frequency that have been applied in the 2D analysis are shown below.

Table 7-2 Wave conditions applied for the analysis of the littoral transport and the definition of the yearly sediment transport.

Case	Hs (m)	Tp (sec)	MWD (deg)	Weight Coeff. / Frequency (%)
1	2.75	8.9	85	8
2	2.75	8.9	105	12
3	3.25	8.5	125	12
4	2.25	8.3	135	10
5	2.25	8.3	155	15
6	4.75	13.8	155	17
7	2.25	8.1	175	11
8	4.25	11.2	175	15

7.5 Two-Dimensional Analysis

7.5.1 Background

The analysis of the sediment transport conditions carried out with LITPACK provides an overview of the sediment transport in the northern part of the study area. At this location the beach profile is nearly parallel and the beach is nearly uniform. Under these types of conditions the longshore currents and the sediment transport processes are generated by the stresses associated with the breaking process of obliquely orientated waves. This current has its maximum close to the breaker line and is generally parallel to the coastline. The position of the breaking line constantly shifts due to the irregularity of the natural wave field.

However, in the vicinity of coastal structures, such as the breakwater at the Hunter River entrance, the current pattern can be significantly influenced. Such structures obstruct the shore-parallel currents by setting up secondary circulation currents. The nature of this obstruction depends on the extent and shape of the coastal structure and is difficult to foresee in advance. At the leeward side of the coastal structures, currents generated by the sheltering effect in the area of diffraction can develop, generating circulation currents a as well as return currents. These are induced as wave set-up in the sheltered areas is smaller than in the adjacent exposed areas thus generates a gradient in the water level towards the sheltered areas.

These processes are likely to occur at Stockton, therefore the one-line model is not fully valid and a dynamic 2-D sediment transport model has to be applied to determine the sediment transport pattern. The application of this model to analyse the sediment transport and subsequently the on-going processes is described in the following sections

7.5.2 2D Modelling Approach

The 2D modelling approach applied for this study is based on the application of the numerical system MIKE 21 and some the information determined in the baseline study. The steps to carry out this analysis can be briefly described as follows:

1. Transform the selected wave conditions (This has been carried out based on the 1-D littoral drift baseline study results) from the offshore buoy at Sydney into the local area using the regional MIKE 21 Spectral Wave model (SW). A description of the model and the transformation procedure has been presented in section 3;
2. A local MIKE 21 PMS wave model is applied to simulate the wave field and the radiation stresses in the local model area using the selected wave events as boundary conditions. This model is applied as it includes the wave processes that are most relevant in the Stockton area such as shoaling, refraction, wave breaking, diffraction, etc. A detailed calibration of this model was carried out in section 5;
3. A local MIKE 21 HD hydrodynamic model is applied to simulate the local flow pattern using boundary conditions from the regional hydrodynamic model (in a nested model set-up) and taking into account the radiation stresses obtained from the local wave model. The data review analysis on Draft Report 1 indicated that the two relevant mechanism in the Stockton area are wave induced currents and the Hunter River flow. These processes are included in the model and a calibration and verification is presented in section 6;
4. A local MIKE 21 ST sediment transport model is applied to simulate the sediment transport capacity in the local area caused by the Hunter River discharges and the wave driven currents;
5. The results of the MIKE 21 ST model are weighted (based on the weighting coefficients) to determine the averaged sediment transport field;
6. Compute the 2-D sediment budgets, which will provide the required information to describe the on-going processes and the coastal hazard lines.

A detailed description of the modelling approach is presented in the figure below**.**

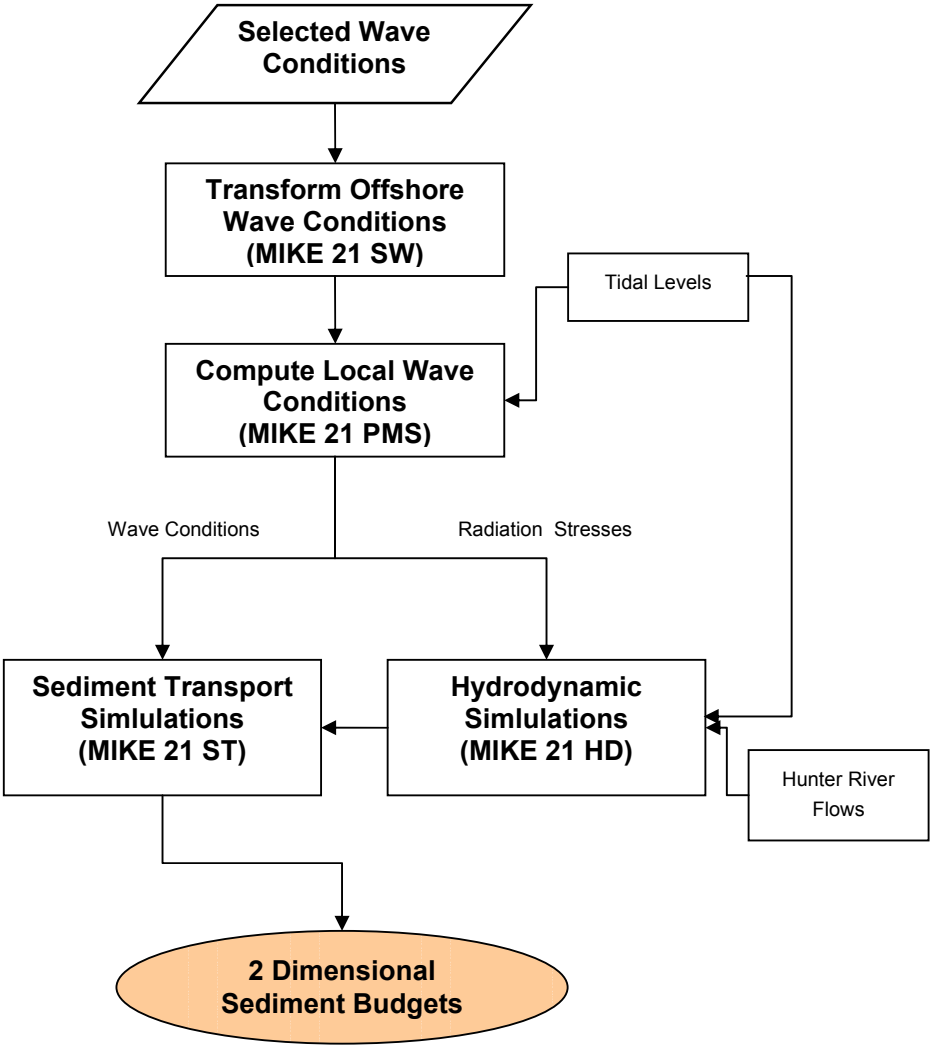


Figure 7-9 Overview of the 2D modelling approach

7.5.3 Model Setup

The local 2D numerical model of the Stockton Beach area has been established. The model domain covers Stockton Beach, the Newcastle Port area, the area south of Nobbys Head and the southern end of the Stockton Bight at the vicinity of Stockton Beach.

The model area extends 12 km from SW to NE and 7.9 km from NW to SE. This model extent has been defined to describe the coastal tidal flow patterns, flow interactions with discharges from the Hunter River and the wave-driven currents. The model domain has been configured so as to include all relevant bathymetric features that may potentially influence the complex sediment transport processes in the area. The maximum water depth observed in this area is -33m AHD. The grid resolution is 15m. An overview of

the study area and model domain is presented in Figure 7-10. A description of the model calibration has been presented in section 6.

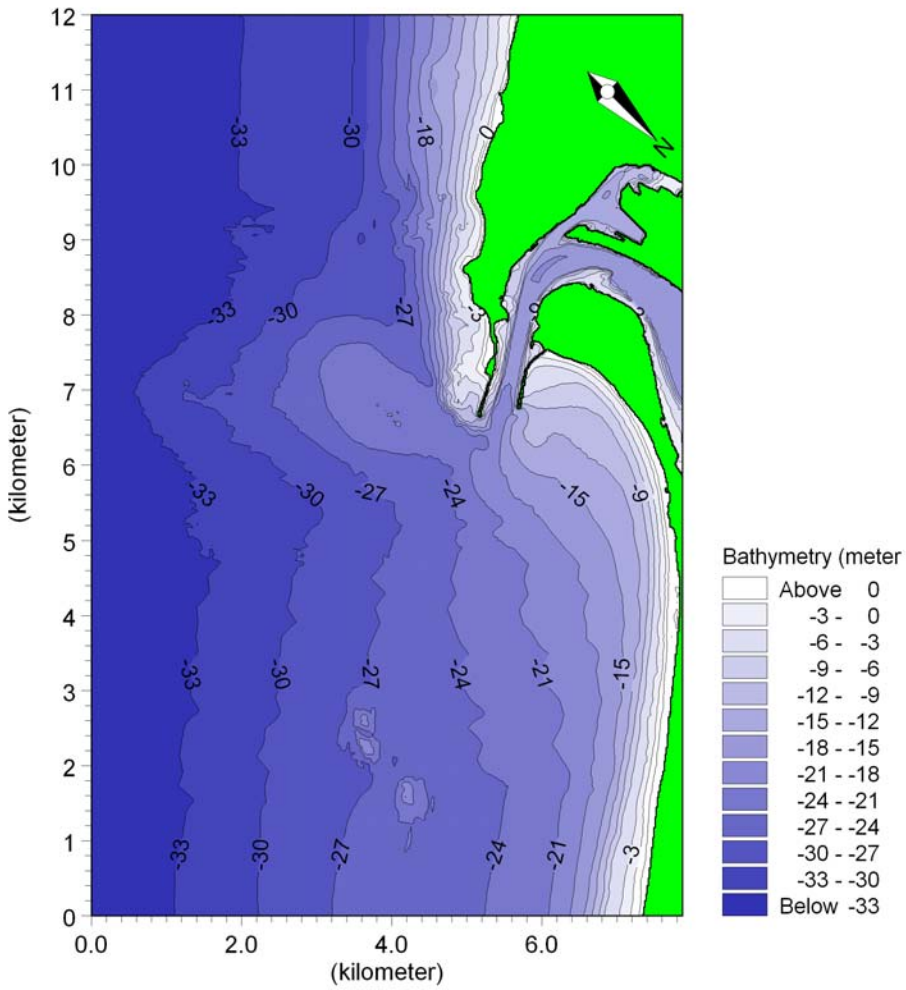


Figure 7-10 Extent of the local model.

The wave and hydrodynamic components of the model have been calibrated and verified against data. A detailed description of the model setup as well as the calibration results are reported in section 5.

To compute the sediment transport in the study area, the sediment transport module MIKE 21 ST has been implemented. This model calculates sediment transport fields based on input from the wave conditions and the hydrodynamic (current field and water levels) models. The extension of the model is the same as that of the wave and hydrodynamic models. A uniform size distribution of d_{50}= 0.25mm and standard deviation of 1.4 has been applied.

The sediment transport is calculated using the sediment transport model STP, which was described for the LITDRIFT calculations. The sediment transport model is able to resolve complex horizontal and vertical sediment transport patterns in the nearshore area.

The results of the 2D sediment transport model are applied to compute sediment rates in complex areas. This is necessary to identify areas and patterns of erosion or accretion which are important to provide an understanding of the on-going sediment transport processes and also to define mitigation strategies.

Selection of Simulation Tidal Period

To obtain the representative conditions responsible for sediment transport at Stockton it is necessary to determine a simulation period. As observed in Figure 7-11, the model simulations have been defined for two tidal periods from 18-12-04 23:34 till 20-12-2004 00:21:00. A warm-up period has been included in the simulation to achieve proper tidal velocities within the model domain and avoid any undesired initial effects.

This tidal period has been selected because it represents mean tidal conditions of the area (average between spring and neap tides), which are the most relevant when evaluating long term events. As it can be observed, the tidal range during the simulations is approximately 1m.

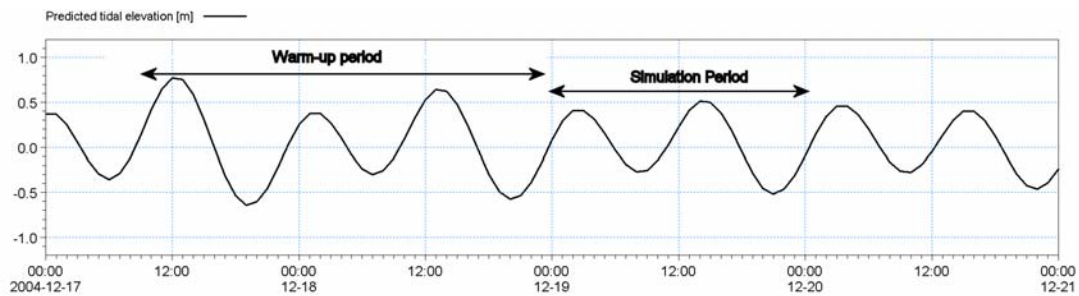


Figure 7-11 Simulation period, including warm-up period.

7.5.4 Model simulations

The calculation of the wave and hydrodynamic conditions and sediment transport capacity in the study area has been carried out for the combined effect of waves and tidal currents. The simulations were carried out for each of the 8 wave events presented in Table 7-2. The results of the simulations for Cases 1, 2, 6 and 8 are presented in Figure 7-12, Figure 7-13 and Figure 7-14. These results provide an estimation of the wave field, mean current conditions and mean sediment transport capacity.

Wave Simulations

The results of the wave simulations for Cases 1, 2, 6 and 8 are presented in Figure 7-12. The results display the predicted wave heights in colours and the significant wave height intensity and direction as arrows.

It is observed that the beach is most exposed to waves propagating from the E-NE (Case 1 and 2) as they are able to reach the Stockton area directly with minimum dissipation. For these cases the wave field is rather uniform along Stockton Beach; however, a short stretch to the south is partially shadowed by the northern breakwater.

For waves propagating from the S- SE the wave field is rather different as the Port breakwaters generate a shadow area in the southern part of beach. This pattern generates

a protected area at the Caravan Park however as we move north the coast is more exposed and the effect of the Harbour breakwaters is less pronounced, eventually becoming negligible. The extension of the lee area varies for different conditions, for waves propagating from the S to SSE the lee area is larger and it reduces for waves propagating from the NE.

A process that is relevant to mention is the focussing of wave energy on the beach for waves propagating from the S to SSE. This issue is well recognised as it was investigated by Treloar et al (1977), which concluded that there is a significant concentration of wave energy at the Port entrance during storms from the S and SE. The focussing of wave energy on the beach is a relevant factor in the analysis of the sediment transport conditions at Stockton because this process tends to produce an uneven wave pattern along the beach, which will undoubtedly have an effect on the sediment transport conditions. This focussing occurs at different locations depending on the wave conditions. For example waves from SSE tend to focus in an area around the sewage treatment ponds, whereas for waves propagating from the South, the area of focus tends to move further north, as shown in Figure 7-12.

The results also show that the predicted wave field at Nobbys Head is quite complex due to the shallow rocky areas and the uneven bathymetry of the area. Waves tend to break in an unstructured way which will induce complex wave-driven currents in this area and in turn the sediment transport processes.

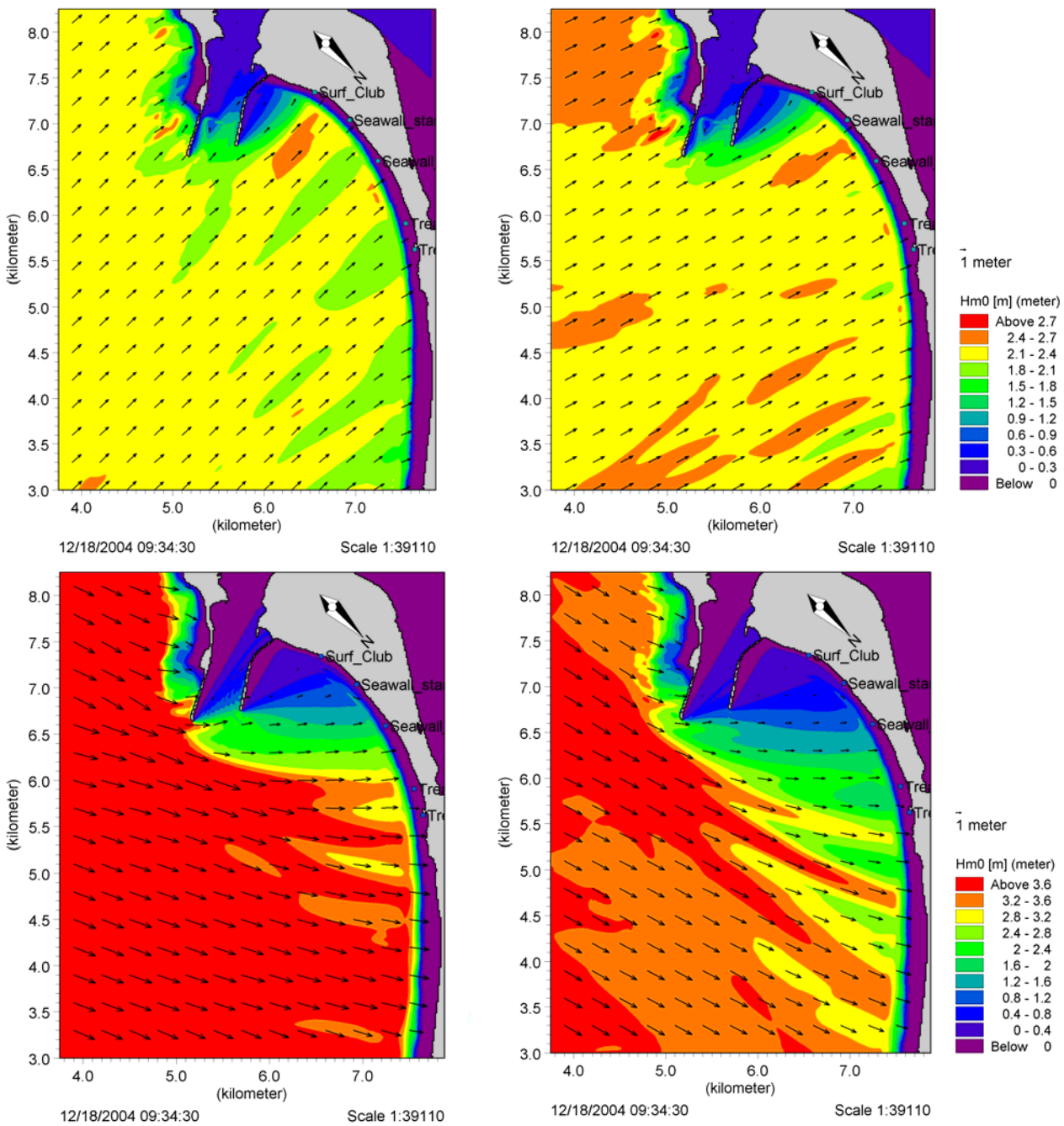


Figure 7-12 Wave predictions at Stockton for Cases 1, 2, 6 and 8

Hydrodynamic Simulations

The residual current conditions (or the average current flow) for a full tidal cycle for the four different wave conditions are presented in figure 7-13. The colours show the average current velocity intensity during this tidal period while the arrows display the resulting velocities (intensity and direction).

As expected the current predictions are quite different for Cases 1 and 2 when compared to 6 and 8. For cases 1 and 2 the waves propagate from the E and ESE and produce a uniform longshore transport along the northern areas of Stockton Beach. In the southern end these currents become small when the waves refract and approach the shore at near perpendicular to the beach. For Case 2, the flow turns offshore just north of the seawall. At this location the waves are exactly perpendicular to the beach. Due to this phenomena the flow direction is different north and south of this point as the angle of approach of the waves with respect to the beach is opposite in the two locations.

The flow predictions at Nobbys Head show a very complex pattern and this is due to the shallow reef and the rather uneven shape of the bathymetry of this area.

For Cases 6 and 8 the waves propagate from the S to SE. It can be seen that the flow is largely influenced by the presence of the breakwaters at the Hunter River entrance and secondary circulation currents are generated. At the leeward side of the coastal structures, currents generated by the sheltering effect of the structure in the diffraction area are developed, generating circulation currents as well as return currents. These are induced by the fact that the wave set-up in the sheltered areas is smaller than in the adjacent exposed areas and this generates a gradient in the water level towards the sheltered areas.

As we move further north, and the effect of the structures is nearly negligible a nearly uniform longshore current is reinstated. A few variations are observed along the northern stretch. These are produced by a focussing of wave energy which induces spatial variation of the wave radiation stresses and the wave driven current. It should be expected that this process will have an effect on the sediment transport conditions in this area.

The flow conditions at Nobbys Head are more uniform generating jet flow which is mainly directed NE. South of Nobbys Head the currents are rather chaotic, similar to what was observed for cases 1 and 2. Undoubtedly the uneven shape of the seabed generates a complex flow pattern.

It should be mentioned that the results show that the tidal currents are not significant in comparison to the wave induced flows. At the entrance the effect of the river inflow and outflows are very small for Cases 6 and 8 and slightly larger for Cases 1 and 2. For the later, the resulting component is directed into the Port.

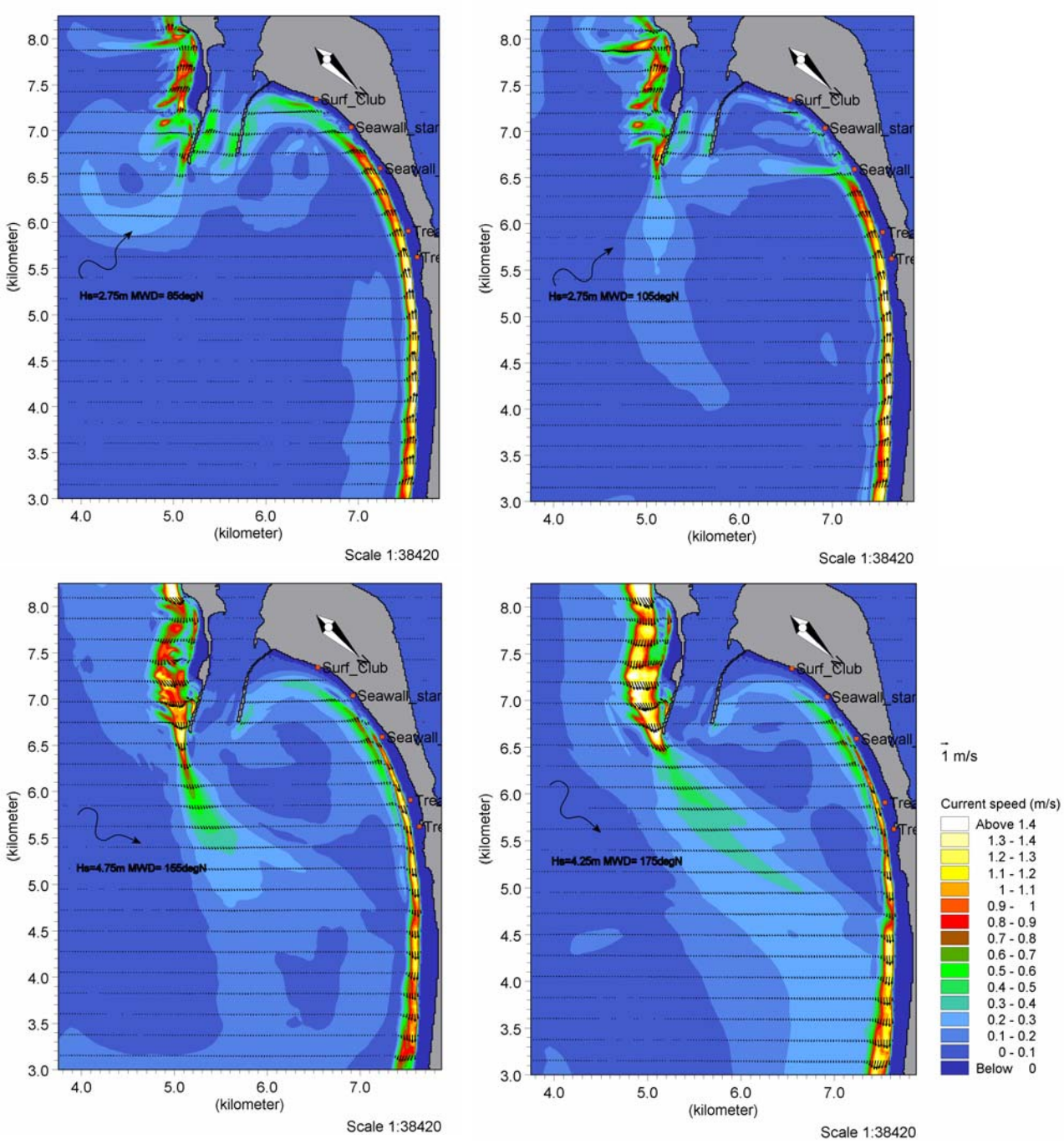


Figure 7-13 Average currents during a full tidal cycle at Stockton for Cases 1, 2, 6 and 8

Sediment Transport Calculations

The sediment transport has been calculated over the simulated tidal cycle for the four wave conditions and is presented in the figure below. The colours show the average sediment transport during this tidal period and the arrows the resulting sediment transport flux (intensity and direction). The results for the 4 cases are presented in Figure 7-14. The results show that the sediment transport occurs mainly in the nearshore areas where wave driven currents are relevant. Outside these areas the sediment transport is quite small.

Cases 1 and 2 (waves propagating from the E and ESE) produce very uniform longshore transport along the northern areas of Stockton Beach. As the waves become perpendicular to the beach, the longshore transport reduces to nearly zero. For Case 1, this occurs just north of the northern breakwater whereas for Case 2 this occurs at the northern end of the Mitchell St seawall, where a large sediment flow is generated offshore. The difference in behaviour for these two cases can be attributed to the variation of the incoming waves.

The results indicate that during easterly events some of the sediment is transported towards the northern breakwater and into the navigation channel. Case 1 also shows some sediment transport inside the navigation channel, in the southern area inside the harbour towards Horseshoe Beach and in the northern part of the channel offshore. Therefore during these events sediment accumulation can occur close to the end of the northern breakwater.

The sediment transport pattern at Nobbys Heads is quite complex, but a resulting northern component is predicted for both cases. The complex sediment transport pattern could be caused by hard seabed areas such as rock outcrops or reefs.

For Cases 6 and 8 the sediment transport pattern is quite different. Again, it is largely influenced by the presence of the breakwaters at the Hunter River entrance. There is a significant variation of the longshore transport along Stockton Beach. The longshore transport is rather small in the southern areas and increases as the southerly waves are able to propagate towards the beach or in other words the shadow effect of the structures diminishes. A return sediment transport pattern (south going) is observed at the leeward side of the coastal structures. This is expected as the wave setup is larger in the exposed areas behind the northern breakwater.

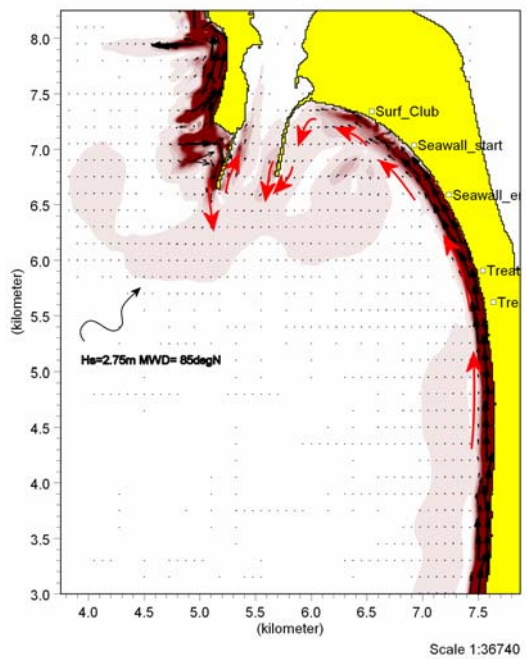
The more exposed nearshore areas show north going transport that increases as the beach is more exposed to the incoming waves. This longshore transport variation indicates (in this case an increase towards north) that there is a transport deficit and therefore potential erosion.

In between these two distinctive areas there is a nodal point or area where the longshore transport is nearly zero. Usually nodal areas are highly erosive, which in these two cases are located in the northern end of the Mitchell St seawall

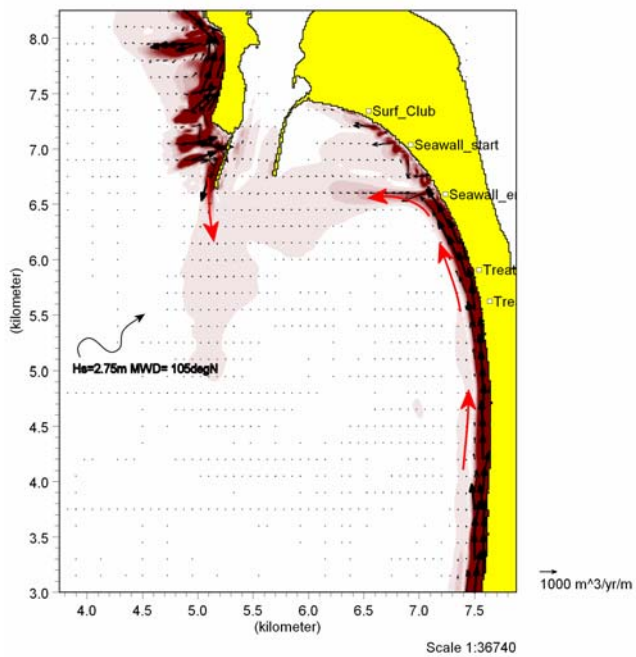
The effect of the wave focussing on the sediment transport north of Stockton is also predicted and varies for different wave conditions. For Case 6 it is observed at the

sewage treatment plants and at Fern Bay and for Case 8 only in Fern Bay. This mechanism generates an uneven sediment transport pattern that has a large influence on the sediment transport pattern in this area. It should be noted, however, that these predictions estimate the sediment transport pattern as if this condition persists throughout an entire year. However, as we have seen in the baseline study a large number of waves influence the littoral transport therefore this effect will be diffused but it will occur in these areas.

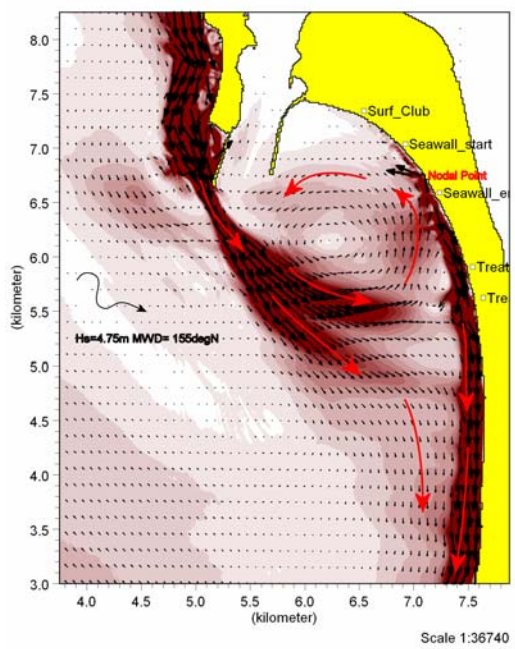
The results show a clear northward sediment transport pattern at Nobbys Head that extends to the northern end of Stockton Beach. This indicates a potential bypassing mechanism, however it should be expected that most of this material is dredged from the navigation channel during maintenance dredging. The predictions do not indicate sediment transport from Nobbys Head to Stockton.



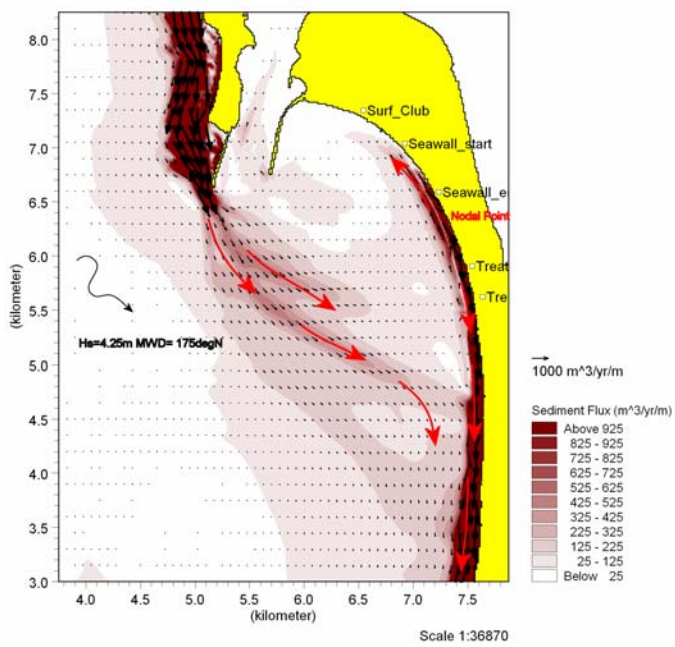
Case 1



Case 2



Case 6



Case 8

Figure 7-14 Average sediment transport predictions for waves and currents during a full tidal cycle at Stockton for Cases 1, 2, 6 and 8

Annually Averaged Sediment Transport Conditions

The results provided in the previous section allow an understanding of how the different wave conditions combined with the tidal currents affect the sediment transport patterns. The results do not take into account the percentage of time that each event occurs and cannot be used to determine the yearly conditions. Nevertheless they provide a description of the long-term sediment transport conditions in the study areas. The 8 simulated events have been averaged or weighted applying the weighting coefficients presented in Table 7-2 .

The computed yearly sediment transport is presented in Figure 7-15. This figure shows the average yearly net transport based on the littoral transport statistics carried out during the baseline study. The main directions of sediment transport have been marked with red arrows. As it can be observed, the dominant waves from the SE-S generate a dominant northward transport.

The results show that the most significant transport occurs in the nearshore areas where wave driven currents occur. Other areas affected by tidal currents show very little transport, except for the Port entrance. No evidence of significant offshore sand transport is observed from these results. Due to the sheltering effect of the breakwaters in the southern areas the longshore transport is rather small but increases to the north. This is a response to the wave action becoming more significant as the shadow effect of the structures diminishes.

Secondary circulation currents at the leeward side of the coastal structures indicate that the area immediately north of the northern breakwater is slightly accreting. This is in agreement with the findings of the PWD (1985) study and the WBM (1998) coastal hazard definition study, which indicated slight accretion in this area. It should be expected that this accretion be minor as the coastline reaches equilibrium conditions.

A nodal point or a point where the sediment transport is zero is observed north of the Mitchell St seawall. North of the nodal point the sediment transport is northward whereas south the sediment transport is southward. This nodal point indicates that this is an erosive area.

From this nodal point towards north the littoral transport increases. This indicates a sand starving process, as more sediment is leaving to the north than being replaced from the south. The results indicate an erosive pattern here, which is in agreement to the PWD (1985), WBM (1998), Umwelt (2003) studies that observed a slight recessional trend between Hereford St and the Stockton sewerage ponds.

A preferential sediment transport path is observed at the sewage treatment ponds and the Fern Bay areas. This path concentrates the incoming energy in different short stretches and induces a variable sediment transport pattern, as opposed to the “expected’ more uniform longshore currents. The results show that the focussing of wave energy is due to the effect of the offshore shoal located west of the Port entrance. It should be expected that this effect be smoother as an averaging processes has been carried out where only 8 events are applied. This is a rather limited number of conditions therefore the results may overestimate this effect and the focusing may be distributed over a

larger stretch, a result that could be attributed to the contribution of more wave conditions than those used for the averaging.

The results also show a northward sediment transport at Nobbys Head bypassing the southern breakwater and partially reaching the Port entrance. No bypassing towards the southern beaches is predicted and most of the material transported from Nobbys Head is directed north. It is unlikely that this material is able to reach the Stockton Areas as it will first reach the Newcastle Port navigation channel and most likely be removed during the channel maintenance dredging operations. If allowed to be transported by the currents and the waves the sediment would require a large morphological time scale to reach the new equilibrium imposed by the Newcastle Port breakwaters.

Sediment transport is predicted on the southern side of the navigation channel, indicating that some of the material is being deposited in this area and ultimately reaching areas such as Horseshoe Beach. This is in agreement with historical information of the area.

