DESIGN AND ASSESSMENT OF PHYSICAL EFFECTS OF MULTI-PURPOSE REEFS FOR BEACH SAND RETENTION AT OREWA BEACH

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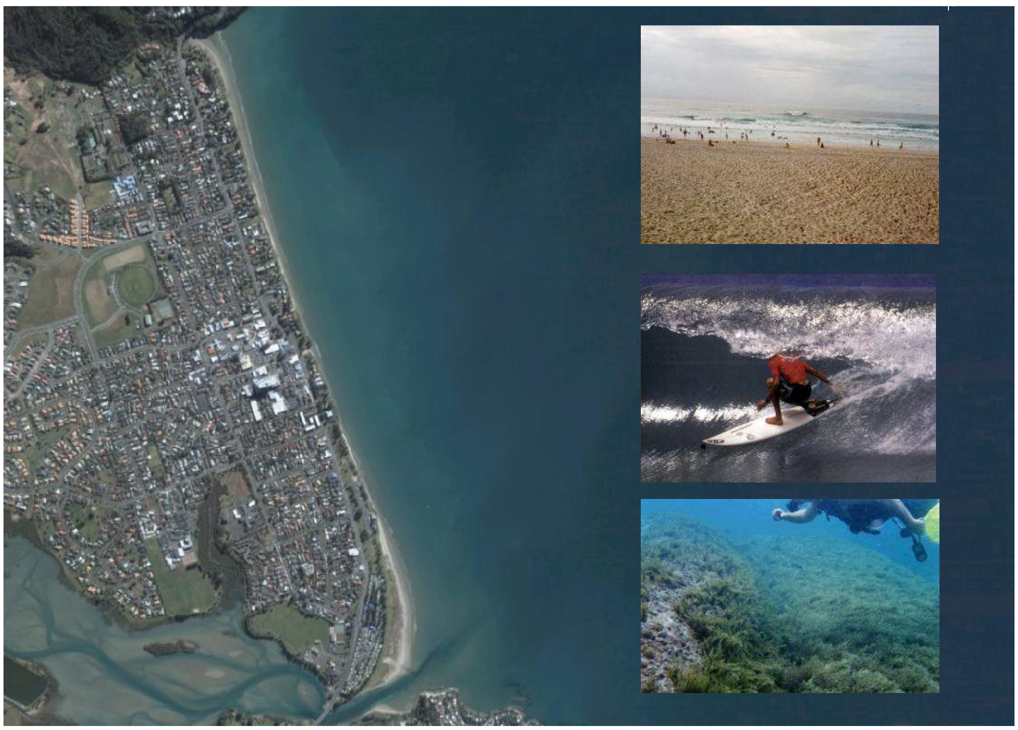

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OREWA BEACH REEFS

Design and Assessment of Physical Effects of Multi-Purpose Reefs for Beach Sand Retention at Orewa Beach

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Prepared for the Orewa Beach Reef Charitable Trust (OBRCT) and Rodney District Council (RDC)

FOREWORD

This technical report supports Resource Consent applications for the construction of submerged breakwaters, or multi-purpose reefs, at Orewa Beach which will modify and dissipate incident wave energy resulting in a reduction of erosive currents and ultimately cause the formation of beachfront salients, or a wider beach i.e. provide a dry breach where presently one does not exist. It has been updated from the previous version in order to provide more detail with respect to environmental impacts, as requested by 2 technical reviewers acting for the Auckland Regional Council. Many of the updated aspects are included in the main body of the report, while specific reviewer queries and the responses are provided as Appendix 6.

The original focus for the first set of multi-purpose reefs was in front of the public reserve and SLCS at the southern end of the beach. However, the highest priority zone of the beach with respect to coastal protection is in the northern part of the beach, where properties are at threat. Thus, the first set of proposed multi-purpose reefs would be built in this area and monitored regularly prior to the construction of further reef systems.

EXECUTIVE SUMMARY

Orewa's long sandy beach is considered its major attraction and the most significant natural resource in the area. However, since early last century a number of contributing factors have dramatically reduced the width of 'dry' beach along much of its length. In recent years, sand has been placed on the beach to increase the dry beach area and protect the back beach from further erosion during storm events; however this nourishment material does not remain where it was place for very long due to the existing beach processes. The aim of this project is to provide a workable design for a system of partially to fully submerged offshore structures ('multipurpose reefs') which will modify and dissipate incident wave energy resulting in a reduction and re-direction of erosive currents and ultimately cause the formation of beachfront salients providing a dry beach where presently one does not exist. The beach enhancement scheme presented here is part of a whole beach solution that would be applied in stages.

This work builds directly on the foundation established by Mead et al., (2004a) in a feasibility study commissioned by the Orewa Beach Reef Charitable Trust (ORBCT) and the Rodney District Council (RDC), and the reader is directed to that report prior to reading this one. This report summarises the key findings from the Mead et al., (2004a) study then uses additional information (measured and modelled) to build on previous work and devise a new design for the Orewa Beach Reef. The feasibility study into the application of multi-purpose reefs for beach protection at Orewa Beach also built upon previous studies of the site, namely the comprehensive modelling of the University of Auckland in 1996 which indicated offshore reefs as the most effective sand retention structure of a variety of options tested.

We begin by presenting a synopsis of Mead et al.'s review of historical information. For the past three decades interested parties have debated over the causes of and the best way to deal with Orewa's ongoing erosion problem, or rather lack of dry high-tide beach and intermittent erosion events. Figure 1 graphically summarises the relevant physical processes that have led to the problem at Orewa. Among the human controlled factors are the sand mining of the onshore dunes, the realignment of the harbour's entrance channel and the tendency to build structures too close to the dynamic foreshore. Natural factors related to the problem are the fact that this is a closed sedimentary system with very little input of new material, the large tide range which allows erosive waves to reach further up the foreshore and the tendency for the most erosive events to be from north-easterly swells which pushes sand southwards towards the harbour entrance.

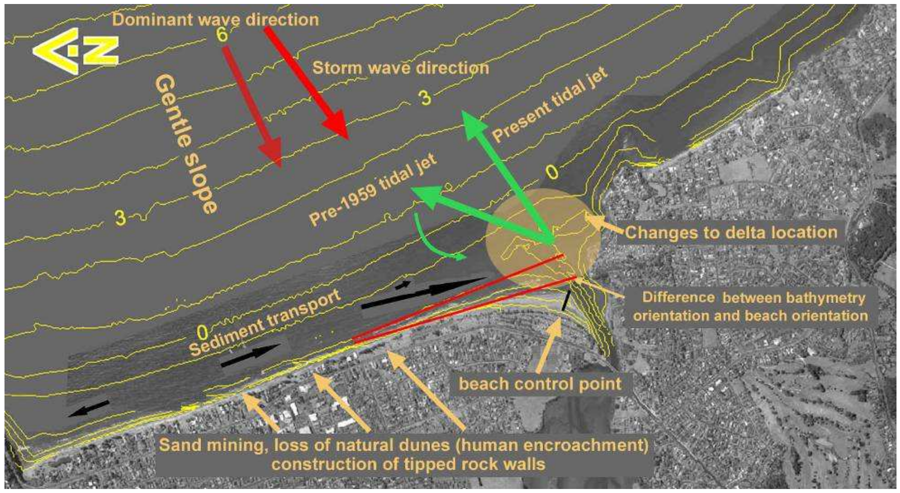


Figure 1. A basic summary of the historic impacts/changes to Orewa Beach and the coastal processes operating in this area.

Chapter 2 continues with a description of the local wind, wave and tide climate. This information is supported by two field studies which collected information on tidal currents and wave-driven currents in the surfzone. Also, a history of shore protection efforts used at Orewa is presented.

Chapter 3 then presents a systematic numerical modelling study which uses the wave climate data and the calibrated numerical models to converge on a radically different design for the Orewa Reef which places equal emphasis on wave dissipation and direct modification of alongshore currents.

The analysis began by comparing simple shore parallel structures to broad crested structures as shown in Figure 2. This set of model runs suggested that the broad crested structures are more effective in dissipating wave energy, however the large tidal range at Orewa would render structures of this sort ineffective if the crest height was set at a level such that the reef was submerged at all tides; the large tidal range effectively varies the surf zone over 300 m of low gradient intertidal sand flats. We therefore abandoned this line of investigation for a different approach, one which could address the wide and varying surf zone present at Orewa Beach during larger wave events.

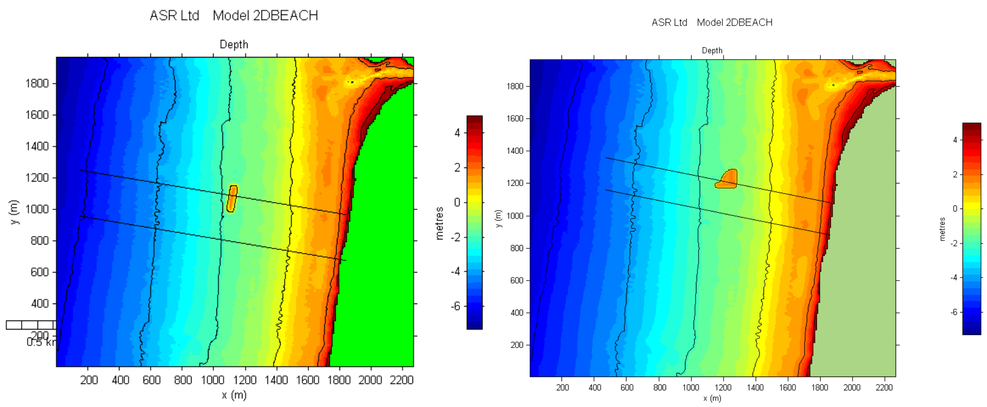


Figure 2. Reef shapes for a narrow crested shore parallel breakwater (left) and a broad crested reef shape (right)

The next series of tests used a different approach aimed at reducing the influence of the long shore currents which transport sediment southward towards the harbour entrance. In order to counteract this southerly directed flow, one common approach is to build a cross-shore structure extending into the surfzone, as has been done at many beaches worldwide in the form of jetties or groynes. Since the public sentiment at Orewa is clearly against the idea of a typical groyne (due to negative aesthetic and access impacts), we attempted to achieve similar result, however with a less intrusive structure which would also dissipate wave energy and reduce the loss of sediment across shore, as well as provide additional benefits such as marine habitat, surfing/wind-surfing/kite-boarding amenity.

Four trial shapes for this type of structure were tested. Each of the four shapes is an obliquely oriented cross shore, submerged structure. The cross shore orientation is designed to affect the alongshore currents while the oblique angle relative to the shore aids in wave shadowing and energy dissipation. For each of the designs, the structure crest height is set to 0.5 m above chart datum (equivalent to the MLWS water level). Thus, these reefs will only be exposed at the lowest of tides. The trial shapes are shown in Figure 3.

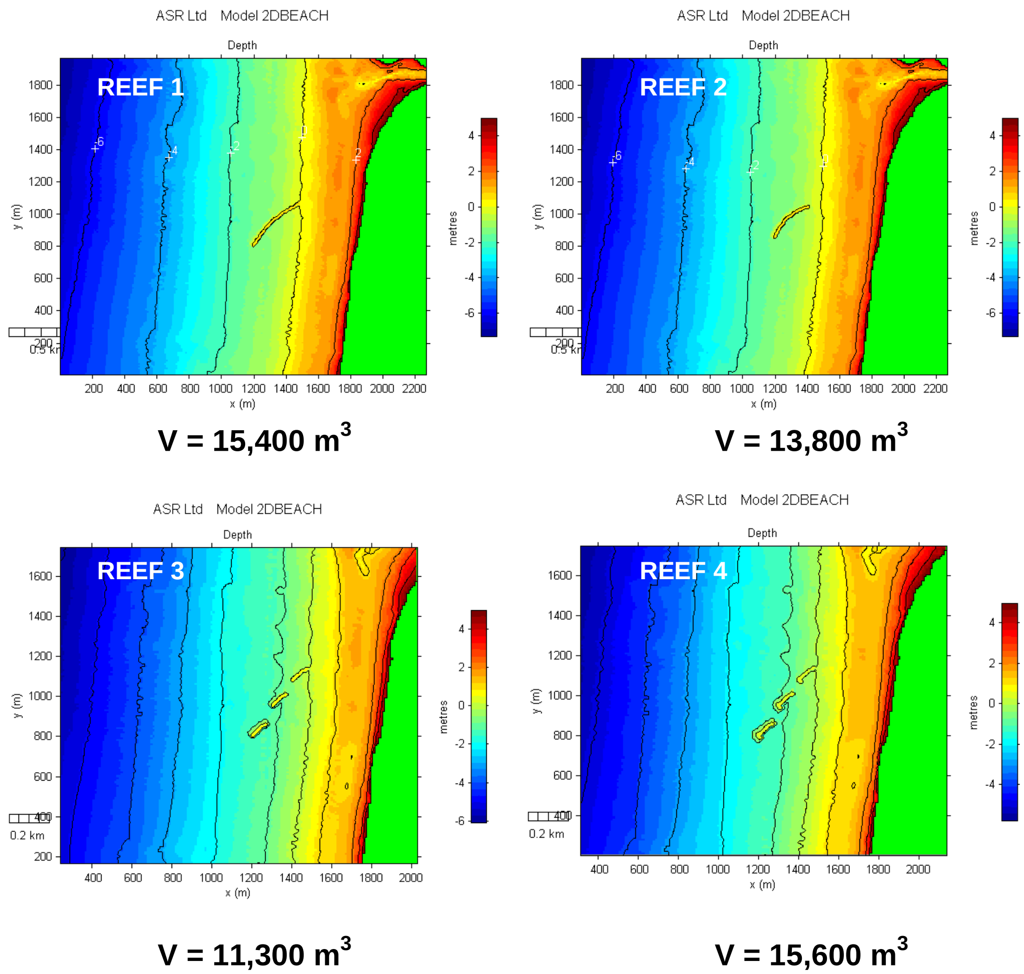


Figure 3. Four reef shapes tested for current modification and wave dissipation.

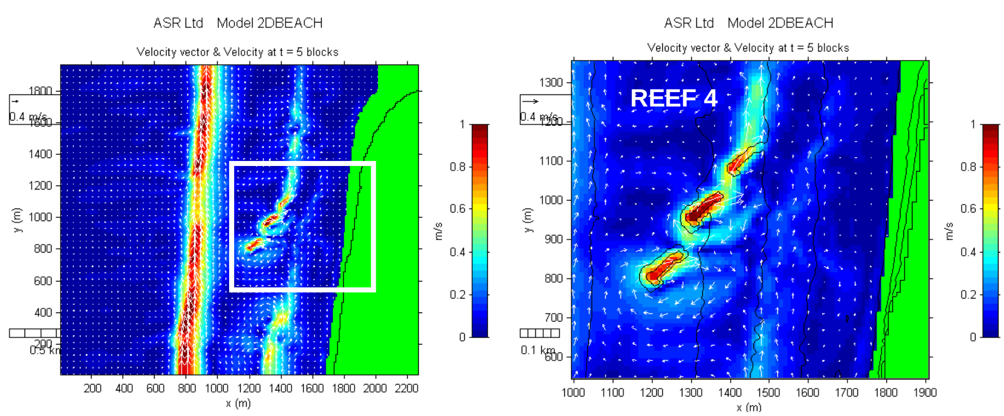


Figure 4. Modelled velocity patterns for 3.1 m wave heights at mid tide (+1.7 m) for reef shape 4.

Of the designs tested, option 4 provided the best combination of wave dissipation and current modification which would lead to an effective shore protection structure. As can be seen in Figure 4, the dominant alongshore currents that move beach sand southwards to the estuary entrance and Waitemata groyne are interrupted and salient forming eddies are created in the lee of the reefs.

Finally, shoreline response modelling using NGENIUS as a preliminary fast tool and the more sophisticated sediment transport model 2DBEACH was performed to quantify the shoreline response from the proposed structure. The results of the NGENIUS modelling are shown in Figure 5. The models suggest that a significant salient will form in the lee of the reef structure. Additional modelling with 2DBEACH using storm wave conditions suggests that the salient shape is dynamically stable. Large waves acting on the reef and salient would cause the saline to retreat, however, the salient represents a large buffer zone and will rebuild in subsequent small wave conditions.

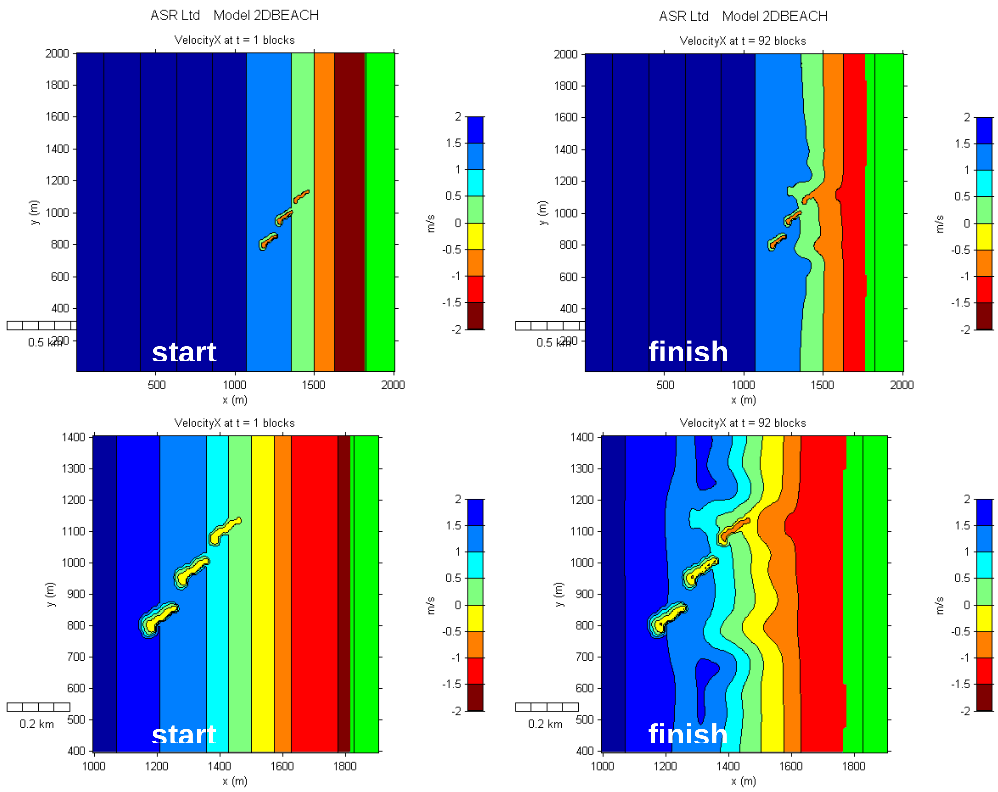


Figure 5. Modelled shoreline response from NGENIUS for the multiple segmented reef design. The full grid is in the top row with a close up on the reef area in the lower row.

A large amount of additional modelling was undertaken to test the influences of 1, 2, 3 and 4 sets of reef systems along Orewa Beach for a range of wave conditions. Each 3 reef system, which extends some 360 m alongshore, has an influence of some 600 m alongshore in which the beach is significantly widened. This distance of 600 m is the length of beach with a definite response, with beach width tapering away each side beyond this. The results indicate that there is some feedback between reef systems, with 2 sets of systems retaining more sand than 2x a single reef system and 3 sets of systems retaining more sand than 1.5x 2 sets of reefs systems.

In simple terms, the 3 reef system spread out over a distance of some 300 m offshore works to dissipate wave energy through the different phases on the tide, while also influencing alongshore currents, especially during larger wave activity at high tide, when most erosion damage occurs. Due to Orewa Beach's flat gradient and tidal range of around 3 m, there can be a very wide surf zone through a tidal cycle during larger wave events. Thus, the beach response to each single reef is somewhat 'blurred' by the varying sealevel. The three reef system helps to dissipate wave action over this wide surf zone, while also modifying currents by deflecting alongshore currents in the surf zone more shoreward and breaking up the dominant alongshore drift. Together, these factors address a relatively rare beach situation.

The final design concept for the first reef system is shown in Figure 6a. This is the first of four proposed reef systems proposed for the length of Orewa Beach. Because each system of reefs can protect approximately 600 m of beach front, 4 reefs would be required to protect and enhance the entire 2.4 km of Orewa Beach, although this would be confirmed with monitoring of the first set of reef systems. This overarching master plan is shown in Figure 6b.

Chapter 4 presents information regarding the construction of submerged offshore reefs using large sand filled geotextile containers (SFC's). In this chapter we present the methodology used to successfully construct similar structures. Specifically, the individual bag units will be deployed from a barge and secured to pre-laid foundation grid. The bags are then pumped full with a water/sand slurry by a pumping system located on the beach.

A

B

Figure 5. Views of the overall projects plan. The first of four reef systems located on the northern part of the beach (A). The larger master plan of 4 reef systems along the length of Orewa Beach is shown in B. Outputs extracted from model simulations with tide at 1.0 m above CD – the transition from grey to sand in panel A is the approximate high tide mark.

Finally Chapter 5 concludes the report by describing the projected effects on the environment. Generally, the physical effects are localised and for short term during construction. These include the disruptions that will be caused during the reef construction process, expected to last between 12 and 16 weeks for each reef system. The long term impacts, however, are largely positive and include wave energy dissipation and wave breaking, as well as a reduction in the strength of the alongshore currents. These in turn then cause the desired permanent effect of widening the beach. Recommended monitoring of the reef is also presented in Chapter 5. These monitoring data will provide information on both the effects of the reef system(s) and an indication of the requirements for future renourishment. As is the case world-wide, beach nourishment (which is designed to fail, i.e. put sand on the beach and once it's been washed away, replace it) are being coupled with structures and dune stabilization to greatly lengthen the duration of the nourish material. At Orewa Beach, nourish material currently has a relatively short residence time, especially when the wave climate is taken into consideration. The presence of the reef systems and appropriate plantings will greatly increase the residence time of any nourishment material, making the solution more sustainable in the long-term, both environmentally and economically.

This proposed solution to enhance the beach amenity at Orewa (namely, provide a wider dry beach) fits very well with the proposed esplanade and beach enhancement plan. While it is noted that the esplanade enhancement strategy is still in the proposal stage, it is important to note that the types of beach planting proposed in the strategy are a crucial component of this proposal – with a wider dry beach it will be imperative to reduce Aeolian sand transport (i.e. sand movement (loss) due to wind) with the planting of appropriate native beach vegetation). Such measures have proven successful in the southern area of the beach, and it is strongly recommended that they are applied to the upper beach areas created by the 'managed advance' that will be achieved by the propose reef systems.