The sediment transport in the areas south of Nobbys Head shows a predominant northward direction. This area is exposed to waves propagating from the southeast; however there is no detailed information available about the sediment distribution in this area. It is important to note that the model predicts the sand transport capacity which is the amount of sediment that the waves and currents are able to transport if a sediment supply is available. However this area includes a number of rocky patches therefore it should be expected that the actual sediment transport will be smaller than the predicted sediment transport capacity.

Erosion at the tip of the northern breakwater was indicated in the previous study of Umwelt (2003). This erosion appeared to propagate back towards Stockton Beach with a drainage pathway from the nearshore zone of Stockton Beach to the entrance channel. The present results do not indicate this pattern, however it should be expected that localised erosion or scouring around the tip of the breakwater should occur as shown in Figure 7-15.

Variations of the predicted sediment transport pattern can occur if the sediment size differs significantly from appointed values in the simulations. Since grain size over the area varies at different locations, some of the results should be carefully evaluated and the 2D modelling results should be interpreted accordingly.

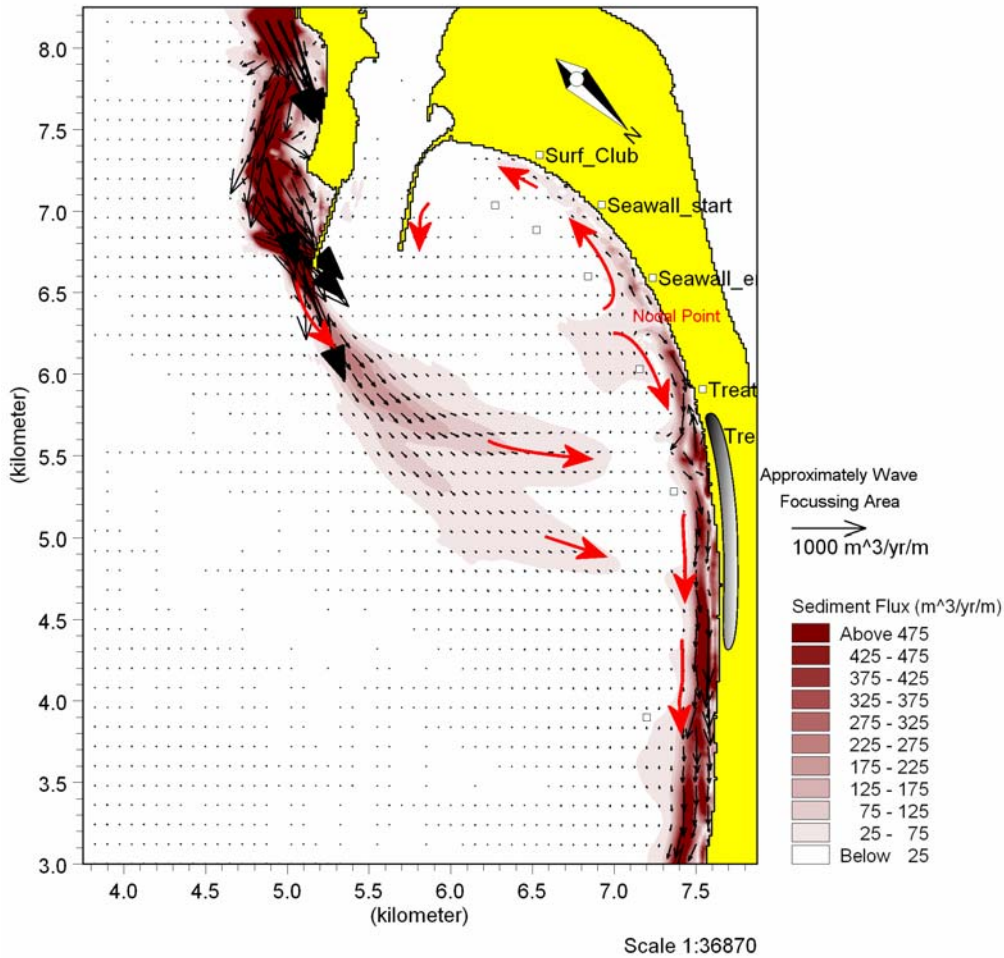
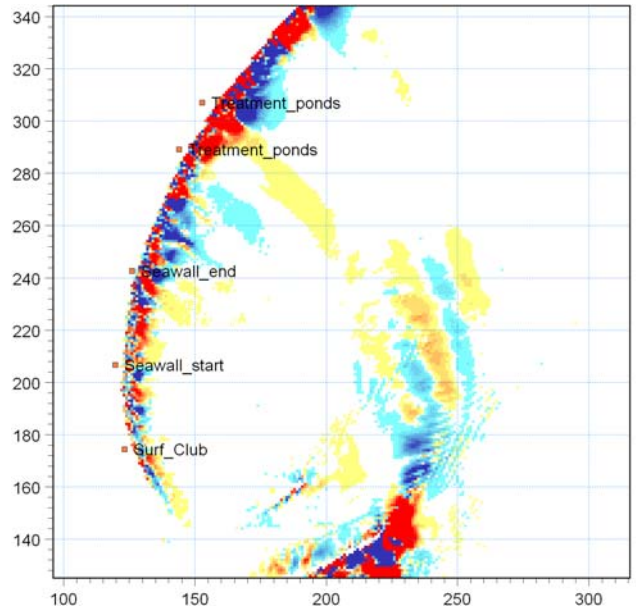


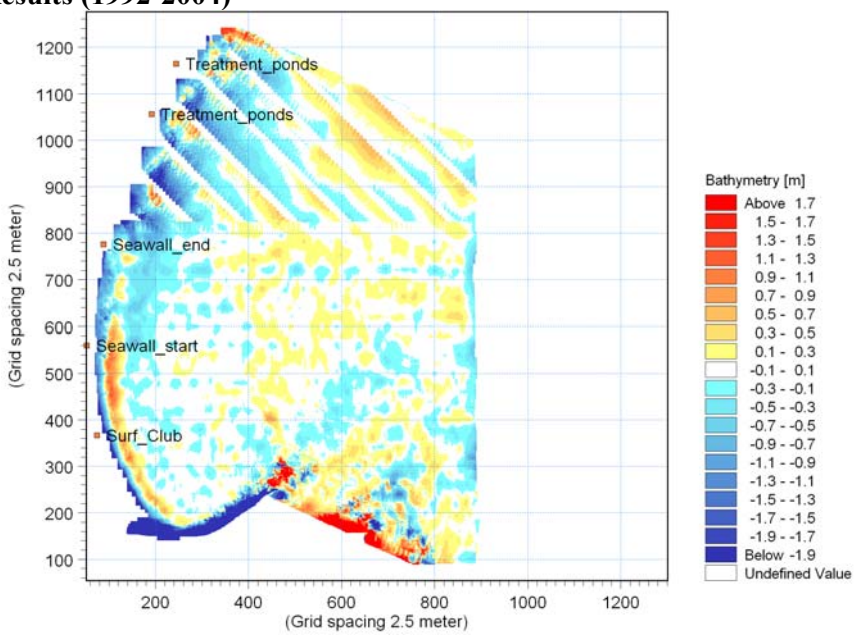
Figure 7-15 Computed yearly transport based on the average conditions between 1992-2004

Predicted erosion accretion pattern

The predicted erosion pattern has been compared to the difference map formulated from hydrographic surveys between 2002 and 2000. As it can be observed the results show an area of erosion in lee of the northern breakwater, similar to what has been measured. The nodal or neutral point is also observed at a similar location, however the model prediction shows a more variable pattern with an eroding and accreting area, but clearly the pattern is similar. Sand deposition in the navigation channel is also observed as well as sediment deposition at the NE end of the northern breakwater. This deposition is due to the result of a returning flow and also a response to the interaction between the wave driven currents and the flow at the river entrance. A slightly accreting area north of the river entrance is observed which may indicate a developing bypassing mechanism. It is important to note that the processes in this area occur slowly due to the water depths that limit the transport capacity of the currents and the wave-driven currents.



Average Model Results (1992-2004)



Bathymetric difference 2002 - 2000

Figure 7-16 Computed and measured morphological changes at Stockton Beach and nearby areas.

7.5.5 Shoreline Change Predictions

Further to this analysis the littoral transport has been computed at different locations along Stockton Beach and Fern Bay. The results are presented in Figure 7-17 and provide a very good overview of the variations of the littoral drift for the average year. The results are presented in thousand cubic metres of sand per year and based on these results it is possible to determine the shoreline movement as the difference of the incoming sediment minus the outgoing sediment divided by the length of the coastal cell and the closure depth.

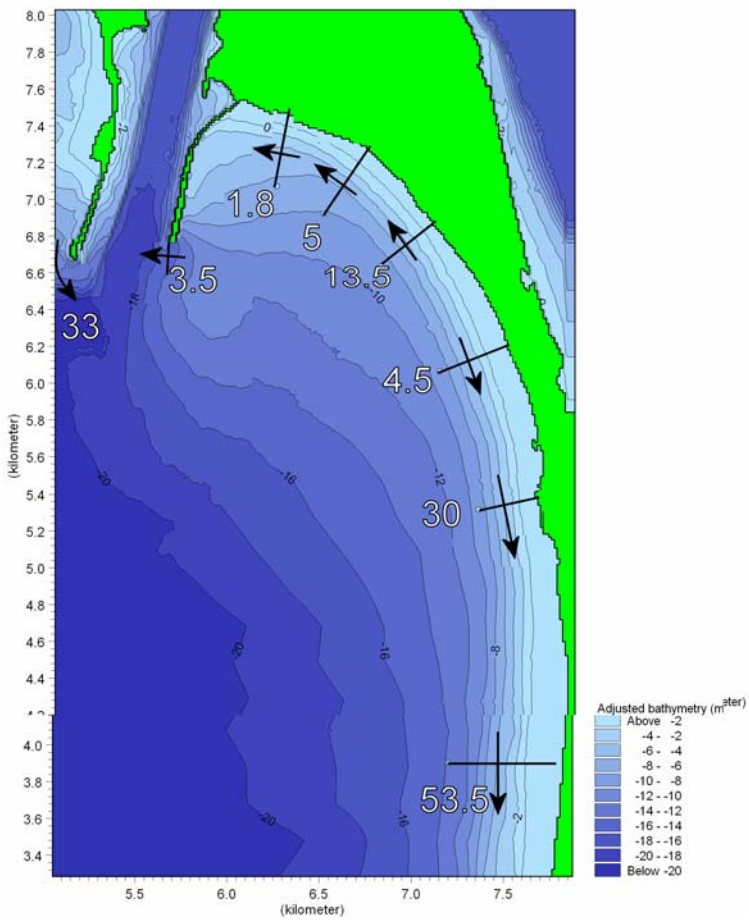


Figure 7-17 Computed yearly littoral drift based on the 1992-2004 conditions (Expressed as 1000 m^3/year).

The yearly shoreline changes based on the 1992-2004 statistics have been computed and compared to the 1992-2001 conditions. As it can be observed, the predicted and measured erosion in the area north of the seawall show similar results. An overestimation is observed at the sewage treatment ponds area. It is likely that this is produced by the wave focussing that produces an overestimation in the shoreline erosion due to the limited number of wave conditions applied to establish the yearly conditions.

The results show slight accretion at the Caravan Park, the Surf Club and the area south of the seawall (monument). The results also show a similar behaviour when compared with the historical shoreline results. Some differences are observed, for example: according to historical data the northern area is quite stable and is even accreting, however the 92-01 shoreline analysis indicates erosion, therefore there is indication that these measurements may be affected by medium term erosion.

Long-term shoreline predictions at the different locations of the Stockton area have been carried out. The results are presented in Table 7-3. The predictions show a similar pattern and trend, which is a very relevant point in the morphological analysis. Some differences are observed, but these should be expected due to the uncertainty of the shoreline analysis that may be influenced by short-term processes such as dune erosion

or rip currents, especially in the southern areas, which have been described as stable or slightly accreting. Another factor that may have contributed to these differences is that the sediment transport averaging has been carried out with a limited number of wave conditions (8 cases).

Table 7-3 Predicted shoreline changes long- term analysis.

Area	Predicted Shoreline Changes 92-04 (m/yr)	Shoreline Changes 92-01 (m/yr)	Historical Shoreline Changes (m/yr)
Caravan Park	0.4	-2.2	0
Surf Club	0.5	-2.8	0.28
South Seawall	1.5	-2.2	0.22
Child Care Centre	-2.6	-2.6	-1.9
Sewage Ponds S	-3.3	-2	-1.8
Sewage Ponds N	-1.7	-2.2	-0.9

8 SHORT TERM BEACH EVOLUTION ANALYSIS

8.1 Dune Erosion

Coastal dunes are often eroded during storm events where large waves in combination with higher water levels are able damage this natural defence. It is of significant value to be able to predict the impact of a storm on a dune in terms of recessional distance, eroded volume and the possibility of breaching. The erosion of the dune is described as an irreversible process as the dune front moves back only as the result of erosion by the waves running up the beach.

Several models have been developed for this purpose, and in this case we will apply the wave impact approach that estimates the sediment transport from the dune and associated profile change as a result of direct wave action. The proposed modelling approach of the dune erosion consists of two parts:

1. A description of the retreat of the dune front, notably the variation in the dune foot level as the dune front is eroded; and
2. An assessment of the rate of sediment loss from the dune front under given conditions in the form of morphology, water level and wave conditions.

The dune geometry is described by the following parameters, Figure 8-1:

- Dune crest level: H_{dune};
- Dune foot level: FL;
- Coastline position, the position of the contour at the MWL: Y; and
- Dune front position, the position of the dune foot: Y_{dune}.

The dune crest level H_{dune} is an input parameter, while the time variation of the other parameters is determined by the model. The dune front is taken to be practically vertical, so the position of the dune foot is nearly the same as the position of the dune crest.

The retreat of the dune front

The rate of retreat of the dune front is determined by the continuity equation from the rate of loss of sediment from the dune front, Q_{dune}. During the time step of Δt the dune front retreats the distance ΔY_{dune} which is determined so that the change in volume equals the rate of loss

$$\Delta vol = Q_d \Delta t \qquad \text{Eq. (8-1)}$$

As the dune front retreats, the dune foot increases its level by moving back along a slope of $\tan(\alpha_{re})$. The actual beach slope $\tan(\alpha)$ is determined from the foot level FL and the difference in the position of the coastline and the dune foot:

$$\tan(\alpha) = (FL - MWL)/(Y - Y_{dune}) \qquad \text{Eq. (8-2)}$$

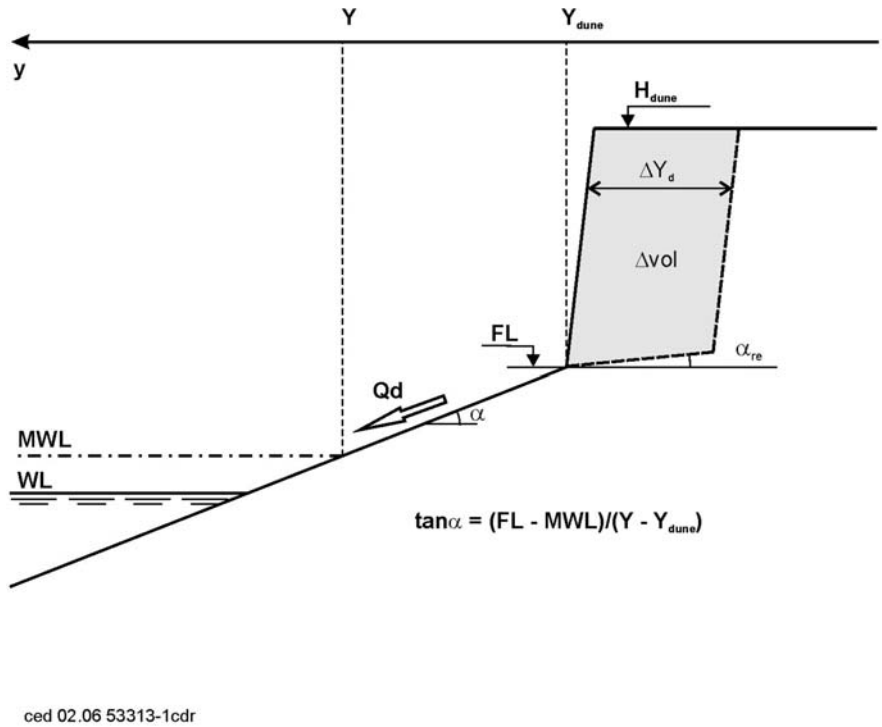


Figure 8-1 Definition sketch for the dune erosion model

The rate of loss of sediment from the dune is calculated from the run-up with formulae based on Larson et al. (2004). The maximum loss, corresponding to a situation with the dune foot level at the water level is found as

$$Q_{d0} = 4C_s R^2 / T \qquad \text{Eq. (8-3)}$$

C_s is a calibration coefficient; R is the wave run-up and T the wave period. The range of values for this calibration factor will be discussed later in the report. As the dune foot level is higher than the water level, the actual rate of sediment loss is found by applying a reduction factor:

$$R_{ed} = \exp(-2(FL - WL)/R) \qquad \text{Eq. (8-4)}$$

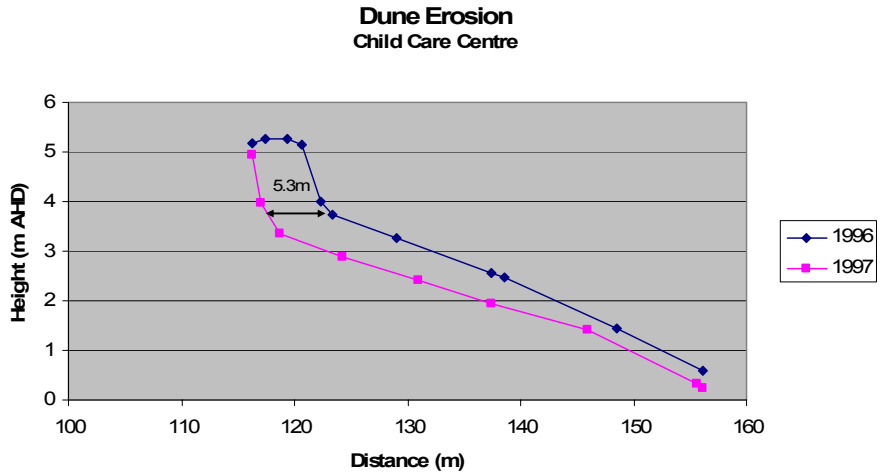
The wave run-up is estimated by applying the formula of Hanslow and Nielsen (1995) which is based on field measurements on natural beaches in Australia. The proposed formulation is:

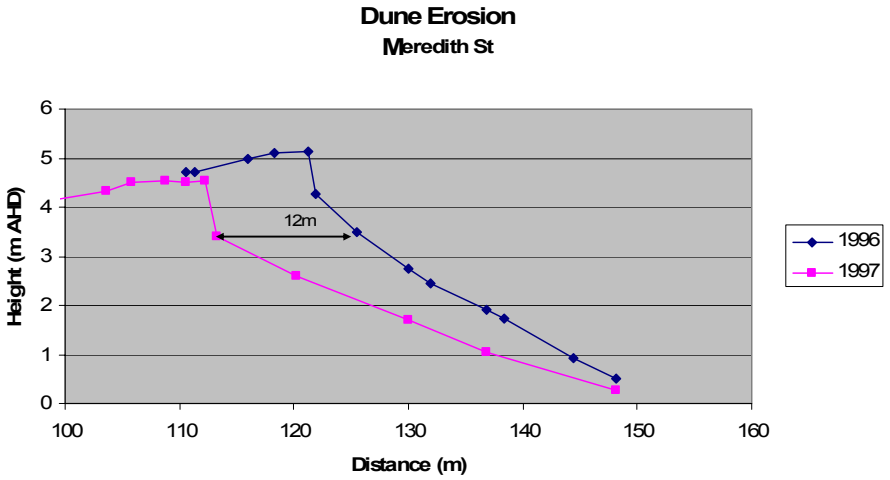
$$R = 0.9 H_s \tan\alpha \sqrt{L_0 / H_s} \qquad \text{Eq. (8-5)}$$

8.2 Calibration of the dune-erosion model

One of key elements to calibrate the dune erosion model is the availability of beach profiles before and after a storm. This information is a key element in the quantification of the volume of sand that is being removed and the understanding of how the dune is retreating during these events. A review of Stockton beach profiles during storms was carried out, and information was found for the May 1997 event. The pre and post beach

profiles are presented in Figure 8-2 . The 1997 profile was measured in the 26/9/97 after the storm.





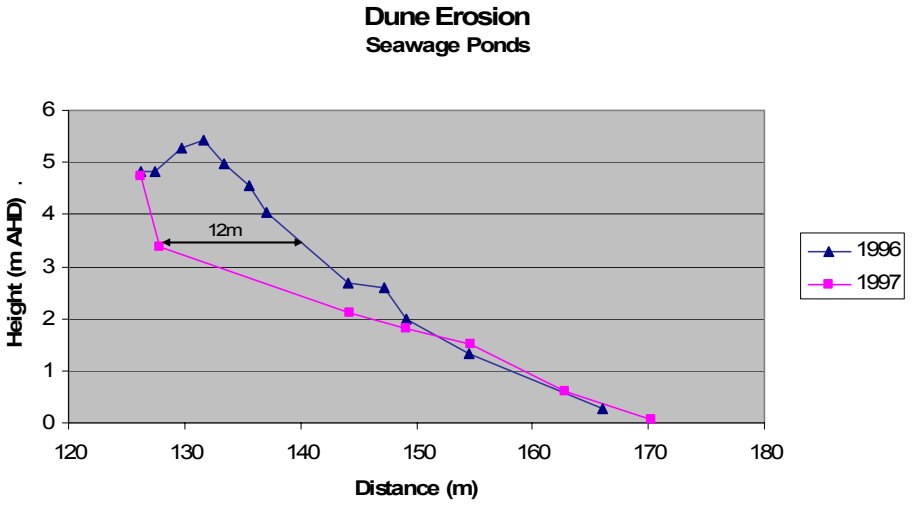


Figure 8-2 Measured dune erosion at three beach profiles for the May 1997 storm.

The geometry of the beach profiles is a key input in the dune erosion analysis but the information provided by the 1996 and 1997 profiles does not provide a detailed description of the beach profile. In order to minimise uncertainty, beach profiles at five locations in the most relevant areas along Stockton were measured from the 21st of March 2006. The profiles were measured at the following locations (from south to north) and used in the calibration procedure:

- Caravan Park;
- Surf Club;
- Monument Area;
- Child Care Centre; and
- Beach area facing Meredith Street.

The profiles are shown in Figure 8-3.

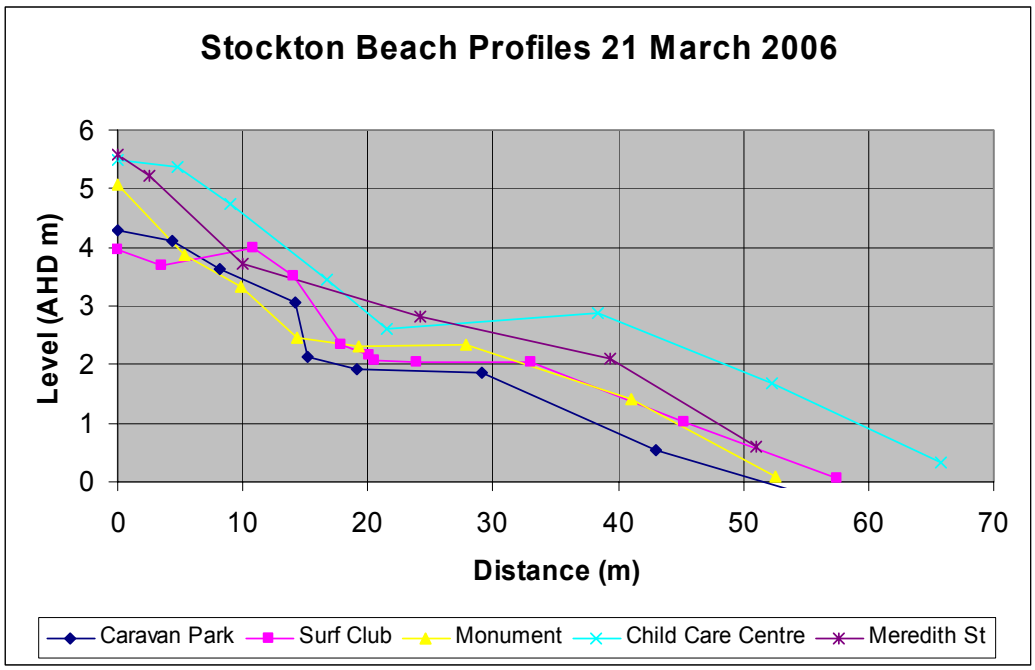


Figure 8-3. Profiles measured on March 21st 2006.

The estimated value of $\tan(\alpha_{re})$ is 0.025, while the definition of the dune height, the dune foot level and the beach width values have been obtained from the profile measurements of March 2006 that are shown in Figure 8-3. The adopted values are presented in Table 8-1.

Table 8-1 Adopted values of the dune height, dune foot height and beach width

Location	Dune height (m AHD)	Dune foot height (m AHD)	Beach width (m)
Caravan Park	4.3	2.1	39
Surf Club	4	2.2	37
Monument	5	2.5	38
Child Care Centre	5.4	2.6	45

Meredith St	5.6	2.8	41
Sewage Treatment Ponds	5.6	2.8	41

Other relevant input to the model are the wave conditions offshore of each profile, therefore wave conditions have been transformed from the Sydney waverider buoy to 5m (AHD) water depth offshore at the locations shown in the table below.

Table 8-2 Location of the wave transformed data at 5m (AHD) water depth (MGA 56)

Location	Easting	Northing
Caravan Park	387,056	6,357,805
Surf Club	386,869	6,357,925
Monument	386,827	6,358,202
Child Care Centre	387,927	6,359,009
Meredith St	387,037	6,359,336
Sewage Treatment Ponds	387,190	6,359,641

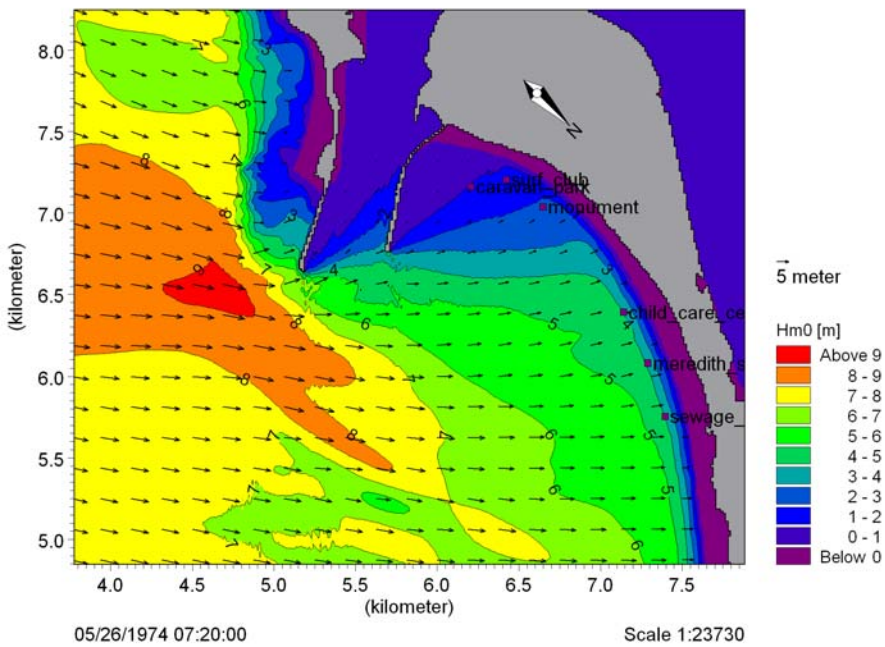


Figure 8-4 Wave field for the May 1974 storm and location of overview of the wave extraction points for the dune erosion analysis.

Before proceeding with the calibration a review of the C_s values was undertaken to estimate the range to be applied in this analysis. Larson et al (2004) derived optimum values of C_s for four data sets by least square fitting the analytical solution to the data. The results of the analysis showed marked scatter in the C_s values. For example, for large wave tank experiments it was found that the mean value of C_s was 1.8 10^{-3} with a standard deviation of 0.89 10^{-3} and about 80% of the data being in the range 1.0 – 2.5 10^{-3}. For small scale laboratory experiments the data showed an estimated optimal C_s of 0.82 10^{-3} and for three experimental sets C_s values of 1.6 10^{-3} and 0.92 10^{-3} were obtained for the first two cases and 1.3 10^{-4} for the last one. The value obtained in this last case is considerably smaller than the values derived in the other data sets. Finally, Larson estimated the overall mean value of the three experiments was 1.4 10^{-3} with a standard deviation of 0.74 10^{-3}, whereas values of the last dataset the mean C_s was found to be 1.7 10^{-4} with a standard deviation of 2.5 10^{-4}.

Based on this information it was decided to establish a range of values of C_s (non-dimensional transport coefficient) as follows:

C_s: $1.0\ 10^{-3}$ - $3.0\ 10^{-3}$

The range of empirical constant f the run-up model was as for defined as:

γ_r: 0.5 -0.7

These two values cover the van der Meer and Hanslow and Nielsen formulations.

Based on these values a calibration of the model was produced. The approach applied is based on the quantification of the short-term beach retreat using the dune erosion model for the May 1997 events and is presented in Figure 8-5. The results are presented for a range of C_s and γ_r values and the calibrated values $C_s = 1.8\ 10^{-3}$ and γ_r=0.6 compared to the measured beach retreat. The comparisons were not carried out south of the Mitchell St seawall because no data was available. The erosion rates were also predicted at these two locations and are presented in Table 8-3.

As it can be observed the predicted and the measured values show good agreement however the beach retreat is slightly over predicted at the Child Care Centre. This has been decided to be slightly conservative due to the uncertainty in the data and the fact that the erosion rates and beach retreat are related to an idealised dune erosion geometry that in reality are more complex as displayed in Figure 8-2.

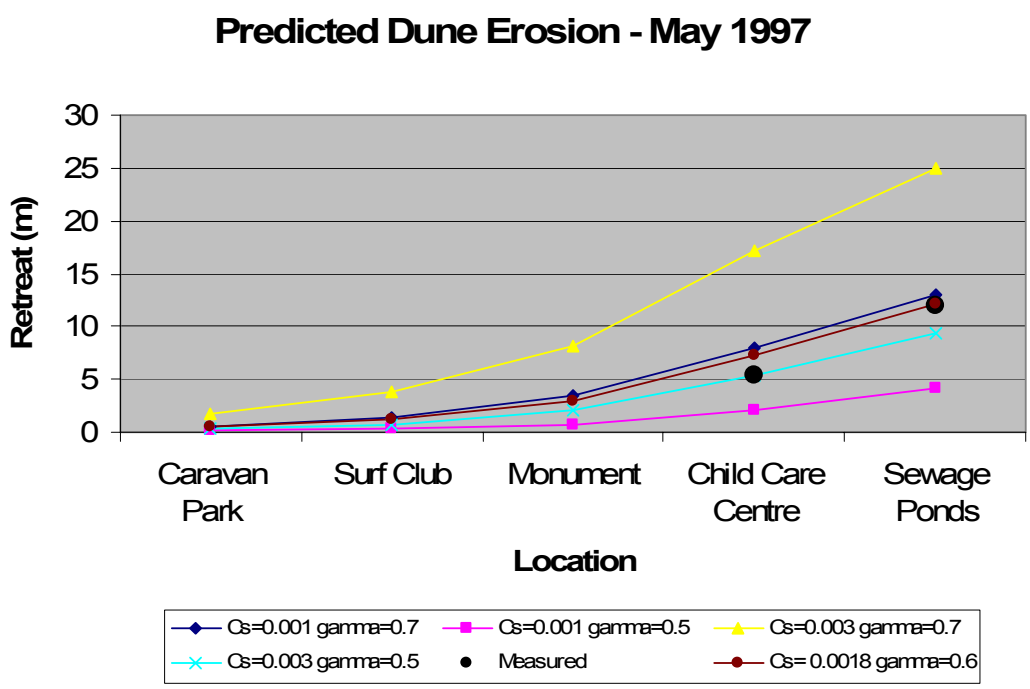


Figure 8-5 Predicted short term retreat for the May 1997 short-term event.

Table 8-3 Measured and predicted erosion rates and erosion distance at Stockton Beach.

Location	Measured		Predicted	
	Erosion rate (m3/m)	Beach retreat (m)	Erosion rate (m3/m)	Beach retreat (m)
Child Care Centre	20	5.3	20	7.25
Sewage Ponds	28	12	32	12.1

8.3 Dune erosion predictions

All the extreme storm conditions shown in Table 8-4 have been modelled and are presented in Figure 8-6 and Figure 8-7. The 1974 events have been analysed independently due to the severity of these two events and their proximity.

Table 8-4 Overview of extreme storm events from May 1974 till June 2004.

Storm Date	Peak Ocean Level (AHD)	Hs (m)	MWD (deg true N)	Duration (hours)
May 1974	1.47	9.2	155*	75
June 1974	1.09	6.7	170*	217.5
Sept 1995	0.86	6.3	165	48
May 1997	1.21	9.9	151	78
July 1999	1.18	6.1	117	66
June-July 2000	1.26	6.1	177	54
July 2001	0.93	6.7	167	36
Jun-Jul 2002	0.81	6.0	174	48

*Directions were estimated based on personal discussions with Mark Kulmar, Dept. of Commerce, Manly Hydraulics Laboratory.

For each event, water levels at the Newcastle Pilot Station were provided. For the 1974 events water levels at the Fort Denison station were extracted from the report presented by DNR.

8.3.1 1974 Storms

The 1974 events occurred in May and June and caused widespread damage to coastal structures and beaches along the coast of NSW. The May storm was induced by a low pressure system cantered SE off the NSW coast which intensified on the 25th of May and peaked in Sydney on May 26th. Waves up to 9.2m were hindcasted offshore Sydney's coast, with rough seas continuing until May 29th. The duration of the storm was approximately three days. During the storm, maximum water levels of 1.47m Standard Datum (2.37m ISLW) were measured at Fort Denison, Sydney, which is the maximum recorded water level. At the peak of the storm, the total residual water level was estimated as 0.47m; it is estimated that 0.24m was provided by the barometric component of the storm and 0.23m by the wind forcing. Following Lord and Kulman's interpretation (2000), this event would be assigned a return interval of 50 years, however the peculiarity is the coincidence with the highest recorded water levels along the NSW coast and this has severe consequences for the erosion of dunes.

The June storm, occurred between the 3rd and 16th of June when a low pressure system developed in the Tasman Sea, creating high wave conditions from S and SSE. The system was very stable, and moved slowly to the east creating a very prolonged period of rough seas and high water levels. Waves exceeding 3m were measured from the 3rd till the 16th of June.

A dune erosion prediction has been carried out for the 1974 storms applying the following calibration coefficients, namely: $C_s = 1.8\ 10^{-3}$ and $\gamma_r = 0.6$.

For this analysis, the repetitive effect of storms was investigated. This is a very relevant issue as storms are not independent of each other and dune retreat produced by one storm does not return to its original position, unless it is done artificially. Therefore, the initial position of the dune foot for the following storm is the final position of the dune after the initial event. This is, in fact, what occurred in 1974 where the two storms occurred at close interval to each other.