CONTENTS

CHAPTER 1 – INTRODUCTION, BACKGROUND AND PROJECT DESCRIPTION

1.1 Report Structure

This document supports the resource consent application for proposed multi-purpose reefs at Orewa Beach, and is structured as follows; Chapter 1 gives an introduction and background to the project and describes the overall rationale for the proposed design including a description of why multipurpose reefs were chosen for Orewa. Chapter 2 gives an overview of the existing beach environment including the regional geology, the beach geomorphology and the relevant wave, tide and climactic issues as they relate to the shoreline stability and the ongoing erosion at Orewa. Chapter 2 also includes a brief description of the marine ecology, water quality and the human environment including community use, access and infrastructural elements. Chapter 3 is a detailed summary of the reef design with Chapter 4 describing the proposed construction methodology. Finally Chapter 5 describes the predicted effects of the proposed structures on all aspects of the local environment, covering each of the areas described in Chapter 1.

1.2 The Project Site

Orewa Beach is located on the north eastern coast of New Zealand's North Island, within the Hauraki Gulf (Figure 1.1). The Hauraki Gulf is bounded by Bream Head to the north, Cape Colville to the east and the Firth of Thames to the south. The Hauraki Gulf can be divided into the Outer, Mid and Inner Gulf regions based on the level of exposure to waves and winds. Exposure, and hence wave height and wind velocity, generally decrease from the Outer to Inner Gulf. Orewa Beach is within the Mid Gulf region while the Inner Gulf region lies south of the Whangaparaoa Peninsula and Cape Colville.

Figure 1.1. Location map of Orewa Beach

Orewa Beach has degraded over the years, which has been attributed to a variety of factors that have contributed to the present state of the beach, i.e. the lack of a 'dry' beach at high tide along many parts of its length.

The contributing factors to this deterioration have been:

- sand extraction for construction in the first half of last century,
- the progressive destruction and reclamation of the coastal dune system
- the progressive construction of tipped rock walls (revetments)
- a realignment of the estuary mouth which occurred in 1959
- construction and modification of a groyne at the southern end of the beach.

Each of these factors has had some impact on the shape and stability of the beach. These factors are shown schematically in Figure 1.2.

These human induced changes have no doubt also occurred within the natural changes of long-term wave climate, oscillations such as the El Nino/La Nina southern oscillation index (ENSO 4-7 year periods), the

Interdecadal Pacific oscillation (IPO ~ 30 year period) as well as larger scale fluctuations such as global climate change.

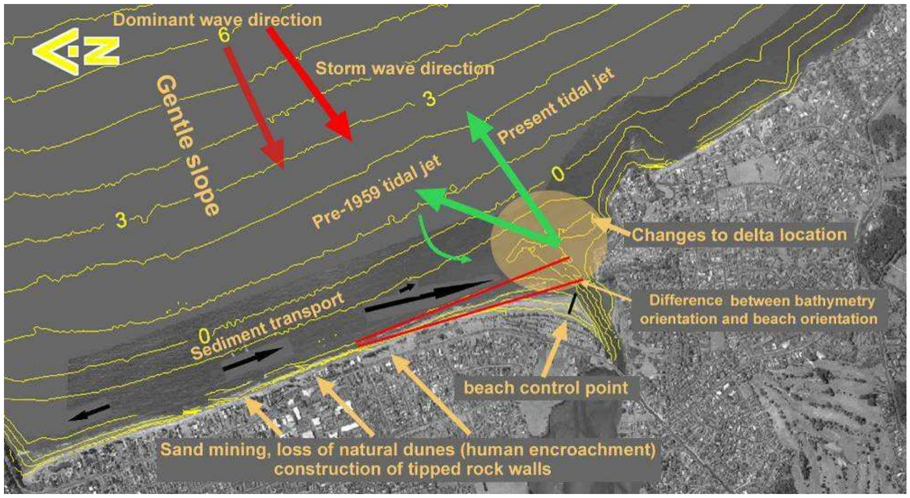


Figure 1.2. Overview of processes and activities which have affected Orewa Beach over the years.

While some reports of the beach refer as far back as the late 19th century, it is the current state of the beach that we are interested in modifying. The analysis is therefore focused on the most recent studies related to Orewa Beach - specifically the period from the early 1980's to present that is also the period spanned by beach monitoring data.

1.3 Project History and Objectives

This recent phase of interest in understanding the processes affecting Orewa Beach and developing a comprehensive long term solution began nearly three decades ago when it became clear that local homeowners' properties were being affected by beach erosion from storm events. During this period a range of reports on Orewa Beach erosion were generated which have useful information with respect to the processes that are operating along this stretch of coast.

Reports by Raudkivi (1980, 1981) ***"Stability of Orewa Beach"*** *and* ***"Orewa Beach Investigation"*** serve as 'foundation' documents for describing present-day coastal protection issues related to Orewa Beach. Many later documents defer to Raudkivi's (1980) earlier assessment, as this report was the first substantial report to be produced following the redirection of the Orewa Estuary entrance in 1959.

Systematic investigations into the coastal erosion problem at Orewa began in the early 1980's as a result of beachfront homeowners seeking to redevelop their properties from low cost single-unit dwellings to larger, more expensive multi-unit dwellings. At the time, the city engineer felt that certain properties, especially those on the southern half of Orewa

Beach were in danger of being inundated during extreme storm events and therefore denied building permits in this area.

Over the intervening years, various storm events have caused beachfront erosion which has threatened several structures along the beach. The community of Orewa has countered these events through a piecemeal approach which has included beach nourishment and the construction of tipped rock (rip-rap) seawalls along sections of the beach. There have also been numerous studies, reports, numerical and physical modelling exercises and monitoring surveys spawned in an attempt to find a solution that is effective and economically viable.

The objective of the projects described in this application represents the latest attempt to provide a comprehensive, holistic and long term solution for Orewa Beach. This phase of the projects began in 2003 when the Orewa Beach Reef Charitable Trust (OBRCT) and Rodney District Council (RDC) commissioned ASR Ltd to undertake a feasibility study for a submerged multi-purpose reef at Orewa Beach.

The OBRCT and RDC brief outlined the following objectives that the construction of a multi-purpose reef at Orewa Beach should primarily aim to achieve:

- Protection of the foreshore, public reserves and estuary against erosion & storm damage through the replenishment and maintenance (retention) of sand on the beach.

- The creation of a multi-use recreational facility for a variety of aquatic activities, i.e. surfing (in all its forms), snorkelling, windsurfing, kite-boarding or kayaking.

- Ecological enhancement, increasing the extent of marine life in the area.

- A reef that is as *multi-beneficial* to the community as possible. The construction of the reef is seen as complimentary to the planned “beautification” program of the foreshore set out by R.D.C., and fits in will with the long-term sustainable management plan of Orewa Beach. It is also recognizes the benefits to other sectors such as Tourism/Economic, Education, Ecological and Social.

Initially the area of interest was tentatively defined as the stretch of beach from the approximate midpoint between the river mouth and surf club, to some 300 m north of the surf club. However, this latest effort extends the area of study to include all of Orewa Beach, with the first priority being the northern part of the beach which is classified as the highest priority with respect to beach protection by the RDC.

1.4 General Description for a system of Multipurpose Reefs to provide shore protection at Orewa

The beach at Orewa has been shown to be generally stable and non-eroding under normal, calm wave conditions. The beach protection strategy for Orewa should focus on reducing the incident wave energy which reaches the beach and causes erosion. This type of event occurs during extreme wave conditions when strong storms coincide with high tides and the sea is able to act directly on the nearshore dunes or rip-rap seawalls which separate the beach from private property.

Since the objective for the project described here is to reduce and redirect incident wave energy which causes erosion, the obvious starting point is to devise a system of structures that will be positioned off shore and reduce the wave energy that reaches the beach. In general terms this falls under the category of an offshore breakwater – a method of shore protection that has been used for centuries. However, the specific method described here incorporates many significant variations on that general concept which makes the project significantly more feasible and potentially successful.

We propose that a system of 1 to 4 partially to fully submerged structures be constructed offshore of Orewa Beach. The structures will each have a unique plan shape which maximises the wave dissipation and works to counteract the erosive long shore currents which arise during storm wave events.

It should be emphasized that this study comes in the wake of nearly 30 years of study, analysis and planning to find a solution for Orewa's ongoing erosion problem. Previous studies have suggested a variety of alternatives including beach nourishment and the construction of sea walls. To date, only the beach nourishment and sand redistribution options have been used with any consistency.

CHAPTER 2 – EXISTING ENVIRONMENT AND LITERATURE REVIEW

2.1 Introduction

This section will describe the existing physical environment at Orewa Beach. The information contained here is synthesized from many sources. A more complete discussion of each section is contained in Mead et al., 2004a *"Feasibility Study for a Multi-Purpose Reef at Orewa Beach, Hibiscus Coast, Auckland New Zealand"* which is included as an appendix to this application.

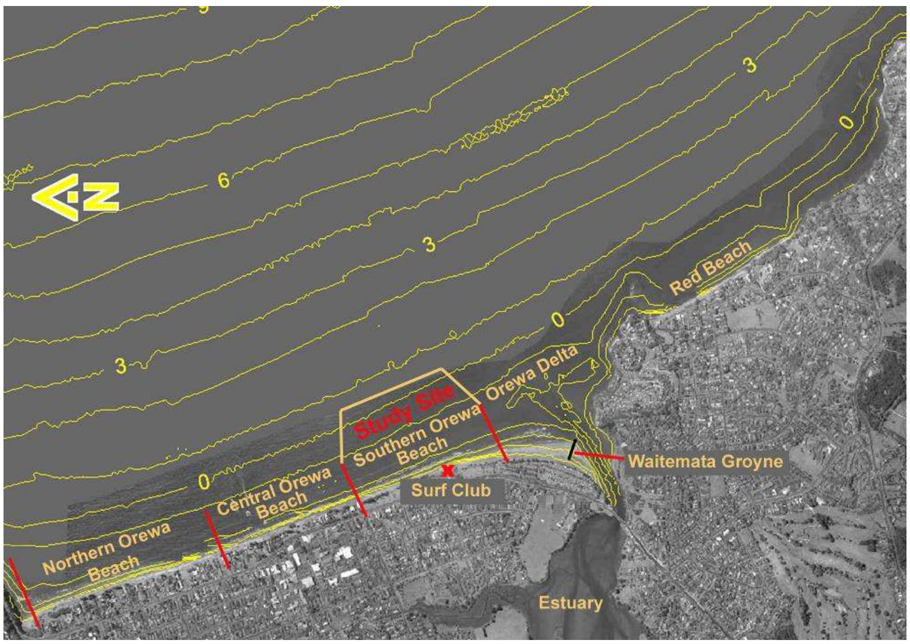


Figure 2.1. Areas and features/landmarks on Orewa Beach referred to throughout this report

2.2 Geology

The Hauraki Gulf was formed by an Early Pleistocene tectonic rift, and the resultant depression has been in-filled with unconsolidated sediments with an average thickness of 250-750 m. In recent pre-glacial times, the Hauraki Gulf was an alluvial plain associated with the Waikato River, and draining the Taupo Volcanic Zone (Healy, 1946). Approximately 20,000 years ago the Waikato River course diverted into the Hamilton Basin, thereafter marking an unconformity in the Gulf sediments. Sea level was approximately 113 m below the modern levels at that time, and the sequential marine transgressions to 6.5 ka B.P. have led to a re-working of the sediments, forming the generally subdued seabed morphology that is characteristic of the Gulf. Over the past 6,500 years, a terrigenous blanket of fine sediments has been accreting in the Firth of Thames/Inner Gulf region, and actively prograding northwards.

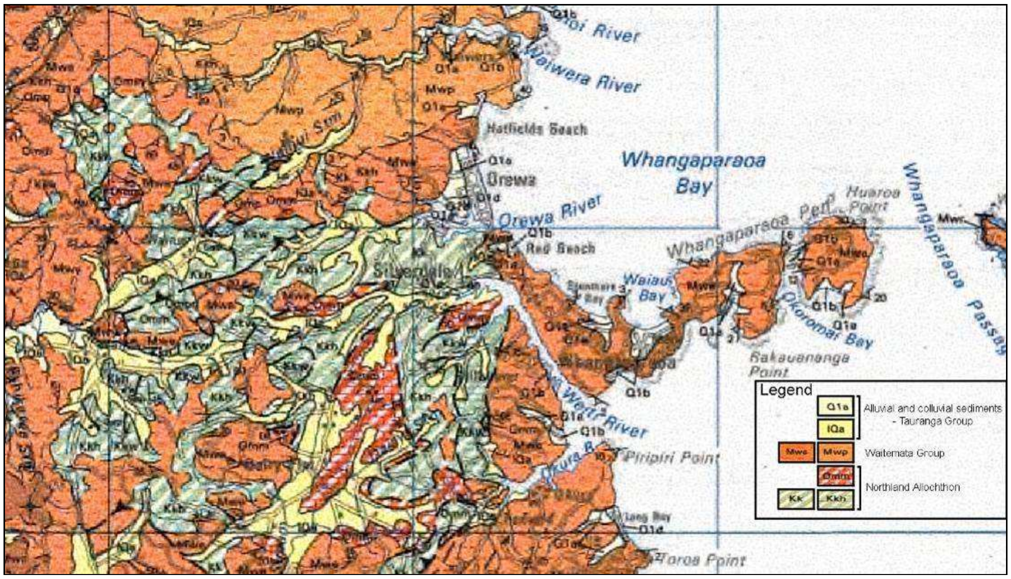


Figure 2.2. Geologic map of the Orewa region (Edbrooke, 2001).

The geologic map of the region (Figure 2.2) shows that the Orewa catchment is dominated by sandstone and mudstone with variable volcanic content from the East Coast Bays and Pakiri Formations of the Waitemata Group and the siliceous and locally calcareous mudstone and limestone from the Northland Allochthon Complex. The Orewa township and estuary lies on Quaternary alluvium and colluvium from the Tauranga Group, shown on the stratigraphic sequence from Edbrooke (2001).

2.3 Coastal Landscape and Surficial Sediment Characteristics

Orewa is a very gently sloping beach that was originally backed by system of low sand dunes. The offshore slopes are on the order of 1:100. Given that Orewa has a tidal range of nearly 3 m this implies that there may be up to 300 m of exposed sand at low tide with virtually no dry beach visible at high tide.

Raudkivi (1981) provides a useful description of the sources of sand in the inner Hauraki Gulf. His suggestion that the sand input is derived mostly from local rivers and creeks is supported anecdotally by the presence of larger sediment grain sizes in the estuary than on the beach. Additional sediment inputs are thought to be derived from eroding cliffs of the Whangaparaoa Peninsula and Waitemata sandstone reefs (Manighetti & Carter, 1999).

Median grain size is given by Raudkivi as 130-140 μm on the beach, 130-170 μm in the estuary decreasing to 75-80 μm at depths of 4-6 m offshore. This is supported by Smith (1986 – 2.2-3.5 ϕ) and Tonkin and Taylor (1994), which report decreasing grain size and increasing silt fraction with increasing depth. This shows the increasingly smaller grain sizes and increasing silt content as depth increases, as described by Manighetti and Carter (1999).

Origins of the Orewa beach sand are not entirely Waikato Sands and apparently Orewa is indicated as an area dominated by fine sediment inputs from rivers (Raudkivi, 1981).

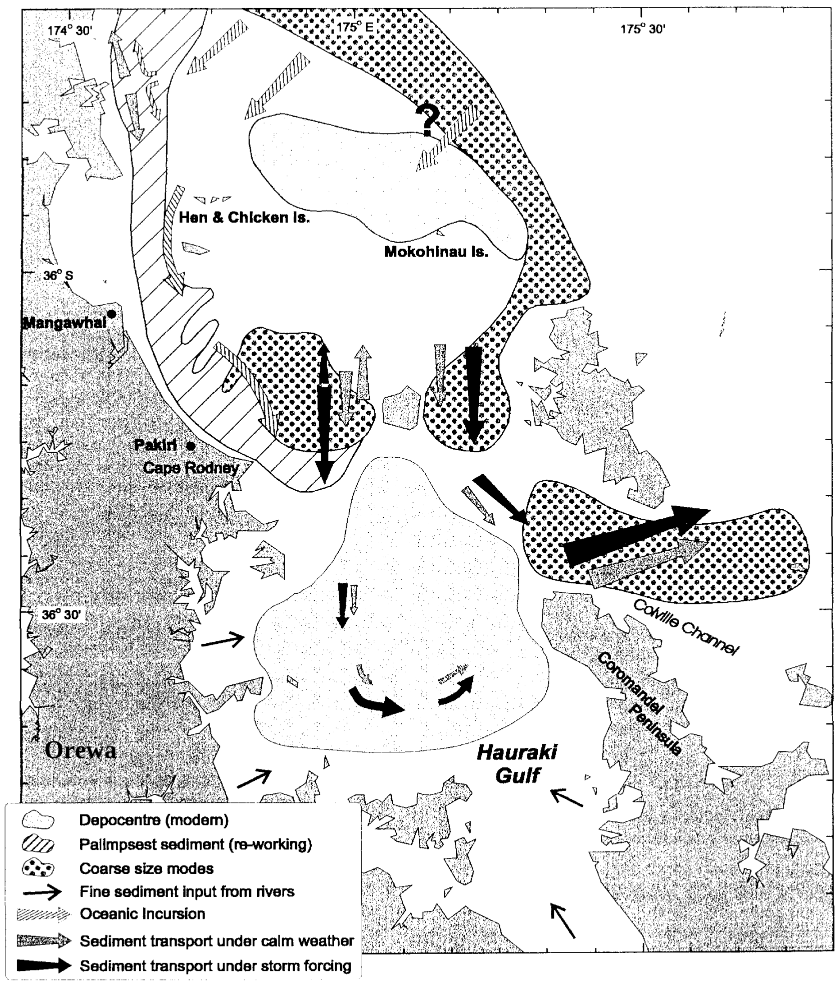


Figure 2.3. The sediment transport and deposition regime in the Hauraki Gulf (after Manighetti & Carter, 1999). Note that Orewa is indicated as an area dominated by fine sediment inputs from rivers.

2.4 Climatic Conditions

This section will describe the overall climatic and weather conditions experienced at Orewa Beach

2.5 Wave and Tide Climate

Sediment transport at Orewa is driven by the alongshore component of wave energy flux. It is therefore very important to have a thorough understanding of the local wave climate in order to understand the nature of the problem and devise an appropriate solution. This section will summarise what is known about the Orewa Beach and Hauraki Gulf wave climates.

2.5.1 Overall Wave Climate

The north-eastern coast has the smallest wave climate of all exposed coasts of New Zealand because of the predominant south-westerly weather patterns (Pickrill and Mitchell, 1979). At Orewa wave energy is further reduced due to sheltering behind Great Barrier Island and the Coromandel Peninsula (Figure 2.4). Generally waves at Orewa Beach are small, short period seas created by fetch limited local winds. Longer period swells and extreme events reach Orewa Beach through the gap between Great Barrier Island and the Coromandel Peninsula (Colville Channel) as well as to the north of Great Barrier Island and the pass between Great Barrier and Little Barrier Island (Cradock Channel) and are the cause of the undesirable loss of beach sand.

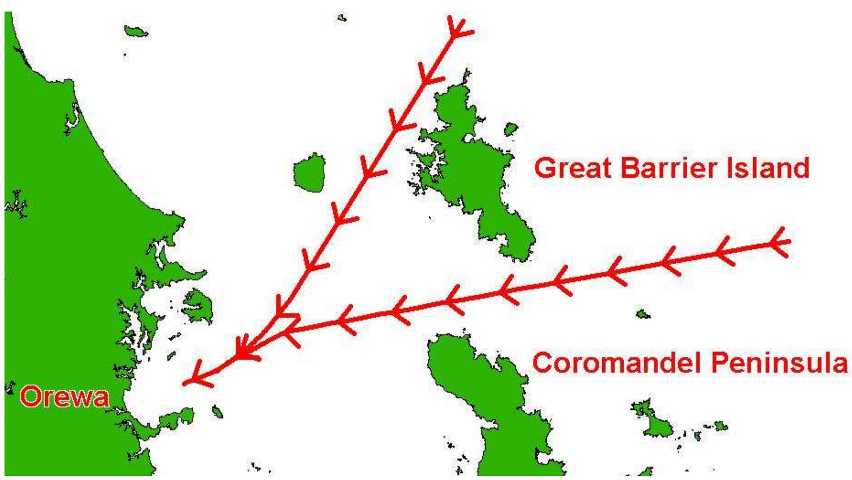


Figure 2.4. The longest period, largest waves to reach Orewa comes from swells that pass north and south of Great Barrier Island and refract in to Orewa.

Until the time of this study, there were no detailed recordings of actual wave heights at Orewa Beach or inside the Hauraki Gulf. Therefore any discussion of the wave climate there was restricted to transformed data from offshore locations or hindcast prediction based on historical weather data. As such, there is some variation in the various wave height metrics obtained from different analysis techniques. The ASR Feasibility and Design report (Mead et al., 2004a) details the hindcast efforts as well as the wave transformation techniques used to develop and inshore wave climate for a location directly in front of Orewa Beach.

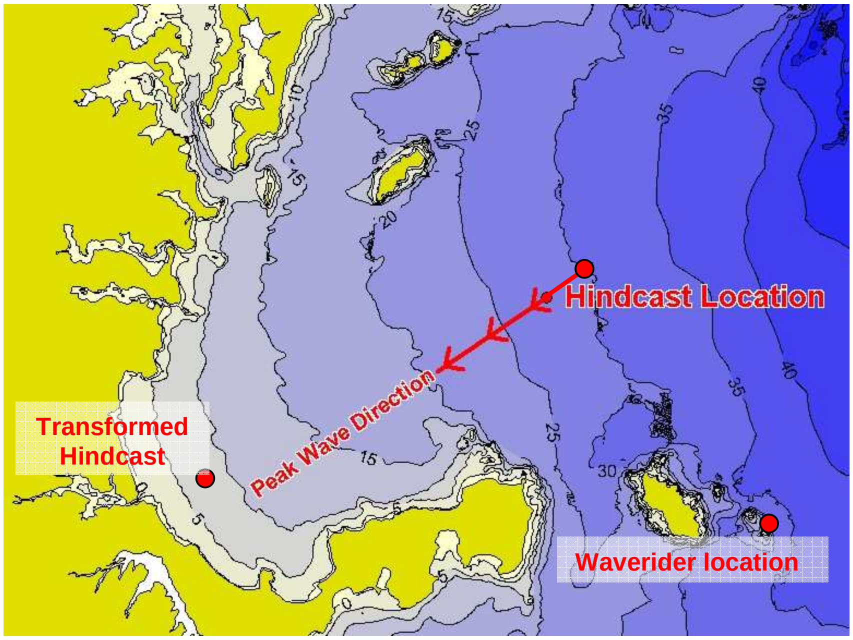


Figure 2.5. Locations for the Gorman *et al.* (2003a and b) 30 m hindcast, the Gorman *et al.* (2003a and b) hindcast transformed to Orewa (7 m depth) and the Waverider buoy data.

As noted before, previous assessments of the wave climate at Orewa relied on hindcast information. A wave hindcast study (Gorman 2003a,b) was used to derive the wave climate for the Hauraki Gulf and Orewa Beach. The Gorman study calculated the wave parameters for a site located at 30 m water depth offshore of Orewa (see Figure 2.5) The results of this study described the wave climate as shown in the joint probability distribution tables for wave height versus direction and wave height versus period (Tables 2.1 and 2.2). The general statistics are then summarised in Table 2.3.

Table 2.1. Joint probability distribution of the significant wave height / mean wave direction occurrence (parts per thousand) for Whangaparaoa hindcast.

Hs (m)		348.75 to 11.25	11.25 to 33.75	33.75 to 56.25	56.25 to 78.75	78.75 to 101.25	101.25 to 123.75	123.75 to 146.25	146.25 to 168.75	168.75 to 191.25	191.25 to 213.75	213.75 to 236.25	236.25 to 258.75	258.75 to 281.25	281.25 to 303.75	303.75 to 326.25	326.25 to 348.75	Sum
0.00	**0.50**	9	18	63	84	38	10	7	5	6	8	17	19	13	7	6	6	**316**
0.51	**1.00**	4	22	110	200	73	6	2	1	1	1	2	2	1	1	1	2	**430**
1.01	**1.50**	0	5	40	83	27	0	0	0	0	0	0	0	0	0	0	0	**155**
1.51	**2.00**	0	1	17	29	11	0	0	0	0	0	0	0	0	0	0	0	**58**
2.01	**2.50**	0	0	8	11	3	0	0	0	0	0	0	0	0	0	0	0	**22**
2.51	**3.00**	0	0	4	6	1	0	0	0	0	0	0	0	0	0	0	0	**10**
3.01	**3.50**	0	0	2	2	1	0	0	0	0	0	0	0	0	0	0	0	**5**
3.51	**4.00**	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	**2**
4.01	**4.50**	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	**1**
4.51	**5.00**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**1**
5.01	**5.50**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
5.51	**6.00**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
6.01	**6.50**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
6.51	**7.00**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
7.01	**7.50**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
7.51	**8.00**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
8.01	**8.50**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
8.51	**9.00**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
9.01	**9.50**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
9.51	**10.00**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Total		**13**	**48**	**244**	**419**	**154**	**17**	**9**	**6**	**6**	**8**	**19**	**21**	**14**	**8**	**7**	**8**	**1000**

Table 2.2. Joint probability distribution of the significant wave height / peak spectral wave period occurrence (parts per thousand) for Whangaparaoa hindcast.

Hs (m)		0 to 2	2 to 4	4 to 6	6 to 8	8 to 10	10 to 12	12 to 14	14 to 16	16 to 18	18 to 20	Sum
0.00	**0.50**	0	75.1	8.4	107	101.8	21.8	2	0	0	0	**316.1**
0.51	**1.00**	0	10	23.2	164.8	164.7	58.7	7.6	0.3	0	0	**429.3**
1.01	**1.50**	0	0	8.1	47.5	58.5	32.5	8.3	0.4	0.1	0	**155.4**
1.51	**2.00**	0	0	0.8	16.4	19.3	14.5	6	0.5	0.1	0	**57.6**
2.01	**2.50**	0	0	0	5.6	7.9	6	2.4	0.3	0	0	**22.2**
2.51	**3.00**	0	0	0	1	4.5	2.9	1.6	0.2	0	0	**10.2**
3.01	**3.50**	0	0	0	0	2.1	2	0.7	0.1	0	0	**4.9**
3.51	**4.00**	0	0	0	0	0.6	1.3	0.3	0	0	0	**2.2**
4.01	**4.50**	0	0	0	0	0.2	1	0.1	0	0	0	**1.3**
4.51	**5.00**	0	0	0	0	0	0.5	0	0	0	0	**0.5**
5.01	**5.50**	0	0	0	0	0	0.1	0	0	0	0	**0.1**
5.51	**6.00**	0	0	0	0	0	0.1	0	0	0	0	**0.1**
6.01	**6.50**	0	0	0	0	0	0	0.1	0	0	0	**0.1**
6.51	**7.00**	0	0	0	0	0	0	0.1	0	0	0	**0.1**
7.01	**7.50**	0	0	0	0	0	0	0.1	0	0	0	**0.1**
7.51	**8.00**	0	0	0	0	0	0	0	0	0	0	**0**
8.01	**8.50**	0	0	0	0	0	0	0	0	0	0	**0**
8.51	**9.00**	0	0	0	0	0	0	0	0	0	0	**0**
9.01	**9.50**	0	0	0	0	0	0	0	0	0	0	**0**
9.51	**10.00**	0	0	0	0	0	0	0	0	0	0	**0**
Total		**0**	**39.9**	**55.1**	**360.2**	**345.8**	**159.1**	**36.8**	**2.4**	**0.6**	**0.1**	**1000**

Table 2.3. Significant wave height statistics for Whangaparaoa, derived from a 20-year wave hindcast (1997-1999).

	Significant wave height (m)
Maximum	7.19
Median	0.66
Mean	0.82
Std. dev.	0.56
Covariance	0.32

In terms of wave direction, Figure 2.6 shows the frequency of occurrence for wave heights from different directions. It can be seen that the majority of waves come from a 30° directional window. A small amount of swell is directed in the offshore direction (towards east-north-east) because of local sea blowing offshore from Orewa as a result of strong southwesterly winds. When peak direction is plotted against significant wave height (Figure 2.7) and peak period (Figure 2.8), the narrow direction bands are also evident.

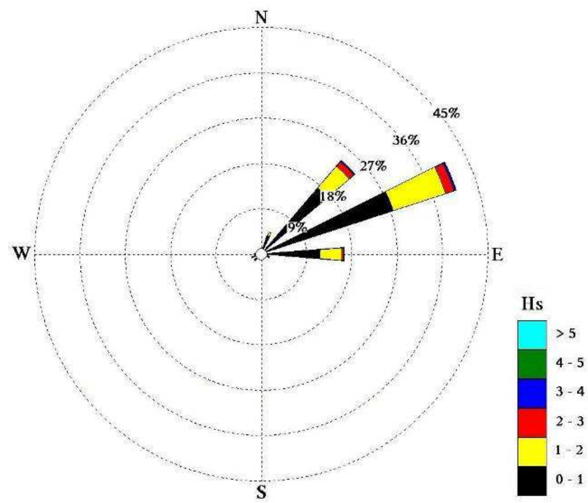


Figure 2.6. Wave 'roses' (height, direction and probability of occurrence) for the 30 m contour offshore of Orewa 01 Jan 1979 to 01 Jan 1999. Wave hindcast data from Gorman *et al.* (2003a and b).

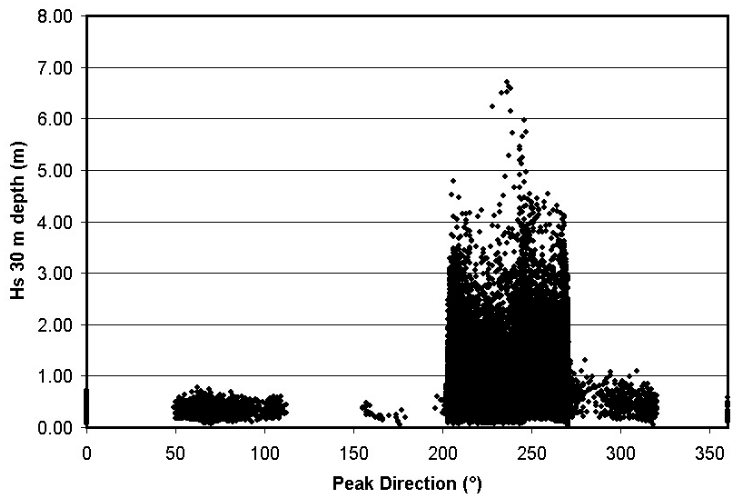


Figure 2.7. Relationship between peak direction and significant wave height at the 30 m hindcast site.

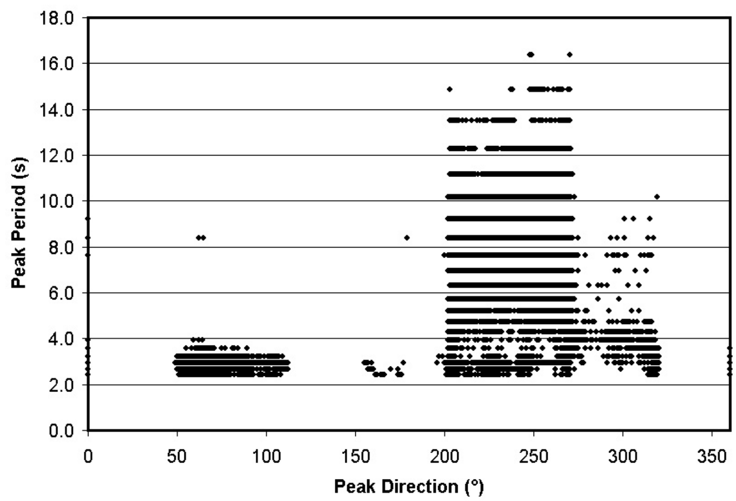


Figure 2.8. Relationship between peak direction and peak period at the 30 m hindcast site.

Because the wave characteristics for waves coming from 50° and 110° show low heights and short periods, this indicates 'sea' conditions rather than swell. For larger swells, the range of wave directions is limited to between 200° and 270° because of sheltering behind Great Barrier Isl and and the northern tip of the Coromandel Peninsula. If the local sea swells generated by offshore wind is discarded the average wave direction is 236° wit h a standard deviation of 22°.

Although the average wave conditions are only 0.82 m (Table 2.3), large cyclone swells occasionally affect the north eastern coast. The maximum significant wave height at the hindcast site during the 20-years was 7.19 m (13.5 s, 236°, 22 June 1996).

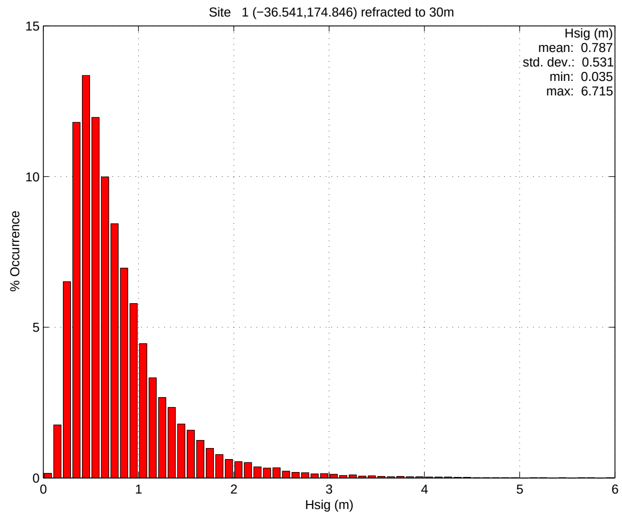


Figure 2.9. Percentage of occurrence of H_{sig} at the 30 m hindcast site.

Analysis of long-term wave data often shows clear seasonal trends. However, this not the case at the Hauraki Gulf hindcast location. Figure 2.10 presents the monthly mean wave height over the hindcast period. Although oscillations in the signal are evident, closer analysis reveals that they are not seasonal. For example, the wave climate during 1989 was much larger than 1992 but there is not a clear seasonal agreement (see Figures 2.11 and 2.12). The average seasonal wave heights are 0.71, 0.78, 0.81, 0.84 m for spring, summer, autumn and winter respectively.

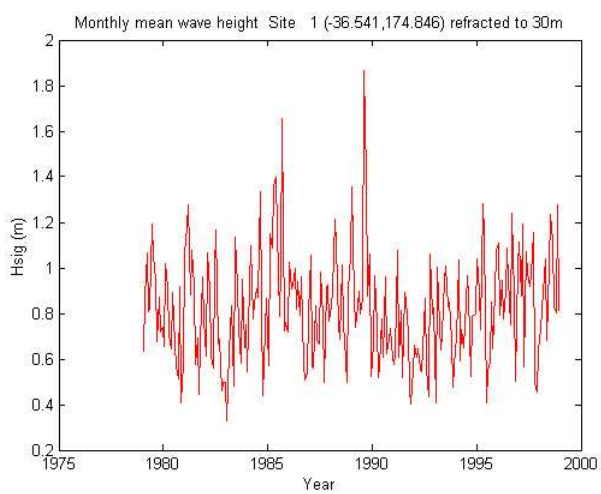


Figure 2.10. Monthly mean Hsig for the 20 year hindcast at Whangaparaoa.

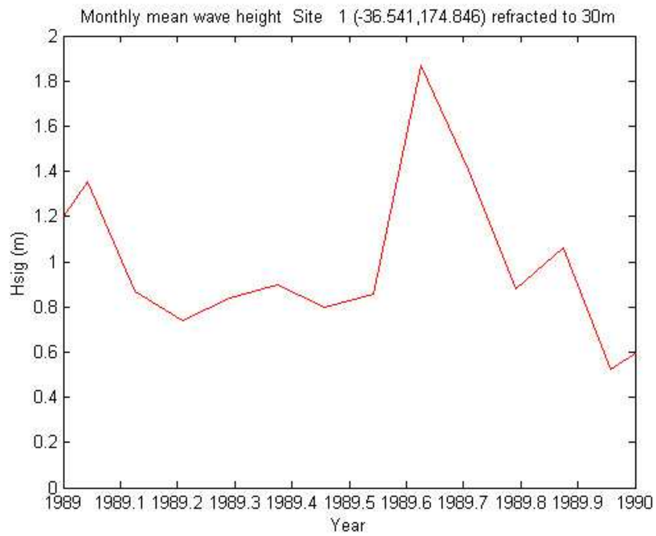


Figure 2.11. Average monthly significant wave height for 1989 at the Whangaparaoa hindcast site.

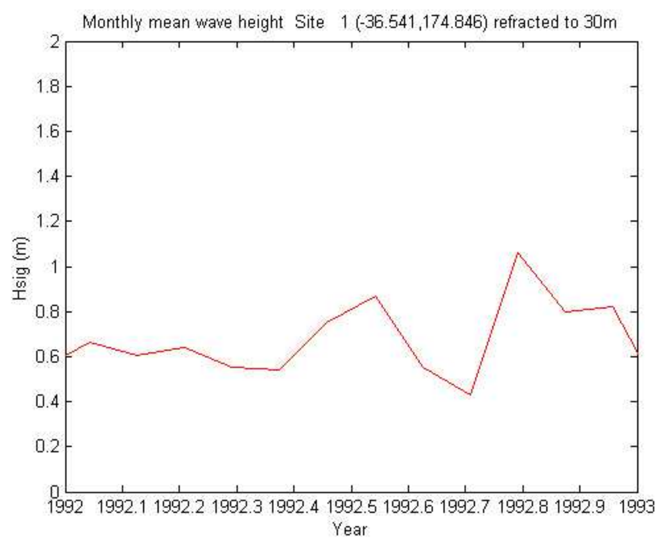


Figure 2.12. Average monthly significant wave height for 1992 at 30 m hindcast site.

The distribution of peak wave periods exhibits a normal bell shaped distribution (Figure 2.13) with a slight peak at 2 s, which is mostly linked with westerly quarter waves (i.e. waves moving away from Orewa Beach. The mean peak wave period of approximately 8 s is indicative of the high proportion of locally generated waves at the site.

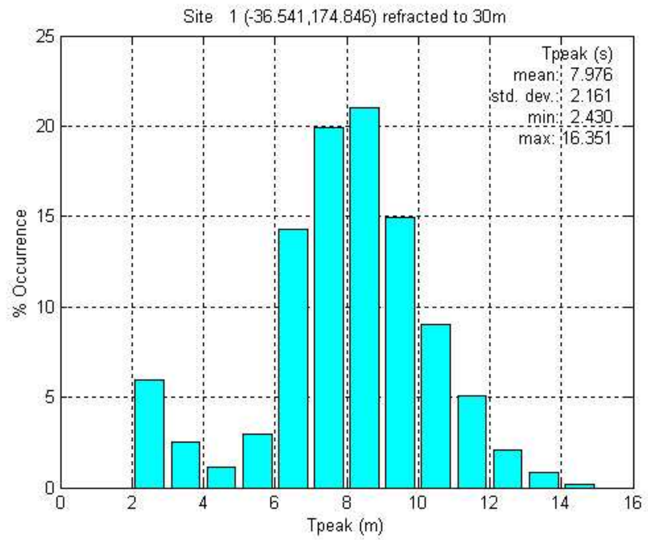


Figure 2.13. Distribution of peak wave period for the 20 year Whangaparaoa hindcast.

2.5.2 Extreme wave conditions

The *Asymptotic Extreme Value* (AEV) method was used for sample selection of wave data. AEV is a robust censoring method that does not require the subjectivity of thresholds or event durations. The censored data were further filtered to remove values less than the median of the original distribution. On this basis, the significant wave heights were sampled for the 30-day maxima, which is within the typical New Zealand 'weather band' frequency ($0.033 < f < 0.6$ cpd).

Two types of model distributions were used; the 3-parameter Weibull distribution and the Fisher-Tippet Type 1 distribution. A *least squares* method was used to find the best-fit of the sampled data to these model distributions. The predicted significant wave height extrema for return periods of 1 – 100 years are given in Table 2.4.

Table 2.4 Predicted significant wave height extrema for specified Return Period Intervals (RPI), derived from Whangaparaoa hindcast data.

RPI (years)	Significant wave height 30 m Whangaparaoa (m)
1	3.94
10	5.86
50	7.17
100	7.74

2.5.3 Inshore Wave Climate

To fully understand the sediment dynamics at Orewa for the purposes of designing an appropriate beach protection strategy, the wave climate from the

20-year hindcast (calculated well offshore at 30 m depth) must be transformed into shallower water for a location offshore of Orewa. Waves reaching the beach at Orewa are transformed significantly by various offshore features as they propagate landward. As already discussed, the Barrier Islands and Coromandel Peninsula block a significant amount of wave energy, but the Whangaparaoa Peninsula as well as land and island bodies to the north of Orewa also play a significant role in limiting the amount of swell reaching the study sites. In addition to blocking of swell, refraction/diffraction and dissipation also modify the waves. A refraction wave model was used to transform waves from the hindcast location to the study site in order to develop an inshore wave climate.

Bathymetry grids were constructed for the transformation modelling by digitising available nautical charts augmented with other available offshore data as shown in Figure 2.14 with a close up of the Hibiscus Coast region shown in Figure 2.15. Note that the grids have been rotated from normal map orientation for the purposes of the numerical model. Near shore, the offshore depth contours are very regular and shore parallel, changing towards the southern end of the beach where the estuary empties in to the sea. It is interesting to note that the estuary delta contours do not extend significantly seaward as compared to the era before re-alignment of the estuary channel (Figure 2.16).

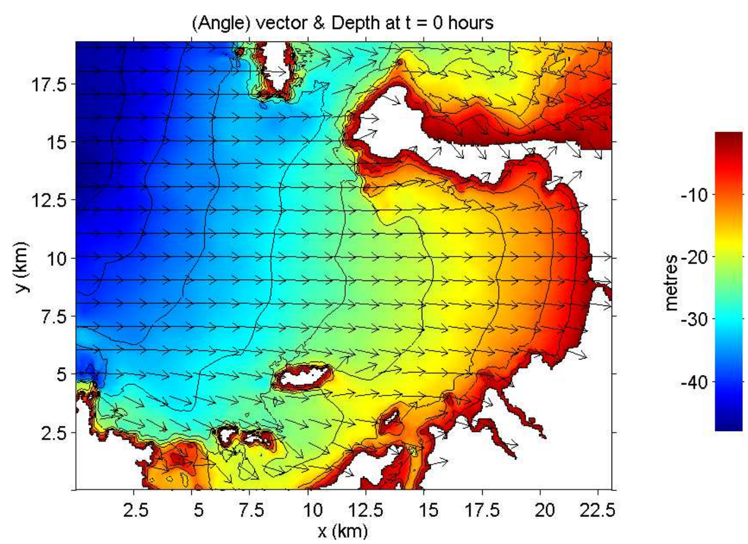


Figure 2.14. 50 m WBEND grid used for hindcast transformation modelling.

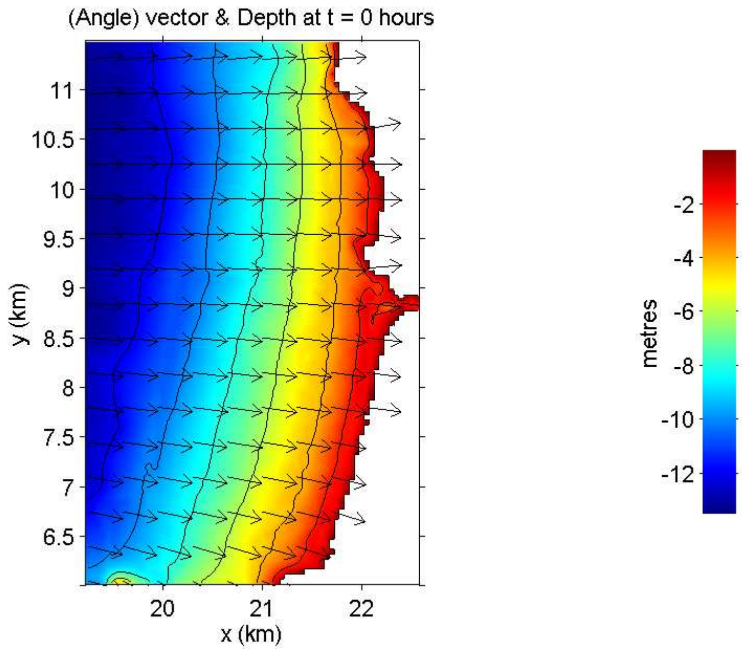


Figure 2.15. Enlargement of the 50 m WBEND grid used for hindcast transformation modelling showing Orewa and Red Beach.

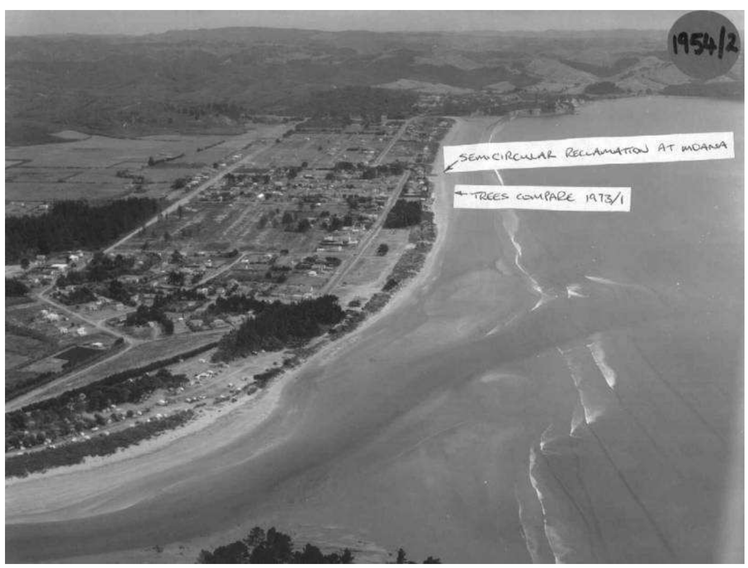


Figure 2.16. Oblique aerial photograph of Orewa showing natural alignment of estuary channel. The ebb delta was a more significant landform before engineering works in 1959.