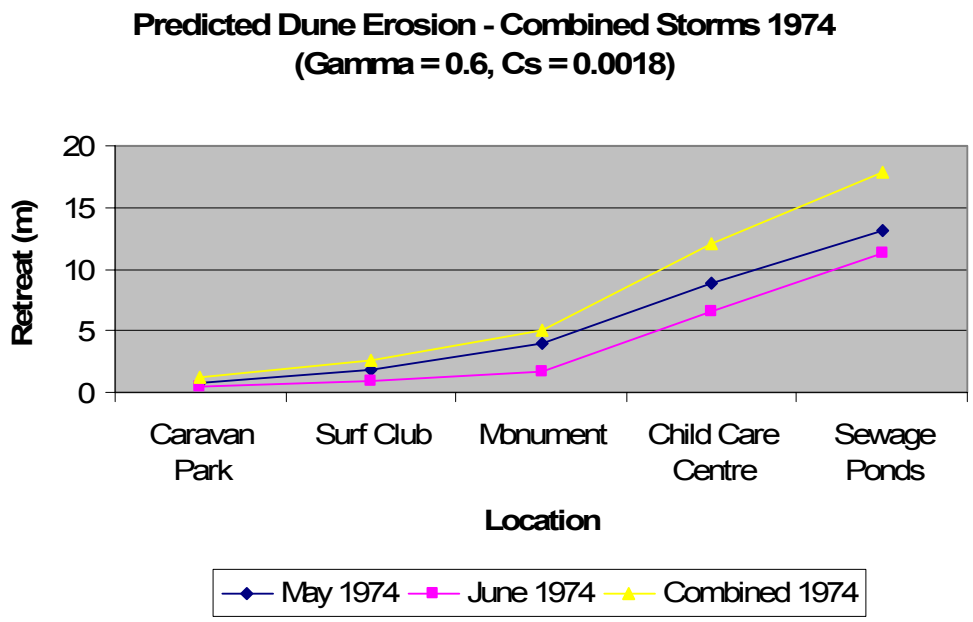


Figure 8-6 Beach retreat prediction of the beach profiles for the May and June 1974 storm events. Gamma: 0.6, Cs: 1.18 10-3

Figure 8-6 shows the beach retreat predictions for the May and June events both independent of each other and also when combined. As it can be observed, the combined effect is larger than each of the individual events, but smaller if the two events are considered separately. This is physically sound because, as the beach retreats, the beach slope decreases, and the capacity of erosion of the waves diminish.

The repetition of events is a very important element when analysing short term beach retreat because during severe storms the majority of sand removed from the berm and frontal dune is transported to the nearshore zone forming offshore storm bars. A significant proportion of the sand is the nearshore area will be returned to the beach within a period of weeks or months during calmer conditions. Onshore winds provide the natural mechanism for the re-establishment of the incipient and frontal dunes. Sand will be blown from the berm against the erosion escarpment and with the aid of

vegetation to trap sand the frontal dune will be gradually rebuilt. These processes may take several months or years.

8.3.2 *All extreme storm events*

All the extreme cases have been simulated and the results included in Figure 8-7. The results show an increase of the beach retreat from south to north. This is expected because the southern areas of the beach are less exposed to the predominant SE to SSE waves. The sewage treatment pond area is clearly the most exposed. This behaviour is very similar for most cases; however the July 1999 event shows a different pattern. The main reason for this can be found in the direction of wave propagation, which in this case is from the E to ESE. This direction of wave propagation is where most of Stockton Beach is exposed.

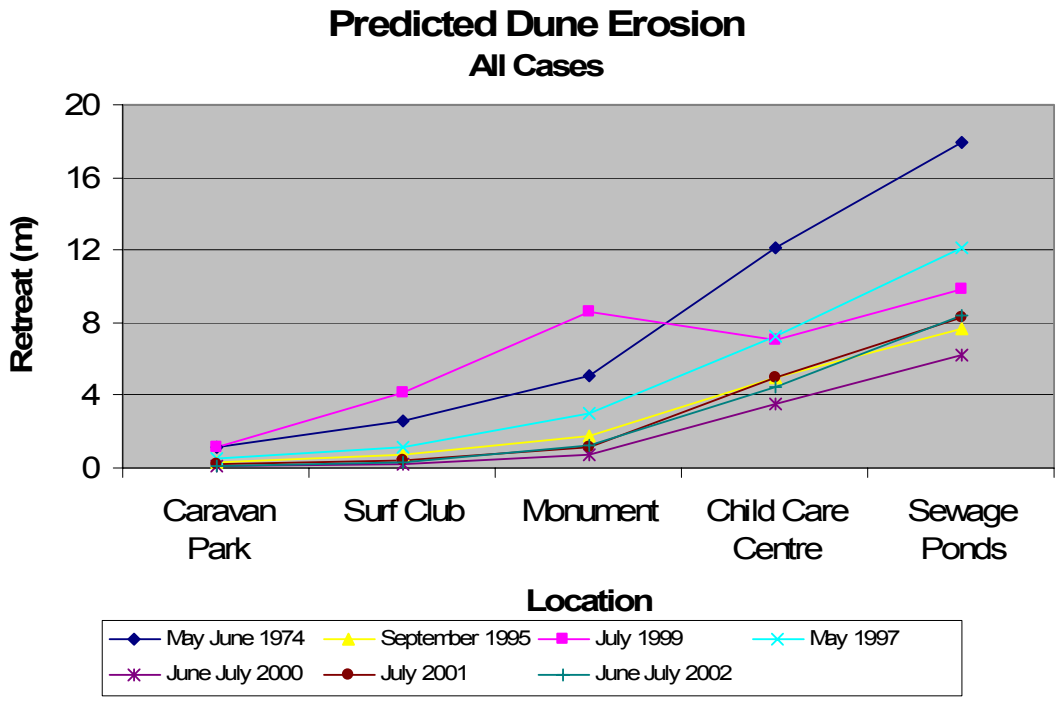


Figure 8-7 Computed retreat from escarpment line for all the severe storm events.

The results show that the largest erosion occurs during the events of May-June 1974 and July 1999. The 1974 event is south easterly therefore the largest rate of erosion is observed at the northern end of the study area, which increases substantially from S to N. The 1999 event shows a different pattern, where less erosion is predicted at the northern end and the erosion estimations are quite uniform, except for the Caravan Park that still is partially protected.

The maximum predicted dune retreat for the analysis period is presented in Table 8-5.

Table 8-5 Predicted maximum dune retreat for Stockton Beach.

Location	Stockton Tourist Park	Stockton Surf Club	Hereford St	Child Care Centre	Meredith St
Erosion (m)	1.2	4.1	8.6	12.1	17.0

Location	Sewage Ponds	Fort Stockton	Fort Wallace	Stockton Centre	Council Boundary
Erosion (m)	17.9	21.9	22.4	23.8	24.5

The model predictions show that large short-term erosion is possible in the Stockton Area. This is in agreement with measurements and anecdotic information of several dramatic erosion situations observed in the Stockton Beach area. Four key elements play a relevant role on dune erosion for this particular event, namely: the wave conditions, the water levels, duration of the storms and the initial geometry of the dune.

The predicted dune erosion model is based on a wave run-up that is a combination of long and short waves. Only short waves are treated in the wave model and the short wave height is used to represent the effect of both waves. The long waves may diffract more effectively into the lee area of the northern breakwater that the short waves, and therefore the reduction in the southern area may be less dramatic than obtained from this analysis.

A further refinement of the results requires information on the beach movement before and after an event, as well as mean ocean conditions (waves, water levels, etc). Anecdotic information from witnesses of any of these events could be applied to further improve the model results. This analysis has been carried out for pure dune erosion processes however a number of processes such as rip currents, localised scour around the Mitchell St seawall, etc can exacerbate the erosion. This is the case for the 1999 event when the area immediately adjacent to the northern breakwater and south of the seawall suffered localised erosion. For these processes an additional retreat value should be included for the estimation of the retreat lines.

8.4 Seabed Variations Analysis

The analysis of the ocean seabed in the nearshore areas offshore of Stockton Beach has shown a persistent deepening whereby large amounts of sand loss have been observed. This process has raised concern over the possible increase of the risk of short term erosion during stormy periods as stated in the Umwelt report (2002). To investigate this issue, an analysis has been undertaken. The approach applied to evaluate and quantify the influence of the nearshore bathymetric variations on the short-term beach fluctuations is based on the application of the dune erosion model. Nearshore seabed variations have also been included in the analysis.

One of the key inputs in this analysis are the wave conditions offshore from the study areas, therefore it is necessary to apply a wave model to describe the nearshore wave field. To achieve this, the numerical model MIKE21 PMS- described in the previous section has been applied, and this model allowed for the inclusion of seabed variations.

The definition of the seabed variations was based on historical bathymetric information. The extension of the area was defined based on the review of beach profiles and historical data carried out in Section 6 of this report which was based on information provided by this and previous studies. The main idea behind this approach is to carry out an analysis of the short term fluctuations of the dune due the modification of the nearshore and offshore areas. This analysis has been designed to provide a general overview of the influence of variations of the seabed on short term erosive events. The approach however, it is not intended at analysing small scale morphological features that may have originated from localised coastal processes.

To carry out this analysis a number of study cases have been predefined as follows:

- **2002 Condition**. Represent conditions on 2002. This is defined as the reference case;
- **Case 1** Water depth increase with respect to 2002 conditions. Maximum variation 1m, average 0.42m;
- **Case 2** Water depth reduction with respect to 2002 conditions, maximum variation 1m, average 0.42m;
- **Case 3** Water depth reduction with respect to 2002 conditions, maximum variation 2m, average 0.85m; and
- **Case 4** Water depth reduction with respect to 2002 conditions, maximum variation 4m, average 1.7m.

To provide a better overview of the seabed variations included in the model, Figure 8-8 shows an overview of the 2002 bathymetry (top left) which is considered a reference case, the proposed modifications for Case 4 (top right) and the modified bathymetry for Case 4 (below). In order to provide a better overview, the extracted profile for the 2002 conditions and the four other cases are presented in Figure 8-9. As it can be observed, while Case 1 has been defined to evaluate further deepening, the other three cases represent shallower conditions. As it can be seen, Case 4 is the case where the most significant decrease of the water depths has been introduced. This case represents a situation where shallow offshore banks are present, as observed in old charts of the area. Cases 2 and 3 represent intermediate previous situations.

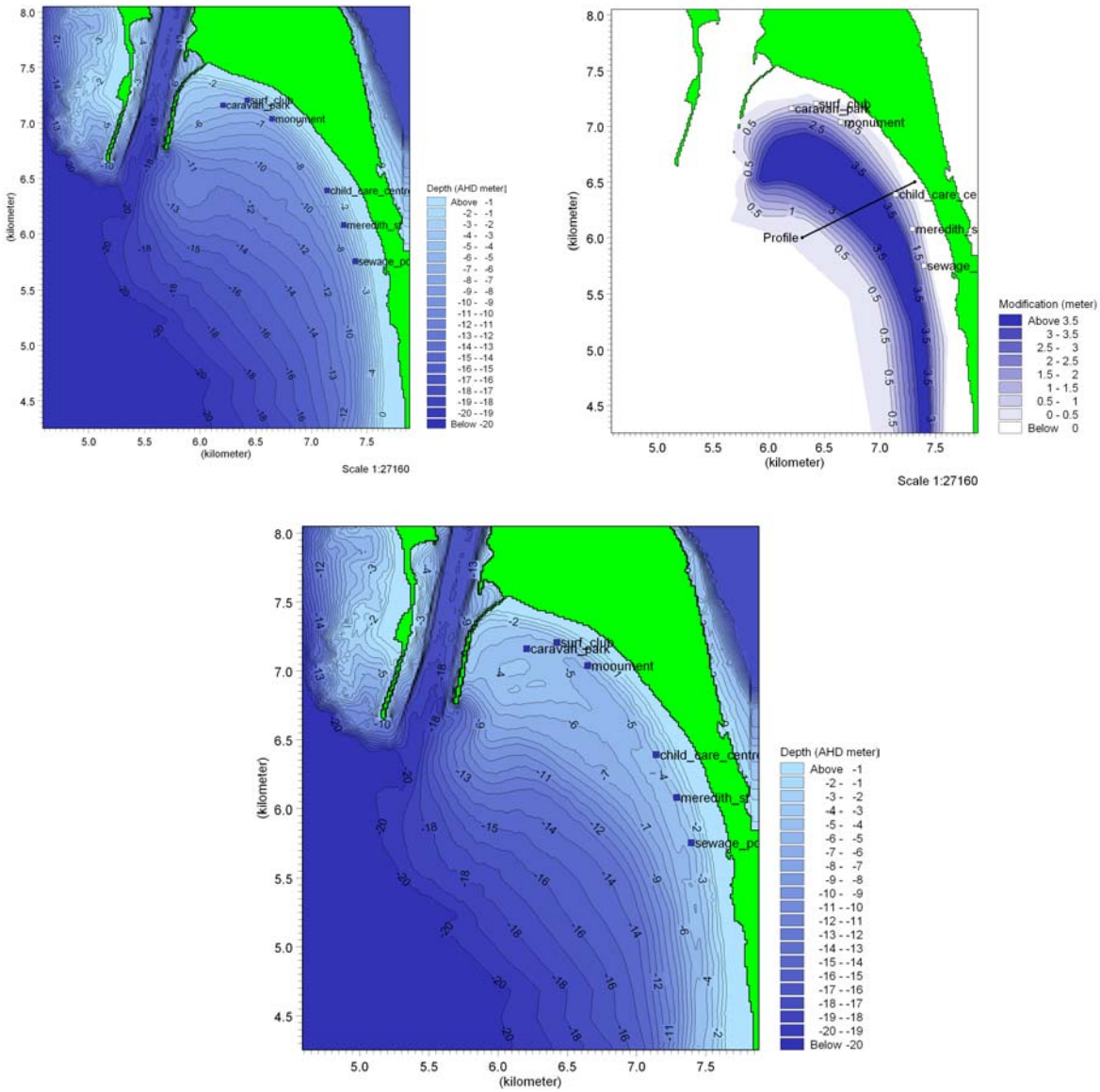


Figure 8-8 Bathymetry 2002 (above left), modifications for Case 4 (above right) and modified bathymetry for Case 4

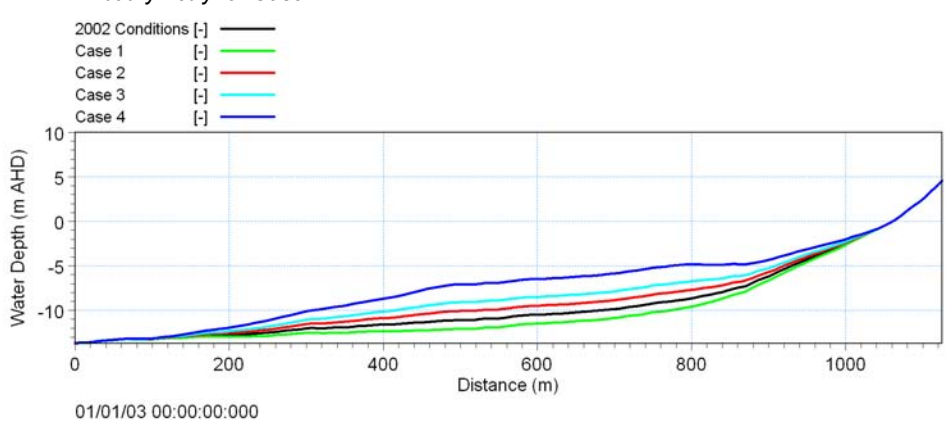


Figure 8-9 Overview of the beach profiles as extracted from the model bathymetries as shown in the modifications file above

As already presented in the dune erosion analysis, two of the most severe ocean events in the NSW coast occurred on May 1974 and July 1999, therefore these two events have been selected for this analysis. These storms represent severe conditions because large wave heights and high water levels persist over a long period of time, and these are the most relevant factors that affect dune erosion. These events also provide a good representation of extreme conditions for waves propagating from SE and E directions.

To perform this analysis, it was necessary to transform the waves offshore from the Sydney waverider buoy to an area offshore Stockton and then apply a local model to estimate the wave conditions in the nearshore area. The numerical models MIKE 21 SW and PMS models were applied with the first one provided boundary conditions to the latter model. The modifications of the nearshore areas were included in the bathymetric input of the nearshore PMS model.

Results of the wave model predictions for all cases for one particular time step of the May 1974 and July 1999 events are presented in Figure 8-10 and Figure 8-11. The model results show the predicted wave field at the Stockton area for the 2002 and the four study cases. As it can be observed, the southern area of the beach is protected from the incoming waves by the northern breakwater generating a shadow area. The extension varies depending on the orientation of the incoming waves, but this should be expected as this region is protected from most incoming waves. As an example of this mechanism, during the 1974 event 9m waves offshore were predicted offshore, with wave conditions at the caravan park, the surf club and the monument area are between 1 and 2.5m.

Small waves in relatively deep areas behave like deep water waves therefore large variations on the seabed are required to induce significant modifications of the wave conditions. The situation is rather different for Cases 3 and 4, where the changes are so dramatic that wave dissipation occurs, extending the area with small waves further north and providing additional protection from the incoming waves.

The variations of the wave conditions on the more exposed northern areas are clearly more significant for the analysed cases. While for Case 1 waves are allowed to propagate and break further onshore, Cases 2, 3 and 4 demonstrate the opposite effect and also a more extended dissipation area compared to the 2002 conditions and Case 1.

Another feature that is characteristic of this area is the focussing of wave energy for waves propagating from the S-SE, as observed for the 1974 event. This phenomenon generates an irregular wave pattern in the northern areas of Stockton Beach.

It is observed that for the 1999 event the wave field is more regular in the Stockton area; this should be expected because the waves approach the beach nearly perpendicular to the coast, which allows for maximum wave penetration as there is minimum dissipation due to refraction and diffraction.

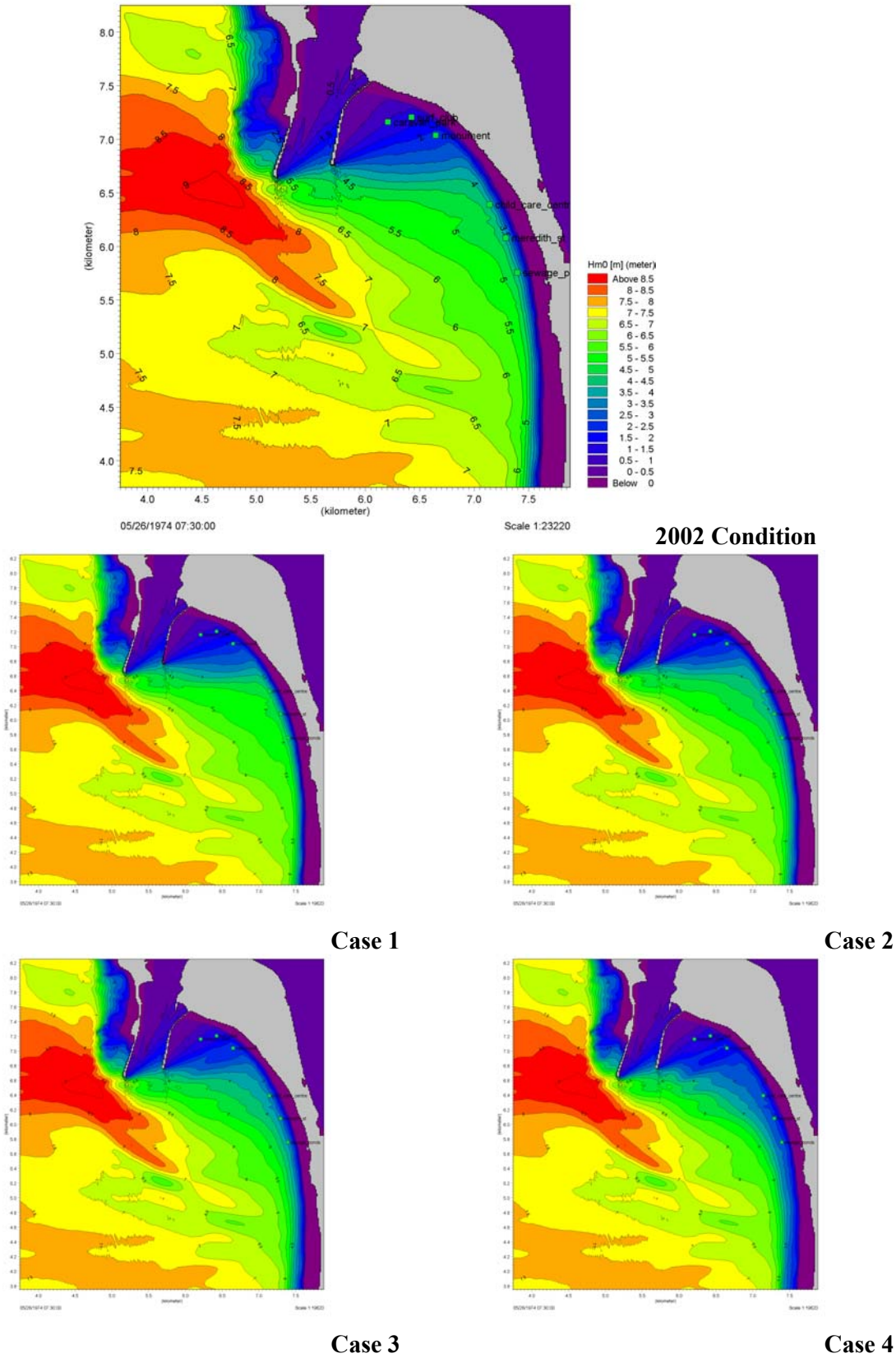


Figure 8-10 Overview of model results for the May 1974 storm for the 2002 conditions and the four study cases.

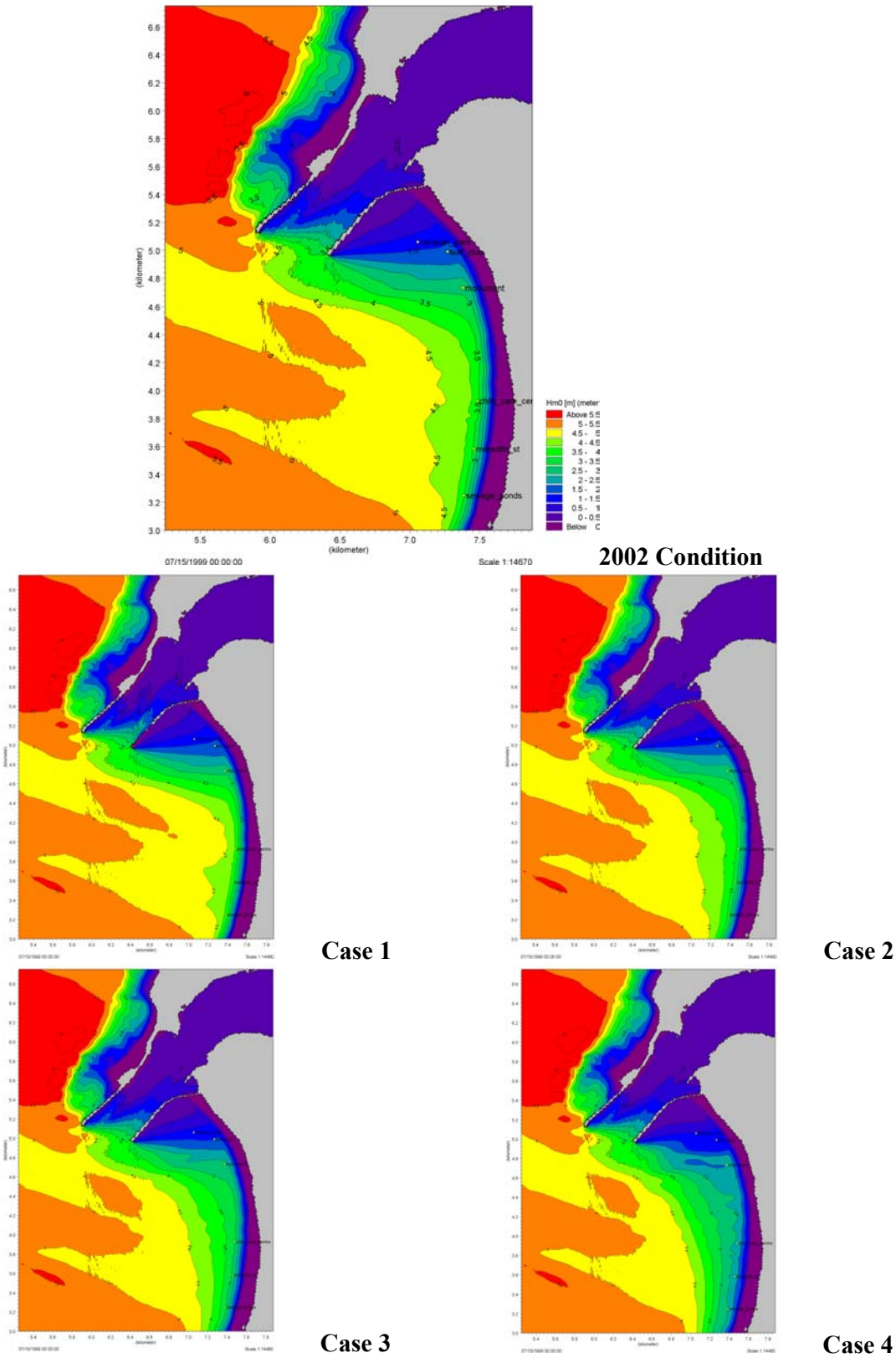


Figure 8-11 Overview of model results for the Jul 1999 storm for the 2002 conditions and the four study cases.

Based on the wave predictions, dune erosion has been predicted for the May 1974 and July 1999 storm events. The predictions are based on the approach described in Section 8.1 which were carried out for the 2002 reference case and for Cases 1, 2, 3 and 4. The results of the simulations are presented in Table 8-6 and Table 8-7.

Table 8-6 Predicted dune erosion at the different analysed locations and the variations (%) for each of the study cases (red: increased erosion, blue: reduced erosion).

Storm Event	Location	Dune Erosion (m)	Variation (%)			
		2002	Case 1	Case 2	Case 3	Case 4
July 1999	Caravan Park	**3.6**	2.8%	-2.8%	-11.1%	-36.1%
	Surf Club	**10.5**	2.9%	-6.7%	-15.2%	-40.0%
	Monument Area	**18.9**	0.0%	-2.1%	-7.4%	-28.0%
	Child Care Centre	**17.0**	6.5%	-7.6%	-17.1%	-40.6%
	Sewage Ponds	**21.6**	4.2%	-4.6%	-11.6%	-30.1%
Average			**3.3%**	**-4.8%**	**-12.5%**	**-35.0%**

The predictions of the 1999 event show a clear pattern, with a general short term erosion increase for Case 1 (deepening) and decrease for Cases 2, 3 and 4. In general the variation is relatively constant for each event, however as expected; there are some variations due to the irregularity of the bathymetry. Clearly the predicted behaviour is fairly uniform for each case and this can be attributed to the way that waves approach the beach being nearly perpendicular to the coast. The general pattern for this event clearly indicates that a further deepening of the seabed leads to an increase of the risk of erosion. The average increase for Case 1 is 3.3%. Cases 2, 3 and 4 show a decrease of erosion, with average decreasing values of 4.8, 12.5 and 35% respectively. The reduction is clearly non linear and this can be related to the wave dissipation processes. A reduction of the water depth will tend to produce additional dissipation of the incoming wave energy. Eventually, if the approaching area is shallow enough, breaking will occur and this process is highly non -linear.

Predictions for the May 1974 event have also been undertaken and are presented in Table 8-7. The results show a more complex pattern than the one observed for the 1999 event. It should be remarked that the main difference between these two cases is the angle of wave approach, which in this case is from the SE.

The results indicate that for Case 1 there is a decrease of the predicted dune erosion at the southern areas (Caravan Park, the Surf Club and the Monument Area). This variation can be attributed to the deepening of the nearshore area that reduces the capacity of the incoming waves to refract towards the area behind the breakwater. As a result of this mechanism, less wave energy is able to reach the lee area behind the northern breakwater, which, in turn, produces a reduction in dune erosion predictions. The behaviour is opposite on the northern areas where a clear increase of the dune erosion is predicted.

Conversely, the predictions for Cases 2, 3 and 4 show the opposite behaviour. Increased erosion is predicted in the southern areas and decreased erosion in the northern region. The northern stretch of the beach is exposed to the incoming waves therefore a reduction of the water depth in the nearshore areas generates additional wave

dissipation, reducing the dune erosion predictions. The effect is opposite in the southern end of the beach where an increase in the erosion pattern is predicted and this can be attributed to the fact that the nearshore profile is shallower and the waves are able to refract more efficiently behind the breakwater.

Table 8-7 Predicted dune erosion at the different analysed locations and the variations (in percentage) for each of the case study (red: increased erosion, blue: reduced erosion).

Storm Event	Location	Dune Erosion (m)	Variation (%)			
		2002	Case 1	Case 2	Case 3	Case 4
May 1974	Caravan Park	**2.6**	-3.8%	3.8%	7.7%	3.8%
	Surf Club	**5.3**	-1.9%	3.8%	3.8%	-13.2%
	Monument Area	**10.1**	-4.0%	5.0%	10.9%	16.8%
	Child Care Centre	**19.8**	5.6%	-7.6%	-16.2%	-36.4%
	Sewage Ponds	**26.6**	5.6%	-6.8%	-14.7%	-33.5%
Average South			**-3.2%**	**4.2%**	**7.5%**	**N/A**
Average North			**5.6%**	**-7.2%**	**-15.5%**	**-35.0**

In order to provide a better overview of the waves approaching the study area, wave heights have been extracted along a line including the study areas. A description of the extraction line is presented in Figure 8-12

As expected the results show a decrease of the wave heights in the southern areas and an increase in the northern stretch of beach. As the southern area deepens the waves reduce their capacity to be refracted into the lee area of the breakwater; therefore less wave energy is able to reach this region. However, this wave energy will instead be able to travel and reach the northern stretch of the beach thus increasing the wave action in this region.

The modifications are even more pronounced for Case 4 where the largest variations are introduced into the bathymetry. The results show a significant and localised modification of the wave pattern, with large focussing in some regions and large reductions in others as observed in the stretch between the surf club and the monument area. This modification of the wave field is produced by the large seabed variations that can induce wave breaking in shallow areas.

Another example of the modification of the wave climate can be found at the Surf Club for Case 4. In this place Cases 2 and 3 show a slight increase of the wave action and therefore of the dune erosion, however Case 4 shows a substantial reduction. The results indicate that the wave conditions, in this particular case, change substantially and a reduction of erosion is predicted. This should not be considered as a general trend; however it provides a good indication of the complexity of the wave field when variations in the nearshore areas occur. However, as stated earlier in this section, this analysis has been designed to provide a general overview of the influence of the variations of the seabed on the short term erosive events. It is not however intended at analysing small scale morphological features that may have originated from localised coastal processes, and this is the case for this particular case.

To summarise, the results show that if the 1974 event is to occur again with the present conditions it is likely that the dune erosion rates would increase between 15 to 35%

compared to cases 3 and 4 (shallower offshore area). If no corrective measures are taken the nearshore deepening process is likely to continue and the dune erosion risk will further increase 5% as shown in case 1 (that is defined to describe possible future nearshore deepening).

Umwelt (2002) based on the analysis of the subaerial beach erosion showed a similar pattern; however their results indicated 200 to 250% increase in the last decade and since the 1950's 500 to 600% increase. The present study indicates significantly smaller values. It is not possible to make a direct comparison to the results of Umwelt since not all modelling details are presented in their report; however the schematised severe storm applied in their analysis were based on a peak significant wave height of 9.1 metres. In the present study similar wave conditions were applied offshore and then transformed nearshore. During this process wave heights reduce significantly as they shoal, refract, break, diffract, etc. Due to these processes the waves and consequently the dune erosion predictions are less affected by the seabed changes in the nearshore areas.

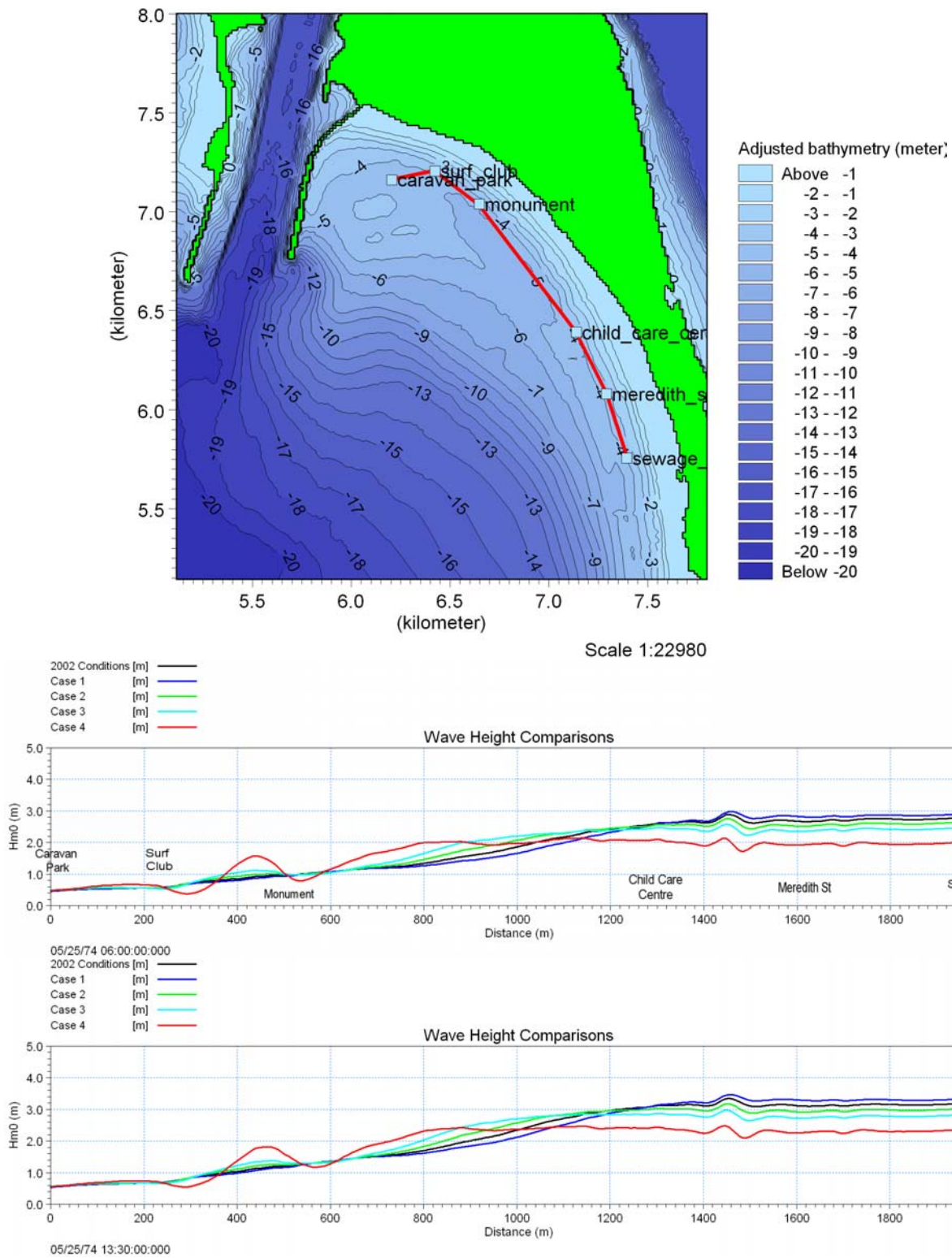


Figure 8-12 Location of the extraction line and the predicted wave heights along the extraction line for two time steps during the 1974 event. The 2002 conditions and the four cases are presented simultaneously.

9 GREENHOUSE EFFECTS

During the 20th century there has been a substantial increase in greenhouse gas concentrations as well as a rise in global temperatures. The Third Assessment Report of the Intergovernmental Panel on Climate Change (IPCC TAR, 2001) indicates that most of the warming observed during the last 50 years is attributable to human activity. Further global warming, sea-level rise and other climatic changes are likely to occur during the 21st century.

The complexity of processes in the climatic system means that it is inappropriate to simply extrapolate past trends in order to forecast future conditions. To estimate future climate change, scientists have instead developed possible "what if" scenarios. Two processes are likely to occur due to green house effects:

- Sea level rise; and
- Variation of the climate conditions.

9.1 Sea level Rise

The TAR (2001) reported that statistically significant associations between increases in regional temperatures and observed changes in physical and biological systems have been documented in freshwater, marine and terrestrial environments on most continents around the world. Surface and satellite-based observations support these findings

Projected warming in the 21st century is dependent on future emissions of greenhouse gases and aerosols. Using the Special Report on Emissions Scenarios (SRES), global average warming projections are expected to range from 1.4 to 5.8 ºC by 2100 relative to temperatures recorded in 1990. These scenarios were regarded as 'plausible' by the IPCC, but not assigned any probabilities.

TAR projections of global average sea level rise range from 9 to 88 cm by 2100, with thermal expansion of sea water contributing to half of this rise and one quarter from the melting of glaciers. A small positive contribution from the Greenland ice melt is expected as well as the possibility of a negative contribution from snow accumulation over Antarctica. The contribution from Antarctica is however especially uncertain, with recent events on the Antarctic Peninsula raising the possibility of an earlier positive contribution from the West Antarctic Ice Sheet (WAIS).