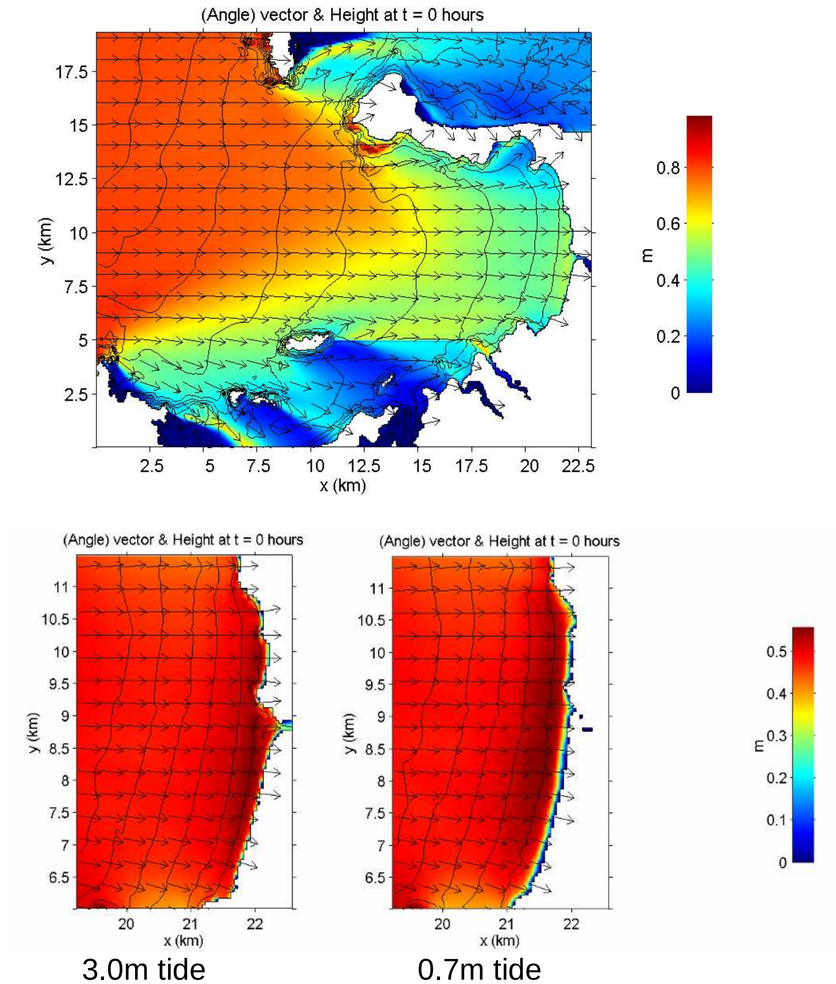


Figure 2.17. Predicted average wave height conditions at based on the WBEND PROB.out file. The top image shows the entire model grid and the bottom two are sub grid showing Orewa and Red Beach for 3.0 and 0.7 m tide levels.

In order to create the inshore climate, the offshore hindcast wave conditions (heights, periods and directions[1]) were run through the refraction model and transformed to the inshore location. The results of the transformation, given in terms of probability of occurrence are listed in Table 2.5.

[1]**Note:** that for this analysis, the wave directions are given relative to the direction the waves are travelling towards, i.e. a wave direction of 225° means the waves are travelling ***towards*** the southwest (225°) or ***from*** the northeast (45°). This opposite to the standard convention for describing wave directions. Since this section is summarising previous work, the figures and discussion are left in that convention, however following sections use the more widely accepted standard whereby directions are referred to as the direction the waves are coming ***from***.

Table 2.5. Final wave climate calculated 7 m off Orewa Beach.

% Occurrence	Hs (m)	Peak Period (s)	Direction towards (°)
16.77	0.5	8.14	235.5
28.97	0.5	8.06	235.5
16.31	0.5	7.78	235.5
8.55	0.5	7.19	235.5
2.05	0.5	7.11	235.5
7.22	1.5	8.55	234.5
7.85	1.5	8.76	234.5
2.17	1.5	8.35	234.5
3.75	1.5	7.87	234.5
1.22	1.5	7.89	234.5
0.93	2.5	9.87	233.5
1.34	2.5	9.26	233.5
0.47	2.5	8.55	233.5
0.96	2.5	8.78	233.5
0.23	2.5	8.92	233.5
0.21	3.5	10.42	232.5
0.31	3.5	9.81	232.5
0.12	3.5	10.42	232.5
0.23	3.5	9.41	232.5
0.04	3.5	9.55	232.5
0.07	4.5	10.71	231.5
0.13	4.5	10.43	231.5
0.02	4.5	11.79	231.5
0.04	4.5	10.18	231.5
0.02	5.5	11.68	230.5
0.01	6.5	12.55	229.5

Figure 2.18 shows the relationship between the offshore wave heights and the computed near shore wave heights. There is a roughly linear fit between the offshore and near shore wave heights. Figure 2.19 then relates this offshore/nearshore ratio to the wave direction and the probability of occurrence for each directional bin.

Based on the model output, the average wave direction of the transformed wave climate at 7 m depth is 242° (direction going towards), which is similar to the 236° average input wave direction at the offshore site. However, the spread of directions has become much narrower as shown in Figure 2.20 and the distribution is significantly different in shape. The range of directions has decreased from 105° to 39° as the wave crests become more shore parallel as they travel up the ramp of the Bay (due to refraction). The Orewa site has a normal bell distribution while the 30 m site has several peaks because offshore barriers block swells from different directions.

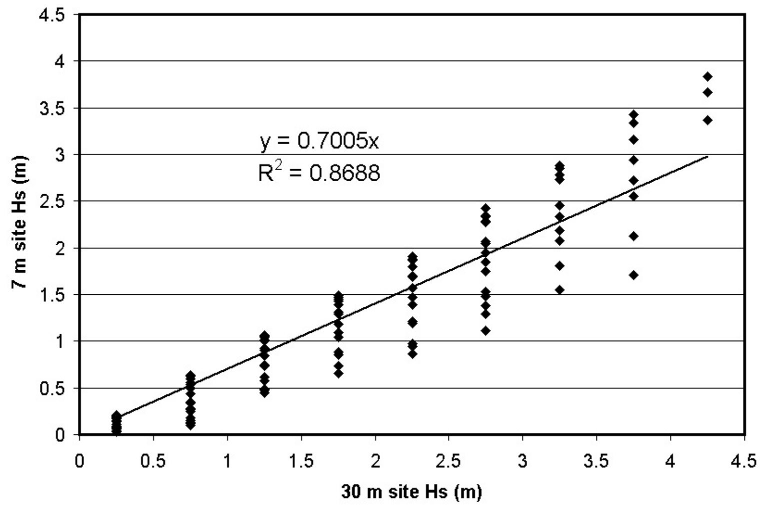


Figure 2.18. Ratio between offshore (30 m site) and inshore (7 m site) wave heights.

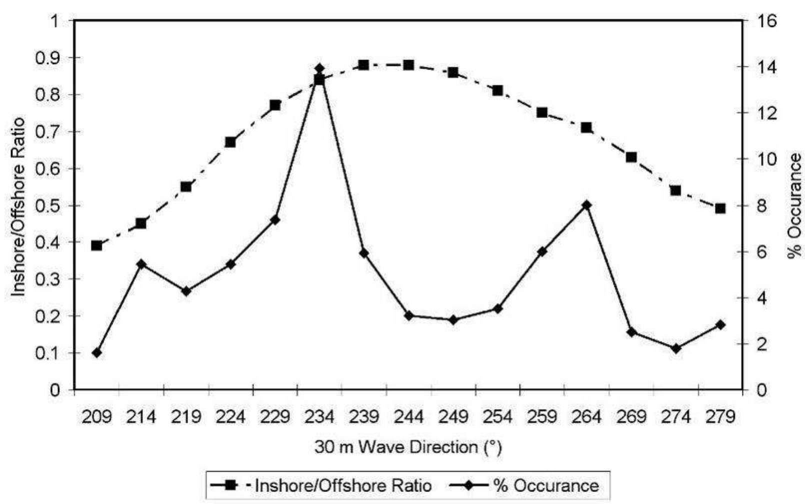


Figure 2.19. Ratio of inshore (Orewa) / offshore (Whangaparaoa) wave heights for different directions with frequency of occurrence for each wave direction.

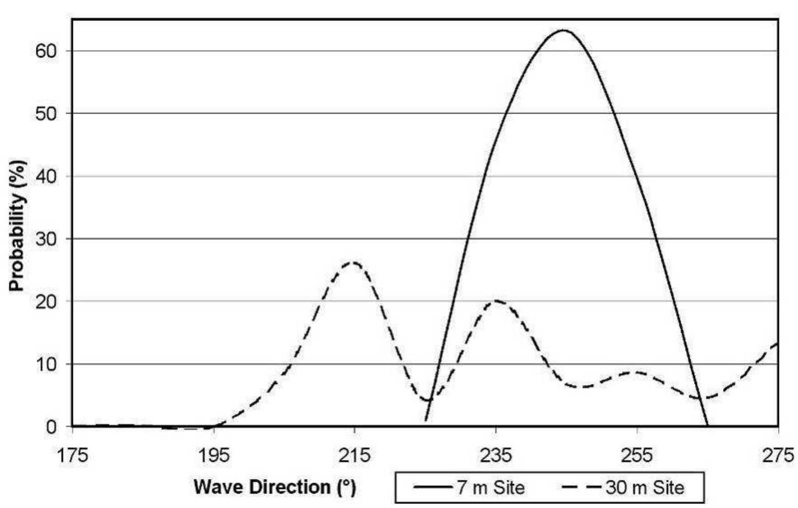


Figure 2.20. Spread of wave directions for Orewa (7 m) and Whangaparaoa (30 m) sites.

Although Figure 2.20 shows the majority of waves are heading towards 240-245°, Figure 2.21 shows that this dominant wave dir ection does not concur with the largest wave events. Figure 2.21 presents the percentage occurrence of different wave directions as well as an estimate of average height of events from the corresponding directions. This estimate is calculated by weighting the height of each directional event by its probability and then averaging all events from similar directions. The graph shows that waves from 235° are three times larger than the waves from the peak direction. The conclusion is that local seas generated by wind within the Hauraki Gulf account for the majority of waves at Orewa and come from a more easterly direction than storm events that are from a more northerly source. According to Figure 2.22, swells heading toward Orewa beach at 235° in 7 m of water had a direction of 215° at the 30 m Whangaparaoa hindcast site and therefore storm events are far more likely to pass though Cradock Channel than Colville Channel (i.e. originate from the north). The impact of this on sediment transport can be inferred from Figure 2.23, with the larger wave event more likely to reach the beach at a more oblique angle to bathymetric contours (shore normal is ~250°), resulting in a stronger alongshore current and increased likelihood of sediment transport to the south.

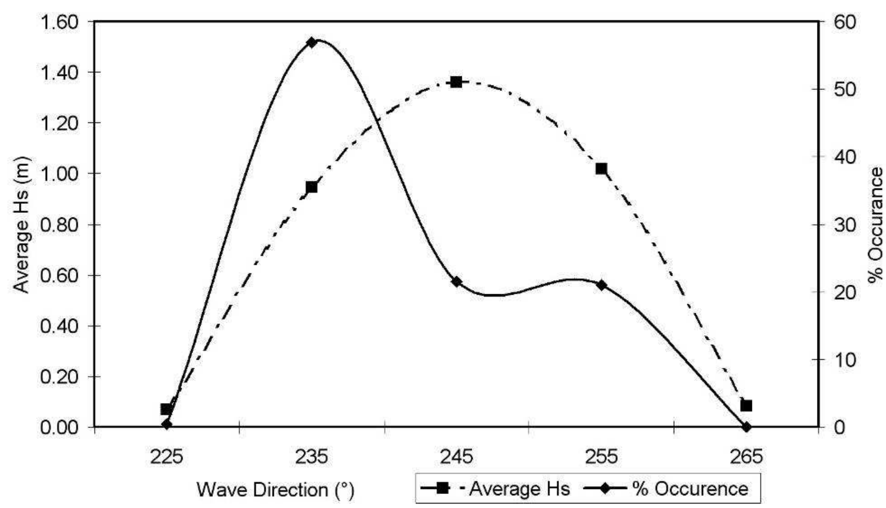


Figure 2.21. Spread of wave directions for Orewa (at 7 m depth contour) plus weight average height (Hs) for each direction

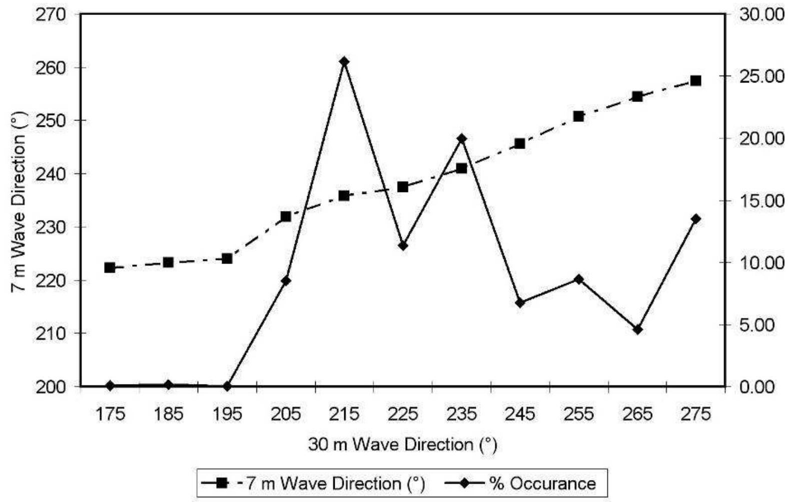


Figure 2.22. Comparison of offshore and inshore wave directions with associated probabilities.

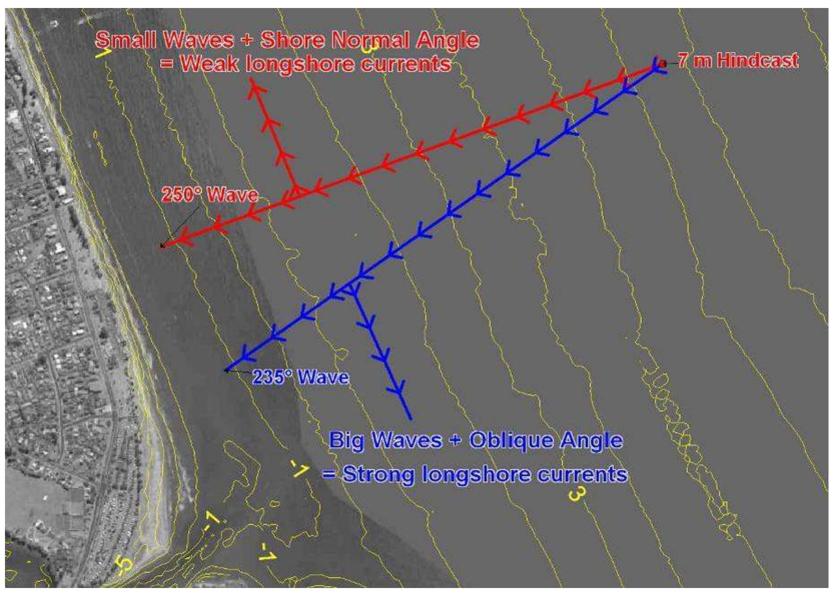


Figure 2.23. Common wave directions at Orewa showing potential effect on sediment transport.

2.5.4 The Inshore wave climate: A closer Look using recent data

For the detailed design study, additional information was available to further refine the inshore wave climate for the purposes of a detailed design. The first source of data that has become available was the ASR MDI (Metocean Data Interface). The MDI contains world-wide wave and wind data dating from January 1997 to the present. The database contains hindcast data of significant wave height, peak period, direction[2] and wind data archived at 3 hour intervals. For Orewa, wave data was extracted from the location 36.5° S

[2] Directions here refer to the direction the waves are coming ***from***. See note 1.

and 175.1° E, as shown in Figure 2.24. The data record for the full 10 years was extracted for the analysis.

In addition to the MDI data, ASR has been providing a local wave forecast for the coasts of New Zealand through its MarineWeather surf forecast system. This system uses offshore wave data that is transformed to nearshore locations through the SWAN wave model. The forecast system has been in operation for just over 2 years (September 2006 – present) and anecdotal reports and observations suggest that the system is accurate in predicting the nearshore wave conditions. This system provides inshore wave data at 12 hour increments. Data was available for the period between January 1, 2007 and September 30, 2007.

SWAN (Simulating WAves Nearshore) is a third generation ocean wave propagation model, incorporating current knowledge regarding the generation, propagation and transformation of wave fields in both deep water and nearshore regions. SWAN solves the spectral action density balance equation for frequency-directional spectra. This means that the growth, refraction, and decay of each component of the complete sea state, each with a specific frequency and direction, is solved, giving a complete and realistic description of the wave field as it changes in time and space.

Physical processes that are simulated include the generation of waves by the surface wind stress, dissipation by white-capping, resonant nonlinear interaction between the wave components, bottom friction and depth limited breaking. The model is described fully in the user manual (Holthuijsen et al., 2004).

The wave spectrum at the outer (offshore) boundaries of the model domain at any particular time is specified, and this in combination with the generation by wind, and transformation within the domain determine the wave field throughout. Dissipation of waves by friction was included in the simulations and used the JONSWAP formula (Hasselmann et al. 1973), with a friction factor of 0.038. Depth limited breaking was included with the method of Battjes and Janssen (1978) with a depth dependent breaking criterion of 0.78. Energy loss by whitecapping was modelled with the Komen et al. (1984) formulation. The water level was set at MSL, and current effects on the wave field were not included.

The MDI data set was first converted to a 'wave rose' diagram for the full 10 year time series as shown in Figure 2.25. Wave rose plots relate the wave height, direction and probability of occurrence. From the full 10 year time series, a subset of the latest 13 months (September 1 2006 – September 30, 2007) was extracted and also converted to a wave rose (Figure 2.26). The similarity in the rose plots suggests that the 1 year data is reasonably representative of the longer tem record. One small difference is the larger wave heights associated with waves coming from the northeasterly direction in the 1 year record (i.e. increase from the 60° sector being relatively minor at <3%). This is an artefact from one extreme event which generated 7 m wave heights at the offshore location in July 2007. This particular event was the largest wave event in the entire 10 year record and is equivalent to a 1 in 150 year wave height event. The slightly greater probability in the 13 month data set than the 10 year data set provides a more conservative or worst case scenario.

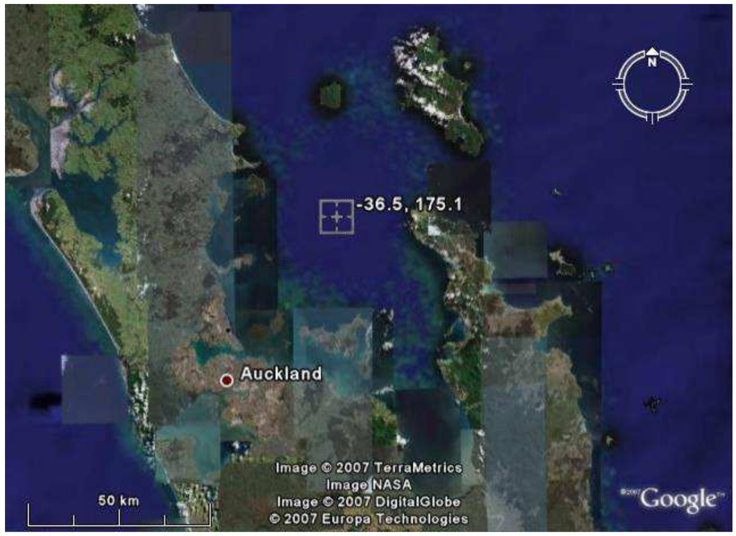


Figure 2.24. MDI data extraction location

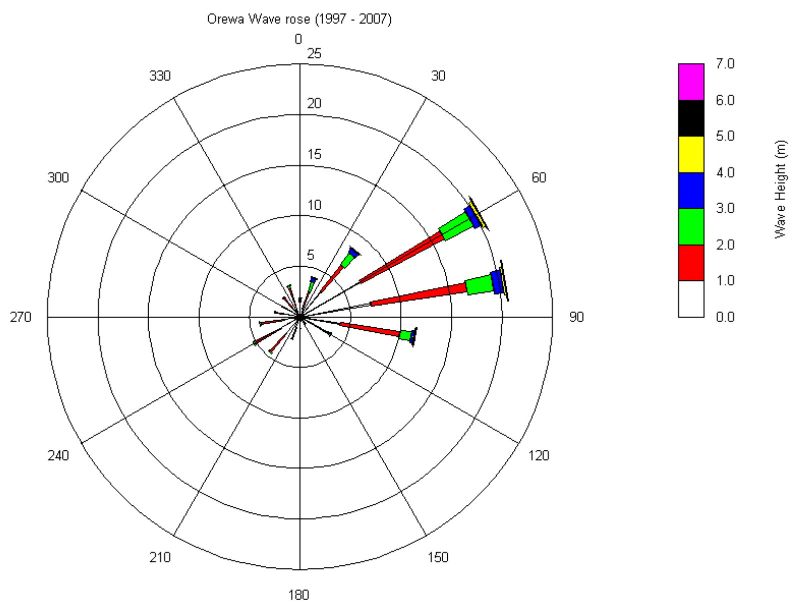


Figure 2.25. Wave Rose for the 10 year data record

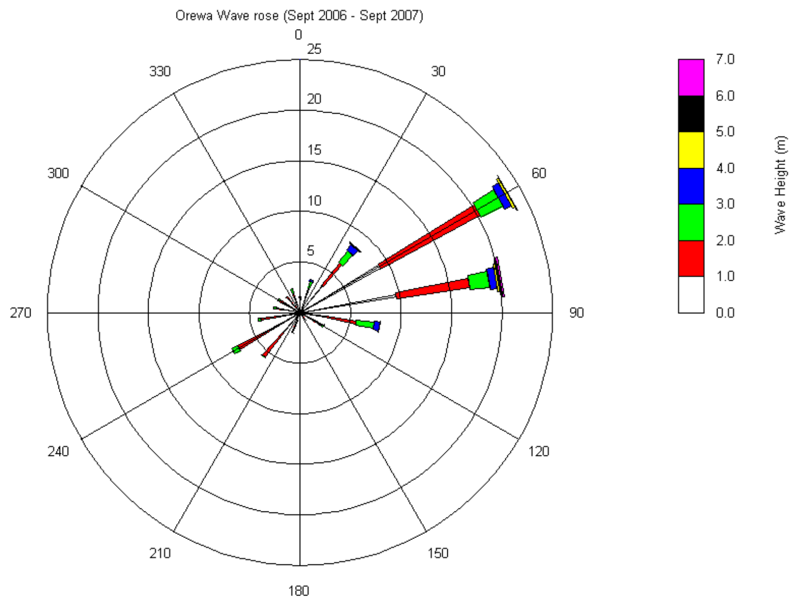


Figure 2.26. 1 year wave rose from the ASR MDI

A similar analysis was performed for the MarineWeather forecast data and the results suggest that the directions and periods for the MarineWeather and MDI data correspond to each other.