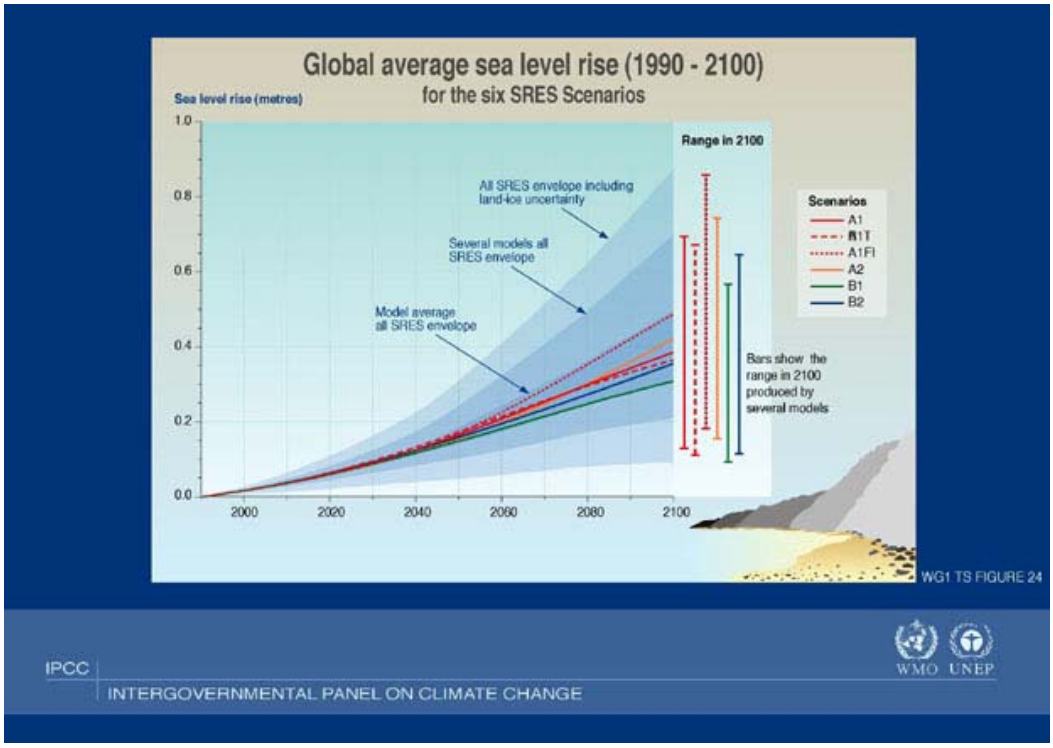


Figure 9-1 Projected sea level changes in the 21st century. IPCC (2001).

Based on these scenarios global-average temperatures and sea-level rise has been predicted and are presented in Figure 9-1.

Table 9-1 Predicted sea level rise due to greenhouse effects in the 21 century (in cm).

Year	Low	Mid	High
2006	1	2.5	4
2026	2	7	14
2040	3	12	23
2050	5	18	32
2056	5	20	36
2070	7	30	52
2100	9	48	88

The estimated relative sea level trends for tide gauge locations around Australia which have at least 25 years of hourly data on the National Tidal Facility archive are shown in Table 9-2. The overall Australian average sea level rise of 0.30 mm per year is substantially lower that the global estimates of IPCC (2001) of 1-2 mm per year over the last 100 years, however the data shows a considerable variation between sites, driven by combinations of the factors. A good example of this regional variation is the sea level fall of 0.19 mm per year at Port Pirie compared to the >2 mm per year sea level rise at nearby Adelaide. The estimated sea level rise in Newcastle is 1.18mm/year.

Table 9-2 The estimated relative sea level trends for tide gauge locations around Australia which have at least 25 years of hourly data on the National Tidal Facility archive. The overall average relative sea level trend of the above list is +0.30 mm per year. Mitchell et al (2000). Obtained from http://www.ozestuaries.org/indicators/sea_level_rise.jsp

Location	**Years Of Data**	**Estimated Trend (mm per year)**
Darwin	34.9	-0.02
Wyndham	26.4	-0.59
Port Headland	27.7	-1.32
Carnarvon	23.9	+0.24
Geraldton	31.5	-0.95
Fremantle	90.6	+1.38
Bunbury	30.2	+0.04
Albany	31.2	-0.86
Esperence	31.2	-0.45
Thevenard	31.0	+0.02
Port Lincoln	32.3	+0.63
Port Pirie	63.2	-0.19
Port Adelaide - inner	41.0	+2.06
Port Adelaide - outer	55.1	+2.08
Victor Harbour	30.8	+0.47
Hobart	29.3	+0.58
Georgetown	28.8	+0.30
Williamstown	31.8	+0.26
Geelong	25.0	+0.97
Point Lonsdale	34.4	-0.63
Fort Denison	81.8	+0.86
Newcastle	31.6	+1.18
Bundaberg	30.2	-0.03
Mackay	24.3	+1.24
Townsville	38.3	+1.12
Cairns	23.6	-0.02

9.2 Variation of the climate conditions

The New South Wales (NSW) Greenhouse Office assessed future change of the NSW climate (Hennessy et al, 2004). Results focused on regionally specific changes such as droughts, extreme temperatures, heavy rainfall, strong winds, extreme weather systems and storm tides.

The study found that average wind-speed projections show a tendency for increases across much of the state. These changes were found to vary not only for the different seasons but also for different regions across the State. The study also projected changes in extreme monthly winds (strongest 5%).

Extreme weather patterns were also investigated and it was found that during the summer half of the year the frequency of Tasman lows that contribute to extreme wind days decreased significantly (from 19% at present to 9% by 2070). The number of cold

fronts increased slightly by 2070 and the number of unclassified systems increased by 2%. In the winter half year, Tasman lows contributing to extreme winds increased in frequency from 26% at present to 31% by 2070. Frontal systems also increased from 25% of extreme wind days at present to 29% by 2070.

The storm systems most conducive to elevated sea levels, through both storm surge and high wave activity along the NSW coast, are slow-moving, low-pressure systems such as cut-off lows or lows of tropical origin that travel south along the New South Wales coast.

Cut-off lows contributing to the top 1% of wind speeds showed small increases in frequency in both the summer and winter halves of the year. While the cause of the increase cannot be determined precisely in the study, the spatial resolution of wind speed change in the model indicates that these systems may exacerbate extreme winds and sea levels along the southern half of the NSW coast.

The changes in frequencies of other patterns suggest a shift of the wave climate with waves form the southeast becoming more prevalent and waves from the northeast and east becoming less prevalent. In summer this change comes about from the reduction in Tasman highs coupled with an increase in intense frontal systems. In winter, there is a smaller reduction in Tasman highs but larger increases in the number of intense winter fronts and Tasman lows. Based on the spatial pattern of change of extreme wind in the future, the increase in waves originating from the south would be expected to have a greater impact on the southern half of NSW.

Therefore it can be concluded that there should be expected variations of the wind conditions and extreme weather patterns in the future due to global warming. This is an important factor to take into consideration when analyzing nearshore processes, conditions and weather patterns.

9.3 Stronger new evidence and the future

Since the release of the Third Assessment Report of the Intergovernmental Panel of Climate Change in 2001 much has changed. A succession of unusually warm years has brought climate change to the forefront of public debate. Such climate-related extreme events include the European heatwave of 2003, destructive tropical cyclones such as Hurricane Katrina, and severe droughts and dwindling water resources. A very good review of the new evidence was presented by Steffen (2006) and some of the most relevant issues associated to this project are presented below.

The initial model-based estimates of the degree of warming by the end of this century lie between 1.4 and 5.8°C. In part, the spread in the range of estimates is due to uncertainty related to the nature and strength of processes that could dampen or amplify the initial greenhouse gas forcing. Most of the emphasis in the past has been on feedbacks associated with water vapour and clouds. Over the past few years, however, research has yielded a better understanding of three additional effects that were recognised as being important in the IPCC (2001) but for which little quantitative information was available at the time.

The first of these effects is based on the radiative properties of aerosols; small particles suspended in the atmosphere that generally scatter incoming solar radiation thus cooling the Earth's surface, acting in opposition to greenhouse gases. Estimates of the magnitude of the aerosol cooling effect have now been made, and the estimates are moving towards a higher value than previously thought.

A second effect is associated with a decrease in the reflectivity of the Earth's surface or albedo which is caused by the melting of snow and ice. The most dramatic example of this effect will likely occur in the Arctic Ocean, which during summer is now projected to become almost totally ice-free. Retreating ice and snow expose darker underlying land and ocean surfaces, leading to enhanced absorption of sunlight and further warming.

The third effect relates to terrestrial carbon cycle dynamics that is expected to change significantly through this century, with strong amplifying feedbacks to climate change. Several processes – oxidation of soil organic matter, the number of carbon pools in wetlands and frozen soil- are all sensitive to climate. As temperature rises, these processes in general release further amounts of carbon into the atmosphere, forming a feedback loop that intensifies the warming.

Steffen estimates that although much uncertainty still surrounds the timing, rate and magnitude of these effects, they all operate to amplify the initial greenhouse warming. Thus, there is now perceived to be greater risk that the upper end of the well known IPCC estimate of 1.4 to 5.8 °C temperature rise will be reached or exceeded by 2100.

New estimates of the rate of historical sea level rise confirm rates reported by the IPCC in 2001; a combination of tide gauge and satellite altimeter data shows a global average sea level rise of 195 mm from 1950 to 2004 (Church and White, 2006). From 1950 to 2000, global averaged sea level rise was 1.8± 0.3 mm per year, but during the 1970's and since 1993 the rate of sea level rise has increased to about 3mm per year. The contemporary rise contrasts with the last 6000 to 7000 years, during which sea level has been relatively stable.

The observed sea level rise is due to both thermal expansion of the ocean and the increase of runoff from the melting of glaciers and ice sheets. The contribution of thermal expansion is estimated to be 0.4mm per year or less from the 1950's to 1990's (overall rate of sea level rise of 1.8mm per year) and about 1.8 mm per year after 1993 (overall rate of sea level rise of 3 mm per year). These figures imply significant contributions to sea level rise from the loss of land based ice (Leuliette et al, 2004; Church et al, 2004; Church and White 2006)

A comparison between the IPCC 2001 prediction band and the new measurements (Leuliette, Nerem and Mitchum, 2004) is presented in Figure 9-2.

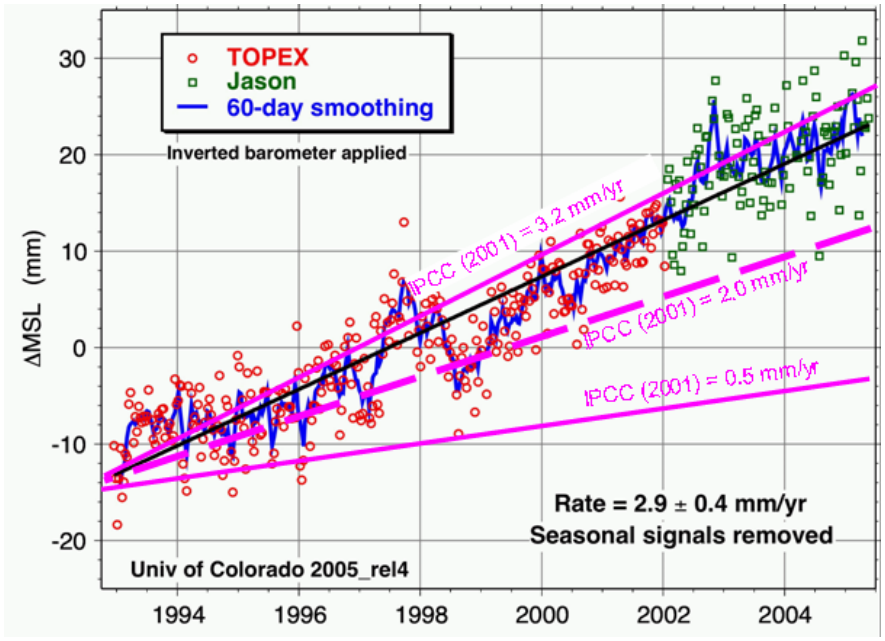


Figure 9-2 Global 10-day mean sea level variations from Topex/Poseidon and Jason no tide gauge calibration applied. A relative bias has been subtracted from the Jason time series. No inverted barometer correction has been applied to the time series, Cowell (2005).

9.4 Predicted Beach Recession due to Greenhouse Effects

There is a range of models which have been formulated to describe the likely shoreline response to sea level rise. The majority predict recession of the shoreline with increasing sea level. The most widely applied model has been developed by Bruun (1962) (1988), and indicates that the volume of sand eroded will be the product of the recession and the dune height above the shoreline. In order to fill the nearshore profile, the profile has to be moved landward by a distance equal to the recession, up to a depth below the shoreline and distance offshore the shoreline to the extent of the littoral sand transport.

The relationship can be expressed as:

$$R = \frac{1}{\tan\theta} S \qquad (9\text{-}1)$$

Where R is the retreat rate, S increase in sea level and $\tan\theta$ is the average slope of the coastal profile. The definition of the active beach area used to define the average value of $\tan\theta$ is a matter of discussion. Most sediment transport modelling relates the transport to the forcing mechanisms such as the orbital velocity beneath the waves. The offshore limit of longshore transport is defined as the area at which sediment transport is negligible. This is an important limit to define the region in which erosion and accretion occur and also to define protection schemes in the beach. It is described as the upper shoreface or the offshore limit of the longshore transport area by Short et al. (1995).

In 1978 Hallemeier (1978) proposed a semi-empirical formula for the determination of the closure depth at which the offshore limit is defined. This analysis was carried out

comparing measurements of beach changes during storms on different coasts. Equation 9-2 shows the relationship of quartz sand (ρ_s= 2.65 g/cm^3)

$$d_l = 2.28\, H_s - 68.5\, \frac{H_s}{T_s^2} \qquad (9\text{-}2)$$

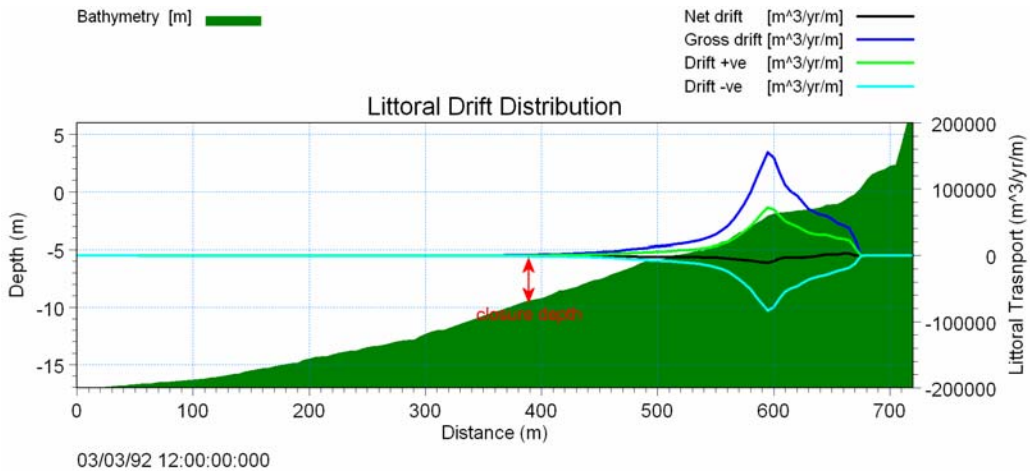


Figure 9-3 Overview of the active profile area, as computed with LITPACK.

Where d_l is the closure depth, H_s is the significant wave height exceeded 12 hours a year and T_s is the corresponding significant wave period. Based on the wave exceedance curve produced in the draft report we can estimate that *Hs*= 5m and *Ts* = 12sec, therefore the predicted closure depth is 9m. This value was very close to the value computed with LITPACK as shown in Figure 9-3 (the plot shows the locations where the longshore transport is negligible).

According to Short et al. (1999) the limiting depth for significant on-offshore transport of sand extends further offshore than the value computed by Equation 9-2. This area is also morphologically active; however, the variation time is much larger than that observed in the upper shoreface. It appears to range from the order of 100 to 1000 years and may be appropriate to apply when calculating sea level rise.

Hallemeier (date) estimated the offshore limit of on-offshore transport as:

$$d_j = (H_s - 0.3\sigma) T_s (g/5000D)^{1/2} \qquad (9\text{-}3)$$

Where H_s is the mean significant wave height, T_s is the mean significant wave period, D is the characteristic grain size and σ is a coefficient that depends on the area of analysis. In the Sydney area in an open beach the offshore limit of the lower shoreface d_j is estimated at 36m. In the Stockton area, however the estimated value of d_j is 20m. This smaller value can be attributed to the port breakwaters which shelter the Stockton area from offshore waves, reducing the mean wave height. This value fits well with the historical beach profile variations at Stockton Beach as shown in Figure 9-4.

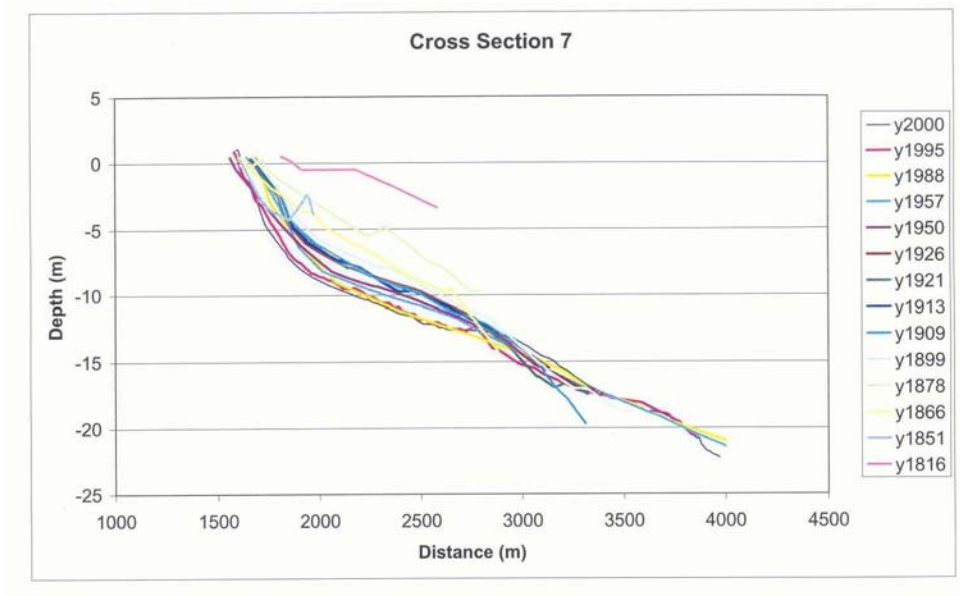


Figure 9-4 Historical variation of beach profiles at Stockton Beach for Umwelt (2003).

Based on the application of the inner closure depth a beach slope of 0.025 was obtained whereas for the offshore limit the slope is 0.011. An intermediate value of 0.0125 was obtained at 15 metres depth. Due to the uncertainties associated with the definition of the beach profile slope the latter value has been applied in the calculations of the shoreline recession due sea level rise. The predicted values based on a $\tan\theta = 0.0125$ and Bruun (1962) (1988) rule are shown in Table 9-3.

Table 9-3 Predicted beach retreat due or shoreline recession due to sea level rise for the next 20 and 50 years at Stockton (in metres).

Year	Low	Mid	High
2026	0.8	3.6	8.0
2056	3.2	14.0	25.6

The estimated weather pattern variations along the NSW coast (Hennessy, 2004) will have implications on the stability of the shoreline. A slight rotation of the wave conditions offshore has been evaluated by using numerical models to transform offshore wave conditions into the nearshore zone. The results indicate that a clockwise rotation of four degrees of the offshore wave conditions induce a rotation of two degrees of the nearshore waves. This process will induce a variation in the net longshore drift and has large implications in the stability of the shoreline. Such an effect could induce a beach rotation process with accreting and erosion areas, similar to what has been observed during interannual fluctuations. This analysis is based on the same weather intensity. In the case that storm intensity is also modified, this will have further implications on the shoreline stability. To estimate shoreline changes associated with these processes it is required to predict the weather patterns as an input to the shoreline model.

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10 DESCRIPTION OF ON-GOING COASTAL PROCESSES

A detailed analysis of the sediment transport conditions at Stockton has been carried out to determine the on-going processes at Stockton Beach. The analysis has been undertaken at a range of time scales including short, medium and long-term. The main findings of Stage I can be summarized as follows:

10.1 Short Term Processes

A detailed analysis of the short-term erosion at Stockton Beach has been undertaken. This analysis was based on the application of a dune erosion model and wave conditions in the nearshore area.

The model was calibrated based on Stockton beach erosion profiles for the May 1997 Storm event. The pre and post beach profiles were analysed and compared to model predictions. A set of coefficients was obtained in this calibration process.

The model was applied to the most severe storm events observed in the Newcastle area. These events combine both large wave conditions and high water levels which are the conditions required for beach erosion. Short-term predictions show an increase in dune erosion risk from south to north for the most frequent events from the south east. For conditions from the E and NE the dune erosion is still severe but occurring more evenly along the beach. The effect of repetition of events was also investigated for the events of May and June 1974, which were the most severe observed on the NSW coast.

The maximum computed dune erosion on Stockton Beach is presented in Table 10-1.

Table 10-1 Predicted maximum dune erosion at Stockton Beach

Location	Stockton Tourist Park	Stockton Surf Club	Hereford St	Child Care Centre	Meredith St
Erosion (m)	1.2	4.1	8.6	12.1	17.0

Location	Sewage Ponds	Fort Stockton	Fort Wallace	Stockton Centre	Council Boundary
Erosion (m)	17.9	21.9	22.4	23.8	24.5

The influence of nearshore deepening on the short term dune erosion was also investigated. The analysis shows that an increase of the dune retreat is predicted with a deepening of the nearshore areas and a decrease for shallower nearshore conditions.

The results indicate that an increase in the dune erosion rate occurs with a deepening of the nearshore areas, therefore if an event as the one in May- June1974 occurs under the present conditions it is likely that the dune erosion rates would increase between 15 to 35%. If no corrective measures are taken the dune erosion risk will further increase 5% for a further deepening of the nearshore areas of 1m. Umwelt (2002) based on the analysis of the subaerial beach erosion showed a similar pattern; however their results indicated 200 to 250% increase in the last decade and since the 1950's 500 to 600%

increase. The present study indicates significant smaller values of dune erosion. It is not possible to make a direct comparison to the Umwelt results since not all modelling details are presented in their report; however it should be mentioned that the schematised severe storm applied in their analysis was based on a significant peak wave of 9.1 metres. In the present study similar wave conditions were applied offshore and then transformed nearshore. During this process wave heights reduce significantly as they shoal, refract, diffract, break etc. Due to these processes the waves and consequently the dune erosion predictions are less affected by the seabed changes in the nearshore areas.

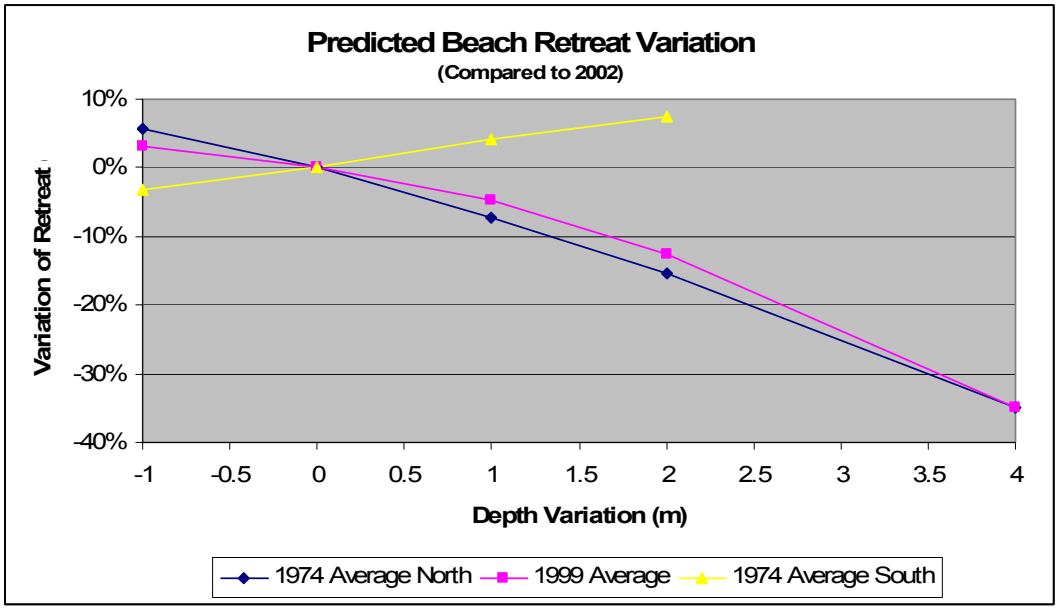


Figure 10-1 Predicted Beach retreat (in percentage) due to a variation of the water depth offshore the study area

10.2 Medium and Long- Term Processes

The medium and long term net littoral drift for the period 1992-2004 was investigated by applying a detailed 2-dimensional modelling approach. The results were compared to historical data between 1816 and 2000 analysed by Umwelt (2003) and longshore calculations by WBM (1998).

Results show that the net transport is a combination of south and northwards events where the first are more episodic and caused by more infrequent NE-E events, while the latter are due to the more frequent SE swell conditions. A large variability is observed in the littoral transport with periods of large northwards transport and other periods where the littoral transport changes direction becoming predominantly southwards.

Medium term effects

The results indicate that there is a medium-term effect in the period 1994-1999. Ranasinghe et al (2004) analysed the processes during this period and described them as beach rotation due to variation of the wave conditions.

Goodwind (2005) investigated the inter- and multidecadal variability in the SE coast of Australia and their impact on coastal systems. This was achieved by hindcasting monthly mid-shelf mean wave direction (MWD) for southeastern Australia, based on the monthly trans-Tasman mean sea level pressure (MSLP) difference between Yamba (NSW) and the north island of New Zealand for 124 years. In this analysis he identified that the MWD varies with strong annual cycle, couple to mean, spectral-peak wave period. Accordingly, months and years where a more southerly MWD occurs are accompanied by an increase in the spectral wave period. The most significant multi-decadal fluctuation in the time series is from 1894 to 1914, when the Tasman Sea surface temperatures were 1 to 1.5 C cooler, monthly and annual wave direction was up to a few degrees more southerly.

In the present study the SOI-longshore transport relationship was analysed by comparing the monthly longshore transport to the SOI index with poor correlation. The analysis shows however that there is a medium-term variability pattern and this is similar or larger to what is observed in the mean long term conditions. Therefore they have to be considered in the definition of the hazard lines as well as in the definition of hazard lines and coastal protection alternatives.

Long-term analysis

A detailed two dimensional analysis of the littoral transport processes was undertaken on the 1992-2004 wave data, and compared to historical data for the period 1866 and 2000. The results show that the wave induced longshore transport is the most significant sediment transport mechanism. It is observed that wave driven currents are quite significant in the nearshore areas shallower than 9m depth AHD. No evidence of major sediment transport in the offshore areas has been found.

The long-term sediment transport mechanism in the Stockton area can be described as follows:

1. Net northward sediment transport. The results indicate an estimated value of 55,000m3/yr in the area north of the sewage ponds for the period 1992-2004. This value is considered as an estimate due to the uncertainties in the wave climate conditions;
2. Based on the historical volumetric analysis there is also a net northward sediment transport between 20,000 and 30,000 m3/year for the period 1866-2004;
3. The difference between the 1992-2000 and 1866-2000 transport can be attributed to the variable wave conditions;
4. A complex two dimensional sediment transport mechanism has been predicted as shown in Figure 10-2;
5. The 2-D results show that the port structures tend to redirect the sediment transport travelling from the south into the deep areas therefore obstructing the bypassing mechanism at the Hunter River entrance;
6. The sediment transport process around the study area is highly two dimensional and can be described as:
    a. The most frequent south-easterly waves diffract around the Port breakwaters;
    b. There is a gradient in the wave set-up along the coastline with the smallest set-up in the sheltered area north of the north breakwater This mechanism drives a local current along the shoreline towards the port

entrance, where an eddy forms and transports sediment behind the breakwater;

c. A nodal or neutral point (area where the sediment transport changes direction) is predicted at the northern end of the Mitchell St seawall. Here the sediment transport splits into two directions, southwards and northwards. It is predicted that this is the major eroding stretch in Stockton Beach;

d. The incoming east and north-easterly waves will refract and produce a uniform longshore transport, allowing for accumulation behind the northern breakwater;

7. There is an sand starving area north of the seawall due to the longshore transport variation in the littoral transport from south to north;
8. At the northern end of the seawall the beach tends to retreat but the seawall imposes a limitation therefore inducing a local deepening of the beach profile;
9. The river is not a significant source of sediment in the Stockton area. Suspended sediment delivered to the sea during floods is too fine to be retained on the beach; coarser catchment sands and gravels are trapped further upstream of the estuary;
10. A northwards transport at Nobbys Head has been predicted. The estimated littoral transport at this location for the period 1992-2004 is 33,000m^3/yr based on the sediment properties at Stockton. Due its exposure to the incoming waves the sediment sizes at Nobbys Head may be larger or rocky outcrops may be present. As a result the computed littoral transport may be overestimated. No sediment bypassing is predicted into the southern end of Stockton Beach;
11. The northward transport at Nobbys Head generates a deposition zone north of the southern breakwater. Part of the sediment is deposited in the navigation channel and part in the southern areas of the river entrance. It is unlikely the sediment will be able to reach Stockton Beach as most of the material deposited in the navigation channel is likely to be removed during maintenance dredging of the navigation channel (153,000 m^3/yr of sand were dredged in 2005 that confirm that the bypassing mechanism is bringing sediment into this area). If this material would not be removed a slow sediment bypassing mechanism would be reinstated. It should be expected that this process would occur slowly over a large morphological time scale (in the order of 10 to 100 years) due to the large water depths that limit the effect of the waves;
12. A wave focussing mechanism north of the Mitchell St seawall has been predicted from the simulations. This process has been already identified by vessels sailing in or out of the Port. The wave focussing mechanism is produced when waves propagate from the S and SE and shoal at the shoal east of the Port entrance. The results indicate that the focussing area extends from the treatment ponds area to Fern Bay as presented in Figure 10-2. It is expected that this process tends to exacerbate erosion in this area; however model results should be considered with care because the analysis was undertaken based on a limited number of conditions which have been applied in the analysis. These processes may be more dispersed across this area because of the larger variety of wave conditions than those applied for the analysis; and
13. The area north of Fern bay is expected to be in equilibrium as the beach orientation reaches the equilibrium angle, as calculated in the baseline analysis. This area receives and accumulates the material transported from the southern beaches; and

14. Erosion at the tip of the northern breakwater was indicated in the study of Umwelt (2002) that was reported to propagate back towards Stockton Beach with a drainage pathway from the nearshore zone of Stockton Beach to the entrance channel. This study does not indicate this pattern however it could be expected that localised erosion or scouring around the tip of the breakwater is possible.

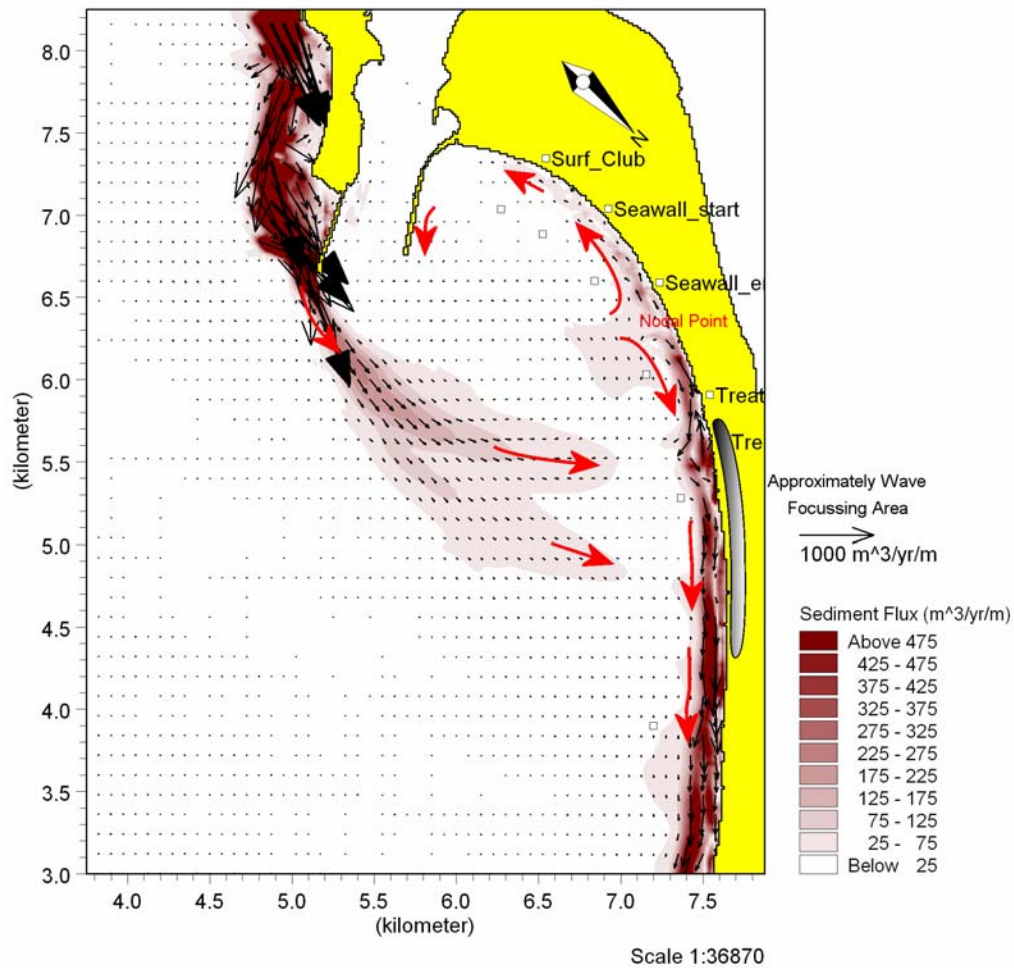


Figure 10-2 Predicted sediment transport mechanism at Stockton based on a typical yearly conditions from 1992-2005

10.3 Greenhouse Effects

The complexity climatic processes and systems means that it is inappropriate to simply extrapolate past trends to forecast future conditions. To estimate future climate change, scientists have developed scenarios, which are not predictions but more of a "what if" analysis. These scenarios provide an estimation of the likely sea-level rise. Based on Bruun (1962, 1988) approximation the estimate retreat for 20 and 50 years is estimated as:

Table 10-2 Predicted beach retreat due or shoreline recession due to sea level rise for the next 20 and 50 years at Stockton (in metres).

Year	Low	Mid	High
2026	0.8	3.6	8.0
2056	3.2	14.0	25.6

It is estimated that an increase in the frequency of particular weather patterns may occur and the changes in frequencies of the weather patterns suggest a shift in wave climate with waves from the southeast becoming more prevalent and waves from the northeast and east becoming less prevalent. This change in the weather pattern will have implications on the stability of the shoreline as the coastline orientation is highly sensitive to variation in wave directions.

11 BEACH EROSION HAZARD ZONE DEFINITION

A beach erosion hazard zone for Stockton Beach was delineated in 2000 as part of the Newcastle Coastline Hazard Definition Study undertaken by WBM for Newcastle City Council. The hazard zone was defined as a hazard band that accounted for the level of uncertainty that was assessed for the definition of the erosion hazards. The hazard lines were based on an estimation of short term effects (dune erosion due to storms), long-term recession and greenhouse effects evaluated based on the analysis of historical data and some numerical investigations WBM (1998). The future trends were estimated based on the assumption that:

- The adopted trend of shoreline retreat will continue unchanged for the next 20 years;
- Shoreline retreat will then progress at a rate of 50% for the following 30 years (to 50 years), reducing to zero thereafter; and
- A transition zone exists south of Mitchell St seawall with shoreline retreat rates reducing both along the beach (towards the south) and in time as outlined above.

During the Newcastle Coastline Management Study Umwelt (2003) analysed the conditions at Stockton Beach and based on the Umwelt (2002) analysis and interpretation of the long term changes in seabed morphology and numerical beach erosion modelling, recognised that the conditions were not consistent with that presented in the Newcastle Coastline Hazard Definition Study produced by WBM. The study indicated that it could be expected that the recent higher-than-average erosion rates were likely to increase at an accelerating rate under large wave activity and the threat to development at Stockton Beach would increase with time. Consequently, it could be assumed that the hazard definition presented in WBM (1998) was likely to be a significant underestimate of what realistically may occur. Therefore in light of the previous investigations the hazard risk at Stockton Beach was likely to be exacerbated and the hazard risk would need to be further assessed as part of additional studies.

Based on the above information, The Hunter Coast and Estuary Management Committee recognised the need for a study update of previous investigations to provide new hazard lines, which are presented in this section.

11.1 Hazard Line Definition

The definition of the beach erosion hazard zones is a relevant element in the establishment of a coastal management plan and requires an understanding of the coastal processes to allow a rational quantification of the shoreline trends. Usually the approach adopted for the definition of the hazard lines and therefore the hazard coastal zones is based on combining short and medium term fluctuation, long -term trends and the expected sea level rise due to greenhouse effects. These processes can be describes as:

1. Beach Fluctuations are associated with oscillation of the shoreline around a mean value, which can be constant if the beach is stable or receding in case of

long-term recession. Depending on the time frame these fluctuations can be classified as:

a. Short term: occur during storm events in response to increased wave height and water sea levels. During these events part of the foredune is eroded, but recovers during mild wave conditions. The dune recovery can take several years. Short-term events have a duration of days to weeks;
b. Medium term: This process is associated to the changes in wave direction due to seasonal or decadal time scales. Shoreline changes are associated with these events;

2. Long term: this is also defined as recession and is usually associated to sand losses produced by a net landward translation of the shoreline over time. Long term processes occur for several years or decades; and
3. Climate change: due to increase greenhouse effects in the atmosphere a sea level rise has been predicted. This process is likely to produce a reaccommodating effect in the beach profile with landward movement of the shoreline and beach recession. Therefore it has to be considered in the beach hazard definition zones.

For a coastline facing shoreline retreat, the prediction of the *t-year* hazard line (S_t) is given as:

$$S_t = S_0 + ST + MT + SER * t + GH(t)$$

Where S_0 is the landward distance from which the hazard line is referenced, *ST* is the shoreline retreat due to short-term events, *MT* is the retreat due to medium term effects, *SER* is the shoreline movement rate (m/year), *t* is the number of years and *GH(t)* is the predicted retreat due to sea level rise induced by sea level rise due to the increase of greenhouse gases.

The coastal processes at Stockton Beach have been estimated based on a number of numerical simulations and this information has been related to the available information on historical trends. In this way it has been possible to determine the future trends of progressive shoreline variations for hazard definition purposes, however there is an uncertainty in the estimations due to:

- The accuracy of hydrographic surveys to determine long term processes (especially before 1866);
- There is a natural variability with cycles where short, medium and long term components superimpose therefore each individual component is difficult to identify; and
- There is a large variability in the sediment transport conditions, which associated with the predicted climate change make it difficult to predict the future long term trends.

The hazard lines will be evaluated for the present conditions (immediate) and for the future (20 and 50 years).In this case the three lines are defined:

- Best estimate or prediction;
- Minimum or seaward limit; and
- Maximum or landward limit.

11.1.1 Present Shoreline Conditions

To produce an accurate quantification of the potential for further erosion and/or recovery of the beach, a survey of the beach was carried out by Newcastle Council on July 2006. The survey extended from the Newcastle Port northern breakwater to the Newcastle Council boundary as presented in Figure 11-1. Measurements were undertaken from the mean sea level line to the top of the beach scarp and a survey reference line was defined. This line is located between 3 and 4 m AHD approximately.

With this information it is possible to evaluate the present state of the beach to determine the S_0 landward distance to include it in the definition of the hazard zones. The landward distance has to be evaluated in the context of the following conditions:

- The effect of any progressive long term or medium term effects; and
- The localised effects which can exacerbate erosion at particular areas as southern areas close to the northern breakwater and the ends of the Mitchell Street seawall

To determine the beach movement, a number of beach profiles were compared to historical beach profiles and measurements obtained from photogrammetric analysis undertaken from the 1950's by DNR. The July 2006 profiles together with the 1952, 1999 and measurements of March 2006 were related to the same coordinate system to determine the profile advance/retreat at several locations along Stockton Beach.

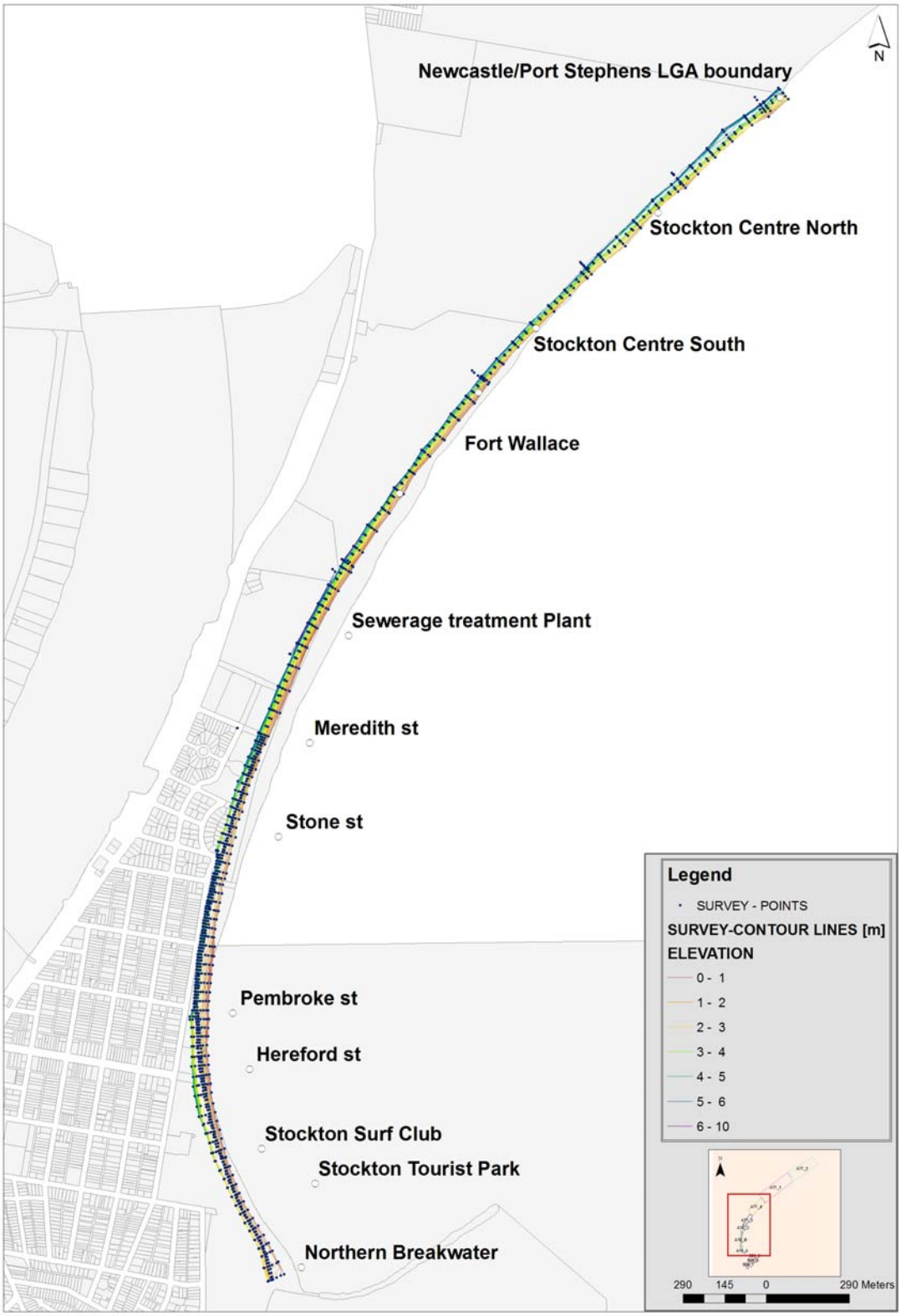


Figure 11-1 Overview of the Stockton beach survey carried out by Newcastle City Council on July 2006.

The comparison at the Stockton Tourist Park (Caravan Park), presented in Figure 11-2, shows that at this location the 1952 escarpment top is the most landward position of the beach for this record. The 2006 beach profile measurements show that the profile has accreted. This is in respect to both the 1952 and 1999 data with a maximum accretion of 25 metres at the top of the escarpment compared to the 1952 data.

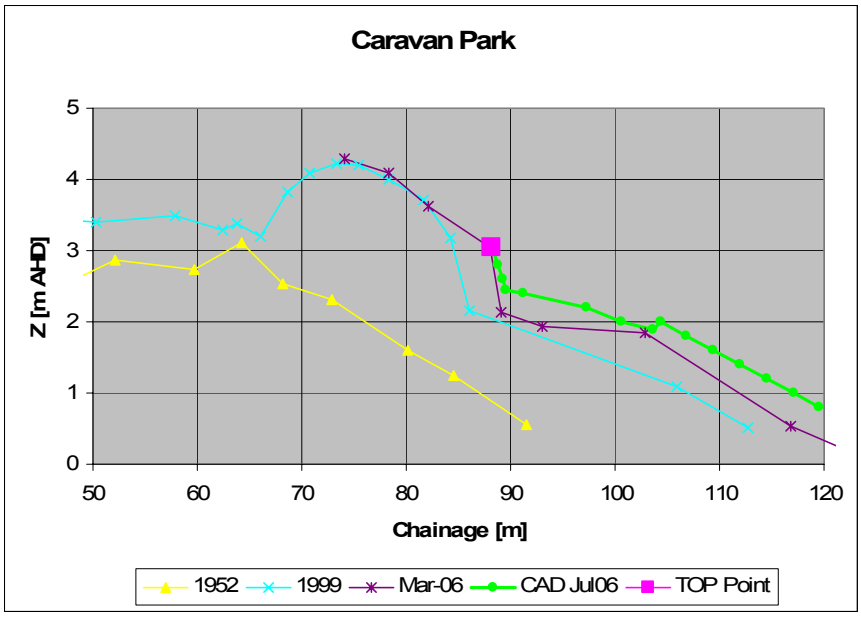


Figure 11-2 Measured beach profile advance/retreat at the Caravan Park (just north of the northern breakwater)

To provide a better historical overview of the shoreline movement at Stockton, the movement of the +3mAHD line has been extracted at four locations to describe the historical behaviour of the most relevant areas of the study, namely:

- **South of Mitchell St seawall**: Stockton Tourist Park (Caravan Park) and the Stockton Surf Club, and
- **North of Mitchell St seawall**: Child Care Centre (between Stone and Meredith St) and the Sewage Ponds.

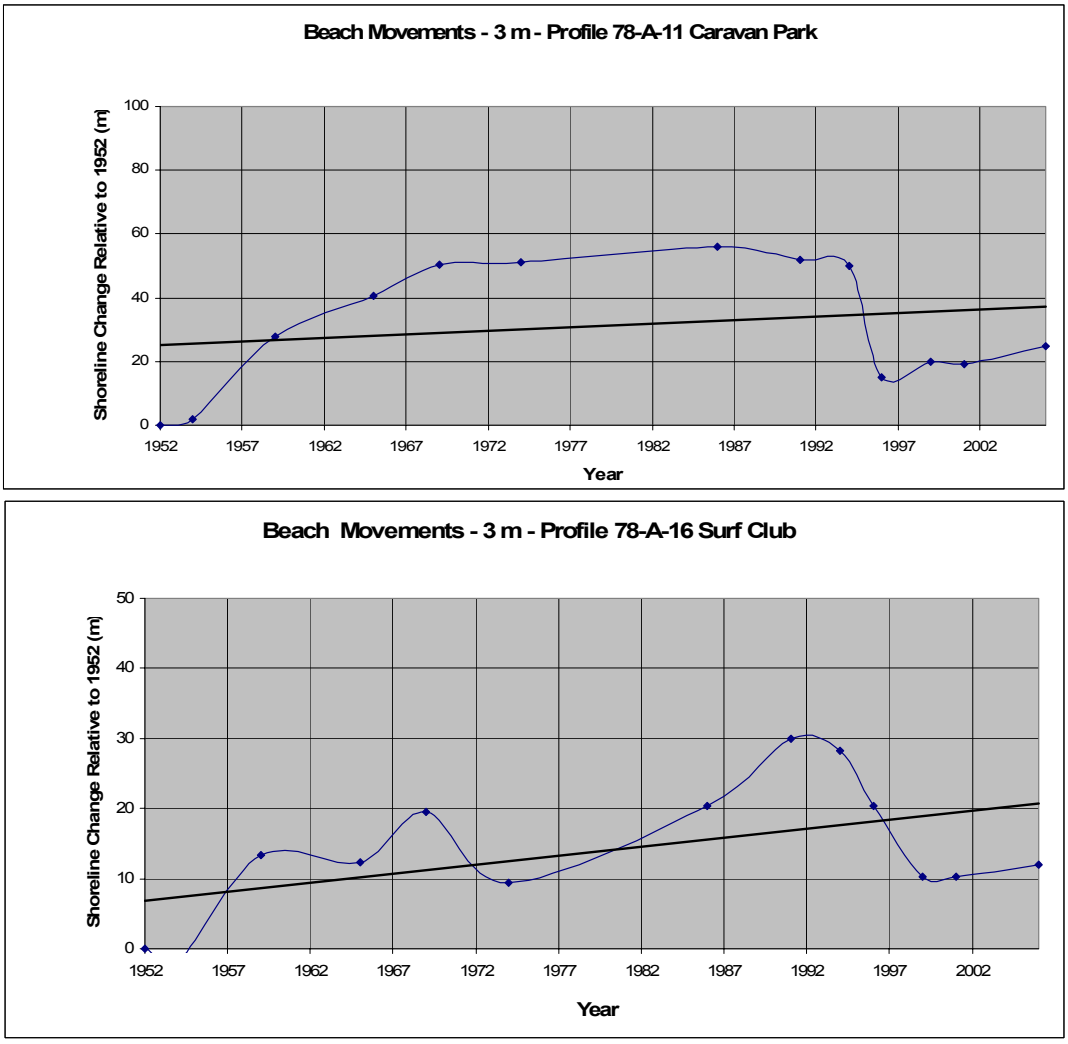
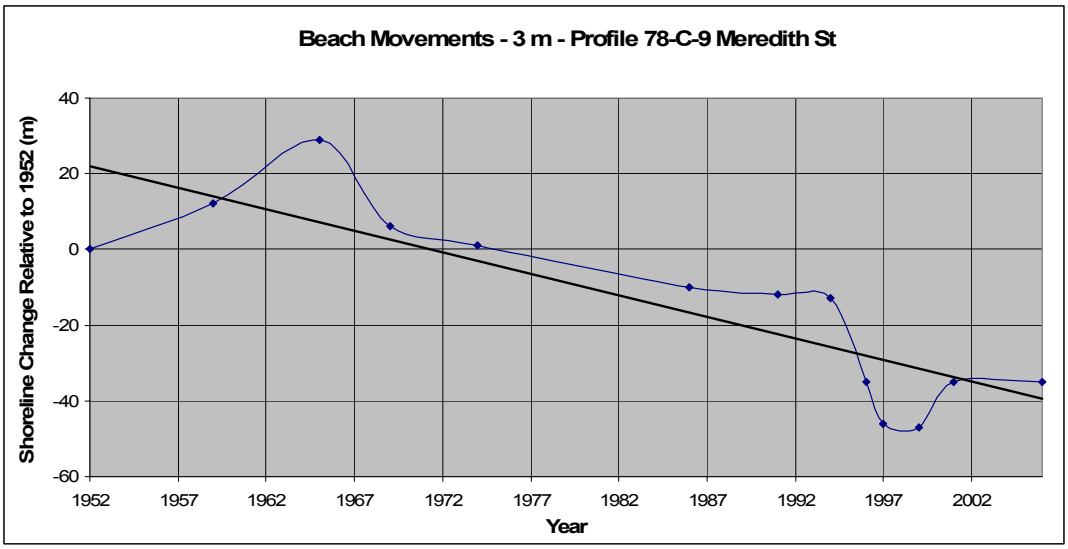


Figure 11-3 Historical shoreline movement at the Caravan Park and the Surf Club.

The analysis shows that the beach at the southern end is stable or undergoing slight long term accretion. The most landward position of the beach escarpment is observed in 1952; since then the beach has recovered steadily. A number of fluctuations are also observed, one of the most significant occurring between 1994 and 1999. During this period the beach receded significantly although it did not reach the position observed in 1952. Since 1999 the beach has recovered between 5 to 7m at the Caravan Park and the Surf Club, but still is slightly eroded when compared to the historical trend. The historical shoreline advance/retreat for the Caravan Park and the Surf Club is presented in Figure 11-3

The area north of the seawall shows a different behaviour with a long term erosional trend. Due to the long term pattern the beach has receded since 1952. Medium and short term fluctuations are superimposed producing significant variation of the beach position. These fluctuations are similar at both the southern and the northern end of the beach. The analysis shows that the 1997-1999 beach scarp is the most landward position for the entire period of analysis. The measurements also show that since this period the

beach has recovered and is placed within or above the average estimated long term movement (black line). For a more detailed illustration the shoreline movement for Meredith St and the sewage ponds areas are presented in Figure 11-4.



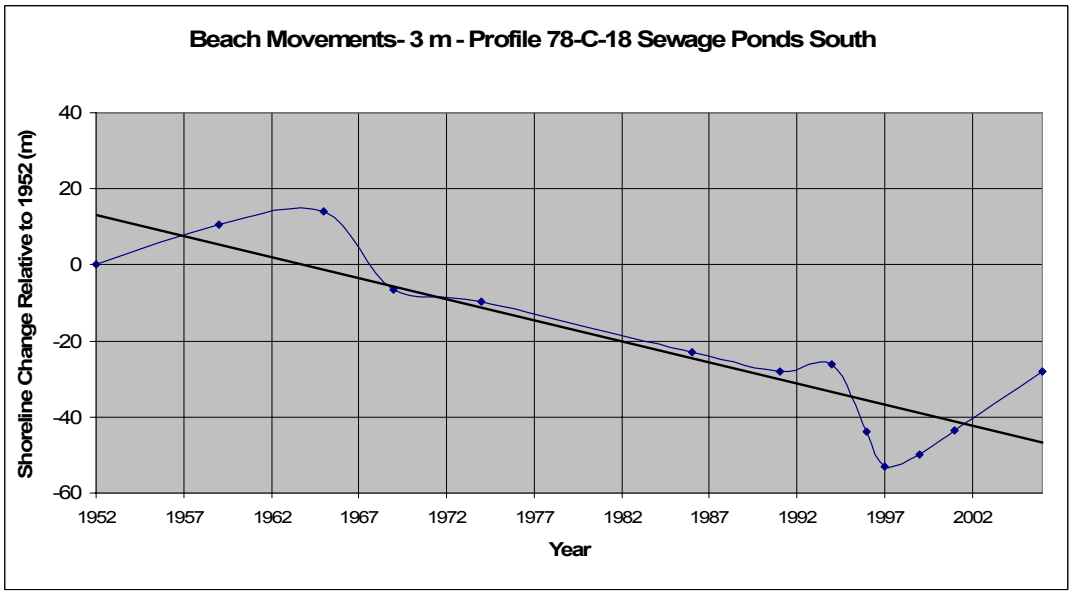


Figure 11-4 Historical shoreline movement at Meredith Street and the sewage ponds.

11.1.2 Shoreline Fluctuations

The beach is a very dynamic area which is affected by the action of waves and currents. Most beaches fluctuate around a mean value (long-term trend) due to seasonal conditions that tend to temporally modify the position of the shoreline. To describe these processes the computed longshore transport at Pt2 north of Stockton Beach for the period 1992-2004 has been presented in Figure 7-4 which include the long term trend and the short and medium term fluctuations, which are significant. The quantification of long-term trends is usually associated to a shoreline rate change (m/yr) whereas the fluctuations are associated with particular events; therefore they are quantified as a total

value (usually in m). Shoreline fluctuations are mainly associated with short and medium term effects and these can be described as follows:

- Short-term events associated with storms. These events occur during increased wave heights and water levels. During these conditions the beach face steepens as sand is transported seaward to form offshore bars. If waves remain large enough for an extended period, a steep erosion escarpment is formed. Erosion continues as long as waves can reach the escarpment base, undercutting vegetation and causing failure of the dune slope. Dune fencing, walkways and other coastal infrastructure or development on the foredune may be undermined and washed away. Buildings landward of the erosion may be damaged through foundation movement. Generally, erosion occurs until the wave action diminishes or due to the erosion the dune retreats landward and is not reached by the waves. During extreme storm events the dune can be over washed and localised dune erosion can occur ; and
- Medium-term events are associated with changes in the wave conditions. Changes in wave conditions on drift-dominated beaches can result in variations in longshore drift rates and rates of headland bypassing. These effects can influence the beach width and shoreline dynamics. This variability can be critical if sediment starvation coincides with the occurrence of storms.

For the definition of the fluctuations in the hazard lines it is important to determine the present state of the beach as it will affect the future erosion potential. If the beach is eroded and the nearshore area has a well established bar from previous storms events, the potential for further erosion of the upper part of the beach and dune is reduced. Conversely, if the nearshore zone has been depleted of sand as a result of longshore transport processes, the potential for erosion from the upper beach is increased. The beach at Stockton is recovering from the erosion period of 1994-1999, however the recovery is different at the northern and southern sections of the beach. The southern section shows recovery but still below the estimated mean long term trend, whereas the northern section shows that sediment is unchanged or slightly accreted. The difference in the present state of the beach is considered in the quantification of the fluctuations in the hazards definition.

Short Term Events

The accurate estimation of storm erosion on beaches requires regular surveys and in particular, surveys before and shortly after a storm. Aerial photography can be applied to quantify the short term erosion but is not always available and if so it is difficult to extrapolate results from one area to another. This difficulty is a response to the particular conditions, e.g. Stockton Beach is protected from the predominant SE waves by the port breakwaters therefore the wave conditions at the beach are significantly different than in an open coast. In order to overcome these issues a modelling approach was applied to estimate short term erosion. The model was calibrated against measurements allowing the determination of dune recession for a number of storms to determine the maximum short term erosion along Stockton Beach.

These values provide a best estimation of the maximum dune erosion for all the simulated cases. Most cases have been considered independently except for the 1974

May and June events that were considered together. The maximum erosion values are presented in Table 8-5.

The shoreline analysis of the beach profiles shows that presently the beach has recovered after a period of significant recession. If the values presented in Table 8-5 are applied the predicted line of erosion may be located seaward of the maximum observed historical recession line. This may be due to the fact that the 1952 storm (most landward dune escarpment line south of the Mitchell St seawall) was more severe or that local erosion effects that are not included in the model. In order to account for this uncertainty the computed hazard line is compared to the conditions of maximum historical erosion (the 1952 or 1999 events) and if located seaward then it is specified at the location of the maximum historical erosion. In this way we account for the uncertainties associated with the definition of the storm events as well as local effects due to the non-uniformity of the beach profiles.

Localised scouring at the vicinity of coastal structures e.g. at the seawall ends and immediately north of the northern breakwater is also an important issue. Little information is available in regards to this topic which adds uncertainty in the calculations. As only historical photos of the area are available an additional erosion value of 25m3/m or 5m is defined to account for these processes.

Additional erosion and accretion patterns often occur along the shore due to spatial variations in wave height and water levels. These variations can result in seaward flowing currents known as “rips” which occur at regular intervals along the shore of most beaches. They often scour deep channels perpendicular to the shoreline and erode the back of the rip embayment due to increased wave penetration and offshore sediment transport processes. Rips may vary in position as wave conditions vary, but are also known to favour certain locations, particularly during storm. A minimum value of 25 m3/m or 5m erosion has been defined in the short term processes to account for these phenomena.

All the short term computations are referred to the dune escarpment at +4mAHD, which do not always agree with the survey reference line provided in the most recent NCC July 2006. The distance between the dune escarpment and the survey reference line is included in the hazard lines definition.

For the 20 and 50 years hazard line definition a maximum dune erosion has been defined to account for any possible further steepening of the beach profiles. Based on the analysis carried out in the dune erosion analysis in Section 8 it was estimated that a further 1m erosion of the sea bed offshore of the beach would increase the dune erosion by 5%. Based on these estimations the maximum dune erosion values are defined 5% larger than those estimated for the present conditions.

Medium Term Events

Medium terms events include inter annual or decadal fluctuations associated with changes in weather pattern and the wave climate. These effects have been observed along the coast of NSW, one of these periods is observed in the period 1994-1999. To analyse the shoreline changes the +3m AHD line movements for the areas Hereford St and Meredith St and are presented in Figure 11-3 and Figure 11-4.

The separation of the short and medium term effects is difficult due to the close interaction between longshore and cross shore effects, however it is clear from the data presented in Figure 11-3 and Figure 11-4 that historical beach fluctuations have been experienced which exceed those predicted for short term processes alone. Some additional allowance is therefore needed to account for the medium term processes described above that were identified as short term processes. This additional component was determined by subtracting the predicted short term erosion from the maximum measured beach fluctuation (1994-1999).

During this period three short term events occurred in Sept 95, May 97 and July 99 which combined with predominant south easterly wave conditions produced significant beach erosion. Total shoreline retreat of 34 and 35 metres was measured at Hereford St. (monument area) and Meredith St., respectively. The values of short-term erosion were predicted at the two locations for all the short term events during the period of analysis. The events were considered interrelated and no beach recovery was allowed in the analysis. Based on this approach the total short term erosion was estimated as 10.7m and 18.6m respectively. These values are slightly larger than those estimated in the short term dune erosion analysis and can be attributed to the fact that the dune is not allowed to recover after each event. This is likely to produce an overestimation of the short term erosion during this period or analysis. In order to provide a more realistic and consistent estimation, the maximum erosion values computed in the short term erosion analysis were applied (8.6m and 17m). Based on these values it is possible to estimate the medium term erosion during this period as the difference between the total erosion in the period 1994-1999 and the maximum single dune erosion values. The predicted medium term values south and north of the seawall are presented in Table 11-1.

Table 11-1 Estimated additional medium-term erosion component.

Erosion (m)\Location	**South of seawall (Hereford St)**	**North of seawall (Meredith St)**
Total 1994-1999 (m)	34	35
Estimated Short-term erosion	8.6	17
Estimated medium-term erosion component	25.4	18

The analysis shows that 25m and 18m fluctuations have occurred south and north of the seawall respectively in addition to that associated with the modelled short term components. The difference in the medium term predictions between the southern and northern part of the beach can be attributed to two factors:

- Medium term effects are associated to temporal variations of the wave conditions that tend to have larger influence at the end of the beach than in the central areas; and
- The northern area has a long-term recession and the medium and long term may be superposed.

The predicted medium term erosion values are based on a beach that is fully accreted, however the analysis of the present state of the beach indicates slight erosion (5m) in the southern area whereas the northern areas are stable or slightly accreting. Therefore a reduction of 5m can be applied for the best estimate medium term erosion south of the seawall, whereas the area north of the seawall shows full recovery and no reduction applies. Based on the present state of the beach the following medium term values are applied:

- South of seawall (Hereford St) Min:0m, best 20m and max 25m
- North of seawall (Meredith St) Min:0m, best 18m and max 18m

11.1.3 Long Term Trends

The definition of long term trends has been based on analysis of historical shoreline change, analysis of nearshore losses as well as a detailed 2-D sediment transport modelling.

The estimated sand volume changes in Area 1 (Figure 6-3) produced by Umwelt provide a very good overview of the historical variations. As it can be observed an average sand loss of 26,400 m^3/y was observed for the period 1886-2000, however there is a large variability in the system and large uncertainty in the data before 1886. This is due to the unknown accuracy of the surveys and as a result the values before 1886 have not been included in the analysis. In order to provide a best estimate of the long-term shoreline rates an additional verification has been undertaken. This has been achieved by comparing the computed 2-D erosion rates to the historical values obtained from the shoreline analysis as presented in Table 11-2. The model results show that a northward longshore transport of 20,500 m^3/yr at Point 2 provided a good match between long-term predictions and measurements; therefore this value has been applied to determine the long-term rates along Stockon Beach. This value is slightly smaller than that measured by Umwelt (2002) but this should be expected as the area of historical analysis extends further offshore where not only longshore transport but cross shore transport occurs which may produce an overestimation of the observed littoral drift.

Table 11-2 Predicted minimum, best estimate and maximum erosion rates at Stockton Beach.

Location	**Shoreline Historical Data (m/yr)**	**Predicted Erosion Rate (m/yr)**
Child Care Centre	-1.3	-1.0
Sewage Ponds	-1.1	-1.3

The band of maximum and minimum erosion rate values was also estimated by allowing a deviation of 20% from the mean values. In this way upper and lower limits of 24,600 and 16,600 m^3/yr were obtained. The predicted minimum, best estimate and maximum erosion rates from the south of Stockton Beach to the Council boundary are presented in the table below.

For the definition of the hazard lines the accreting areas will be considered as stable or erosion rate equals to zero.

Table 11-3 Predicted minimum, best estimate and maximum erosion rates at Stockton Beach. Based on the 2D simulation.

Location	Erosion Rates		
	Min	Best	Max
Stockton Tourist Park	0	0	0
Stockton Surf Club	0	0	0
South Seawall / Hereford St	0	0	0
Child Care Centre	-0.8	-1.0	-1.2
Sewage Ponds	-1.0	-1.3	-1.5
Fort Wallace / Stockton Centre	-0.7	-0.8	-1.0

11.1.4 Greenhouse effects

With increasing greenhouse gas concentrations there is considerable potential for future changes in the climate system which make it inappropriate to simply extrapolate past trends to forecast future conditions. To estimate future climate change, the United Nations Intergovernmental Panel on Climate Change has developed a range of emission scenarios, which have been used in conjunction with global climate models to provide an indication of potential future sea level change. Coastal response to these sea level rise scenarios has been calculated in the present study using Bruun Rule (1962, 1988) to estimate potential future retreat for 20 and 50 years.

Table 11-4 Predicted beach retreat due or shoreline recession due to sea level rise for the next 20 and 50 years at Stockton (in metres).

Year	Low	Mid	High
2026	0.8	3.6	8.0
2056	3.2	14.0	25.6

11.1.5 Zone of reduced foundation capacity

In addition to the previous values, it is necessary to define a zone of reduced foundation capacity for building foundations. This zone would need to take into account the reduced bearing capacity of the dune sand adjacent to the erosion escarpment.

Typical geotechnical properties for normally consolidated dune sands have been provided by Nielsen (1992). Based on beach scour levels and the reduced levels of the back beach areas at Stockton the width of the zone of reduced foundation capacity has been determined as 13 metres for areas north of the surf club reducing to 9 metres at the Caravan Park, Umwelt (2003). These values are not included in the hazard lines but should be considered for planning purposes.

11.2 Uncertainty and Natural Variability

It is important to address the uncertainty and the natural variability of the hazard lines. In this respect the section that described these issues on the Newcastle Hazard Definition by WBM (1998) has been included in this report.

Hazard zones are typically defined by single lines representing the best estimate of the projected landward limit of the backbeach erosion escarpment for various planning periods. There is considerable uncertainty and natural variability in the processes at Stockton Beach which needs to be incorporated into the assessment of such zones. Adopting an estimation approach inherently incorporates a risk that the limit of erosion will extend beyond the projected line or in fact will never reach it.

A conservative approach could be adopted whereby the upper limits of possible erosion are utilised in the assessment such that there is a low probability of the projected line being reached within the specified planning period. Such an approach may be appropriate in planning future developments. However, in presently developed areas, a conservative hazard line may encompass many properties with only a low probability of being threatened within the planning period. Being located within a hazard zone has many implications with respect to the value and usage of properties and therefore adopting a purely conservative approach is not always appropriate.

Accordingly it is considered that at Stockton Beach where uncertainty and natural variability exist, the hazard zones should be identified in such a manner to illustrate the variable risk associated with erosion reaching certain limits within each planning period. This has been achieved by adopting a band of the extent of possible erosion for each planning period rather than a single line. The seaward boundary of the band represents the minimum distance considered appropriate and by definition, erosion has a relatively high probability of reaching this line within the specific planning period. Conversely, the landward limit of the band represents the maximum likely erosion extent has a low probability of being reached.

With a band of possible erosion extent defined, future coastal management planning can consider the risk and consequences of erosion reaching certain limits in deciding appropriate management strategies. However, to assist coastal management decisions where a preferred line is required, a best estimate within that band has been provided for planning purposes. By definition, this has been based on the best estimate from the available information of the various erosion components combined in defining the hazard zones.

In defining the hazard bands, the uncertainty and variability possible within each of four main categories of erosion (short term, medium term, long term and climate change) have been assessed. The major contribution to variability is associated with the longer trends which have been assessed in the form of an annual recession rate. As such, for the longer planning periods, the band widths are wider reflecting the greater uncertainty into the future. This leads to an overlapping of bands for the different planning periods.

11.3 Prediction of Erosion Hazard Lines

The various components outlined above have been combined to determine the projected landward limit of backbeach erosion escarpments for immediate, 20 and 50 year planning periods. The various erosion components for each planning period as described above have been combined at the different locations along the beach and for the hazard band limits considered as presented in Table 11-5, Table 11-6, and Table

11-7. The zone of reduced foundation has not been included in the tables. The predicted erosion hazard lines are presented in Appendix A.