In terms of wave height, the data shows that the MDI consistently over-predicts the wave heights relative to the MarineWeather system (Figure 2.27). We explain this by noting the MDI data is resolved on a very coarse spatial grid using offshore wind information. Features such as Great Barrier Island are not present in the MDI wave model and thus the wave heights are over predicted. MarineWeather on the other hand takes in to account more detailed bathymetric features and is therefore more accurate for the wave heights. This was confirmed when MDI data from February 1997 was compared to a record of wave data recorded east of Whangaparaoa Peninsula (see Figure 2.5 'Waverider Location'). The average wave height for the February 1997 MDI data was 28% larger than the Waverider record. Looking at the record for February 2007, the MDI data was 34% larger than the Marine Weather model output. While this is not a conclusive test, it is indicative that the MDI may be only slightly over predicting inshore wave heights. We are therefore confident in using the MarineWeather model output data for generating our inshore wave climate for the purposes of reef design and assessment of the functional performance.

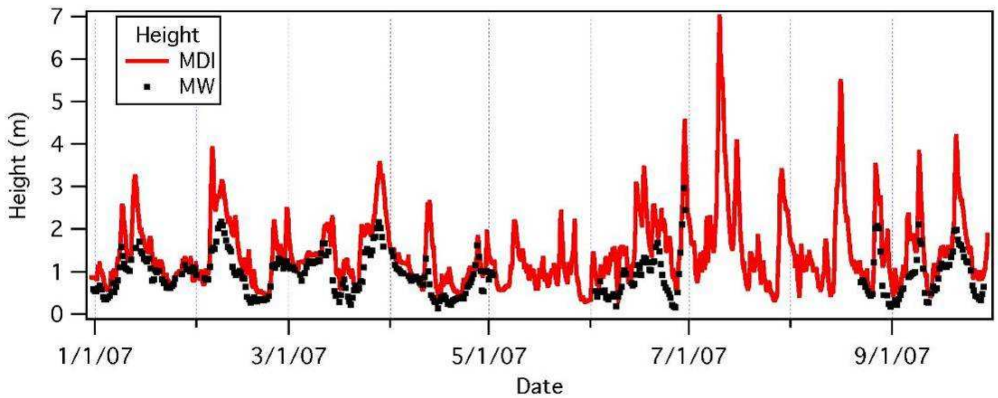


Figure 2.27. Comparison between the MDI wave height data and the MarineWeather forecast model output.

Additional confidence in the development of the inshore wave climate is provided when the measured data from 3 separate deployment periods is validated by the MarineWeather data. Tables 2.6 to 2.8 compare measured and modelled MarineWeather data wave heights, periods and directions (Note, Direction bins are 10° in the mod el output for ease of plotting to the website). The measured wave statistics can be seen to be similar to the modelled wave outputs (sometimes a little higher sometimes a little lower, which mainly due to the differences in the WW3 offshore wave data).

Table 2.6. Measured and modelled wave statistics for Orewa Beach on 10 October 2007.

Hs		Tp		Dir	
Modelled	Measured	Modelled	Measured	Modelled	Measured
0.7	0.54	3.5	3.23	55	55.57
0.5	0.5	8.1	8.58	75	67.23
0.5	0.43	8.1	8.71	35	40.29
0.6	0.43	8.1	8.73	65	57.11
0.5	0.42	8.1	8.63	65	59.58
0.5	0.45	8.1	9.39	65	59.28
0.4	0.4	8.1	9.32	75	57.71

Table 2.7. Measured and modelled wave statistics for Orewa Beach on 21 February 2008.

Hs		Tp		Dir	
Modelled	Measured	Modelled	Measured	Modelled	Measured
0.5	0.64	4	3.87	75	74
0.5	0.68	4	4.08	75	68.4
0.6	0.71	4	4.35	75	67.6
0.6	0.57	4.2	3.67	75	68.1

Table 2.8. Measured and modelled wave statistics for Orewa Beach on 22 February 2008.

Hs		Tp		Dir	
Modelled	Measured	Modelled	Measured	Modelled	Measured
1.2	1.34	6.1	4.82	65	70.12
1.1	1.26	6.1	4.66	65	70.99
1.2	1.32	6.1	5.51	65	71.18
1.3	1.19	6.1	5.65	65	67.78

2.5.4 Tides and Tidal Model Calibration

The mean sea level (MSL), mean spring tide, and mean neap tide were determined from the hydrographical chart at Tiritiri Matangi Island approximately 17 km of Orewa Beach (Table X1). The MSL is 1.7 m above Chart Datum. The mean spring-tide represents a range from 0.5 m to 2.9 m and the mean neap-tide represents a range from 0.8 m to 2.6 m.

The overall tidal range near Orewa is approximately 2.4 m as shown in Table 2.9 (data from New Zealand hydrographic chart no. NZ5321). The tide range experienced at Orewa combined with the very shallow beach slope (<1:100) means that large expanses of beach are exposed during low tide and small swell conditions, however this area becomes fully submerged with the rising tide and additional wave setup.

Table 2.9. Tide levels from Nautical Chart NZ5321 for Tiritiri Matangi,

Tide	MLWS	MLWN	MSL	MHWN	MHWS
Level (m)	0.5	0.8	1.7	2.6	2.9

Comparison of predicted tides (tables), measured tides and modelled tides (the Hauraki Gulf model) is very close, and so, Chart Datum, as used for modelling purposes (either to CD, or MSL measured from CD) is considered to be the same as Lowest Astronomical Tide[3]. Thus, the crest heights of the structures for design purposes are based on the measured tidal data relatively to CD, i.e. we have confidence on the crest heights in relation to the actual tidal levels at the site.

[3] The datum was originally determined from tidal observations at the Port of Auckland, where it was expected that there were be some small difference at Orewa because of tidal phase and amplitude differences (Mead *et al*., 2004), however, confirmation of Chart Datum was attained via comparison to measured water levels.

2.5.4.1 Tidal Inputs

The 3DD model grid has two open boundaries, occurring along the eastern and northern edges in the Hauraki Gulf. For these boundaries, the tidal input is a sea level elevation time series at an hourly interval. The boundary conditions shown in Figure 2.28 were extracted from a global coarse tidal model of the Hauraki Gulf from a process called “Nesting”. Nesting utilizes modelled sea level and velocity conditions at locations within the larger grid, in this case the broad Hauraki Gulf model with a 500 m cell size, and applies the extracted data over the open boundaries of the nested model grids. The Orewa estuary model simulation ran for 20 days to ensure robust coverage of a full 15-day spring-neap tidal cycle allowing for an adequate initial buffering time where the current dynamics stabilize to the driving forces. The bathymetry grid shown in Figure 2.29 was set to chart datum (~Lowest Astronomic Tide) and the mean sea level was determined to be 1.7 m above chart datum in line with measurements and predictions.

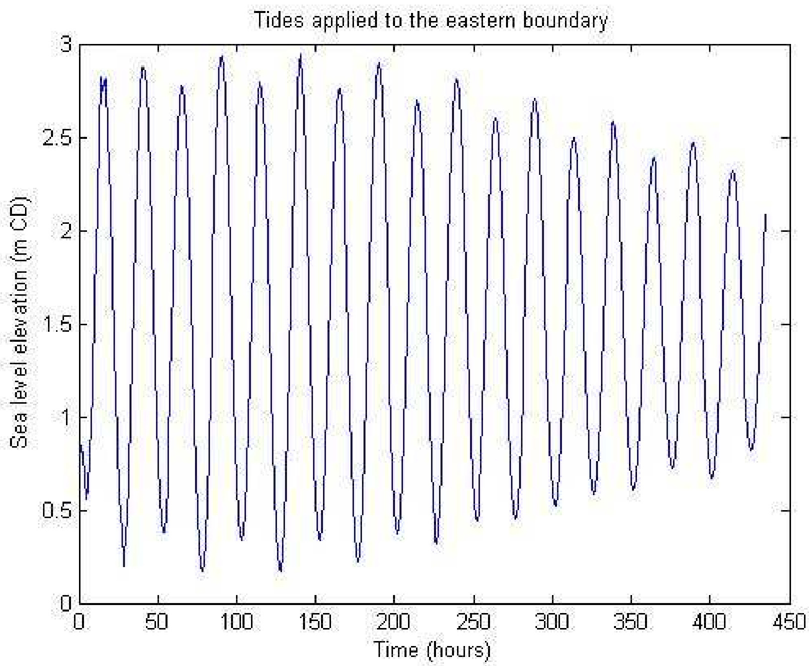


Figure 2.28. Plot of the tidal time series applied to the eastern boundary of the model grid for Orewa estuary model calibration

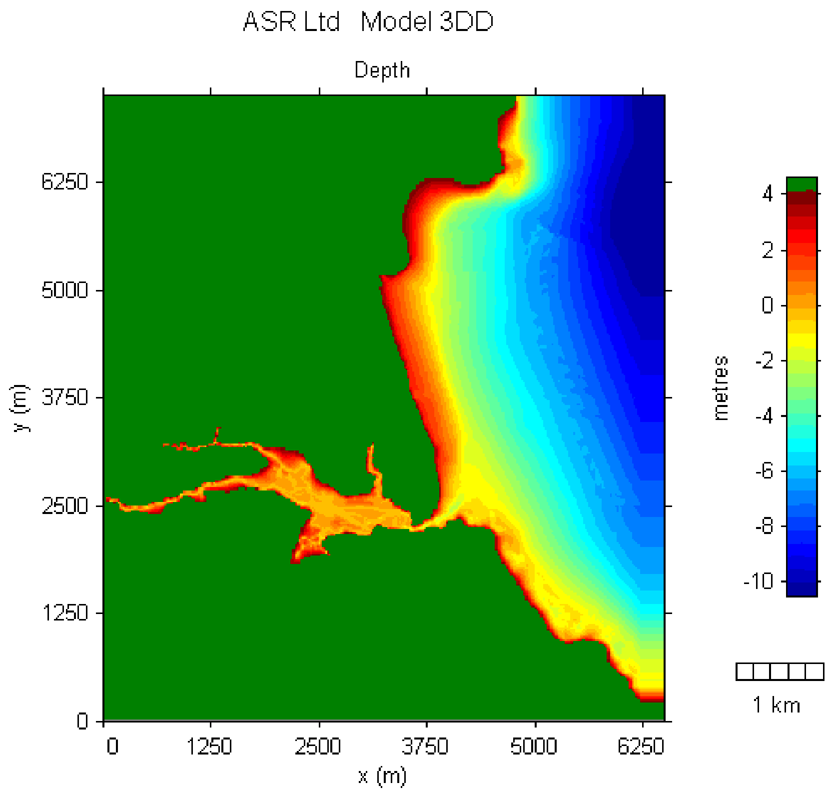


Figure 2.29. The 3DD bathymetry grid of Orewa estuary at 25 m resolution

2.5.4.2 ADV Current Measurements

During a field study of the tidal conditions at Orewa, an Aquadopp current meter was placed at three different locations; the Orewa Estuary channel, the ebb tidal delta of the estuary and in front of the Surf Life Saving Club (SLSC) as shown in Figure 2.30 with positions listed in Table 2.10.

Table 2.10. Position of the aquadopp current meter during the study of the Orewa tide conditions. Positions are indicated in Figure 2.30.

Site	Easting (m)	Northing (m)
Channel	294391.88	5947175.11
Ebb Delta	294893.99	5947403.42
SLSC	294674.16	5948198.27

Figure 2.30. Location of field measurements for the tidal model calibration.

2.5.4.3 Calibration/Validation of the model

The tidal numerical model was calibrated to match the measured current velocities and water levels at those sites where data was recorded by the ADV by adjusting the bed-friction parameter ("resistance length," also called the roughness length) on the northern boundary and setting the edges of the model.

During the field data collection in October 2007, the observed tidal range was approximately 2.1 m. The equivalent tidal range is observed after 374 hours in the model. To allow comparisons between the measured data and the model output the low tide at 2:00am on the 11th of October is set up as the low tide at 374 hours (model time).

Good model calibration of the Orewa estuary numerical model was achieved. The predicted current speeds from the model reasonably match the measurements. Pressures recorded by the ADV were converted to water levels and corrected to the depth of the measurement. Figures 2.31 to Figure 2.36 illustrate the calibration at measurement sites. There is a variation between the current speeds modelled and those predicted at the ebb delta site (Fig. 2.33), which is due to the presence of the unstable eddy formed during the outgoing tide, i.e. the lateral movement of this eddy during the out-going tide, combined with the 3-dimensional complexity of this kind of feature means that the velocity changes as the eddy 'wobbles' over the delta – which is not well reflected at a single point (cell) in a 2D model, and at <2 m deep in this location, is not easily modelled in 3-dimensions. However, the peak and mean velocities, as well as the overall trend in velocities during the out-going tide, are represented in the model output and are of similar magnitudes. The 'shedding' of eddies in the region of the ebb tidal instrument deployment is visible in Figure 2.37.

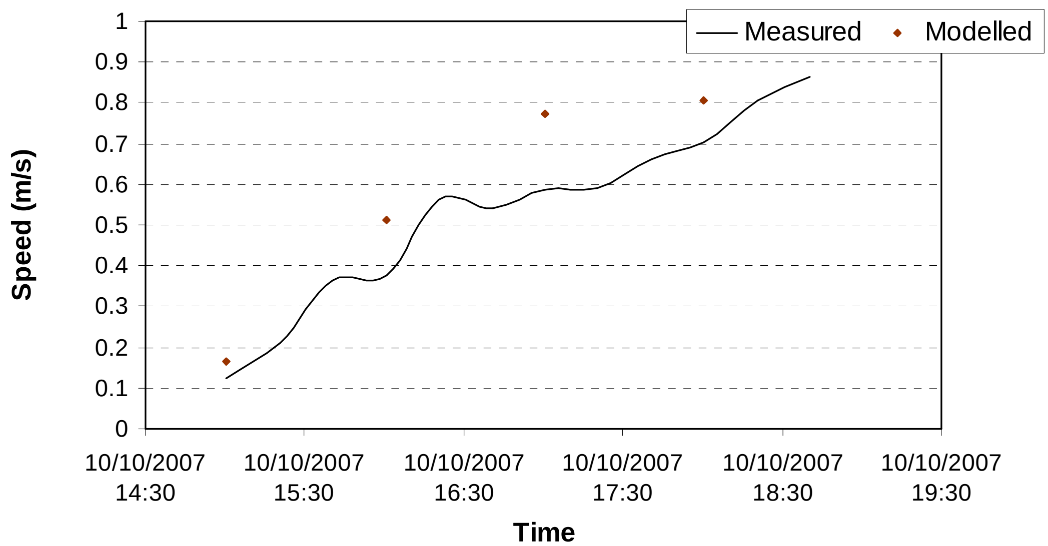


Figure 2.31. Calibration plot of current velocity in the channel.

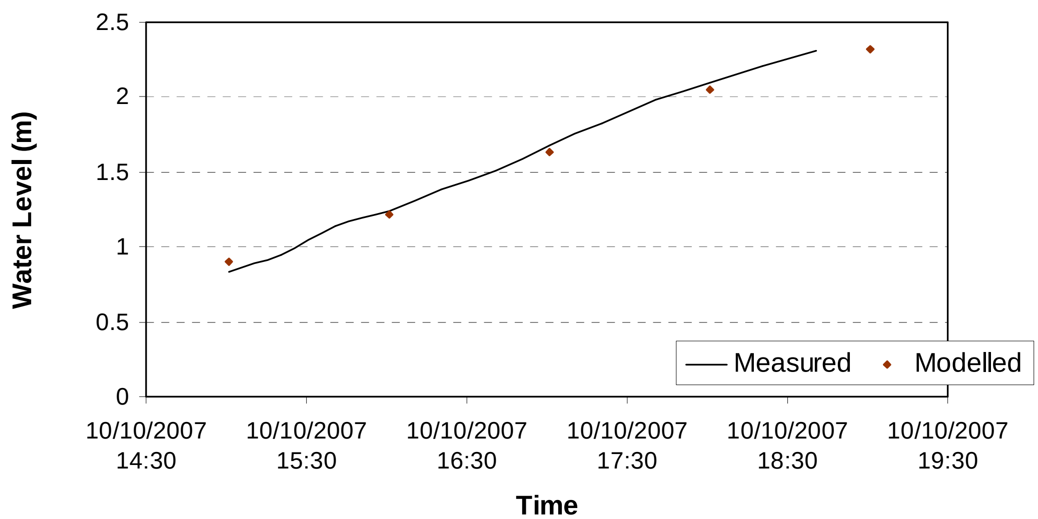


Figure 2.32. Calibration plot of water level in the channel.

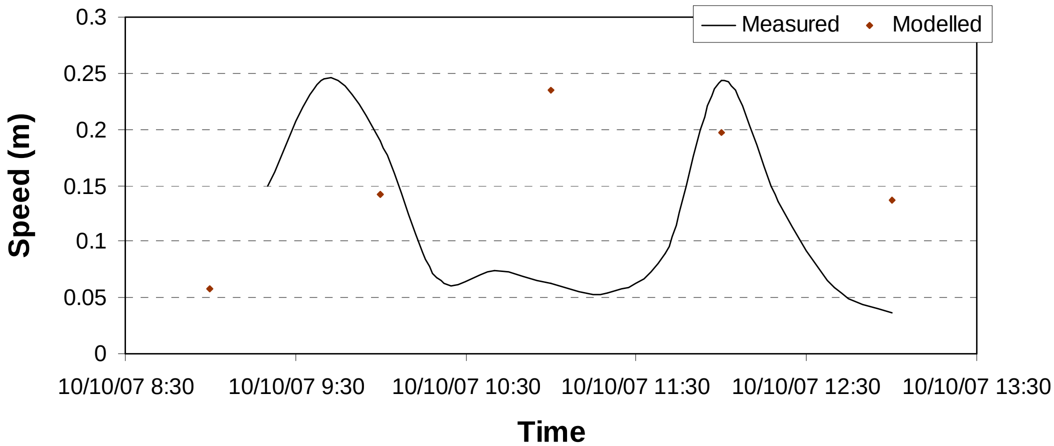


Figure 2.33 Calibration plot of current velocity in the ebb tidal delta.

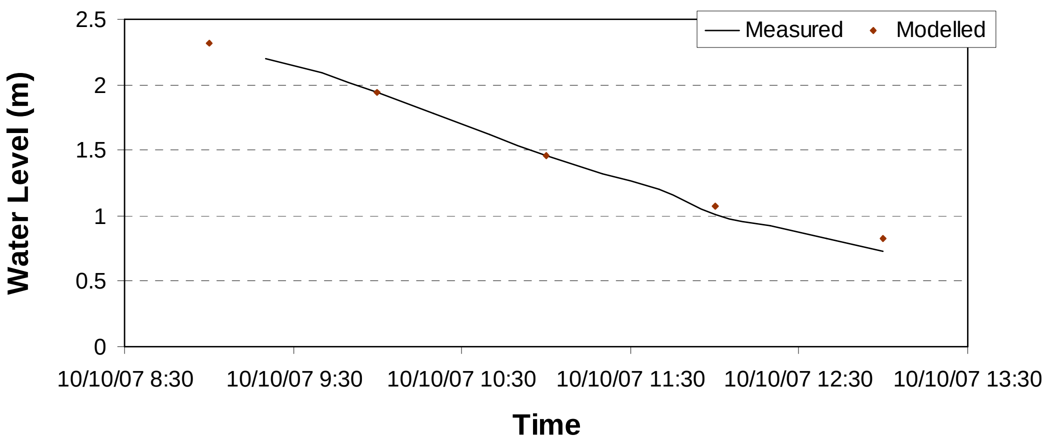


Figure 2.34 Calibration plot of water level in the ebb tidal delta.

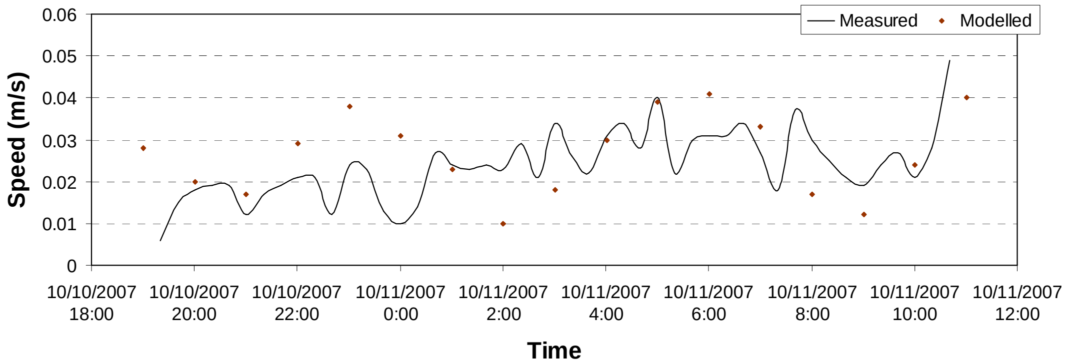


Figure 2.35 Calibration plot of current velocity in front of the SLSC.

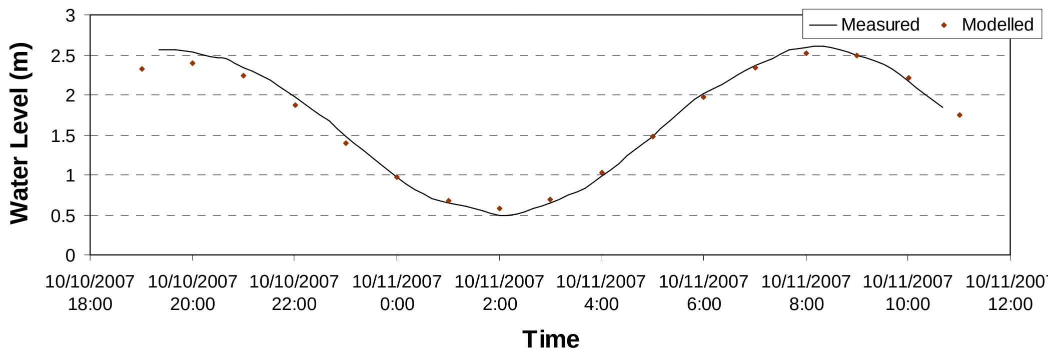


Figure 2.36. Calibration plot of water level in front of the SLSC.