Table 11-5 Stockton Beach immediate erosion hazard definition

Locations	Process	Min	Max
North of Breakwater	Reference line (m)	12*	
	Short term (m)	5	5
	Medium term (m)	N/A	N/A
	Long term (m(	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	0	5**
	Total	***17***	***22***
Stockton Tourist Park	Reference Line (m)	*0*	
	Short term (m)	*5*	*5*
	Medium term (m)	N/A	N/A
	Long term (m(	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	*0*	*5****
	Total	***5***	***10***
Stockton Surf Club	Reference Line (m)	0	
	Short term (m)	5	5
	Medium term(m)	N/A	N/A
	Long term (m	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	0	12
	Total	***5***	***17***
Hereford St	Reference Line (m)	0	
	Short term (m)	5	8.6
	Medium term (m)	N/A	N/A
	Long term (m	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	0	0
	Total	***5***	***8.6***
Child Care Centre	Reference Line (m)	3	
	Short term (m)	5	12.1
	Medium (m)	N/A	N/A
	Long term (m	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	0	0
	Total	***8***	***15.1***
Meredith Street	Reference Line (m)	0	
	Short term (m)	5	17
	Medium term (m)	N/A	N/A
	Long term (m)	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	0	0
	Total	***5***	***17***

* Distance between the measured reference line and the scarp of the dune (+4mAHD)
** To account for local scour due to the presence of structures
*** To reach the maximum historical erosion line
N/A non applicable

Sewage Ponds	Reference Line (m)	0	
	Short term (m)	5	17.9
	Medium term (m)	N/A	N/A
	Long term (m)	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	0	0
	Total	***5***	***17.9***
Fort Stockton	Reference Line (m)	0	
	Short term (m)	5	21.9
	Medium term (m)	N/A	N/A
	Long term (m)	N/A	N/A
	Green House (m)	N/A	N/A
	Add Effects (m)	0	0
	Total	***5***	***21.9***
Fort Wallace	Reference Line (m)	0	
	Short term (m)	5	22.4
	Medium term (m)	N/A	N/A
	Long term (m)	N/A	N/A
	Green House (m)	N/A	N/A
	Local Effects (m)	0	0
	Total	***5***	***22.4***
Stockton Centre	Reference Line (m)	0	
	Short term (m)	5	23.8
	Medium term (m)	N/A	N/A
	Long term (m)	N/A	N/A
	Green House (m)	N/A	N/A
	Local Effects (m)	0	0
	Total	***5***	***23.8***
Council Boundary	Reference Line (m)	*0*	
	Short term (m)	5	24.5
	Medium term (m)	N/A	N/A
	Long term (m)	N/A	N/A
	Green House (m)	N/A	N/A
	Local Effects (m)	0	0
	Total	***5***	***24.5***

N/A non applicable

Table 11-6 Stockton Beach 20 year erosion hazard definition

Locations	Process	Min	Best Estimate	Max
North of Breakwater	Reference Line (m)	12*		
	Short term (m)	5	5	5
	Medium term (m)	0	20***	25
	Long term (m)	0	0	0
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	0	5**	5**
	Total	***17.8***	***45.6***	***55***
Stockton Tourist Park	Reference Line (m)	*0*		
	Short term (m)	*5*	*5*	*5*
	Medium term (m)	0	20***	25
	Long term (m(	0	0	0
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	*0*	*0*	*0*
	Total	***5.8***	***28.6***	***38***
Stockton Surf Club	Reference Line (m)	0		
	Short term (m)	5	5	5
	Medium term(m)	0	20***	25
	Long term (m	0	0	0
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	0	0	0
	Total	***5.8***	***28.6***	***38***
Hereford St	Reference Line (m)	0		
	Short term (m)	5	8.6	9
	Medium term (m)	0	20***	25
	Long term (m	0	0	0
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	0	0	0
	Total	***5.8***	***32.2***	***42***
Child Care Centre	Reference Line (m)	0		
	Short term (m)	5	12.1	12.7
	Medium (m)	0	18	18
	Long term (m)	16	20	24
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	0	0	0
	Total	***21.8***	***53.7***	***62.7***
Meredith St	Reference Line (m)	0		
	Short term (m)	5	17	17.9
	Medium term (m)	0	18	18
	Long term (m)	18	23	27
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	0	0	0
	Total	***23.8***	***61.6***	***70.9***

* Distance between the measured reference line and the scarp of the dune (+4mAHD)

** To account for local scour due to the presence of structures

*** Medium term effects in the southern areas has been estimated as 25m however due to the eroded state of the beach is reduced 5m

Sewage Ponds	Reference Line (m)	0		
	Short term (m)	5	17.9	18.8
	Medium term (m)	0	18	18
	Long term (m)	20	26	30
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	0	0	0
	Total	***25.8***	***65.5***	***74.8***
Fort Stockton	Reference Line (m)	0		
	Short term (m)	5	21.9	23
	Medium term (m)	0	18	18
	Long term (m)	17	21	25
	Greenhouse (m)	0.8	3.6	8.0
	Add Effects (m)	0	0	0
	Total	***22.8***	***64.5***	***74***
Fort Wallace	Reference Line (m)	0		
	Short term (m)	5	22.4	23.5
	Medium term (m)	0	18	18
	Long term (m)	14	16	20
	Greenhouse (m)	0.8	3.6	8.0
	Local Effects (m)	0	0	0
	Total	***19.8***	***60***	***69.5***
Stockton Centre	Reference Line (m)	0		
	Short term (m)	5	23.8	25
	Medium term (m)	0	18	18
	Long term (m)	14	16	20
	Greenhouse (m)	0.8	3.6	8.0
	Local Effects (m)	0	0	0
	Total	***19.8***	***61.4***	***71***
Council Boundary	Reference Line (m)	*0*		
	Short term (m)	5	24.5	25.7
	Medium term (m)	0	18	18
	Long term (m)	14	16	20
	Greenhouse (m)	0.8	3.6	8.0
	Local Effects (m)	0	0	0
	Total	***19.8***	***62.1***	***71.7***

Table 11-7 Stockton Beach 50 year erosion hazard definition

Locations	Process	Min	Best Estimate	Max
North of Breakwater	Reference Line (m)		12*	
	Short term (m)	5	5	5
	Medium term (m)	0	20***	25
	Long term (m)	0	0	0
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	0	5**	5**
	Total	***20.2***	***56***	***72.6***
Caravan Park	Reference Line (m)		*0*	
	Short term (m)	*5*	*5*	*5*
	Medium term (m)	0	20	25
	Long term (m(	0	0	0
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	*0*	*0*	*0*
	Total	***8.2***	***39***	***55.6***
Surf Club	Reference Line (m)		0	
	Short term (m)	5	5	5
	Medium term(m)	0	20***	25
	Long term (m	0	0	0
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	0	0	0
	Total	***8.2***	***39***	***55.6***
Hereford St	Reference Line (m)		0	
	Short term (m)	5	8.6	9
	Medium term (m)	0	20***	25
	Long term (m	0	0	0
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	0	0	0
	Total	***8.2***	***42.6***	***59.6***
North of Seawall (Child Care Centre)	Reference Line (m)		0	
	Short term (m)	5	12.1	12.7
	Medium (m)	0	18	18
	Long term (m	40	50	60
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	0	0	0
	Total	***48.2***	***94.1***	***116.3***
Meredith Street	Reference Line (m)		0	
	Short term (m)	5	17	17.9
	Medium term (m)	0	18	18
	Long term (m)	50	62	75
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	0	0	0
	Total	***58.2***	***111***	***136.5***

* Distance between the measured reference line and the scarp of the dune (+4mAHD)

** To account for local scour due to the presence of structures

*** Medium term effects in the southern areas has been estimated as 25m however due to the eroded state of the beach is reduced 5m

Sewage Ponds	Reference Line (m)	0		
	Short term (m)	5	17.9	18.8
	Medium term (m)	0	18	18
	Long term (m)	50	65	75
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	0	0	0
	Total	***58.2***	***114.9***	***137.4***
Fort Stockton	Reference Line (m)	0		
	Short term (m)	5	21.9	23
	Medium term (m)	0	18	18
	Long term (m)	42.5	52.5	62.5
	Greenhouse (m)	3.2	14.0	25.6
	Add Effects (m)	0	0	0
	Total	***50.7***	***106.4***	***129.1***
Fort Wallace	Reference Line (m)			
	Short term (m)	5	22.4	22.4
	Medium term (m)	0	18	18
	Long term (m)	35	40	50
	Greenhouse (m)	3.2	14.0	25.6
	Local Effects (m)	0	0	0
	Total	***43.2***	***94.4***	***116***
Stockton Centre	Reference Line (m)	0		
	Short term (m)	5	23.8	25
	Medium term (m)	0	18	18
	Long term (m)	35	40	50
	Greenhouse (m)	3.2	14.0	25.6
	Local Effects (m)	0	0	0
	Total	***43.2***	***95.8***	***118.6***
Council boundary	Reference Line (m)	*0*		
	Short term (m)	5	24.5	26
	Medium term (m)	0	18	18
	Long term (m)	35	40	50
	Greenhouse (m)	3.2	14.0	25.6
	Local Effects (m)	0	0	0
	Total	***43.2***	***96.5***	***119.6***

12 GLOSSARY OF COASTAL TERMS

In order to ensure sound communication it is important to define the coastal terms that are used in coastal engineering and shoreline management. The definitions of coastal terms and processes are described below. Figure 12-1 represents the coastal profile and has been obtained from Mangor's (200X)shoreline management guidelines.

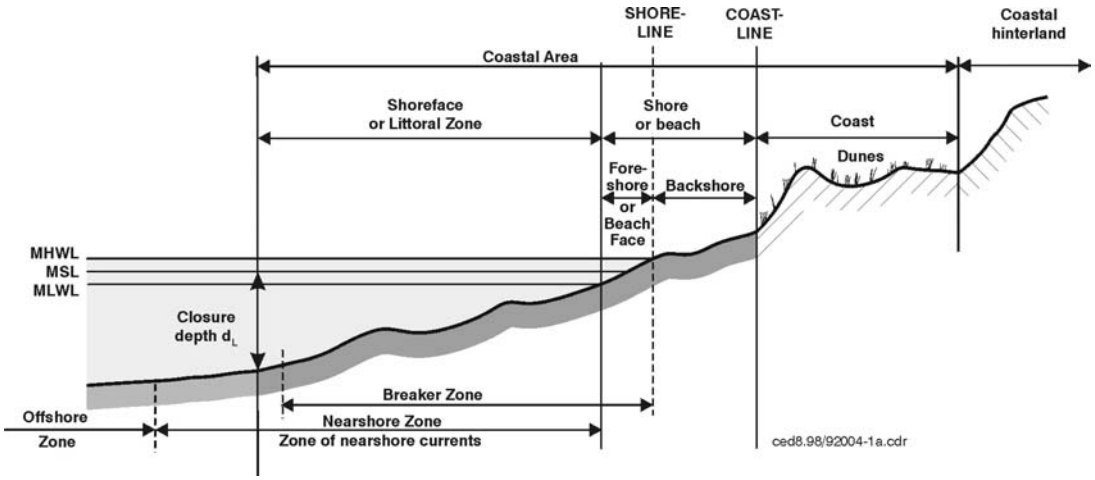


Figure 12-1 Definition of coastal terms, adapted from Shore Protection Manual, 1984.

Definition of coastal terms:

- COASTAL AREA: The land and sea areas bordering the shoreline.

- COAST: The strip of land that extends from the coastline inland to the first major change in the terrain features. The main types of coast features are the following:
  - Dune areas
  - Cliff areas
  - Low lying areas, possibly protected by dikes or seawalls etc.

- COASTAL HINTERLAND: The land that extends landward of the coast and which is not influenced by coastal processes.

- COASTLINE: Technically the line that forms the boundary between the COAST and the SHORE, i.e. the foot of the cliff or the foot of the dunes. Commonly, the line that forms the boundary between the land and the water.

- SHORE or BEACH: The zone of unconsolidated material that extends from the low water-line to the line of permanent vegetation (the effective limit of storm waves). The shore can be divided into the foreshore and the backshore. The foreshore, also called the beach face, is the area between mean low water spring and mean high water spring plus the uprush zone. In the case of a high tidal range the foreshore can be very wide, in which case the foreshore is sometimes referred to as a TIDAL

FLAT. The backshore is the part of the shore which is dry under normal conditions. The backshore is often dominated by berms and it is without vegetation. The backshore is only exposed to wave action during extreme events; the width of the backshore depends on the beach material, the magnitude of the storm surge, and the wave exposure at the site. Fine sandy beach material, high storm surge, and high wave exposure provide the conditions for a wide backshore; the width of the backshore, however, also depends on the geology at the site. The landward limit of the backshore towards the COAST is the place where there is a marked change in material or physiographic form, e.g. the foot of a cliff, the foot of the dunes, or the line of permanent vegetation. A shore of unconsolidated material (sand) is usually called a BEACH.

- SHORELINE: The intersection between the mean high water-line and the shore.

- SHOREFACE or LITTORAL ZONE: The zone extending seaward from the low water-line to some distance beyond the breaker-zone. The littoral zone is the zone in which the littoral processes take place; these are mainly long-shore transport, also referred to as the littoral drift, and cross-shore transport. The width of the instantaneous littoral zone of course depends on the wave conditions. In this context, we will define the littoral zone as the zone valid for the yearly wave climate. The width of the littoral zone can thus be defined as the width of the transport zone for the significant wave height, which is exceeded 12 hours per year, $H_{S,\ 12\ h/y}$.

- BREAKER-ZONE or SURF-ZONE: There is no clear definition of the breaker-zone, but it can be defined as the zone extending seaward from the shoreline that is exposed to depth-limited breaking waves. The outer limit of the breaker-zone is called the BREAKER-LINE. The instantaneous width of the surf-zone varies of course with the instantaneous wave conditions. In this context we will define the surf-zone as the zone valid for the yearly wave climate defined by the significant wave height $H_{S,\ 12\ h/y}$, which is the wave exceeded 12 hours per year. The width of the breaker/surf-zone can thus be defined as the width of the zone within which $H_{S,\ 12\ h/y}$ breaks. The breaker/surf-zone is somewhat narrower than the littoral zone, as the transport starts at greater depth than the breaking. It is evaluated that 80 to 90% of the yearly littoral transport takes place within the breaker or surf-zone. The depth at the outer limit of the breaker-zone is close to the 1.8 times $H_{S,\ 12\ h/y}$.

- NEARSHORE ZONE: The zone extending seaward from the low water-line well beyond the breaker-zone; it defines the area influenced by the nearshore currents. The nearshore zone extends somewhat further seawards than the littoral zone.

    - CLOSURE DEPTH: The depth beyond which no significant longshore and cross-shore transports take place due to littoral transport processes.

- OFFSHORE ZONE: The offshore zone is not well defined. In relation to beach terminology, it is thus not clear if it starts from the littoral zone, from the breaker or from the nearshore zone. In the present context, the offshore zone is defined as the zone off the nearshore zone.

In parallel with the above definition of coastal terms for coast and shore, there is a distinction between coast protection and shore protection, as measures protecting the coast against coastline retreat and the shore against shore degradation, respectively. Furthermore, the term sea defence is defined as the measure which protects a low-lying coast and hinterland against flooding. The complete definitions are presented below:

- COAST PROTECTION: Measures aiming at protecting the coast against coastline retreat, thus protecting housing, infrastructure, the coast and the hinterland from erosion often at the expense of losing the beach and the dynamic coastal landscape. Coast protection often consists of hard structures such as revetments or groynes.

- SHORE PROTECTION: Measures aiming at protecting, preserving or restoring the shore and the dynamic coastal landscape as well as protecting against coastline retreat to the extent possible. Shore protection often consists of a combination of structures (such as detached breakwaters) and nourishment or nourishment alone.

- SEA DEFENCE: Measures aiming at protecting low-lying coast and coastal hinterland against flooding caused by the combined effect of storm surge and extreme astronomical tides. Sea defence often consists of dikes of some kind, either as traditional vegetated sand dikes, possibly partly reinforced by a revetment, or in the form of artificial dunes.

- BATHYMETRY: Depth distribution, where depth is the distance between the seabed and mean sea level.

- MIKE21 SW (Spectral Wave Model).

- MIKE21 PMS (Parametric Slop Model).

- Beach escarpment: steep slope at the edge of the foredune.

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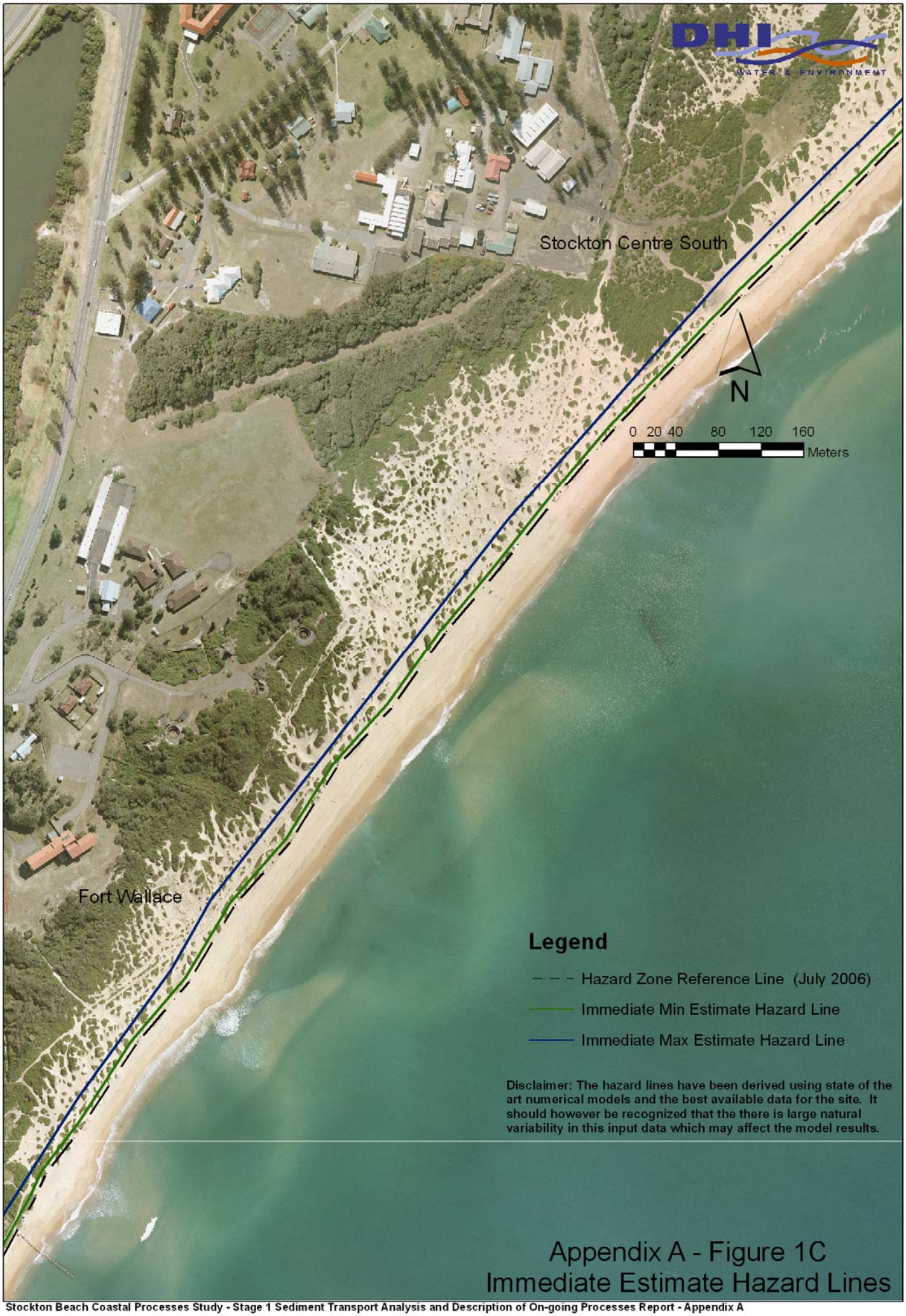
A P P E N D I X A

Erosion Hazard Lines

Appendix A - Figure 1A
Immediate Estimate Hazard Lines

Appendix A - Figure 1B
Immediate Estimate Hazard Lines



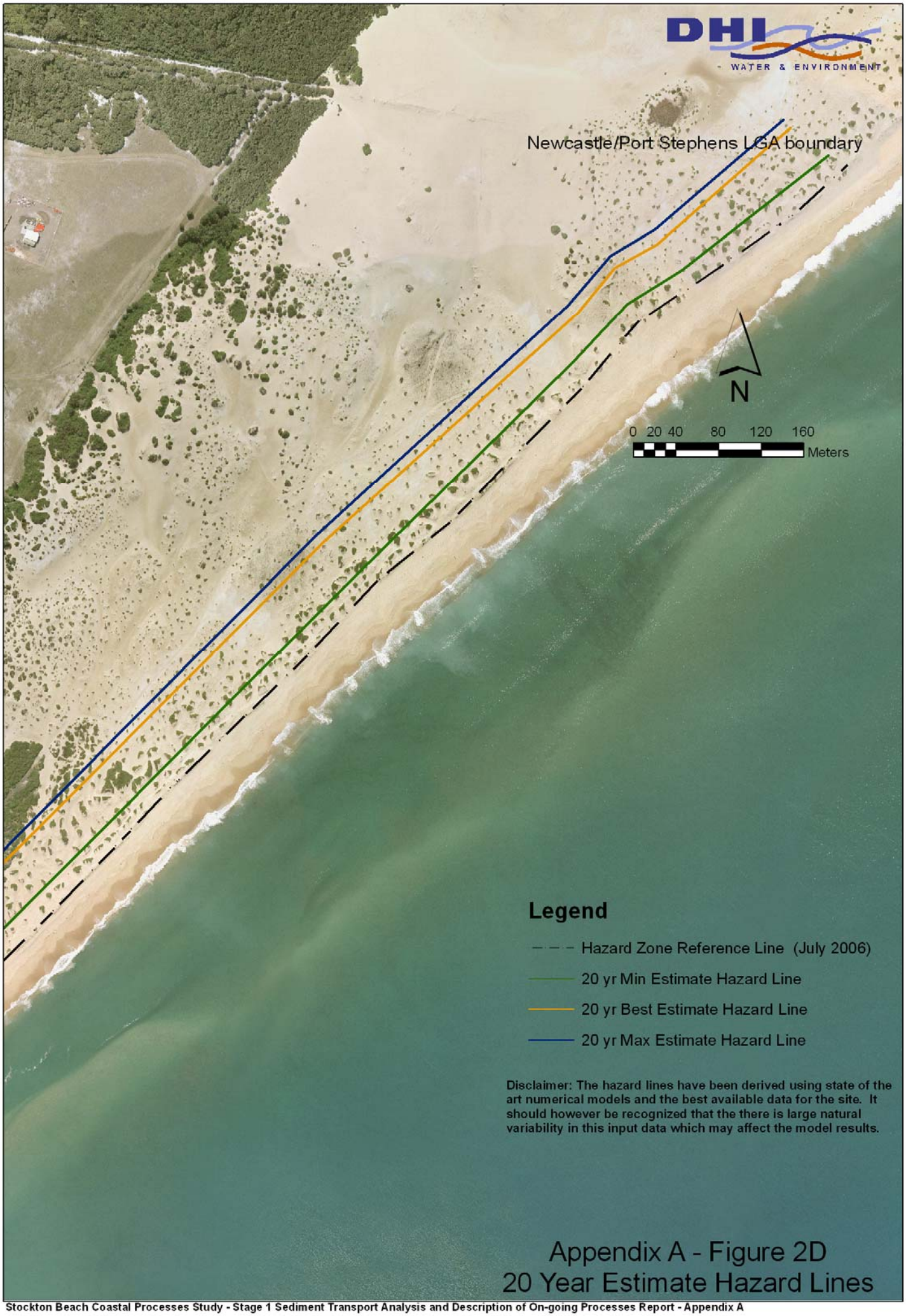
Appendix A - Figure 1C
Immediate Estimate Hazard Lines

Appendix A - Figure 1D
Immediate Estimate Hazard Lines

Appendix A - Figure 2A
20 Year Estimate Hazard Lines

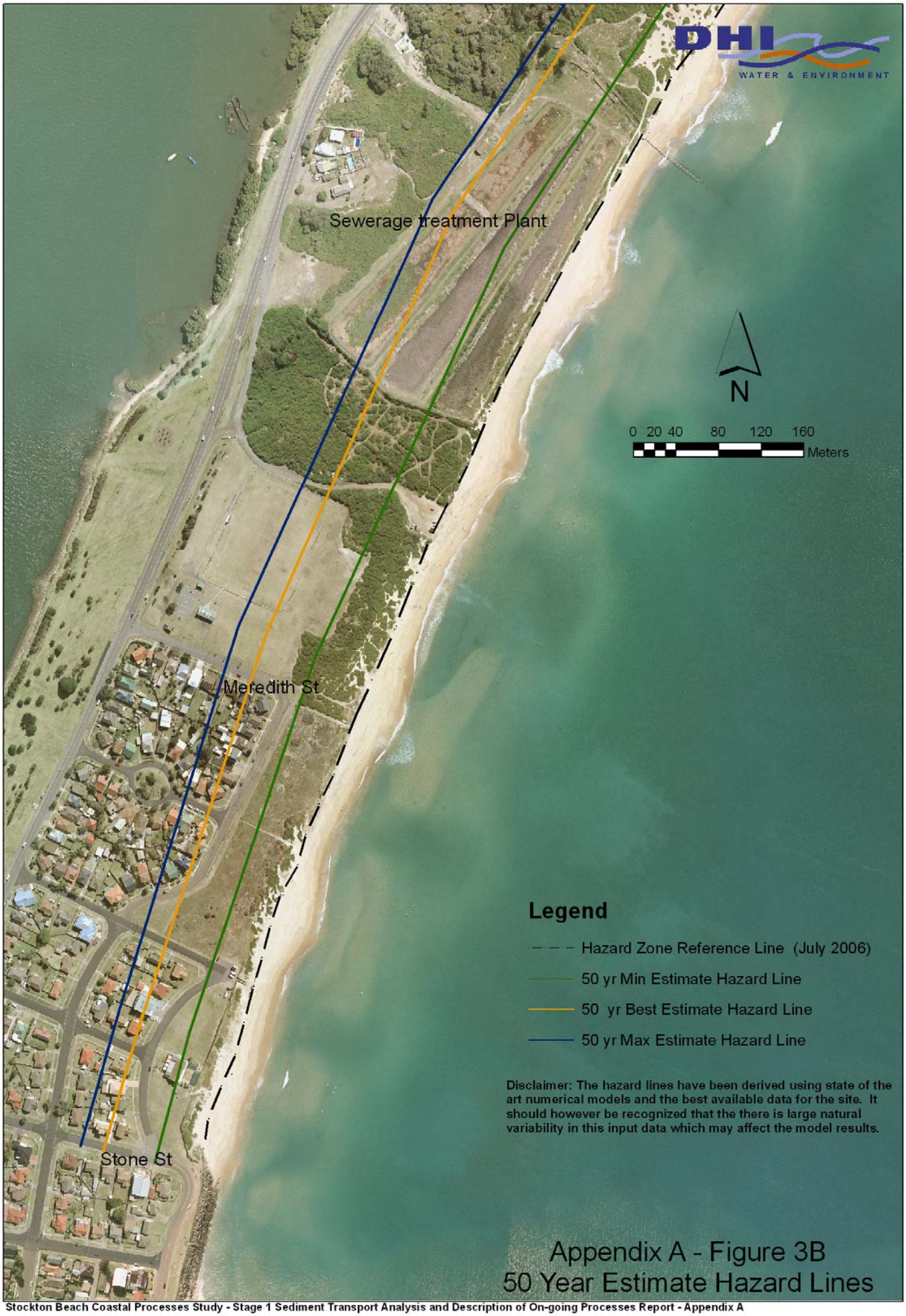
Appendix A - Figure 2B
20 Year Estimate Hazard Lines

Appendix A - Figure 2C
20 Year Estimate Hazard Lines

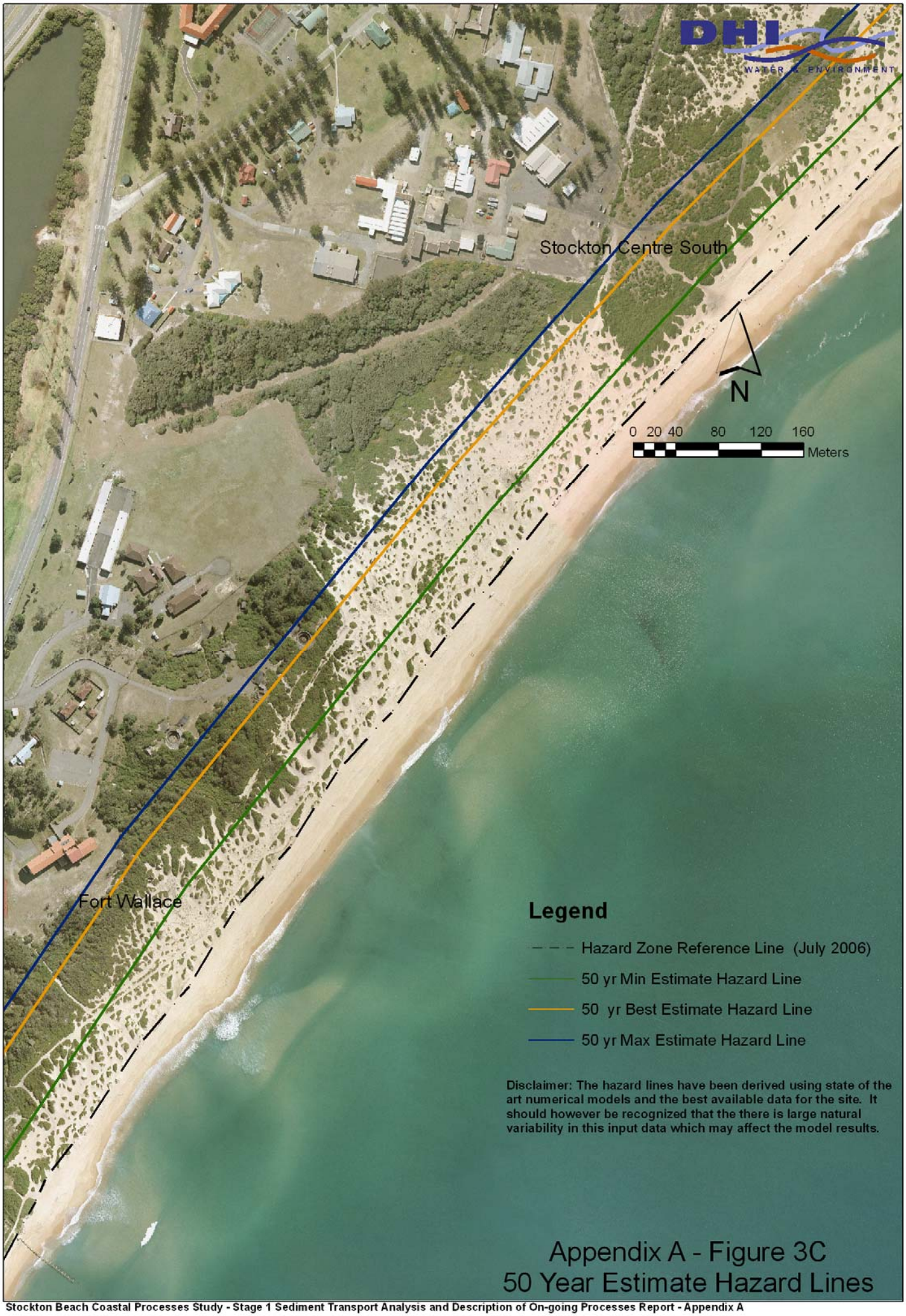


Appendix A - Figure 2D
20 Year Estimate Hazard Lines

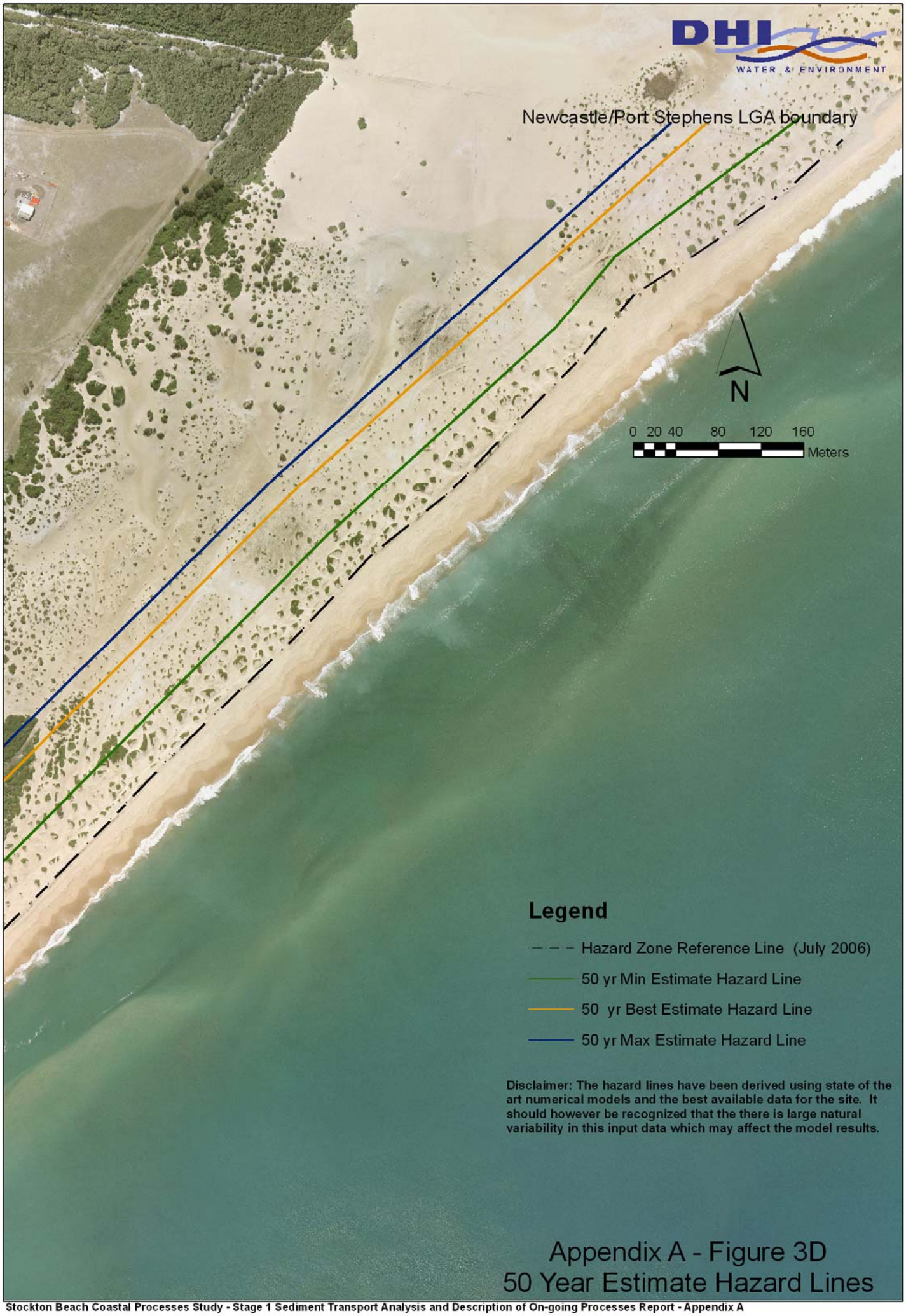
Appendix A - Figure 3A
50 Year Estimate Hazard Lines



Appendix A - Figure 3B
50 Year Estimate Hazard Lines



Appendix A - Figure 3C
50 Year Estimate Hazard Lines



Appendix A - Figure 3D
50 Year Estimate Hazard Lines

A P P E N D I X B

Historical storms in the NSW coast

Storm Date	Peak Ocean Water Level (m,MSL)	Sydney Waverider		Storm Classification
		Hs (m)	MWD (deg)	
May 1974	1.48	9.1	155	X
June 1974	1.26	6.7	170	X
9-13 June 1994	0.89	5.6	157	A
22-25 July 1994	0.87	3.7	157	B
7-12 August 1994	0.84	4.2	169	B
2-5 January 1995	1.02	3.6	180	B
1-13 March 1995	0.85	5.4	106	A
16-23 June 1995	1.03	5.2	180	A
25-29 September 1995	0.86	6.3	165	X
21-24 December 1995	1.05	4.6	180	B
12-18 February 1996	0.79	4.7	157	B
2-7 May 1996	0.76	3.7	57	B
14-19 July 1996	0.88	3.7	147	B
19-21 August 1996	0.66	5.5	180	B
29 Aug. - 3 September 1996	0.84	6.0	135	A
18-25 November 1996	0.74	4.6	145	B
7-10 January 1997	0.97	4.4	167	B
9-13 May 1997	1.21	9.9	151	X
7-9 March 1998	0.82	5.1	159	A
24 March 1998	0.60	3.4	173	B
4-9 June 1998	0.99	4.7	166	B
1-2 July 1998	0.64	4.8	131	B
10-15 July 1998	1.00	4.0	160	B
6-8 August 1998	1.23	5.0	127	B
7 April 1999	0.67	3.7	162	B
21-30 April 1999	1.00	5.7	101	A
25 May 1999	0.60	4.5	59	B
12 June 1999	1.18	4.4	159	B
2-3 July 1999	0.82	4.1	155	B
14-17 July 1999	1.18	6.1	117	X
15-16 August 1999	0.56	4.4	127	B
11-13 September 1999	0.79	4.2	141	B
29 Dec. - 1 January 2000	0.72	4.3	174	B
6-8 January 2000	1.07	4.1	180	B
7-9 March 2000	0.78	4.3	128	B
31 May - 3 June 2000	1.21	4.5	171	B
30 June - 2 July 2000	1.26	6.1	177	X
17-18 July 2000	0.96	4.0	139	B
16-17 August 2000	0.80	3.7	190	B
5-10 March 2001	1.14	4.2	101	B
6-12 May 2001	1.17	4.2	157	B
19-22 July 2001	1.19	3.8	143	B
27-29 July 2001	0.93	6.7	167	X
26-27 September 2001	0.59	3.8	173	B
8-9 October 2001	0.82	4.6	178	B
13-14 November 2001	0.75	3.7	174	B
19-22 November 2001	0.87	5.7	156	A
30-31 March 2002	1.01	3.7	176	B
28-30 May 2002	0.87	4.1	143	B
18-19 June 2002	0.84	4.5	152	B
29 Jun. - 1 July 2002	0.83	6.0	174	X
14-17 August 2002	0.67	5.3	146	A
23-24 August 2002	0.71	3.8	169	B
9-10 January 2003	0.57	4.8	171	B
3-6 April 2003	0.73	3.7	185	B
16-19 April 2003	1.06	4.5	126	B
27-28 June 2003	0.88	4.3	94	B
31 Jul. -2 August 2003	0.96	4.8	174	B
4 September 2003	0.61	4.8	171	B
11-13 October 2003	0.81	4.5	174	B
25-27 February 2004	0.64	5.4	81	A
18-20 July 2004	0.95	5.9	171	A

Note:
1a: Ocean Water Level data was obtained from Tomaree Head (30km north) in the period 1994-1997
1b: Ocean water levels obtained from Newcastle Pilot Station for the period 1998-2004
2: Sydney Waverider 150km south Newcastle
3: Storm classification -

"X" extreme larger than 6 metres
"A" severe between 5 and 6 metres
"B" Moderate between 3 and 5 metres

The 1974 event was reconstructed from the paper "The storms of May-June 1974, Sydney, NSW" by Foster. Water Levels were provided for the May event.