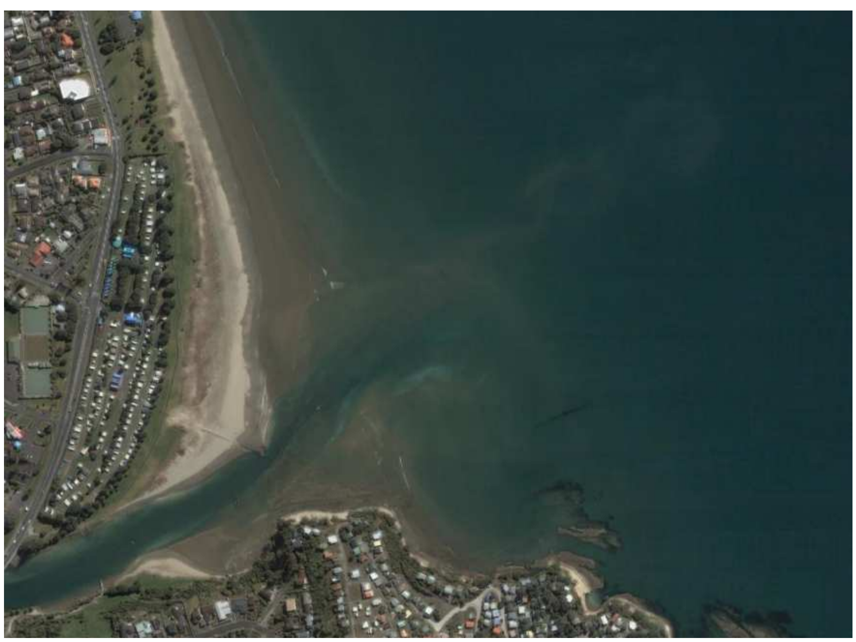


Figure 2.37. Google Earth image showing the variable eddies moving out over the ebb tidal delta (31 August 2004). Such eddies result in unsteady currents as seen in Figure 2.33, which are not replicated in a 2-dimensional model.

2.5.4.4 Output of the calibrated tidal model

Two water level time series were extracted from the calibrated tidal model at the entrance of the estuary channel and offshore of the surf lifesaving club. These time series are used as input sea level boundaries for 2DBeach model simulations. Tide levels from the Orewa estuary channel are applied on the eastern boundary of the 2DBeach model grid and tide levels from the offshore location of the SLSC are applied on the western boundary of the 2DBeach model grid (Figure 2.38).

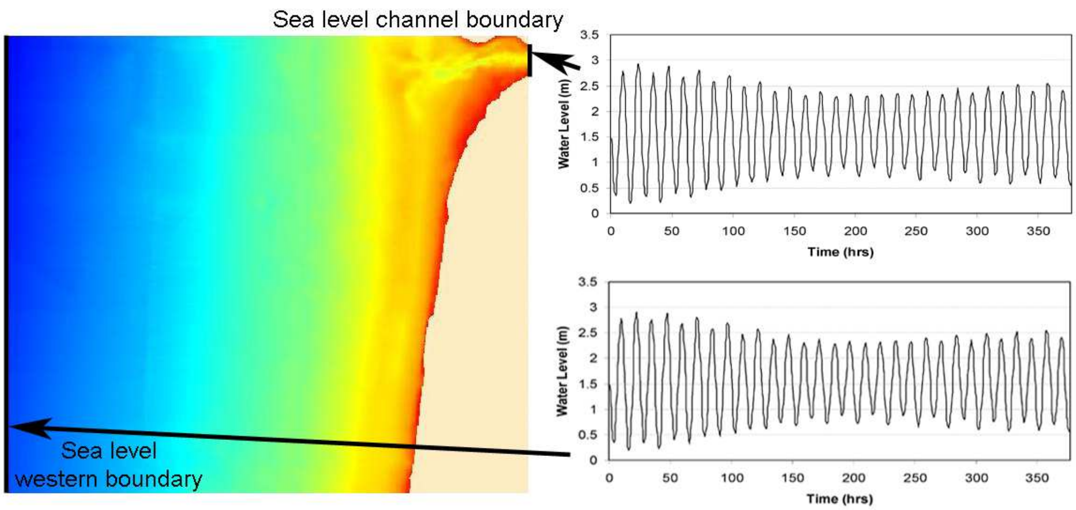


Figure 2.38. Plot of sea level boundaries from the calibrated tidal model on the 2DBeach model grid

2.5.5 Model Calibration to Field Measurements of Current Velocities in the Surfzone

In order to verify the validity of the numerical models, we compared model data to field data collected offshore of Orewa Beach. On February 21 and 22, 2008 a field data campaign was conducted at Orewa beach. The objective was to collect wave and current data during storm conditions to get an indication of the actual velocities experienced in the surfzone.

To accomplish this, a surfzone sled (Figure 2.39) was fitted with an Aquadopp current meter and pressure sensor. A reflecting prism was fitted to the mast and the sled was towed into the surfzone using a Personal Water Craft (PWC, 'Jet-Ski'). The sled was retrieved by using a 6 tonne winch mounted onto a truck. At 20 min increments, the winch was pulled towards shore ~50 m. At each stationary location, the location of the sled was surveyed using a total station. The instrument recorded wave and current data at intervals throughout the day. These data are listed in Table 2.9 and 2.10 and summarized in Figure 2.40.

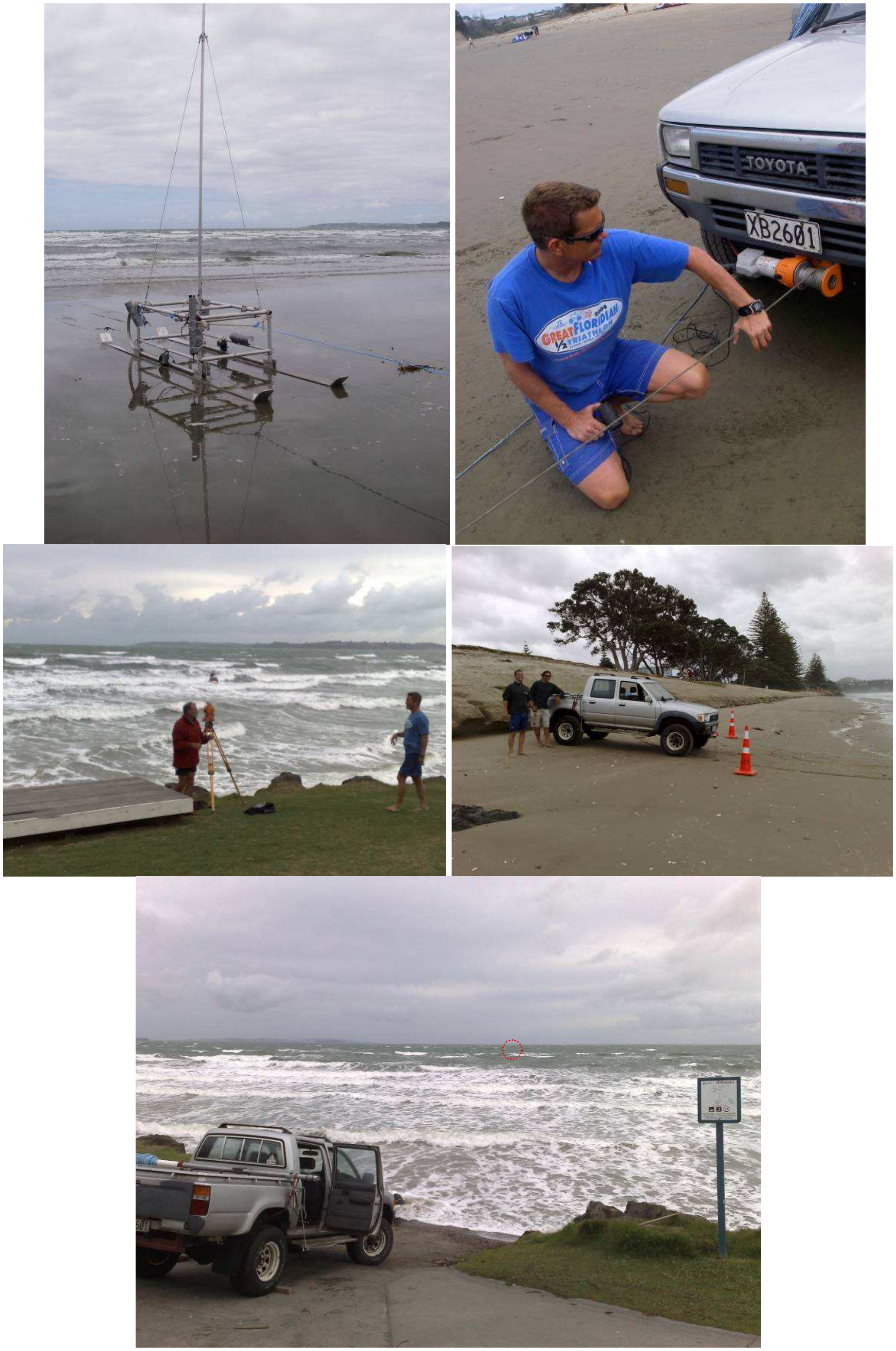


Figure 2.39 – The surfzone sled (top left) with mast and Aquadopp attached. Retrieving the sled with a winch (top and middle right). Deployment of sled and surveying (middle left) and conditions on 21 Feb 2008 (bottom) – the PWC and sled are located in the red circle.

Table 2.11 Measured Wave Data

Time	Hs	Tm	Tp	Dir from
	(m)	(s)	(s)	(deg)
21 Feb				
11:10	0.64	3.0	3.8	74
11:30	0.68	3.2	4.1	68
11:50	0.72	3.4	4.2	68
12:10	0.74	3.7	4.4	64
12:30	0.71	3.5	4.4	68
12:50	0.72	3.4	4.5	62
13:10	0.57	3.2	3.7	68
22 Feb				
10:10	1.26	3.7	4.7	71
10:30	1.34	4.2	4.9	70
10:50	1.35	4.0	5.1	69
11:10	1.32	3.9	5.5	71
11:30	1.19	3.9	5.7	68
11:50	1.08	4.1	5.4	71
12:10	0.9	4.0	5.8	65
12:30	0.76	4.1	4.9	66

Table 2.12 Measured Current Data

Time	Point (in Fig. 2.39)	Speed	Dir to
		(m/s)	(deg)
21 Feb			
11:40	1	0.293	356
12:00	2	0.081	340
12:40	3	0.167	60
13:20	4	0.181	2
22 Feb			
10:00	1	0.097	44
10:20	2	0.167	24
11:00	3	0.333	18
12:00	4	0.434	36
12:20	5	0.327	8

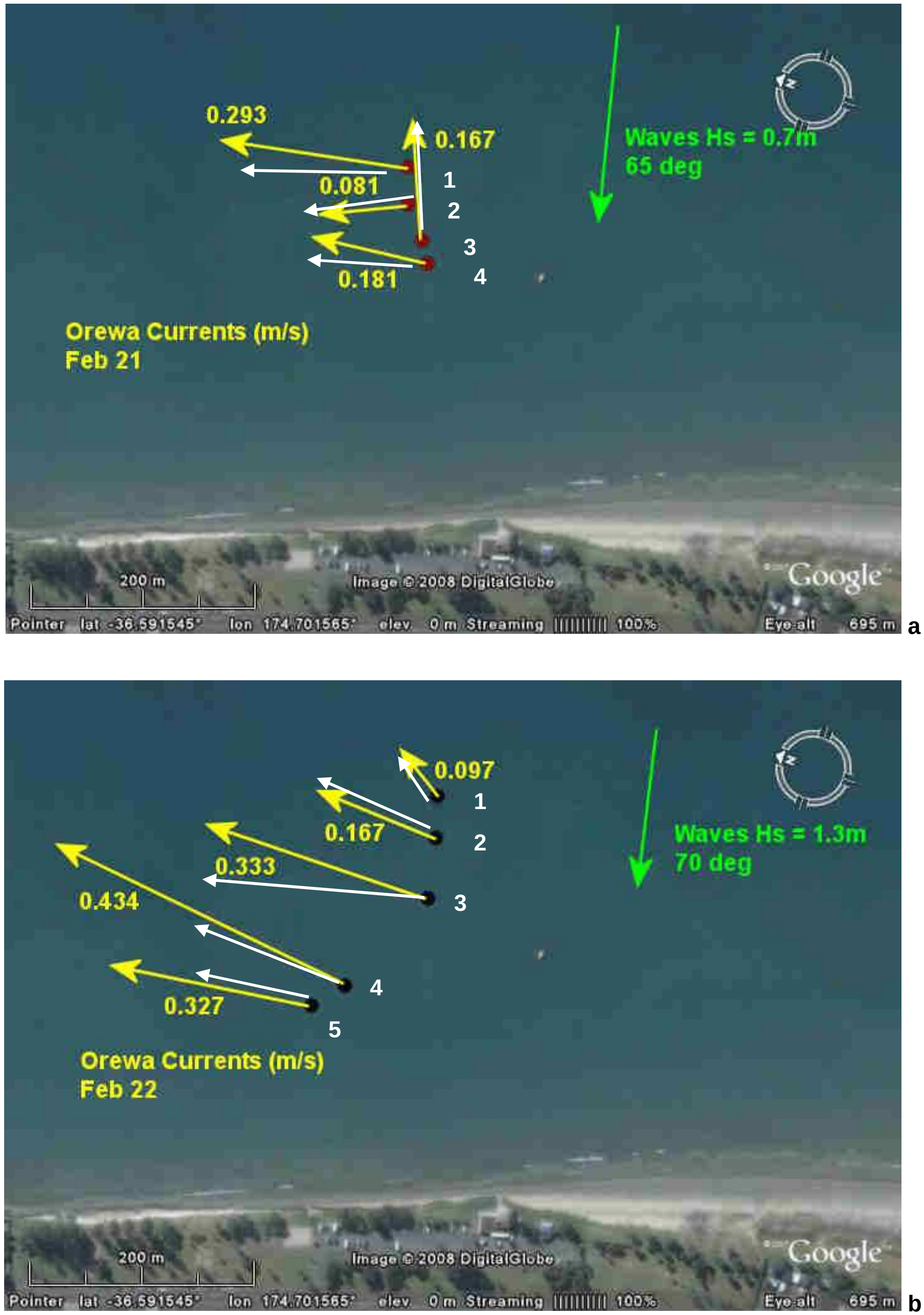


Figure 2.40 (a,b). Graphical summary of measured field data from Orewa for February 21 (upper) and February 22, 2008 (lower). The white arrows show the results of current speeds and velocities from the 2DBEACH model simulations. It can be seen that there is very good agreement between the measured velocities and directions, with the exception of the velocities of the 22nd February's 2 inshore data points, which is likely due to the differences in actual and modelling bathymetry and the intermittent exposure of the Doppler instrument.

During the course of the field work, the wave heights increased through the day along with the wind speed. This is reflected in the recorded wave data (Table 2.11) and meteorological data recorded at nearby airports (Table 2.13). The significant wave height on February 21 was 0.7 m coming from a direction of 65 deg. On February 22, significant wave heights reached 1.3 m[4] with a slightly more easterly approach direction from 70°. With th e wave heights, the measured currents also increased, as shown in Table 2.12. Current speeds on February 21 ranged from 0.08 m/s to 0.29 m/s while on February 22, current speeds were measured between 0.09 and 0.43 m/s. As described in Section 2.5.3, current directions are related primarily to the wave approach direction, with secondary effects due to perturbations of the bathymetry. On both days the wave approach was shore normal to slightly south of shore normal. This forced the wave-driven currents to flow northward along the beach. It is noted that many of the severe erosion events occur with waves from the northeast (as happened the following day at the peak of the storm (Section 2.5.3), which resulted in severe erosion and the consequent need for renourishment in the reserve area), with the short period waves moving sand *offshore* of the beach, and the alongshore currents driving the sand along the shore (either north or south). The data from the fieldwork are used to calibrate the models, which can then be given any input data (e.g. a 5 year return period storm event from the northeast) for a simulation, with good calibration providing confidence in the outputs.

Table 2.13. Weather information recorded at Auckland Airport and Great Barrier Island during the field study.

Auckland Airport

Time (NZDT)	Temp (C)	Pressure (mbar)	Wind dir from (deg)	Wind Speed (km/hr)	Wind Speed (knots)
21Feb					
8:00 a.m.	17	1024	90	5.6	3.0
9:00 a.m.	20	1024	67.5	14.8	8.0
10:00 a.m.	22	1024	90	22.2	12.0
11:00 a.m.	22	1024	90	24.1	13.0
12:00 p.m.	23	1024	45	24.1	13.0
1:00 p.m.	24	1023	45	24.1	13.0
2:00 p.m.	23	1023	67.5	22.2	12.0
3:00 p.m.	24	1022	22.5	24.1	13.0
4:00 p.m.	23	1021	67.5	24.1	13.0
5:00 p.m.	23	1021	67.5	25.9	14.0
22 Feb					
8:00 a.m.	19	1016	67.5	20.4	11.0
9:00 a.m.	20	1016	67.5	27.8	15.0
10:00 a.m.	20	1015	67.5	37.0	20.0
11:00 a.m.	21	1015	67.5	44.4	24.0
12:00 p.m.	21	1015	67.5	38.9	21.0
1:00 p.m.	21	1014	67.5	44.4	24.0

[4] Significant wave height represents the average of the top 1/3rd of wave heights, with the maximum wave heights being 1.86x the significant wave height (i.e. 2.4 m), which relates well to observed wave heights while in the surf zone.

2:00 p.m.	22	1013	67.5	40.7	22.0
3:00 p.m.	22	1012	67.5	46.3	25.0

Great Barrier Island

Time	Temp	Pressure	Wind dir	Wind speed	
(NZDT)	(C)	(mbar)	from (deg)	(km/hr)	(knots)
21 Feb					
7:00 a.m.	19	1024	90	31.5	17.0
10:00 a.m.	21	1024	90	33.3	18.0
1:00 p.m.	22	1024	90	33.3	18.0
4:00 p.m.	20	1022	90	37.0	20.0
22 Feb					
7:00 a.m.	20	1016	67.5	46.3	25.0
10:00 a.m.	21	1016	67.5	51.8	28.0
4:00 p.m.	20	1012	67.5	51.8	28.0

Table 2.14. Tides near Orewa during the field survey.

Date	Time	Tide (m)	Date	Time	Tide (m)
21/02/2008	8:30	**2.88**	22/02/2008	8:30	**2.91**
21/02/2008	9:00	**2.77**	22/02/2008	9:00	**2.94**
21/02/2008	9:30	**2.60**	22/02/2008	9:30	**2.87**
21/02/2008	10:00	**2.37**	22/02/2008	10:00	**2.73**
21/02/2008	10:30	**2.12**	22/02/2008	10:30	**2.52**
21/02/2008	11:00	**1.86**	22/02/2008	11:00	**2.28**
21/02/2008	11:30	**1.61**	22/02/2008	11:30	**2.01**
21/02/2008	12:00	**1.35**	22/02/2008	12:00	**1.75**
21/02/2008	12:30	**1.10**	22/02/2008	12:30	**1.48**
21/02/2008	13:00	**0.88**	22/02/2008	13:00	**1.21**
21/02/2008	13:30	**0.70**	22/02/2008	13:30	**0.96**
21/02/2008	14:00	**0.58**	22/02/2008	14:00	**0.75**

To determine the accuracy of our simulations, the wave and tide conditions during the instrument deployment were used as initial condition for model simulations. The model 2DBEACH was used to determine the overall, steady state current speeds and directions induced during that period.

Model conditions for February 21 were:

Hs = 0.7 m
T = 4 Sec
Tide = Low (+0.5), Mid (+1.7 m) High (+2.9 m)

Model conditions for February 22 were:

Hs = 1.3 m
T = 6 sec
Tide = Mid (+1.7 m) High (+2.9 m)

Figure 2.41 shows the velocity field for the wave conditions of February 21 for the mid tide simulation while Figure 2.42 for February 22. Following the calibration process, the model is in close agreement with the field data, both in terms of velocities and directions. This is particularly evident in the current direction as seen when comparing Figure 2.40 a,b to Figures 2.41 and 2.42 (in Figure 2.40, modelling results are overlaid on field measurements. The models show a trend of current flow towards the north as seen in the field data. In terms of velocities, the magnitude of the velocities computed in the simulations fits well within the range of measured velocities – some discrepancies are expected between the measured and modelled velocities, since the model's bathymetry is not exactly the same as that present during the field work period, which means that rip-cells will be located at different positions along the beach, the tide was continually dropping, and at the 2 inshore sites, the shallow water depth meant that the Doppler instrument was sometimes exposed (reducing average velocities).

Following the calibration of the tide, wave and current components of the models, the design and assessment of functional performance and physical impacts could be undertaken.