A P P E N D I X C

MIKE 21 Model Description

Brief Description of *DHI Software* for Coastal Applications

Introduction

Since the early 1970s, DHI has been at the forefront in the development and application of advanced numerical modelling software to providing solutions for a large number of coastal engineering projects. These projects include shoreline management, sedimentation in harbours and navigation channels, tidal inlet stabilisation and environmental impact assessment of dredging and reclamation. The main software models applied in these projects are LITPACK and MIKE 21 modelling systems.

LITPACK

LITPACK is a modelling system consisting of several modules that are used to investigate coastal processes on a straight or nearly straight coastline.

At the core of LITPACK is the module that calculates sand transport **STP**. STP includes a state-of-the-art description of sand transport processes. The hydrodynamics e.g. wave boundary layer, vertical variation of flow velocities and turbulence are described in detail, including additional turbulence generated by wave breaking. The variation of the suspended sediment concentration over the wave period and depth is calculated. The model takes into account a moderate gradation of the bed material. This deterministic approach makes the model considerably less sensitive to calibration parameters, as is the case for empirical or semi-empirical formulas.

The **LITDRIFT** module in LITPACK is used to determine the yearly littoral drift at a given section of the coast. By using this module at several sections along the coast, and adding the effect of sand transport from other sources and sinks (e.g. rivers) a regional sand budget can be developed for the coast.

The **LITLINE** module is used to simulate coastline changes as a function of variation in the littoral drift, which is due to the presence of structures, variations in the wave conditions, orientation of the coastline or change in the coastal profile. It is often used to evaluate the long-term (tens of years) shoreline changes due to the impact of various shore protection strategies, such as groin fields, offshore breakwaters, revetments, beach nourishment etc.

The **LITPROF** module in LITPACK is used to model the evolution of a beach profile during storms. It describes various cross-shore processes, such as wave asymmetry, streaming and undertow in the surf zone. It is applied for instance, to assess the potential of exposure for buried pipelines near the coast or to assess setback of the shoreline during rough conditions.

For more information on LITPACK, see
http://www.dhisoftware.com/litpack/Description/index.htm

MIKE 21

MIKE 21 is a general numerical modelling system for the study of waves, currents, sediment transport, both cohesive and non-cohesive, and water quality on a complex bathymetry. All modules use the same (rectangular) grid and all modules are general tools, which can be applied in many different areas to study different problems with different scales.

The modules frequently used for coastal modelling applications in MIKE 21 are:

MIKE 21 NSW (Near Shore Waves): This is a stationary, parameterised, spectral wind wave model. The model includes all dominant wave transformation mechanisms such as shoaling, refraction and breaking. The model also includes local wave generation by wind, and current refraction to a certain degree. The output of this model is the spatial variation of wave height, -period and mean direction. Further, the wave induced radiation stress field is calculated.

MIKE 21 PMS (Parabolic Mild Slope): This is a refraction/diffraction model based on the parabolic approximation to the mild slope equation. It is an efficient tool for the determination of wave fields in larger coastal areas where backscattering can be neglected. The model includes all dominant wave transformation mechanisms such as shoaling, refraction, diffraction and breaking. The output of this model is the spatial variation of wave height, -period and mean direction. Further, the wave induced radiation stress field is calculated.

MIKE 21 HD (Hydrodynamics): This model simulates the water levels and depth-integrated fluxes driven by wave breaking, wind and/or tide. MIKE 21 HD includes flooding and drying of inter-tidal areas. The model results are the spatial and temporal variations of water levels and flow velocity fields.

MIKE 21 ST (Sand transport): This model is used for calculating the sand transport rates and associated initial rates of bed level changes. The calculation can be done for pure currents and combined currents and waves. Several formulations are implemented in the model for calculating sand transport in pure currents. The STP formulation (detailed sand transport model used also used in LITPACK) and Bijker's method are available for calculating sand transport rates in combined currents and waves.

MIKE 21 MT (Mud transport): This model describes the erosion, transport and deposition of cohesive sediments (mud, silt or clay) under the action of waves and currents. The model also takes into account the consolidation of the bed. The lack of a universally applicable, physically based formulation for cohesive sediments means that any model of this type will, to some extent, be empirical. Consequently, the mathematical descriptions of erosion and deposition are essentially empirical, although they are based on sound physical principles. The advection-dispersion scheme used in the MIKE 21 AD Module is used to describe the transport and dispersion of suspended sediments. The model can be used to determine the siltation of cohesive materials in harbours, lagoons or coastal areas and to determine the fate of dredged spoils.

In coastal engineering practice, MIKE 21 is used for the detailed study of waves, tidal and wave driven currents and the sediment transport in the coastal zone or around a tidal inlet.

Examples are: simulation of rip currents and associated rip channels near groins or groin bays; simulation of two-dimensional wave driven flows and associated sediment transport patterns in the vicinity of breakwaters (submerged or surface piercing); simulation of the complex flow and sediment transport processes at tidal inlets and navigation channels etc.

The detailed two-dimensional flow and sand transport maps calculated with MIKE 21 can be used to optimise the planform of proposed coastal harbours in order to minimise sedimentation at the entrance. It can also be used to optimise the planform of coastal protection measures, in order to reduce adverse impacts due to the complex 2-dimensional flow and sediment transport pattern.

INTEGRATED APPLICATION OF MIKE 21 and LITPACK

In several coastal projects, both MIKE 21 and LITPACK are used in a complementary manner. LITPACK is used to evaluate the overall sediment budget and the long-term (tens of years) impact of various shore protection options, assuming the coastline is quasi-uniform. Thereafter, a few selected options are investigated in detail to evaluate the 2D waves, flow and sediment transport patterns in the study area. In this case, LITPACK is used to select a number of characteristic storms (for the year or season) important for coastal sediment transport. MIKE 21 is then used to simulate these characteristic storms (for the year or season), and the results integrated to obtain the seasonal/yearly sediment transport. The two results are used in combination to determine the 'best' option.

MIKE 21 CAMS

Until recently, application of coastal area morphological models has been limited to a few coastal laboratories. The release of MIKE 21 CAMS in the latest version of MIKE 21 software package (Release 2001) now makes this technology available to professional practising engineers.

MIKE 21 CAMS is a coastal area morphological modelling shell based on MIKE 21 modules. It integrates the waves, flow and sediment transport models into a full morphological model to model the time-evolution of bed level changes at a given coastal area. MIKE 21 CAMS includes full feedback of the bed level changes on the waves and flow calculations. Hence, it is a truly dynamic coastal area morphological model.

MIKE 21 CAMS is used for investigating the morphological evolution of coastal areas due to the impact of engineering works (coastal structures, dredging works etc.) in areas subjected to the action of waves and/or current. It is most suitable for medium-term morphological investigations (several weeks to months) over a limited coastal area. Typical dimensions are about 10km in the alongshore direction and 2km in the offshore direction. The computational effort can become quite large for long-term simulations (several years), or for larger areas.

For more information on MIKE 21, see:
http://www.dhisoftware.com/mike21/Description/index.htm

MIKE INFO COAST

MIKE INFO COAST is a GIS based data management tool that is used to integrate and visualise coastal data in a way that is easily accessible to coastal managers and decision makers. It is a powerful tool for the management of all relevant data monitoring of the morphological evolution in the coastal zone, for instance in connection with evaluation of the impact of nourishment schemes, harbours and coastal protection works.

MIKE INFO Coast (MICoast) is application software built within a Geographic Information System (GIS) specially designed for shoreline management applications.

It is a powerful tool for the management of most relevant data involved in the monitoring of the morphological evolution in the coastal zone. This can be for instance in connection with evaluation of the impact from shore protection schemes, harbours or other coastal engineering works.

MICoast 2001 is used to:

- Store various data used in coastal engineering applications in a project based data structure. This makes it easy for engineers and managers to quickly obtain an overview of the data available at different locations for the project site.
- Easily access the stored data and obtain additional information and knowledge about the shoreline using facilities available in MICoast. This includes displaying the various shoreline types (colour coded), obtaining transects from surveys, calculation of accretion and erosion from shoreline data, displaying oblique photographs analysis and display of time series data with the Timeseries toolbox, etc.

For more information on MIKE INFO Coast, see:
http://www.dhisoftware.com/mikeinfo/Description/index.htm

Contact:

For further information please contact:

DHI Software Support Centre

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MIKE 21 WAVE MODELLING

MIKE 21 SW - Spectral Waves FM

Short Description

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Fax: +45 4516 9292

E-mail: dhi@dhi.dk

Web: www.dhi.dk

MIKE21_SW_FM_Short_Description.doc/HKH/MSD0412.lsm - 2004-12-16

MIKE 21 SW - SPECTRAL WAVE MODEL FM

MIKE 21 SW is a state-of-the-art third generation spectral wind-wave model developed by DHI Water & Environment. The model simulates the growth, decay and transformation of wind-generated waves and swell in offshore and coastal areas.

MIKE 21 SW includes two different formulations:

- Fully spectral formulation
- Directional decoupled parametric formulation

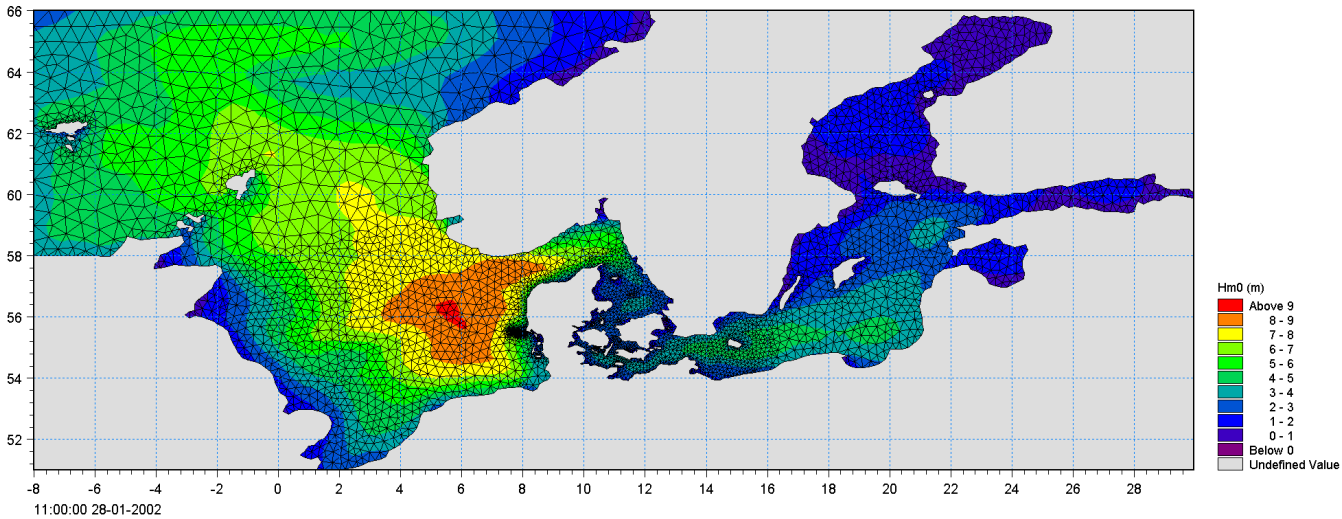
The fully spectral formulation is based on the wave action conservation equation, as described in e.g. Komen et al (1994) and Young (1999). The directional decoupled parametric formulation is based on a parameterisation of the wave action conservation equation. The parameterisation is made in the frequency domain by introducing the zeroth and first moment of the wave action spectrum. The basic conservation equations are formulated in either Cartesian co-ordinates for small-scale applications and polar spherical co-ordinates for large-scale applications.

The fully spectral model includes the following physical phenomena:

- Wave growth by action of wind
- Non-linear wave-wave interaction
- Dissipation due to white-capping
- Dissipation due to bottom friction
- Dissipation due to depth-induced wave breaking
- Refraction and shoaling due to depth variations
- Wave-current interaction
- Effect of time-varying water depth
- Effect of ice coverage on the wave field

The discretisation of the governing equation in geographical and spectral space is performed using cell-centred finite volume method. In the geographical domain, an unstructured mesh technique is used. The time integration is performed using a fractional step approach where a multi-sequence explicit method is applied for the propagation of wave action.

MIKE 21 SW is a state-of-the-art numerical modelling tool for prediction and analysis of wave climates in offshore and coastal areas. © BIOFOTO/Klaus K. Bentzen



A MIKE 21 SW forecast application in the North Sea and Baltic Sea. The chart shows a wave field (from the NSBS model) illustrated by the significant wave height in top of the computational mesh. See also www.waterforecast.com

Computational Features

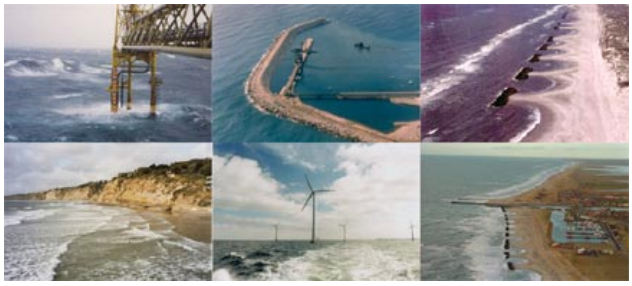
The main computational features of MIKE 21 SW - Spectral Wave Model FM are as follows:

- Fully spectral and directionally decoupled parametric formulations
- Source functions based on state-of-the-art 3rd generation formulations
- Instationary and quasi-stationary solutions
- Optimal degree of flexibility in describing bathymetry and ambient flow conditions using depth-adaptive and boundary-fitted unstructured mesh
- Coupling with hydrodynamic flow model for modelling of wave-current interaction and time-varying water depth
- Flooding and drying in connection with time-varying water depths
- Cell-centred finite volume technique
- Fractional step time-integration with an multi-sequence explicit method for the propagation
- Extensive range of model output parameters (wave, swell, air-sea interaction parameters, radiation stress tensor, spectra, etc.)

Application Areas

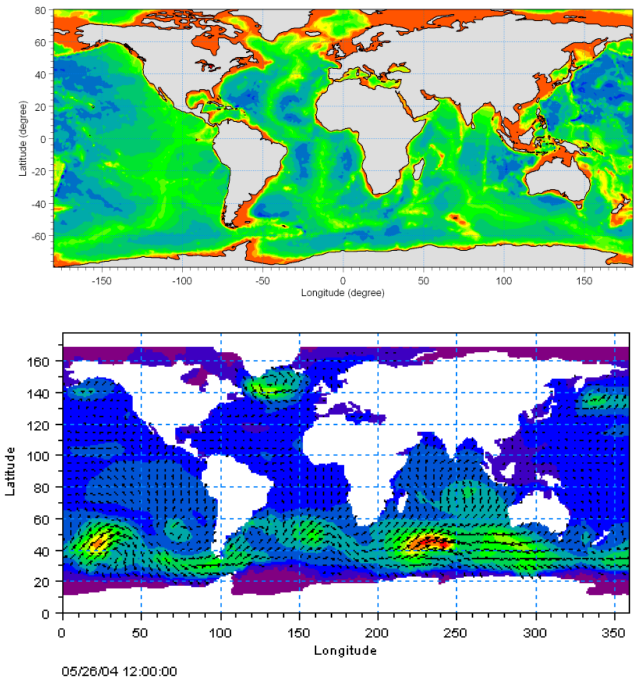
MIKE 21 SW is used for the assessment of wave climates in offshore and coastal areas - in hindcast and forecast mode.

A major application area is the design of offshore, coastal and port structures where accurate assessment of wave loads is of utmost importance to the safe and economic design of these structures.



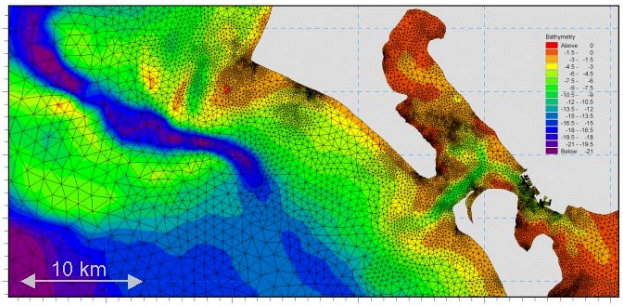
Illustrations of typical application areas of DHI's MIKE 21 SW – Spectral Wave Model FM

Measured data are often not available during periods long enough to allow for the establishment of sufficiently accurate estimates of extreme sea states. In this case, the measured data can then be supplemented with hindcast data through the simulation of wave conditions during historical storms using MIKE 21 SW.



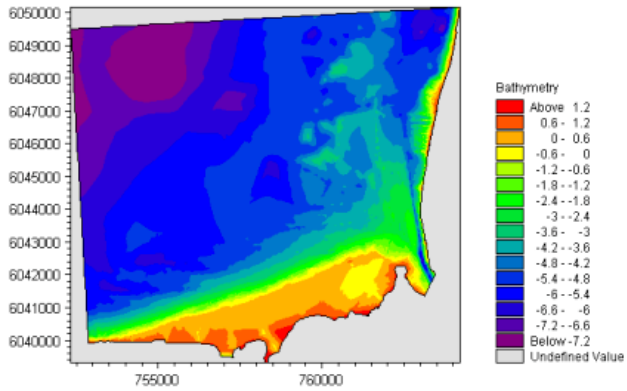
Example of a global application of MIKE 21 SW. The upper panel shows the bathymetry. Results from such a model (cf. lower panel) can be used as boundary conditions for regional scale forecast or hindcast models. See http://www.waterforecast.com for more details on regional and global modelling

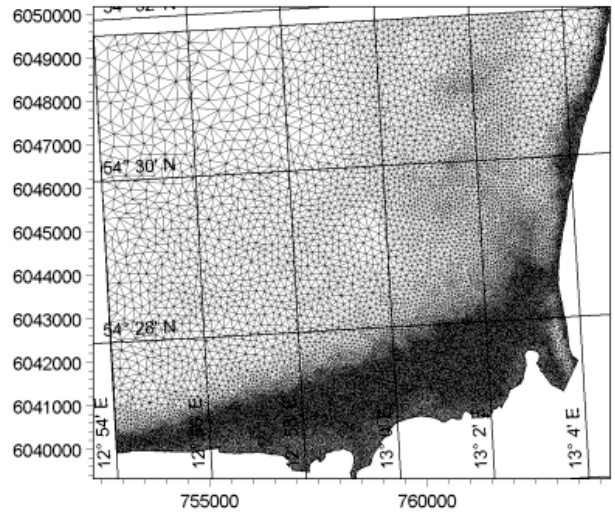
MIKE 21 SW is particularly applicable for simultaneous wave prediction and analysis on regional scale and local scale. Coarse spatial and temporal resolution is used for the regional part of the mesh and a high-resolution boundary and depth-adaptive mesh is describing the shallow water environment at the coastline.



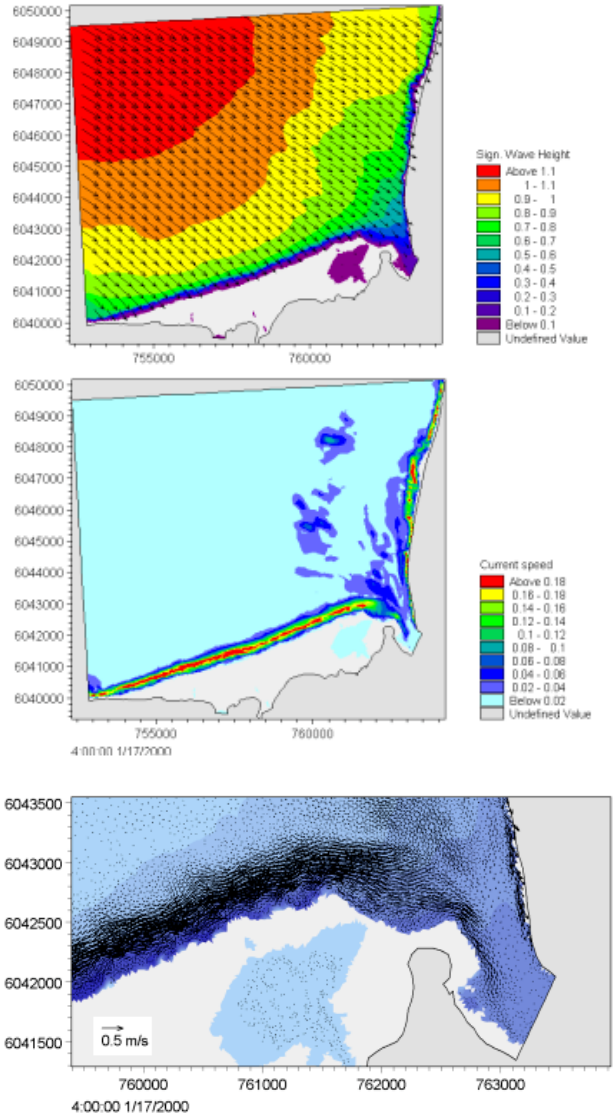
Example of a computational mesh used for transformation of offshore wave statistics using the directionally decoupled parametric formulation

MIKE 21 SW is also used for the calculation of the sediment transport, which for a large part is determined by wave conditions and associated wave-induced currents. The wave-induced current is generated by the gradients in radiation stresses that occur in the surf zone. MIKE 21 SW can be used to calculate the wave conditions and associated radiation stresses. The long-shore currents and sediment transport are then calculated using the flow and sediment transport models available in the MIKE 21 package. For such type of applications, the directional decoupled parametric formulation of MIKE 21 SW is an excellent compromise between the computational effort and accuracy.





Bathymetry (upper) and computational mesh (lower) used in a MIKE 21 SW application on wave induced currents in Gellen Bay, Germany



Map of significant wave height (upper), current field (middle) and vector field (lower). The flow field is simulated by DHI's MIKE 21 Flow Model FM, which is dynamically coupled to MIKE 21 SW

Model Equations

In MIKE 21 SW, the wind waves are represented by the wave action density spectrum $N(\sigma,\theta)$. The independent phase parameters have been chosen as the relative (intrinsic) angular frequency, $\sigma = 2\pi f$ and the direction of wave propagation, θ. The relation between the relative angular frequency and the absolute angular frequency, ω, is given by the linear dispersion relationship

$$\sigma = \sqrt{gk\tanh(kd)} = \omega - \bar{k}\cdot\bar{U}$$

where g is the acceleration of gravity, d is the water depth and $\bar{U}$ is the current velocity vector and $\bar{k}$ is the wave number vector with magnitude k and direction θ. The action density, $N(\sigma,\theta)$, is related to the energy density $E(\sigma,\theta)$ by

$$N = \frac{E}{\sigma}$$

Fully Spectral Formulation

The governing equation in MIKE 21 SW is the wave action balance equation formulated in either Cartesian or spherical co-ordinates. In horizontal Cartesian co-ordinates, the conservation equation for wave action reads

$$\frac{\partial N}{\partial t} + \nabla\cdot(\bar{v}N) = \frac{S}{\sigma}$$

where $N(\bar{x},\sigma,\theta,t)$ is the action density, t is the time, $\bar{x} = (x,y)$ is the Cartesian co-ordinates, $\bar{v} = (c_x, c_y, c_\sigma, c_\theta)$ is the propagation velocity of a wave group in the four-dimensional phase space $\bar{x}$, σ and θ. S is the source term for energy balance equation. ∇ is the four-dimensional differential operator in the $\bar{x}$, σ, θ-space. The characteristic propagation speeds are given by the linear kinematic relationships

$$(c_x, c_y) = \frac{d\bar{x}}{dt} = \bar{c}_g + \bar{U} = \frac{1}{2}\left(1 + \frac{2kd}{\sinh(2kd)}\right)\frac{\sigma}{k} + \bar{U}$$

$$c_\sigma = \frac{d\sigma}{dt} = \frac{\partial\sigma}{\partial d}\left[\frac{\partial d}{\partial t} + \bar{U}\cdot\nabla_{\bar{x}} d\right] - c_g\bar{k}\cdot\frac{\partial\bar{U}}{\partial s}$$

$$c_\theta = \frac{d\theta}{dt} = -\frac{1}{k}\left[\frac{\partial\sigma}{\partial d}\frac{\partial d}{\partial m} + \bar{k}\cdot\frac{\partial\bar{U}}{\partial m}\right]$$

Here, s is the space co-ordinate in wave direction θ and m is a co-ordinate perpendicular to s. $\nabla_{\bar{x}}$ is the two-dimensional differential operator in the $\bar{x}$-space.

Source Functions

The source function term, S, on the right hand side of the wave action conservation equation is given by

$$S = S_{in} + S_{nl} + S_{ds} + S_{bot} + S_{surf}$$

Here S_{in} represents the momentum transfer of wind energy to wave generation, S_{nl} the energy transfer due non-linear wave-wave interaction, S_{ds} the dissipation of wave energy due to white-capping (deep water wave breaking), S_{bot} the dissipation due to bottom friction and S_{surf} the dissipation of wave energy due to depth-induced breaking.

The default source functions S_{in}, S_{nl} and S_{ds} in MIKE 21 SW are similar to the source functions implemented in the WAM Cycle 4 model, see Komen et al (1994).

The wind input is based on Janssen's (1989, 1991) quasi-linear theory of wind-wave generation, where the momentum transfer from the wind to the sea not only depends on the wind stress, but also the sea state itself. The non-linear energy transfer (through the resonant four-wave interaction) is approximated by the DIA approach, Hasselmann et al (1985). The source function describing the dissipation due to white-capping is based on the theory of Hasselmann (1974) and Janssen (1989). The bottom friction dissipation is modelled using the approach by Johnson and Kofoed-Hansen (2000), which depends on the wave and sediment properties. The source function describing the bottom-induced wave breaking is based on the well-proven approach of Battjes and Janssen (1978) and Eldeberky and Battjes (1996).

A detailed description of the various source functions is available in Komen et al (1994) and Sørensen et al (2003), which also includes the references listed above.

Directional Decoupled Parametric Formulation

The directionally decoupled parametric formulation is based on a parameterisation of the wave action conservation equation. Following Holthuijsen et al (1989), the parameterisation is made in the frequency domain by introducing the zeroth and first moment of the wave action spectrum as dependent variables.

A similar formulation is used in the MIKE 21 NSW Near-shore Spectral Wind-Wave Model, which is one of the most popular models for wave transformation in coastal and shallow water environment. However, with MIKE 21 SW it is not necessary to set up a number of different orientated bathymetries to cover varying wind and wave directions.

The parameterisation leads to the following coupled equations

$$\frac{\partial(m_0)}{\partial t}+\frac{\partial(c_x m_0)}{\partial x}+\frac{\partial(c_y m_0)}{\partial y}+\frac{\partial(c_\theta m_0)}{\partial \theta}=T_0$$

$$\frac{\partial(m_1)}{\partial t}+\frac{\partial(c_x m_1)}{\partial x}+\frac{\partial(c_y m_1)}{\partial y}+\frac{\partial(c_\theta m_1)}{\partial \theta}=T_1$$

where $m_0(x, y, \theta)$ and $m_1(x, y, \theta)$ are the zeroth and first moment of the action spectrum $N(x, y, \sigma, \theta)$, respectively. $T_0(x, y, \theta)$ and $T_1(x, y, \theta)$ are source functions based on the action spectrum. The moments $m_n(x, y, \theta)$ are defined as

$$m_n(x,y,\theta)=\int_0^\infty \omega^n N(x,y,\omega,\theta)d\omega$$

The source functions T_0 and T_1 take into account the effect of local wind generation (stationary solution mode only) and energy dissipation due to bottom friction and wave breaking. The effects of wave-current interaction are also included. The source functions for the local wind generation are derived from empirical growth relations, see Johnson (1998) for details.

Numerical Methods

The frequency spectrum (fully spectral model only) is split into a prognostic part for frequencies lower than a cut-off frequency σ_{max} and an analytical diagnostic tail for the high-frequency part of the spectrum

$$E(\sigma,\theta)=E(\sigma_{\max},\theta)\left(\frac{\sigma}{\sigma_{\max}}\right)^{-m}$$

where m is a constant (= 5) as proposed by Komen et al (1994).

The directional decoupled parametric formulation in MIKE 21 SW is used extensively for calculation of the wave transformation from deep-water to the shoreline and for wind-wave generation in local areas

Space Discretisation

The discretisation in geographical and spectral space is performed using cell-centred finite volume method. In the geographical domain an unstructured mesh is used. The spatial domain is discretised by subdivision of the continuum into non-overlapping elements. Triangle and quadrilateral shaped polygons are presently supported in MIKE 21 SW. The action density, $N(\sigma,\theta)$ is represented as a piecewise constant over the elements and stored at the geometric centres.

In frequency space either an equidistant or a log-arithmic discretisation is used. In the directional space, an equidistant discretisation is used for both types of models. The action density is represented as piecewise constant over the discrete intervals, $\Delta\sigma$ and $\Delta\theta$, in the frequency and directional space.

Integrating the wave action conservation over an area A_i, the frequency interval $\Delta\sigma_l$ and the directional interval $\Delta\theta_m$ gives

$$\frac{\partial}{\partial t}\int_{\Delta\theta_m}\int_{\Delta\sigma_l}\int_{A_i} N d\Omega d\sigma d\theta - \int_{\Delta\theta_m}\int_{\Delta\sigma_l}\int_{A_i}\frac{S}{\sigma} d\Omega d\sigma d\theta$$
$$= \int_{\Delta\theta_m}\int_{\Delta\sigma_l}\int_{A_i}\nabla\cdot(\bar{v}N) d\Omega d\sigma d\theta$$

where Ω is the integration variable defined on A_i. Using the divergence theorem and introducing the convective flux $\bar{F} = \bar{v}N$, we obtain

$$\frac{\partial N_{i,l,m}}{\partial t} = -\frac{1}{A_i}\left[\sum_{p=1}^{NE}(F_n)_{p,l,m}\Delta l_p\right]$$
$$-\frac{1}{\Delta\sigma_l}\left[(F_\sigma)_{i,l+1/2,m} - (F_\sigma)_{i,l-1/2,m}\right]$$
$$-\frac{1}{\Delta\theta_m}\left[(F_\theta)_{i,l,m+1/2} - (F_\theta)_{i,l,m-1/2}\right] + \frac{S_{i,l,m}}{\sigma_l}$$

where NE is the total number of edges in the cell, $(F_n)_{p,l,m} = (F_x n_x + F_y n_y)_{p,l,m}$ is the normal flux trough the edge p in geographical space with length Δl_p. $(F_\sigma)_{i,l+1/2,m}$ and $(F_\theta)_{i,l,m+1/2}$ is the flux through the face in the frequency and directional space, respectively.

The convective flux is derived using a first-order upwinding scheme. In that

$$F_n = c_n\left(\frac{1}{2}(N_i + N_j) - \frac{1}{2}\frac{c}{|c|}(N_i - N_j)\right)$$

where c_n is the propagation speed normal to the element cell face.

Time Integration

The integration in time is based on a fractional step approach. Firstly, a propagation step is performed calculating an approximate solution N^* at the new time level (n+1) by solving the homogenous wave action conservation equation, i.e. without the source terms. Secondly, a source terms step is performed calculating the new solution N^{n+1} from the estimated solution taking into account only the effect of the source terms.

The propagation step is carried out by an explicit Euler scheme

$$N^*_{i,l,m} = N^n_{i,l,m} + \Delta t\left(\frac{\partial N_{i,l,m}}{\partial t}\right)^n$$

To overcome the severe stability restriction, a multi-sequence integration scheme is employed. The maximum allowed time step is increased by employing a sequence of integration steps locally, where the number of steps may vary from point to point.

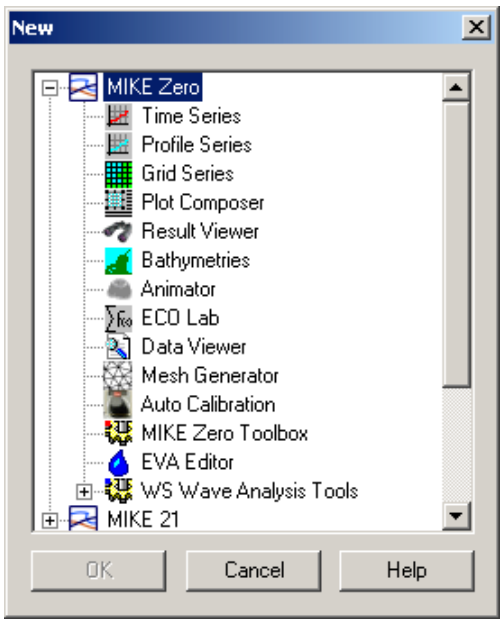
A source term step is performed using an implicit method (see Komen et al, 1994)

$$N^{n+1}_{i,l,m} = N^*_{i,l,m} + \Delta t\left[\frac{(1-\alpha)S^*_{i,l,m} + \alpha S^{n+1}_{i,l,m}}{\sigma_l}\right]$$

where α is a weighting coefficient that determines the type of finite difference method. Using a Taylor series to approximate S^{n+1} and assuming the off-diagonal terms in $\partial S/\partial E = \gamma$ are negligible, this equation can be simplified as

$$N^{n+1}_{i,l,m} = N^n_{i,l,m} + \frac{(S^*_{i,l,m}/\sigma_l)\Delta t}{(1-\alpha\gamma\Delta t)}$$

For growing waves ($\gamma > 0$) an explicit forward difference is used ($\alpha = 0$), while for decaying waves ($\gamma < 0$) an implicit backward difference ($\alpha = 1$) is applied.



Overview of the common shell MIKE Zero containing entries for common data file editors, plotting facilities and a toolbox for tools/utilities as the Mesh Generator and Data Viewer

Model Input

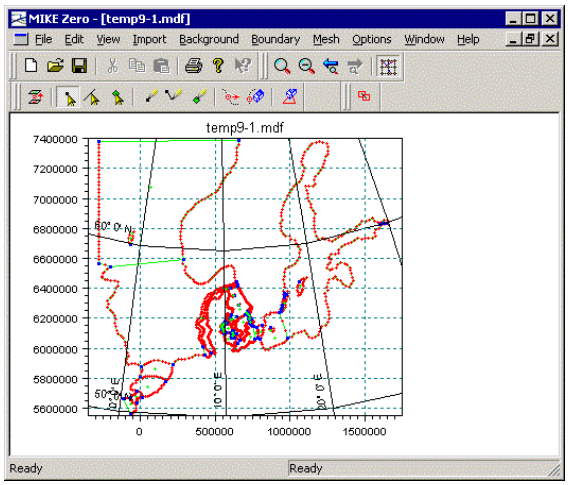
The necessary input data can be divided into following groups:

- Domain and time parameters:
  - computational mesh
  - co-ordinate type (Cartesian or spherical)
  - simulation length and overall time step
- Equations, discretisation and solution technique
  - formulation type
  - frequency and directional discretisation
  - number of time step groups
  - number of source time steps
- Forcing parameters
  - water level data
  - current data
  - wind data
  - ice data
- Source function parameters
  - non-linear energy transfer
  - wave breaking (shallow water)
  - bottom friction
  - white capping
- Initial conditions
  - zero-spectrum (cold-start)
  - empirical data
  - data file
- Boundary conditions
  - closed boundaries
  - open boundaries (data format and type)

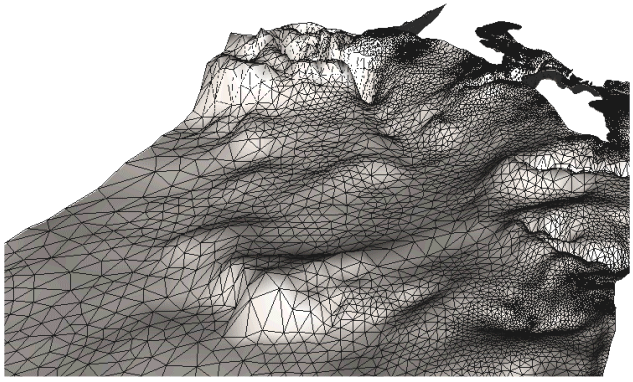
Providing MIKE 21 SW with a suitable mesh is essential for obtaining reliable results from the model. Setting up the mesh includes the appropriate selection of the area to be modelled, adequate resolution of the bathymetry, flow, wind and wave fields under consideration and definition of codes for essential and land boundaries.

Furthermore, the resolution in the geographical space must also be selected with respect to stability considerations.

As the wind is the main driving force in MIKE 21 SW, accurate hindcast or forecast wind fields are of utmost importance for the wave prediction.

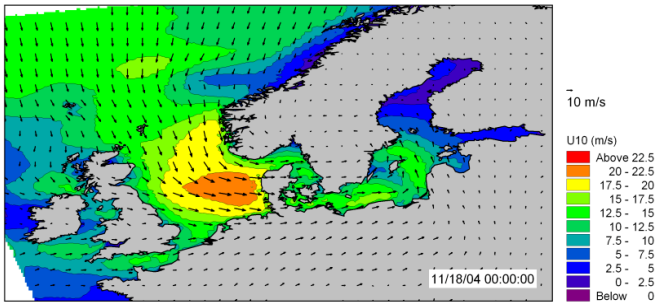


The Mesh Generator is an efficient MIKE Zero tool for the generation and handling of unstructured meshes, including the definition and editing of boundaries



3D visualisation of a computational mesh

If wind data is not available from an atmospheric meteorological model, the wind fields (e.g. cyclones) can be determined by using the wind-generating programs available in MIKE 21 Toolbox.

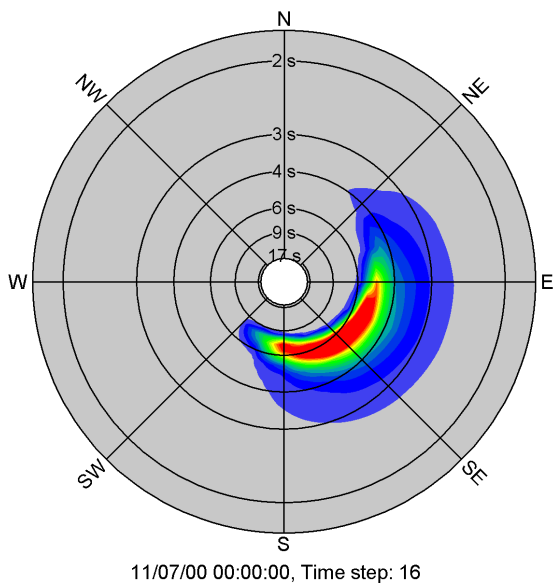


The chart shows an example of a wind field covering the North Sea and Baltic Sea as wind speed and wind direction. This is used as input to MIKE 21 SW in forecast and hindcast mode

Model Output

At each mesh point and for each time step four types of output can be obtained from MIKE 21 SW:

- Integral wave parameters divided into wind sea and swell such as
  - significant wave height, H_{m0}
  - peak wave period, T_p
  - averaged wave period, T_{01}
  - zero-crossing wave period, T_{02}
  - wave energy period, T_{-10}
  - peak wave direction, θ_p
  - mean wave direction, θ_m
  - directional standard deviation, σ
  - wave height with dir., $H_{m0} \cos\theta_m$, $H_{m0} \sin\theta_m$
  - radiation stress tensor, S_{xx}, S_{xy} *and* S_{yy}



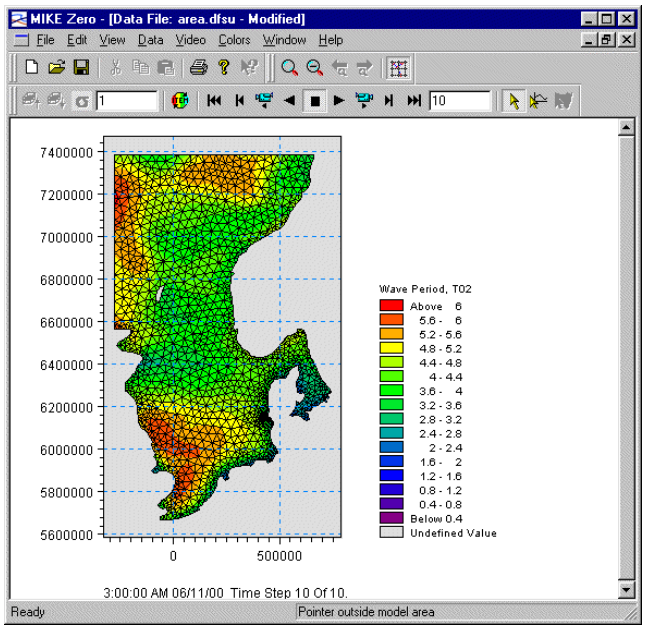
Example of model output (directional-frequency wave spectrum) processed using the Polar Plot control in the MIKE Zero Plot Composer

The distinction between wind-sea and swell can be calculated using either a constant threshold frequency or a dynamic threshold frequency with an upper frequency limit.

- Input parameters
  - water level, h
  - current velocity, $\overline{U}$
  - wind speed, U_{10}
  - wind direction, θ_w
- Model parameters
  - bottom friction coefficient, C_f
  - breaking parameter, γ
  - Courant number, Cr
  - time step factor, α
  - characteristic edge length, Δl
  - area of element, a
  - wind friction speed, u_*
  - roughness length, z_0
  - drag coefficient, C_D
  - Charnock parameter, z_{ch}
- Directional-frequency wave spectra at selected grid points and or areas as well as direction spectra and frequency spectra

Output from MIKE 21 SW is typically post-processed using the Data Viewer available in the common MIKE Zero shell. The Data Viewer is a tool for analysis and visualisation of unstructured data, e.g. to view meshes, spectra, bathymetries, results files of different format with graphical extraction of time series and line series from plan view and import of graphical overlays.

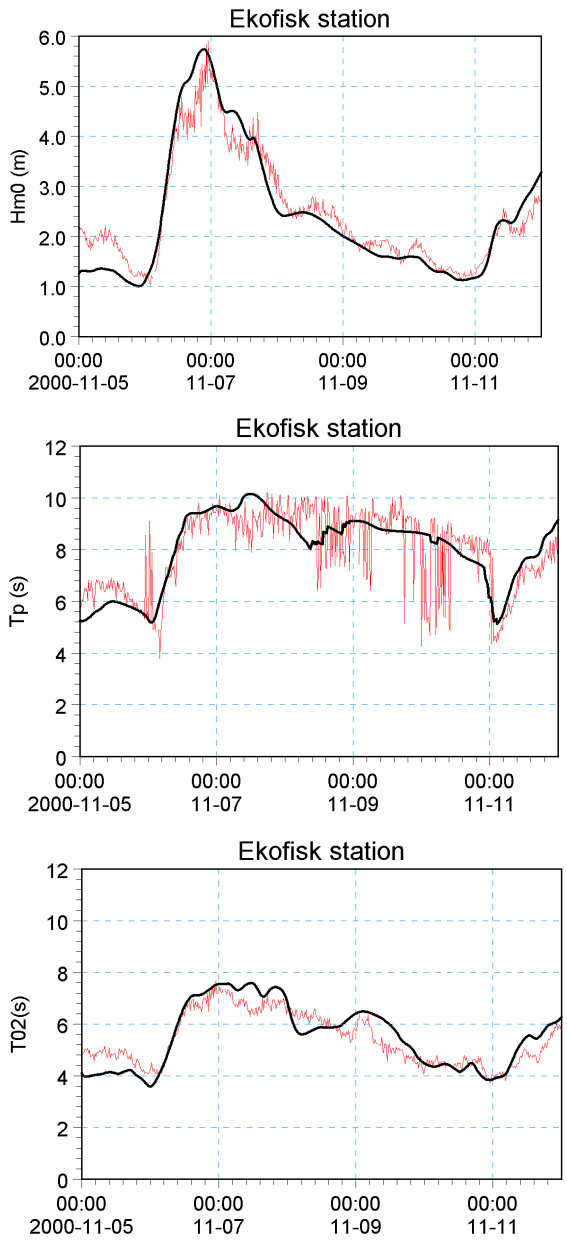
Various other editors and plot controls in the MIKE Zero Composer (e.g. Time Series Plot, Polar Plot, etc.) can be used for analysis and visualisation.



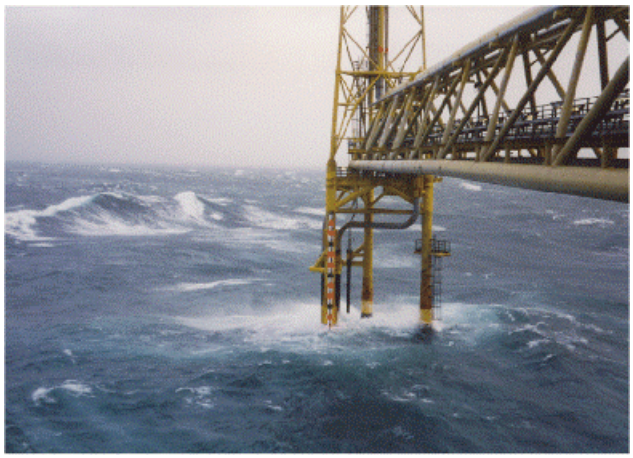
The Data Viewer in MIKE Zero – an efficient tool for analysis and visualisation of unstructured data including processing of animations

Validation

The model has successfully been applied to a number of rather basic idealised situations for which the results can be compared with analytical solutions or information from the literature. The basic tests covered fundamental processes such as wave propagation, depth-induced and current-induced shoaling and refraction, wind-wave generation and dissipation.

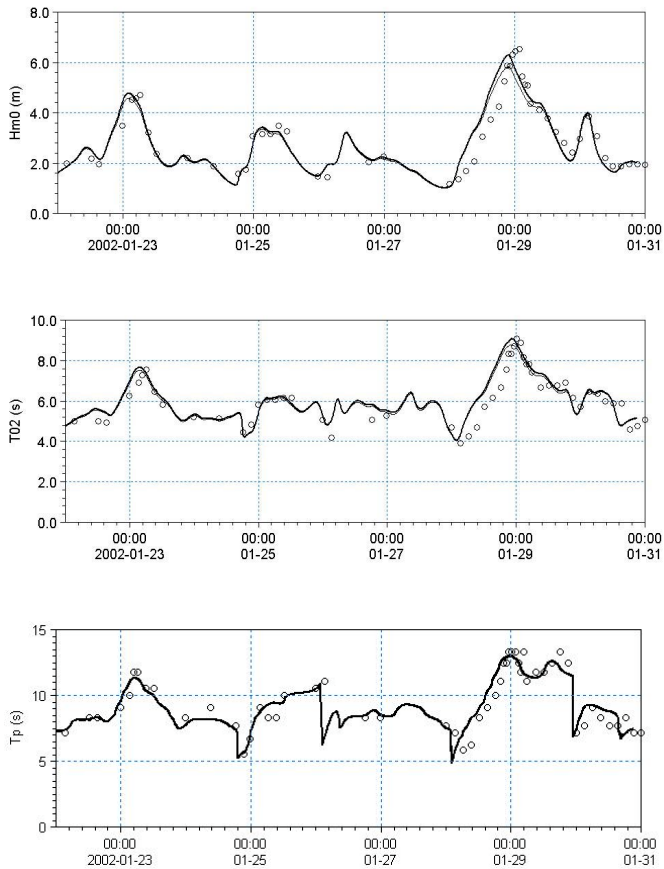


Comparison between measured and simulated significant wave height, peak wave period and mean wave period at the Ekofisk offshore platform (water depth 70 m) in the North Sea). (——) calculations and (——) measurements



A major application area of MIKE 21 SW is in connection with design and maintenance of offshore structures

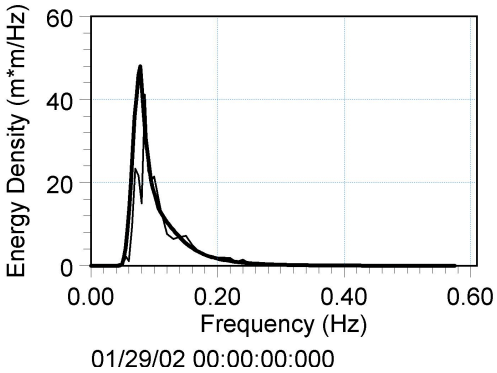
The model has also been tested in natural geophysical conditions (e.g. in the North Sea, the Danish West Coast and the Baltic Sea), which are more realistic and complicated than the academic test and laboratory tests mentioned above.

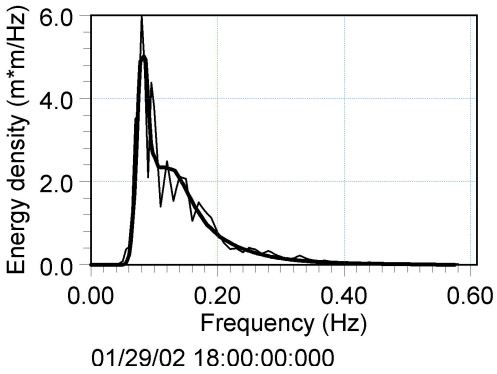


Comparison between measured and simulated significant wave height, peak wave period and mean wave period at Fjaltring located at the Danish west coast (water depth 17.5 m).(——) calculations and (o) measurements

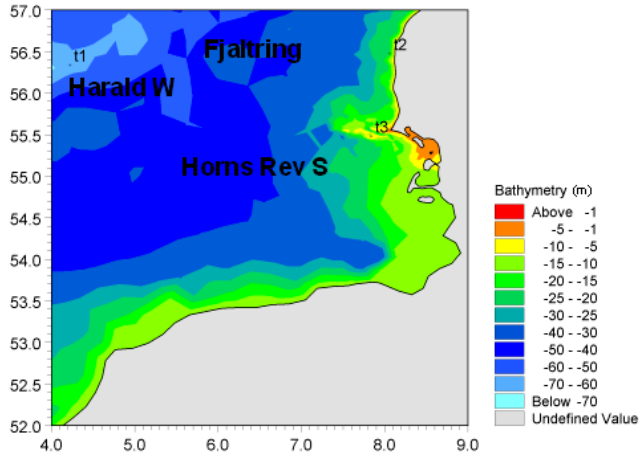
The Fjaltring directional wave rider buoy is located offshore relative to the depicted arrow

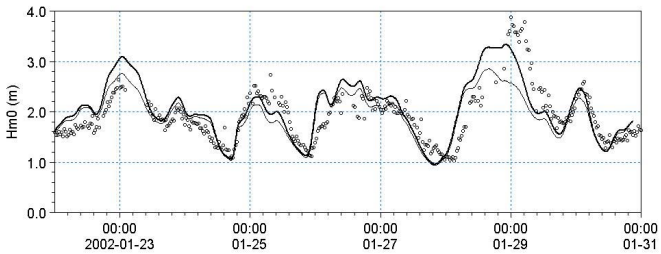
MIKE 21 SW is used for prediction of the wave conditions at the complex Horns Rev (reef) in the southeastern part of the North Sea. At this site, a 168 MW offshore wind farm with 80 turbines has been established in water depths between 6.5 and 13.5 m.





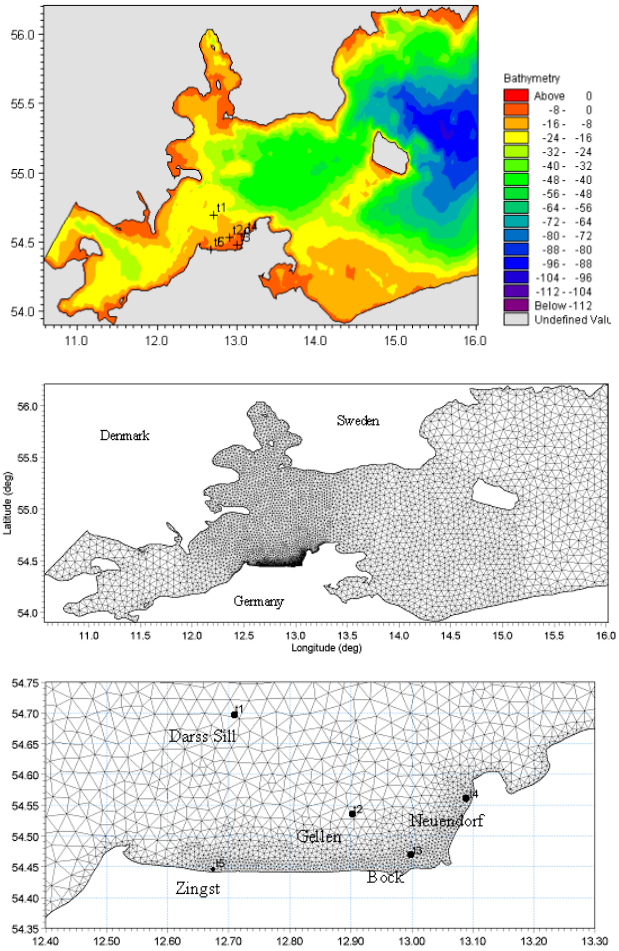
Comparison of frequency spectra at Fjaltring. (——) calculations and (—-)measurements



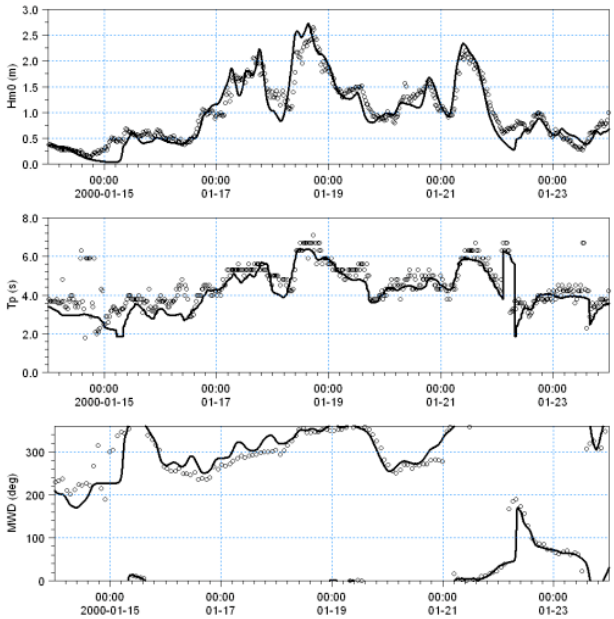


The upper panels show the Horns Rev offshore wind farm and MIKE C-map chart. 'The middle panel shows a close-up of the mesh near the Horns Rev S wave rider buoy (t3, 10 m water depth. The lower panel shows a comparison between measured and simulated significant wave height at Horns Rev S, (——) calculations including tide and surge and (—-) calculations excluding including tide and surge, (o) measurements

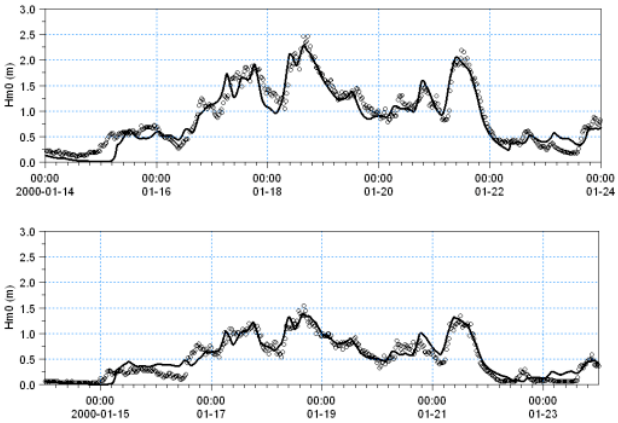
The predicted nearshore wave climate along the island of Hiddensee and the coastline of Zingst located in the microtidal Gellen Bay, Germany have been compared to field measurements (Sørensen et al, 2004) provided by the MORWIN project. From the illustrations it can be seen that the wave conditions are well reproduced both offshore and in more shallow water near the shore. The RMS values (on significant wave height) are less than 0.25m at all five stations.



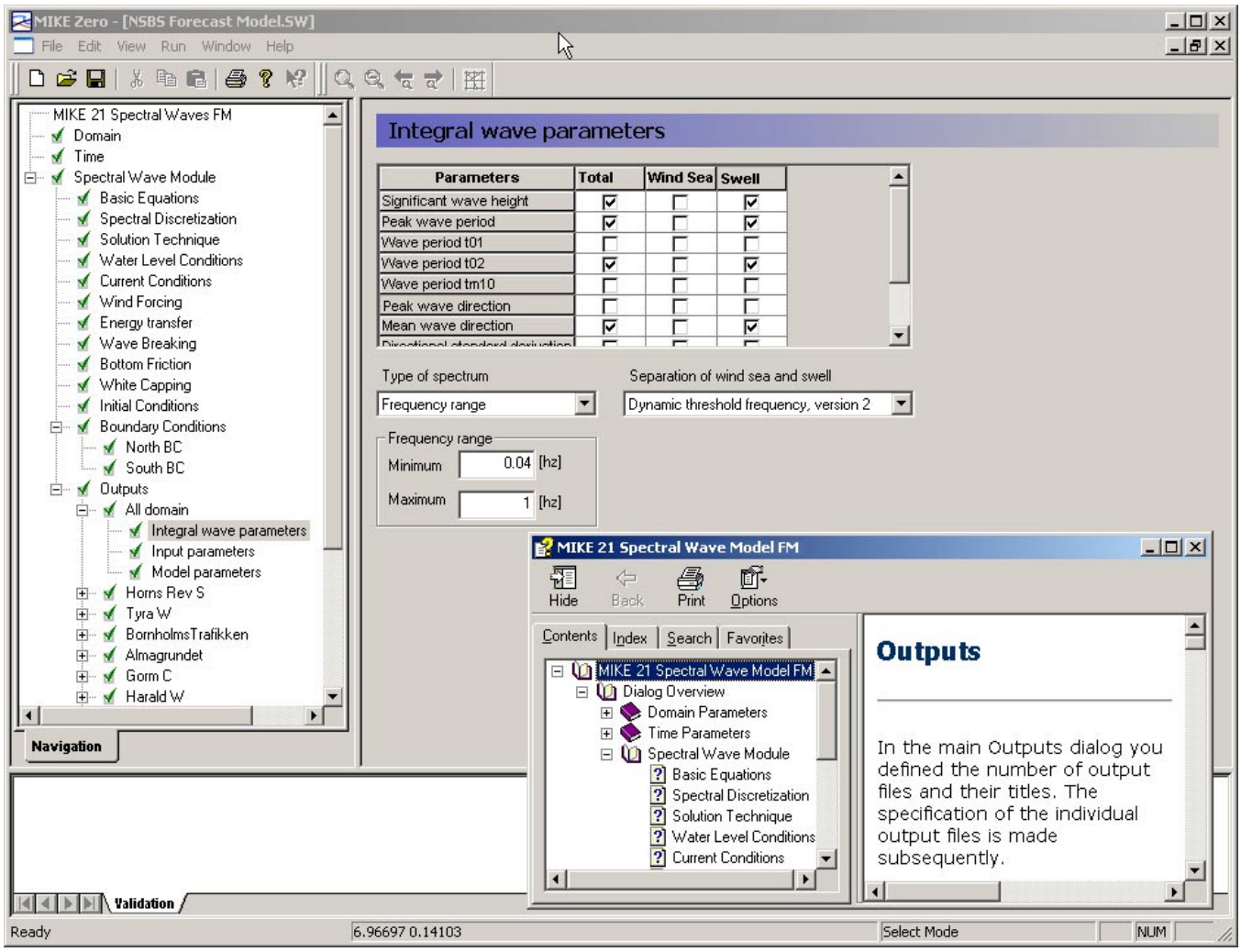
A MIKE 21 SW hindcast application in the Baltic Sea. The upper chart shows the bathymetry and the middle and lower charts show the computational mesh. The lower chart indicates the location of the measurement stations



Time series of significant wave height, H_{m0}, peak wave period, T_p , and mean wave direction, MWD, at Darss sill (Offshore, depth 20.5 m). (——) Calculation and (o) measurements. The RMS value on H_{m0} is approximately 0.2 m



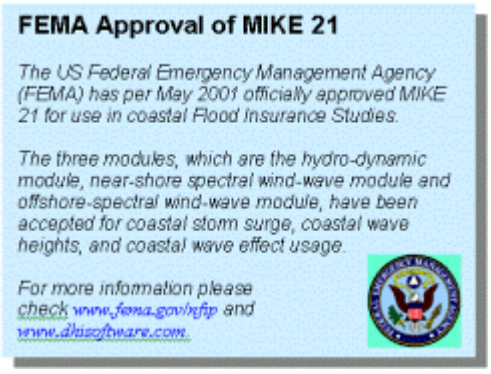
Time series of significant wave height, H_{m0}, at Gellen (upper, depth 8.3m) and Bock (lower, depth 5.5 m). (——) Calculation and (o) measurements. The RMS value on H_{m0} is approximately 0.15 m



The graphical user interface of the MIKE 21 SW model, including an example of the extensive Online Help system

Graphical User Interface

MIKE 21 SW is operated through a fully Windows integrated Graphical User Interface (GUI). Support is provided at each stage by an Online Help system.



FEMA approval of the MIKE 21 package

Hardware and Operating System Requirements

The module supports 2000/XP. Microsoft Internet Explorer 4.0 (or higher) is required for network license management as well as for accessing the Online Help.

The recommended minimum hardware requirements for executing MIKE 21SW are:

Processor:	Pentium, AMD or compatible processor; 2 GHz (or higher)
Memory (RAM):	512 MB (or higher)
Hard disk:	20 GB (or higher)
Monitor:	SVGA, resolution 1024x768
Graphic card:	32 MB RAM (or higher), 24 bit true colour
CD-ROM/DVD drive:	for installation of software

Support

News about new features, applications, papers, updates, patches, etc. are available at the MIKE 21 Website located at:

http://www.dhisoftware.com/mike21

For further information on MIKE 21 SW, please contact your local DHI Software agent or Software Support Centre at DHI:

DHI Software Support Centre
DHI Water & Environment
Agern Allé 5
DK-2970 Hørsholm
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Rugbjerg, M., Nielsen, K., Christensen, J.H. and Jacobsen, V., (2001). Wave energy in the Danish part of the North Sea. In: Proc of 4th European Wave Energy Conference, 8 pp.

MIKE 21 SW is also applied for wave forecasts in ship route planning and improved service for conventional and fast ferry operators

A P P E N D I X D

Dune Erosion Model Description

The rate of retreat of the dune front is determined by the continuity equation from the rate of loss of sediment from the dune front, Q_{dune}. During the time step of Δt the dune front retreats the distance ΔY_{dune} which is determined so that the change in volume equals the rate of loss

$$\Delta vol = Q_d \Delta t \qquad \text{Eq. (D-1)}$$

As the dune front retreats, the dune foot increases its level by moving back along a slope of $\tan(\alpha_{re})$. The actual beach slope $\tan(\alpha)$ is determined from the foot level FL and the difference in the position of the coastline and the dune foot:

$$\tan(\alpha) = (FL - MWL)/(Y - Y_{dune}) \qquad \text{Eq. (D-2)}$$

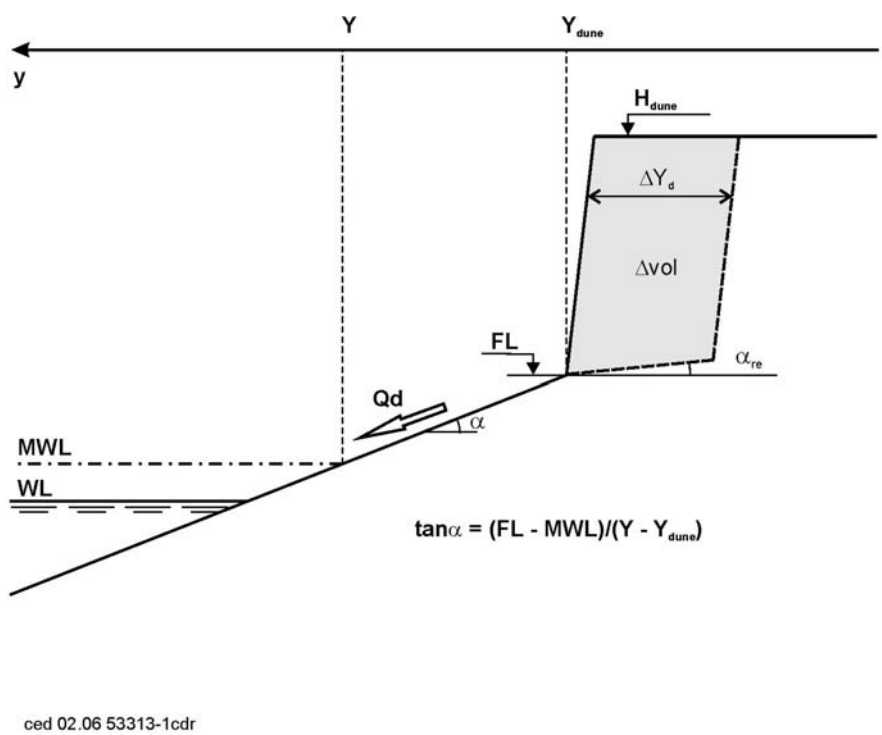

Figure 13-1 Definition sketch for the dune erosion model

The rate of loss of sediment from the dune is calculated from the run-up with formulae based on Larson et al. (2004). The maximum loss, corresponding to a situation with the dune foot level at the water level is found as

$$Q_{d0} = 4C_s R^2 / T \qquad \text{Eq. (D-3)}$$

C_s is a calibration factor; R is the wave run-up and T the wave period. The range of values for this calibration factor will be discussed later in the report. As the dune foot level is higher than the water level, the actual rate of sediment loss is found by applying a reduction factor:

$$R_{ed} = \exp(-2(FL - WL)/R) \qquad \text{Eq. (D-4)}$$

The wave run-up is estimated applying Hanslow and Nielsen (1995) formulation based on field measurements on natural beaches in Australia. The proposed formulation is:

$$R = 0.9\, H_s \tan\alpha \sqrt{L_0 / H_s} \qquad \text{(D-5)}$$

The rate of sediment loss from the dune front is caused by erosion of sand from the dune front and is therefore associated with the intensity of the wave run-up impinging on the front. The formulation of the erosion rate is based on Larson et al. (2004).

The run-up height above the instantaneous water level WL is determined from the model by van der Meer and Janssen (1994), and by Hanslow and Nielsen (1995).
The run-up height at a dike exceeded by 1 in 50 waves has been estimated van der Meer and Janssen as:

$$\frac{R_{2\%}}{H_{m0}} = 1.77 \times \gamma_f \gamma_b \gamma_\beta \xi_0 = 1.77\gamma \frac{\tan\alpha}{\sqrt{H_{m0} / L_0}} \qquad \text{Eq (D-6)}$$

Where $\gamma_f, \gamma_b, \gamma_\beta$ are reduction factors due to the presence of a berm, roughness of the slope and the wave angle. Due to the complexity of the present profile no attempt is made to assess the individual reduction factors, in stead all these factors have been lumped into single $\gamma -$factor. The Irribarren number ξ_0, (or surf-similarity parameter) giving a measure of the steepness of the beach compared to the wave steepness.

H_{m0} is the significant wave height near the toe of the dike and L_0 is the deep water wave length based on the spectral wave period (approx. 0.9Tp). For this analysis H_{m0} has been extracted at 5m depth AHD.

Adjustment to other run-up values may be obtained by changing the coefficient in eq. (3). For example is $R_{1/3}$ found with a coefficient of 1.31 instead of 1.77. A simple estimate of the run-up is therefore obtained from the offshore wave height H_s as

$$R = 1.77\gamma_r \tan\alpha \sqrt{H_s L_0} \qquad \text{Eq (D-7)}$$

with γ_r as a single calibration factor, usually defined as 0.7.

Hanslow and Nielsen (1995) proposed a run-up formulation based on field measurements on natural beaches in Australia. The proposed formulation is:

$$R = 0.9\, H_s \tan\alpha \sqrt{L_0 / H_s} \qquad \text{Eq (D-8)}$$

As it can be observed, the two formulations are very similar and the only difference is in the coefficient value. If we apply a calibration factor value of γ_r=0.7 in van der Meer's formulation, the resulting calibration coefficient is 1.24, compared to the recommended value of 0.9 by Hanslow and Nielsen. It should be noted, however, that there is some uncertainty in the definition of the wave heights to compare both formulations as Hanslow and Nielsen apply a scale factor to the offshore wave height. This scaling depends on the angle between the offshore waves and the orientation of the beach.

The rate of loss of sediment from the dune is calculated from the run-up with formulae based on Larson et al. (2004). The maximum loss, corresponding to a situation with the dune foot level at the water level is found as

$$Q_{d0} = 4C_s R^2 / T \tag{D-9}$$

C_s is a calibration factor. The range of values for this calibration factor will be discussed later in the report. As the dune foot level is higher than the water level, the actual rate of sediment loss is found by applying a reduction factor:

$$R_{ed} = \exp(-2(FL - WL)/R) \tag{D-10}$$

Giving a rate of dune loss of

$$Q_d = \exp(-2(FL - WL)/R)4C_s R^2 / T \tag{D-11}$$

Q_d is always positive corresponding to a retreating dune front. If the dune foot moves so far back and up that it reaches the level of the dune crest the rate of loss Qd is nearly zero. In that case there is no dune front to attack and if the waves are high enough they will cause overtopping.

A P P E N D I X E

Comments on the Present Hazard Line Predictions Compared to Previous Assessments

The approach adopted for the definition of the hazard zones within the current study as well as that of WBM (1998) is based on the quantification of shoreline fluctuations, long -term trends and the expected sea level rise due to greenhouse effects.

Shoreline fluctuations identified within the current study are of similar magnitude to that quantified by WBM (1998), however in the present study an additional allowance to address possible localised scouring at the vicinity of the seawall ends has been included, this allowance has been defined based on historical information of the area.

WBM associated these fluctuations with short term storm related process. The modelling undertaken in the current study, however clearly indicates the significance of variations in both wave direction and wave energy over time scales varying from weeks to years as key drivers for these processes. As a consequence they have been separated into short and medium term components. The former being related to variation in wave energy (storms) while the latter is the product of variations in wave direction and energy over periods of years.

Umwelt (2003) raised some concern that the magnitude of storm related erosion estimated by WBM (1998) may have been underestimated due to the effects of offshore steepening of the beach profile. This issue was examined using a nearshore storm cut model with various nearshore profiles. The results of this modelling indicate that if no corrective measures are undertaken the dune erosion risk will further increase by 5% for a further deepening of the nearshore areas of 1m. This estimate is less than that suggested by Umwelt (2003) due to the use of nearshore wave characteristics. The effect of beach profile steepening has been included in the 20 and 50 years hazard line definition with a small additional allowance for ongoing profile steepening.

The current study also identifies a similar rate of long term sediment loss to that identified by WBM (1998). In both studies higher rates of retreat are observed to the north of Stockton township. Modelling undertaken in the current study demonstrates that this is the product of imbalance in alongshore sediment transport with more sediment being transported northwards than is supplied. The predicted extent of future hazard associated with this trend is a point of difference between the current study and that of WBM (1998). WBM hazard line predictions were based on the assumption that the long term trend would continue unchanged for the next 20 years and reduce to 50% for the following 30 years reducing to zero thereafter. In the present study the computed rate of erosion has been considered to continue unchanged from now onto the next 50years. This assumption has been made on the basis of the modelling which suggests imbalance in sediment transport and analysis of historical data of the area that does not show any evidence of a reduction of the rate of erosion. This difference in the application of the long-term rate results in similar predictions for the immediate and 20 year hazard lines but a significant difference for the 50 year predictions.

Allowance for potential beach recession associated with predicted sea level rise also varies slightly between the current study and WBM due to different planning time frames and the inclusion of the additional data from Umwelt (2003) on profile change and depth of closure.