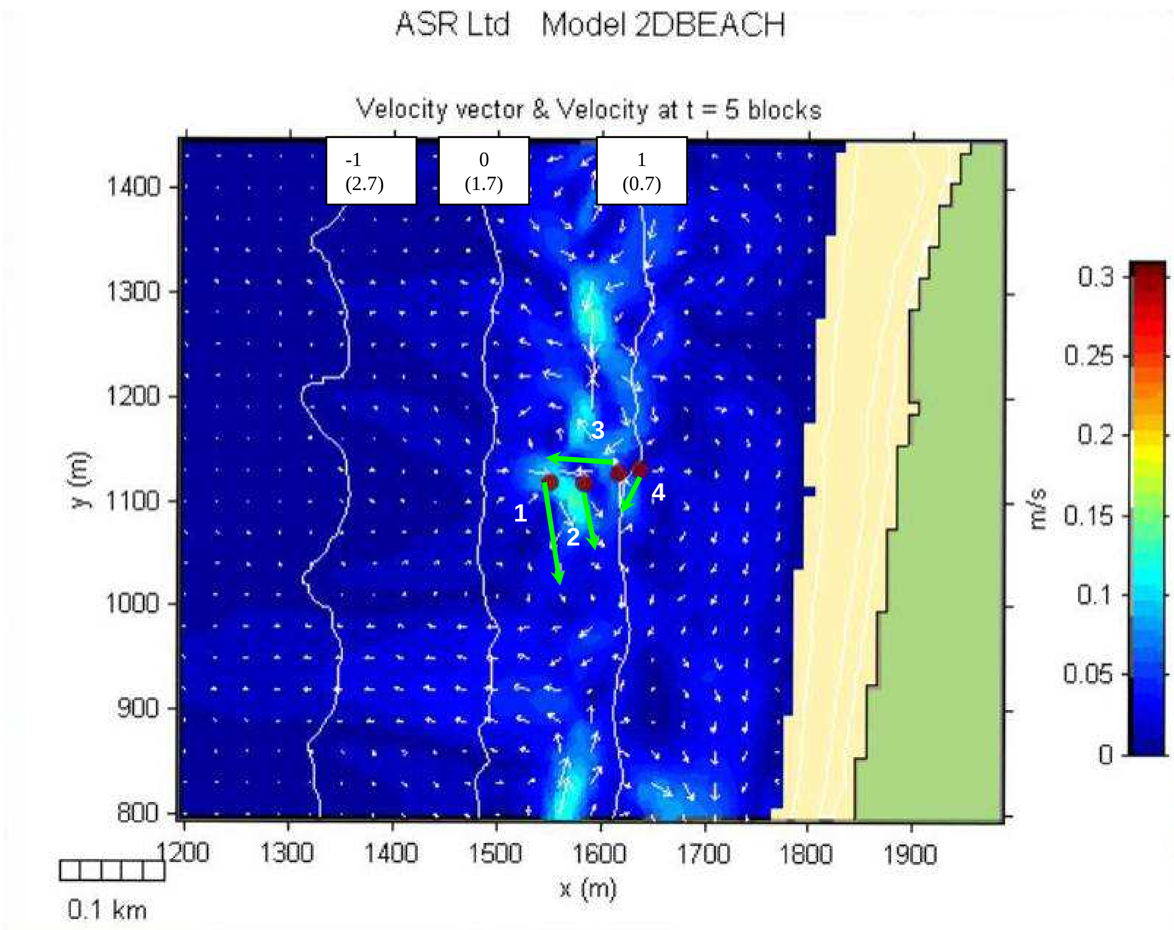


Figure 2.41 – Model results for surfzone velocities on February 21, mid tide. Model water depths are indicated on the figure, which reflect a model sea level of +1.7 m relative to Chart Datum. Thus, the -1.0 m, depth contour would have a total water depth of

2.7 m. The measured current speeds were in the range of 0.1 to 0.3 m/s (refer to Table 2.10) while the model predicts current speeds between 0.1 and 0.2 m/s.

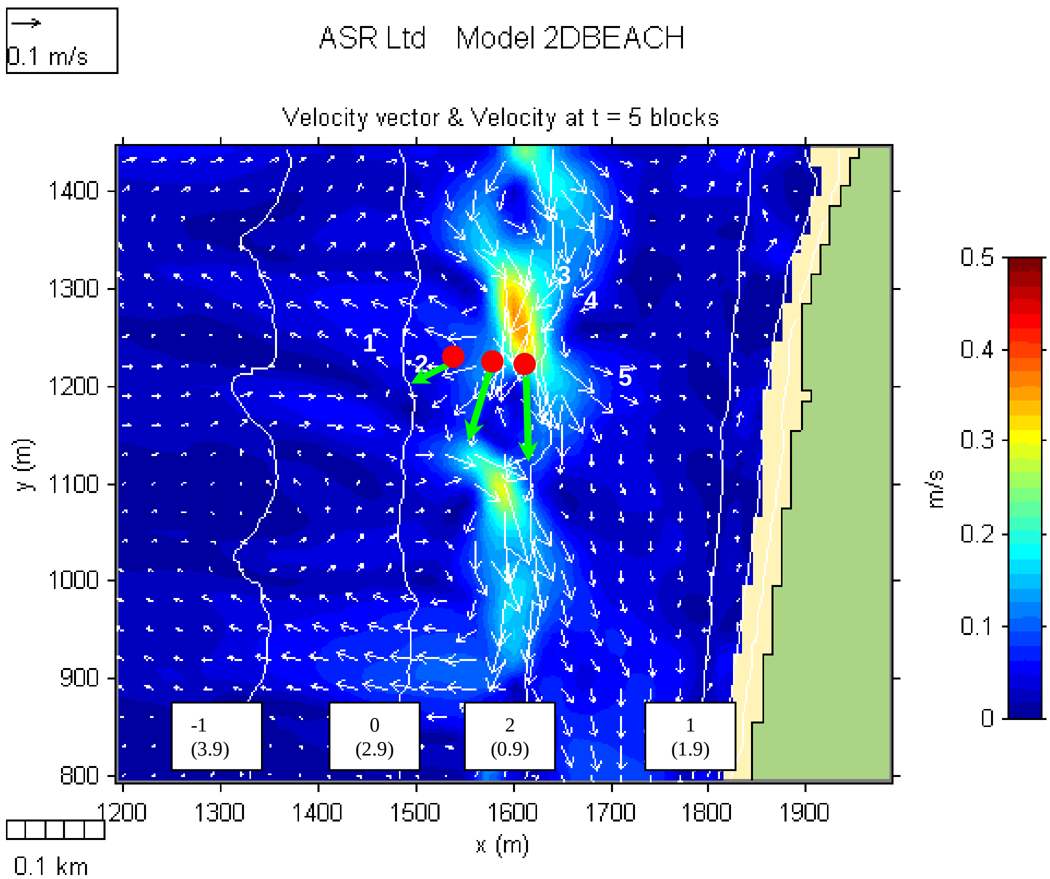


Figure 2.42a. High tide model simulation of the 22nd February 2008.

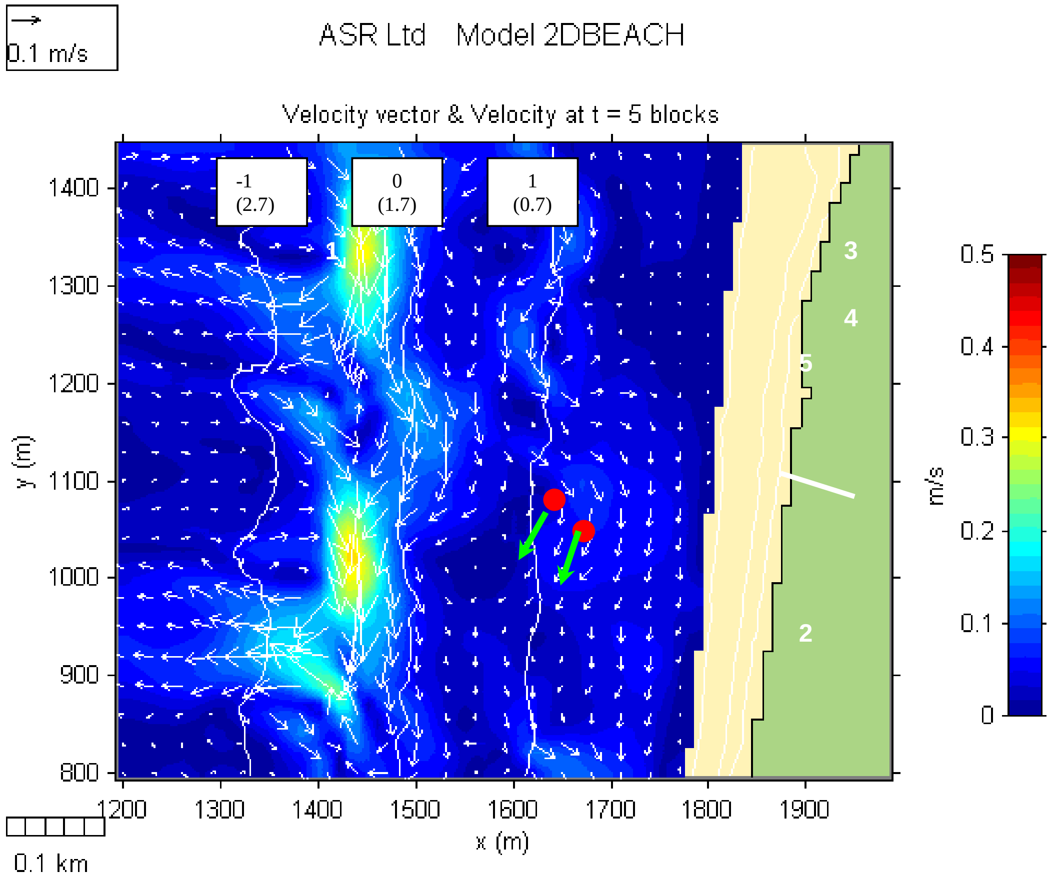


Figure 2.42b – Model results for surfzone velocities on February 22nd 2008, high tide (Fig. 2.42a, upper) and mid tide (Fig. 2.42b, lower). Model water depths are indicated on the figures which reflect a model sea level of +1.7 m relative to Chart Datum for mid tide and +2.9 m relative to CD for high tide. Measured current speeds were between 0.1 m/s and 0.4 m/s (refer to Table 2.10) which is the range of values predicted by the model for the surfzone.

2.6 Shoreline Stability and History of Coastal Protection Works at Orewa

An important conclusion that can be drawn from the available information is that Orewa Beach suffers from erosion only as a result of significant storm events and that under normal conditions the beach is in a state of ***'dynamic equilibrium'***. It is the long history of human modifications to the natural environment (sand mining, building on the dunes, the estuary realignment, construction of rock sea walls and the construction of the Waitemata Groyne) that has fundamentally changed what is ***'normal'*** for this beach. Indeed, the beach profile data do not show an erosion trend, only a loss of renourished material to the ***'normal'*** beach profile.

Perceived erosion did not become an issue at Orewa until the 1960's when construction and development began in the central section of the beach, seaward of main road. When strong storms generated waves, tides and surge which threatened these early structures, the community responded by building seaside barriers made of tipped rock (rip-rap, rubble, etc…). Indeed, the central section of the beach was the first part to experience documented erosion (Tonkin and Taylor, 1992, 1993).

Raudkivi (1981), and following authors (e.g. Tonkin and Taylor, 1992) describe Orewa Beach as an isolated pocket beach, with little input of 'new' material, rather sediment is 'redistributed' along its length. Following Raudkivi's (1981) report, beach profile monitoring was initiated to help determine the cause of any ongoing erosion problems on Orewa Beach. In 1986 the southern groyne was modified and strengthened and in 1988 the first nourishment of Orewa Beach was undertaken, utilising sand from the estuary. This was the first of a series of sand nourishment projects, which have continued up to the present day.

In 1991 the RDC, recommended putting sand on the beach and stated that it was willing to accept continual nourishment to maintain a dry beach area. Monitoring at the time indicated that the nourishment was only slowly being lost, and suggested that there was a reduction of infilling of the estuary, which was initiated by the re-alignment of the estuary in 1959 and possibility enhanced by the modifications to the Waitemata groyne in 1998.

By 1993, it was suggested that the major cause of erosion of Orewa Beach was due to the realignment of the estuary mouth. In 1959, the natural estuary channel opening was realigned using explosives by the New Zealand Navy. This was done to modify and reduce the tidal currents which were responsible for a number of drownings north of the estuary entrance. As a result, there was a change in the circulation patterns that naturally deposited sediment to the north. Although it was noted that the rock walls and reduction of the estuary tidal prism (due to construction of oxidation ponds) were also part of the equation, it was the change in the estuary mouth alignment which resulted in the estuary becoming a sediment 'sink'. An average accumulation of 7,500 m^3/year was estimated between May 1989 and November 1992.

Parkin (1994) was the first to take a holistic view of the reasons behind the beach erosion at Orewa e.g. tipped rock walls, estuary re-alignment, loss of natural dunes, changes in wave climate (e.g. ENSO, IPO, etc.). Even so, it is suggested that the beach was stable until rock walls were constructed and that the change to the estuary entrance impacted on the overall coastal processes of the area. Parkin (1994) suggests that the northward flow of ebb-tide countered the littoral drift to the south from the middle of the beach. When the ebb-flow was lessened, it resulted in a reduction in the return of attendant sand to the southern and central part of the Orewa Beach. This is supported by the filling of the estuary and numerical modelling of the differences in the channel position (Mead *et al.,* 2004a) – lower currents through the ebb-tidal delta allow sediment of move into the estuary.

In 1995, the University of Auckland undertook a physical modelling study of Orewa Beach to test a variety of coastal protection methods (groynes, offshore reefs/breakwaters, removal of south groyne, re-alignment of the estuary and combinations of all). Two further reports analysed the results in greater detail and looked at additional modelling cases arising from the most successful in the initial model runs (i.e. offshore reefs, new groyne, removal of old groyne, filling estuary channel and sand transfer). The interesting points that came out of the physical modelling exercise included a broad assessment of the existing sediment transport regime, which demonstrated a possible effect of the channel re-alignment and an indication that the most effective solution to retain beach sand is offshore reefs.

The modelling also indicated that returning the channel to the pre-1959 alignment had a positive impact on sediment retention on the beach. This then became one of the main recommendations of Tonkin and Taylor (1996). However, factors associated with filling the estuary channel such as the overall cost of such a project, the re-creation of the safety hazard that previously existed in front of the campground and the fact that the beach seems to have found a new equilibrium shape in the intervening years make this recommendation difficult to rationalise. Indeed, it was eventually concluded that an offshore reef was the best option in that such a structure would retain sand on the beach more effectively than a new groyne part-way between the surf club and the south groyne.

Tonkin and Taylor's (1995) report provides some useful information with respect to sediment movements of nourishment material placed on the central part of Orewa Beach. The notable comments include a confirmation of slow loss of beach sand in a southern direction, with monitoring showing accretion of sand in the area north of the groyne from where it was taken. The report also pointed out the need for new coastal 'control' points (i.e. offshore reefs) since the overall sediment dynamics have changed since the realignment of the estuary and the original groyne construction.

Tonkin and Taylor's 1996 and 1997 reports follow on from the results of the University of Auckland physical modelling. As Davis (1999) pointed out, there are some significant inconsistencies with the results of the monitoring programme and the recommended beach protection strategy. Even though it is repeatedly stated that there is no indication of continuing erosion of Orewa

Beach, it is suggested that the estuary channel is filled to return the flow pattern to pre-1959, that rock walls should be built in parts of the beach where they are not already present (even though earlier reports conclude that the rock walls are causing the erosion) and that nourishment should also be continued. These measures may have been recommended to provide a dry beach (although this intention is not made clear in the report), but we considered them extreme if the beach is in dynamic equilibrium.

Contrary to the results of the University of Auckland physical modelling, it is suggested that an offshore reef(s) will not work unless sand is placed on the beach. With the results of the physical modelling, continued beach nourishment and the need for a 'new' coastal control point, it is difficult to understand why continued investigation into the application of an offshore reef(s) was not recommended.

Davis (1999a) concluded storm damage occurs during north easterly to northerly storm events, which is well supported by the earlier nourishment monitoring. It is noted that the groyne (sink) and channel diversion provide a good source of sand for beach management, and recommended that the beach monitoring and nourishment be reduced to target storm damage (i.e. the loss of dry beach during storm events), nourishment as a remediation. In this regard it is noted that ARC are not opposed to sand transfer, if justified. However, this approach raises the question of sustainability and accumulative costs of renourishment.

Davis (1999a) describes rock revetments as a primary method of protection, rather than nourishment, i.e. if the sand is removed the rock walls will protect the land behind them. It is suggested that nourishing the beach for amenity reasons should be considered at a later date. While this makes sense when the continued cost of nourishment is considered, especially since loss of beach sand is directly related to particular storm events, developing a coastal protection scheme that unifies beach protection and enhanced amenity is likely to have greater benefits, since many methods of coastal protection (e.g. the rock walls at Orewa) lead to increased loss of the beach.

Davis (1999b) describes the long-term sustainable management strategy for Orewa Beach. It is noted that there are 8 storm water outlets on Orewa Beach, 7 in the northern half of the beach, and although they have been implicated in coastal erosion problems at other sites in the past, it is concluded that other than local scour, this is not likely to be the case at Orewa. The main goals of the long-term sustainable management strategy include continued 'soft-engineering' for beach protection and enhancement of the beach amenity.

This review of beach erosion at Orewa indicates that the beach is currently stable (in dynamic equilibrium) after changes over the last century caused by a variety of human actions. It is clear that an appropriately designed multi-purpose reef is in harmony with the long-term sustainable management strategy since it works in with the preference for 'soft-engineering' by increasing the effectiveness (i.e. an offshore reef can retain material placed on the beach) and supports the goal to enhance the beach amenity in a sustainable way (beach is Orewa's greatest natural asset, but is presently virtually non-existent at high tide). Indeed, such an approach is complimentary to the proposed Beach Enhancement /Esplanade strategy for Orewa Beach.

Table 2.15. Summary of Physical works on Orewa beach, reproduced from Davis, 1999a. and Beca Carter Hollings and Ferner Ltd., 2006.

Action	Quantity of Material (m^3)	Date
Sand Mining		1941 – 1944
Low Tech Groynes		1954 - 1957
Estuary channel realignment		1959
Tipped rock walls (ad-hoc)		1960 – 1980
Widening of estuary channel and construction of Waitemata rock groyne		1961
Waitemata groyne modified		1986
Central beach sand renourishment from estuary	20,000	Sept/Oct 1988
Sand renourishment of central beach, ex groyne ex groyne	32,000 18,000	June 1994 October 1994
Sand renourishment of central beach, ex groyne ex groyne	17,500 10,000	June 1995 October 1995
Sand renourishment of central beach, ex groyne	28,000	October 1996
Sand renourishment of central beach, ex groyne	25,000	June 1997
Sand renourishment of central beach, ex estuary ex groyne	17,500 12,000	August 1998 October 1998
Sand renourishment of central beach, ex groyne	25,000	1999
Sand renourishment of central beach, ex groyne	25,000	2000
Sand renourishment of central beach, ex groyne	25,000	2001

2.7 Summary

Orewa's long sandy beach is considered its major attraction and the most significant natural resource in the area. However, since early last century a number of contributing factors have dramatically reduced the width of 'dry' beach along much of its length. It is very difficult to implicate any single factor as the primary agent for the existing state of the beach, it is most likely to be a combination of all. In recent years sand has been placed on the beach to increase the dry beach area and protect the back beach from further erosion that can occur during storm events, but the nourish material does not stay in place in the long term. Beach profile monitoring over the past two decades indicates that the beach is currently stable (in dynamic equilibrium), with the equilibrium beach orientation controlled by the northern headland and the southern groyne. The monitoring also shows that sand borrowed from the southern groyne 'sink' and placed to the north (centre of the beach), is transported back to the south and into the sink area. Figure 2.43 summarises the impacts and physical processes at Orewa Beach.

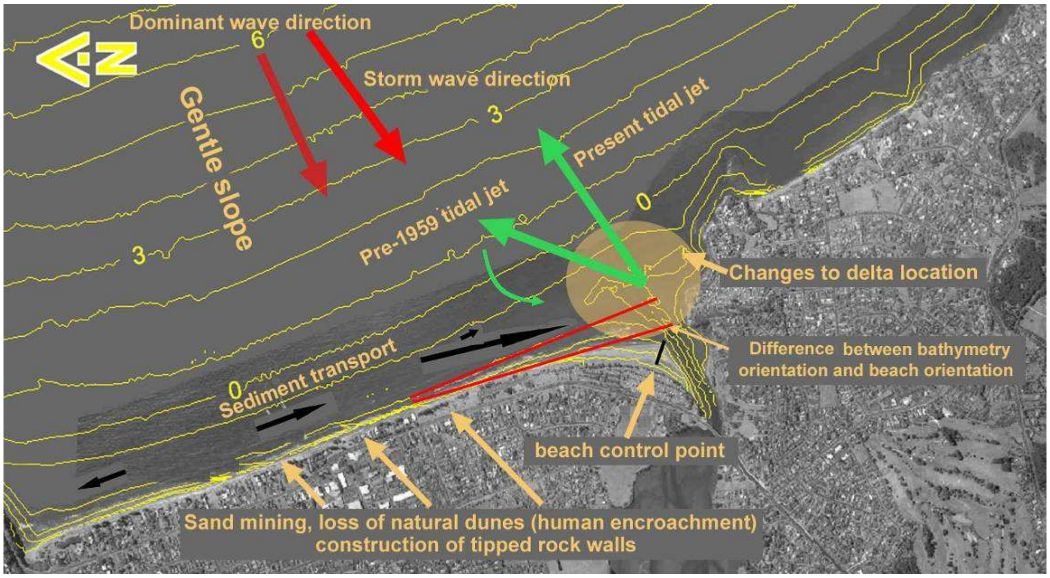


Figure 2.43. A basic summary of the historic impacts/changes to Orewa Beach and the coastal processes operating in this area. The red arrows indicate wave attack, with the bolder arrow representing the storm events from the east to northeast that drive sediment to the south. Sediment transport modelling supports this southerly sediment transport and suggests that it is the shallower beach contours that are more susceptible to impacts during storm events, which is supported by the monitoring of nourishment. The re-direction of the estuary entrance in 1959 has resulted in a tidal jet that is directed out to sea rather than close to the shore and northwards as it was prior to re-direction. While this has likely had some impacts beach sand (in conjunction with a range of human impacts over the past century), an important point in the present study is that this suggests that the Waitemata, or Southern, groyne is the main beach control point, rather than the ebb-tidal delta.

In terms of project options and alternatives that have been previously explored, a 1995 study by the University of Auckland undertook physical modelling studies of Orewa Beach to test a variety of coastal protection methods (groynes, offshore reefs/breakwaters, removal of south groyne, re-alignment of the estuary and combinations of all). While these studies have several limitations, the modelling provides a comparative assessment of the various combinations of coastal control options, and gives an indication of which will be the most effective at retaining beach sand in the central and southern areas of the beach. <u>The results indicated that the most effective solution to retain beach sand at Orewa Beach is an offshore reef(s)</u>. Placement of an appropriately designed submerged reef(s) is in harmony with the long-term sustainable management strategy since it works in with the preference for 'soft-engineering' by increasing its effectiveness (i.e. an offshore reef can be used retain material placed on the beach) and supports the goal to enhance the beach amenity in a sustainable way.

CHAPTER 3 – MULTIPURPOSE REEF DESIGN

3.1 Introduction

This section describes the overall design for the Orewa Beach Multipurpose Reef System. Three numerical models from the 3DD Suite of Coupled Models were used for preliminary reef design and assessment of the functional performance – 3DD, N-GENIUS and 2DBEACH – with a large number of model simulations utilizing the sophisticated model 2DBEACH to consider both hydrodynamic and sediment transport impacts of 1, 2, 3 and 4 sets of reefs under 'normal', 1, 10 and 100 year wave events. The detailed design study continues on from the basic reef parameters determined in the preliminary design.

For the detailed design, additional wave climate and tidal information was available to further fine-tune the models and hence the reef position and shape. Additionally, advances in the numerical modelling tools as well as design guidance from recent reef projects and research publications in the open literature have all contributed to the design presented here.

The design process focused on three main issues, wave attenuation, wave rotation and salient formation/widening of the existing beach. Wave attenuation is the degree to which offshore wave heights are reduced inshore of the reef. This quantity is primarily dependent on the reef geometry – cross shore width, along shore length and depth of submergence (Tajziehchi and Cox, 2007).

Wave rotation refers to redirecting wave crests in order to reduce the alongshore component of wave energy flux that generates alongshore currents and removes sand from the beach (Black and Mead 2001). Wave rotation can also promote sand retention during periods when waves pass over the reef without breaking. At the Orewa site, the erosive alongshore currents are directed to the south. Thus, waves need to be rotated in a more northerly direction to decrease this southerly directed current. A detailed description of wave rotation, including case studies, is included as an Appendix.

Salient formation occurs as a result of the combined effects of wave attenuation and wave rotation. On sandy shores, natural reefs and islands as well as man made structures such as breakwaters create wider beaches, termed salients[5] or tombolos[6] (Figure 3.1), due to sediment deposition in their lee. While manmade structures have previously been built offshore to afford coastal protection, a thorough understanding of salient formation and impacts

[5] A salient is a build up of sand in the lee of an offshore structure that *does not* attach to the structure that formed it and so enables sediment to bypass between the obstacle and the shore and is therefore less likely to cause erosion on the adjacent coastline.
[6] A tombolo is a build up of sand in the lee of an offshore structure that *does* attach to the structure that formed it, blocking sediment movement alongshore and thus usually resulting in erosion of the downcoast shoreline.

has often been incomplete resulting in over-design and negative impacts in the aesthetic and amenity value of the coast. Recent, studies have identified the critical parameters that govern the formation of salients and tombolos and defined methods to predict the shoreline response in the presence of offshore obstacles of known dimensions (Black and Andrews, 2001a & b; Andrews, 1997).

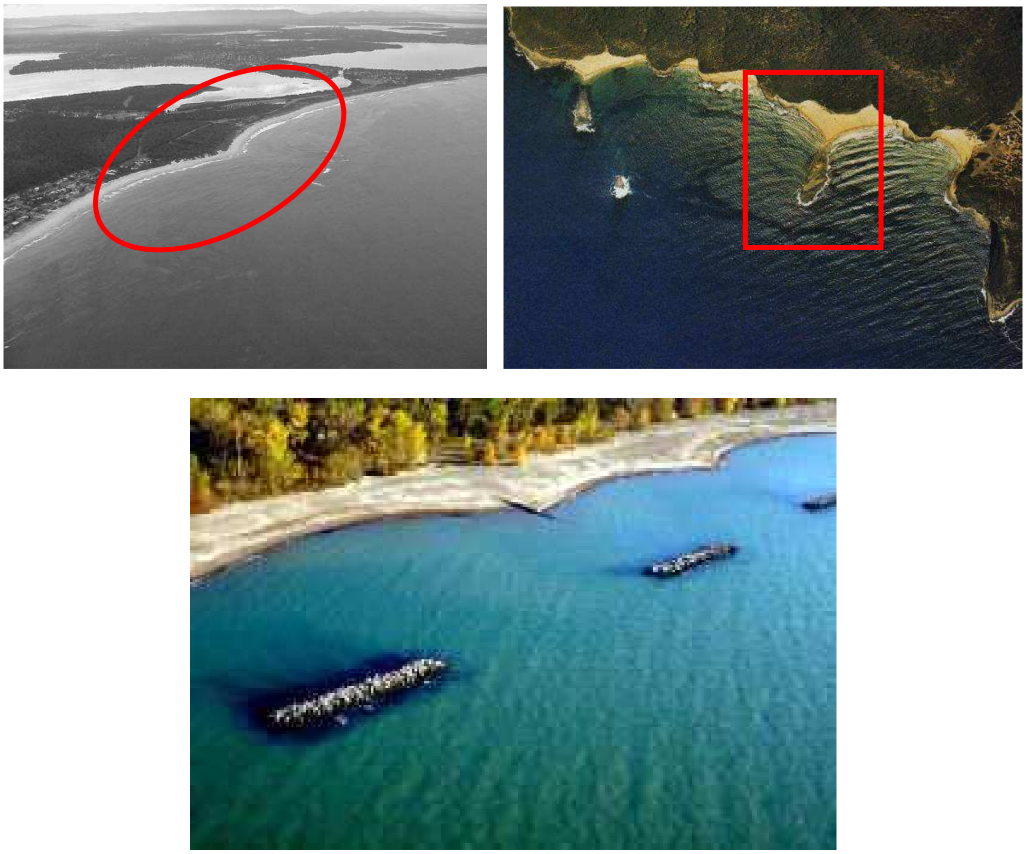


Figure 3.1. A natural example of a salient shoreward of a submerged reef (top left) and a tombolo in the lee of an emerged reef (right). (From Black and Andrews, 2001a). Lower image is salient formation in the lee of man made, shore parallel breakwaters (US Army Corps of Engineers).

For coastal protection, structures that lead to salient formation are preferred because the gap between the offshore reef and the shore still allows alongshore transport of sediment (e.g. Black *et al*., 1997; Black *et al*., 1998; Black *et al*., 2000a & b; Mead *et al*., 2001), unlike a tombolo, which effectively acts as a groyne and leads to negative down-coast impacts (Bush *et al*., 1996; Bruun, 2000; Nielsen, 2001). Over 350 natural cases of offshore coastal protection, such as those presented in Figure (3.1), were identified on the New Zealand and eastern Australian coastlines from aerial photographs (Andrews, 1997). To confidently amalgamate the recreational and coastal protection aspects, accurate predictions of outcomes prior to construction of offshore reefs are required, including the expected adjustments of the beach (Black and Andrews, 2001a). Care is required both to optimise the benefits of the structures and to minimize or eliminate any negative shoreline impacts (Black, 1999; Black and Andrews, 2001a). On the Gold Coast in Queensland, Australia, the world's first multi-purpose submerged reef has achieved coastal protection by salient formation with no down coast impact (Figure 3.2) (Turner, 2006 – a short case study of this project is presented in Section 1.8 of the Feasibility and Preliminary Design Study (Mead et al., 2004a).).

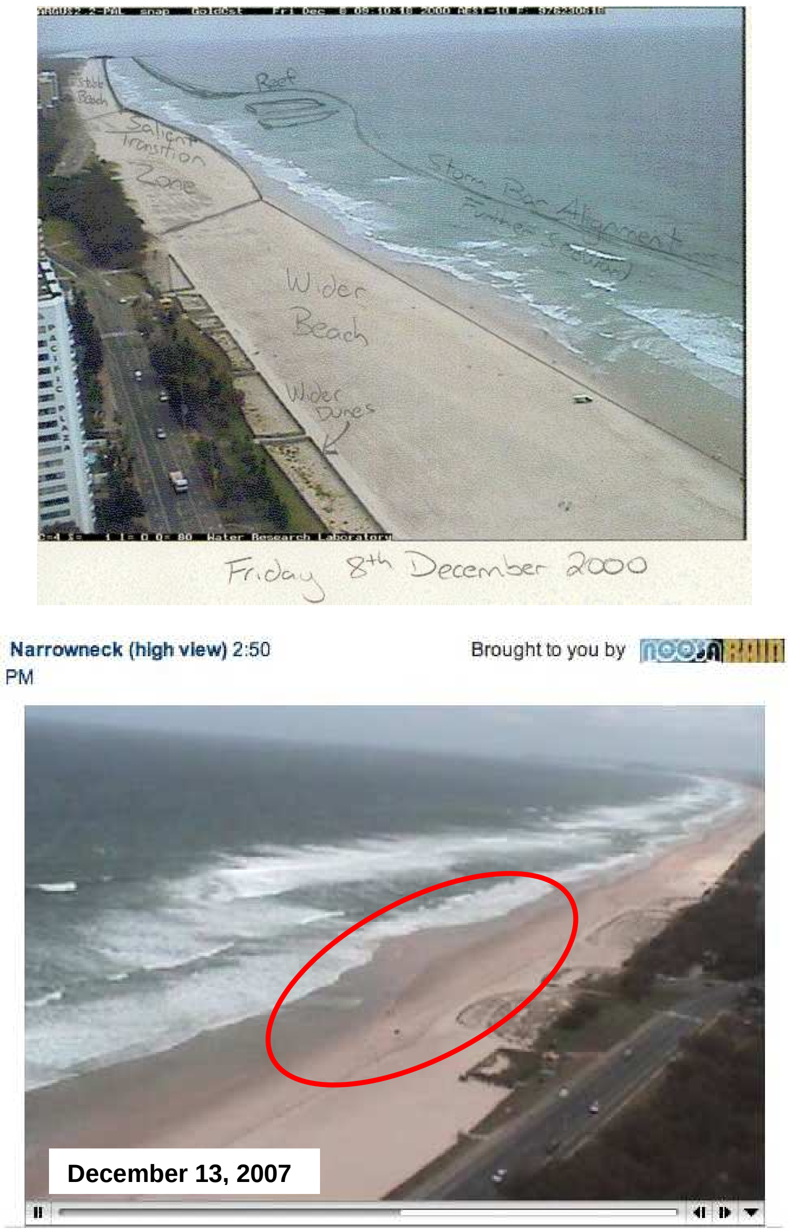


Figure 3.2. Two views separated by 7 years of the shoreline response at Narrowneck. The upper photograph shows the reef area under small wave conditions and is annotated by John McGrath of the Gold Coast City Council. In the lower image, the wave conditions are somewhat larger. The salient can be clearly seen in the nearshore, swash zone area.

The basic premise for the reef at Orewa is to devise a system that will widen the beach ('managed advance') and provide protection to the beach during high tides and storm wave conditions; the current conditions in which erosion occurs. The approach taken here is a combination of direct and indirect actions which provide a holistic solution to the episodic erosion problem at Orewa.

In natural coastal beach and dune systems, it is the beach that provides protection for the shoreline. As waves break, energy is dissipated and their erosive power is lost. When the incident wave energy exceeds the dissipative capability of the natural beach slope, the waves will cut into the dunes causing localised erosion. The sand that is taken from the dune however, is then redistributed into the nearshore surfzone and provides protection through additional wave dissipation. During calmer periods, natural processes rebuild the dunes and some of the material that has moved offshore is able to move back on to the dry portion of the beach.

In the case of Orewa, human activities such as sand mining have removed the large reservoir of 'emergency sand' that was available to nourish the beach during storms. This was followed by construction on the dunes which imposed an arbitrary line which residents sought to protect through the use of rock walls. The rock wall then caused increased wave reflection during large storms which added to the severity of the beach erosion. Additionally, the redirection of the ebb tidal jet from the Orewa estuary further reduced the amount of sand that was naturally returned to the beach. Thus Orewa became a system with sand moving out, and none moving in. The result was a net erosion and retreat of the dry beach line.

The design presented here calls for structures to be built between 20 and 300 m offshore of the low water line. These structures will work during smaller wave conditions to encourage the deposition and redistribution of sand in the shadow of the reef, i.e. a widening of the beach, or managed advance. This deposition of sand is called a 'salient' and is a very common and well documented consequence of wave dissipating structures placed offshore of sandy coasts. Once salients have formed, they will become the primary protective feature for the beach at Orewa – not just the reefs themselves – acting as the buffer zone and supply of sand during storm events.

This point should be reiterated:

"The reefs form the salients and the salients protect the beach."

In the current case, they also provide a dry high tide beach that is presently often not present at Orewa.

3.2 Review of the Preliminary Design

The preliminary design and feasibility study (Mead *et al*., 2004a) focused on a design which would promote sand retention, salient formation and in turn beach protection. Over 35 reef designs were tested with WBEND, 3DD and 2DBEACH, resulting in several hundred model simulations that assessed the size, location and basic plan shape that would most efficiently achieve the project goals.

The gentle seabed gradient (<1:100 vert.:horiz.) and the small wave climate (mean wave height 0.8 m) relative to the tidal range (~3 m) were specific issues which presented problems in the overall design process.

Designs were tested for mean and storm wave events (both calculated from the hindcast and measured data at Orewa), wave directions and tidal ranges from mean low water spring (MLWS) to MHWS with storm surge (up to 0.5 m). While this approach considered a wide range of events, the focus was on storm events at higher tidal levels, which are the most erosive conditions.

The preliminary design considered the ideal offshore position, the effect of wave rotation and the dimensions of the salient expected to form as a result of the reef. Figure 3.3 summarises the preliminary recommendations for the reef position at Orewa Beach. The reef was positioned with the inshore end between 300 and 400 m offshore of low tide. The alongshore width of the reef is ~200 m. The reef shown in this figure is a generic reef (i.e. a simple 'V' shape) and is not the final plan shape devised in that study. In the initial study, the water depth at the reef was between 1.5 and 2.5 m (relative to chart datum (CD)). The crest height at 0.0 m (to CD). which is 0.5 m below mean low water spring (MLWS) and 1.7 m below mean sea level (MSL). The initial designs called for a total reef volume of 10,000-15,000 m^3 covering an area of 8,000-12,000 m^2. These initial design guidelines were then used to initialize the detailed design process

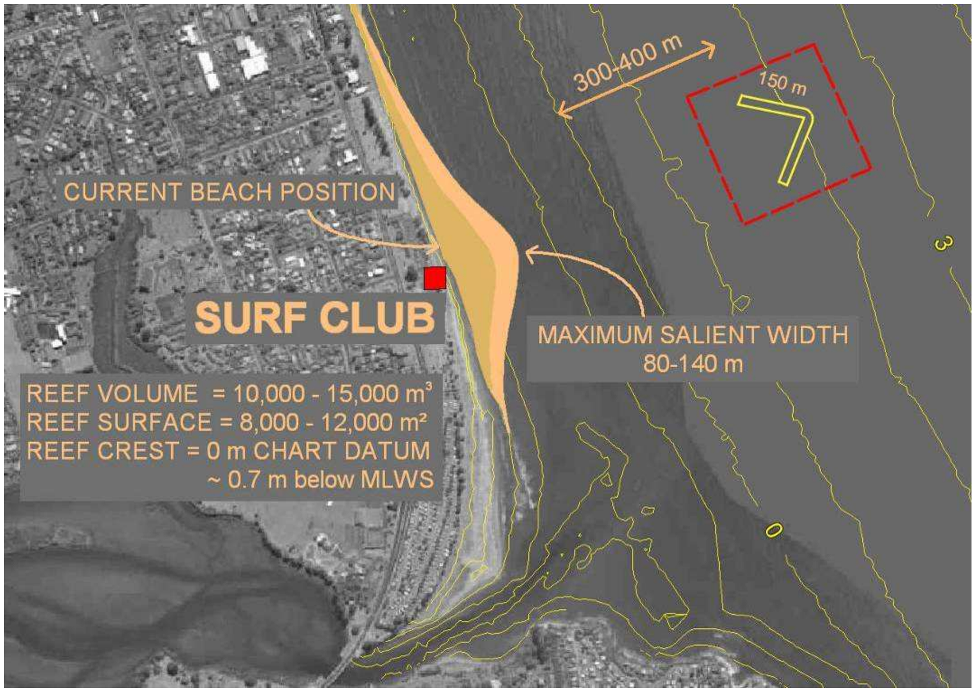


Figure 3.3. Recommended location of the Orewa multi-purpose reef from the feasibility report (Mead et al., 2004a). A generic reef is shown within the recommended reef area (red box).

3.3 Revised Design Scenarios

The Orewa multipurpose reef preliminary design described above was refined and revised for the purposes of the Resource Consent application, i.e. a final design, performance and impact assessment was required in order to apply for Resource Consents. This detailed design effort examined in greater detail the ability of the preliminary design to sufficiently dissipate wave energy and modify wave driven currents to promote accretion along the beach.

3.3.1 Offshore Distance and Crest Height

The first series of tests examined the relative differences between shore parallel 'breakwater' type reefs and broad crested reef designs. Examples are shown in Figure 3.4. For the shore parallel reefs, single and double reef systems were simulated at two different crest levels (+0.5 m and +1.7 m relative to chart datum). For the broad crested reef shapes, two different designs were tested at two distances off shore (300 m and 400 m), each with a crest level of +0.5 m relative to Chart Datum.

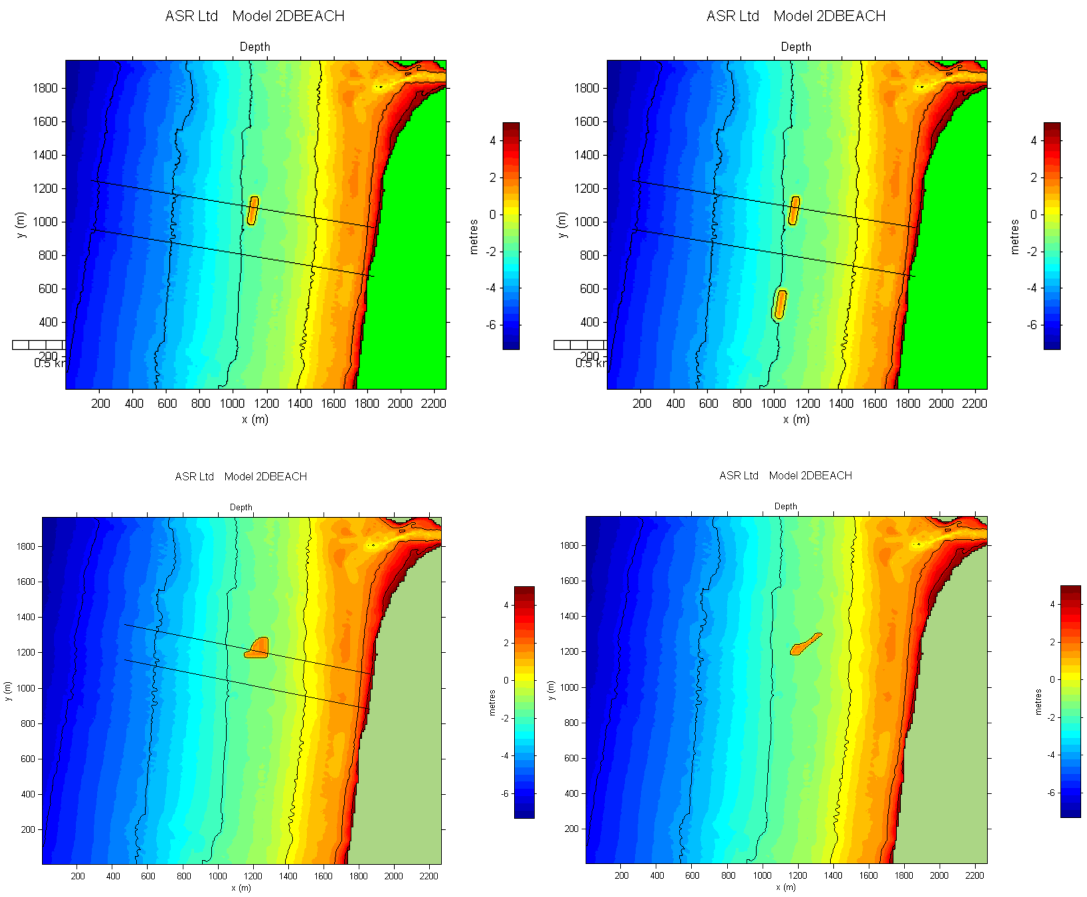


Figure 3.4. Reef shapes tested in the first phase of the revised design study.

It was shown in the feasibility study that by increasing the reef crest height from 0.0 m to 0.5 m a significant increase in beach protection could be achieved, which is supported by other studies of reef crest sensitivity (e.g. Mead et al., 2004b). At Orewa, discounting atmospheric effects, reefs with a crest height of 0.5 m above chart datum (CD) will be emergent for <1% of the time (MLWS is 0.5 m at Orewa Beach).

Each of these reef designs were tested using the calibrated hydrodynamic module of 2DBEACH. This allowed for a rapid assessment of the current patterns and wave height attenuation that each reef would create. The wave and tide parameters used are shown in Table 3.1. These values were chosen to cover the range of tidal conditions as well as average wave, 1-year storm wave and 10-year storm wave conditions. Thus, for each reef shape there were 7 model runs. Each reef shape had two variations; for the shore parallel reefs the crest height was set at either +0.5 or +1.7 m relative to Chart Datum at a fixed distance offshore. For the other reef shapes, the crest height was set to +0.5 m and the offshore distance was either 300 m or 400 m. The total number of simulations between the 4 reef shapes was therefore 56 cases (4 reefs x 2 variations x 7 wave conditions). An example of the model output for one scenario is shown in Figure 3.5. A subset of the complete model results is also provided in Appendix 1.

Table 3.1. Summary of model scenarios for the 2DBEACH Hydrodynamics cases.

Case	H (m)	T (sec)	Tide Level (m)
1 – Average	0.5	8	MLWS - +0.5
2 – Average	0.5	8	MSL - +1.7
3 – Average	0.5	8	MHWS - +2.9
4 – 1 year	3.1	10	MLWS - +0.5
5 – 1 Year	3.1	10	MSL - +1.7
6 – 1 Year	3.1	10	MHWS - +2.9
7 – 10 Year	4.7	12	MHWS (storm) - +3.4

Figure 3.5 shows representative results for the case of one shore parallel reef with a crest height of +1.7 m relative to Chart Datum. Panel B shows the wide surfzone that is created as a result of the gently sloping bathymetry. The rust collared region indicated in Panel B is the area subject to wave breaking under these conditions. As a result, the offshore reef design does not strongly influence the wave height as shown in Panel C. Similarly, Panel D indicates very little effect on the nearshore wave-induced currents. Transects of wave height and sea level across the reef are compared in Panels E and F. These plots indicate a sharp drop off in wave height as a result of the wave breaking on the reef (Point 1, Panel E), however this effect is negated after the wave passes over the reef, shoals again and breaks near shore, resulting in an increase in the wave height immediately adjacent to the shoreline (Point 2, Panel E). The large reduction in wave height, indicated in Figure 3.5, Panel C by the colour transition from red to blue, is due to the wave breaking at the 3 m depth contour. Wave breaking dissipates energy and reduces overall wave heights. This effect can be clearly seen in, Panels E and F as the wave height drops sharply approximately 800 m from the shoreline.

The complete results for the single, shore parallel, +1.7 m crest height reef case are contained in Appendix 1. A similar analysis was completed for the other three reef shapes indicated in Figure 3.4, however, the plots are not included in this document to reduce repetition. The results of this series of tests suggested that the shapes with the wider crests located further offshore resulted in the greatest wave dissipation. The problem however is related to the large tidal variation and the gentle beach slope, which creates a very wide intertidal and surf zone that tends to ‘smear’ the positive effects of this wave dissipation, i.e. the reduction in wave heights is reduced over the full tidal range. Indeed, given the local gradient ~3 m tidal range, it is not feasible to put in a structure that is always outside the surf zone width during all tides in storm events, and with a single reef structure, the ‘smearing’ effect of bar formation is not addressed. Therefore this type of reef design was abandoned in favour of an alternative approach, one which considered modification of current patterns and extending the wave dissipation effects across a wider distance to help address the wide horizontal fluctuations caused by the low beach gradient and relatively large tidal range.

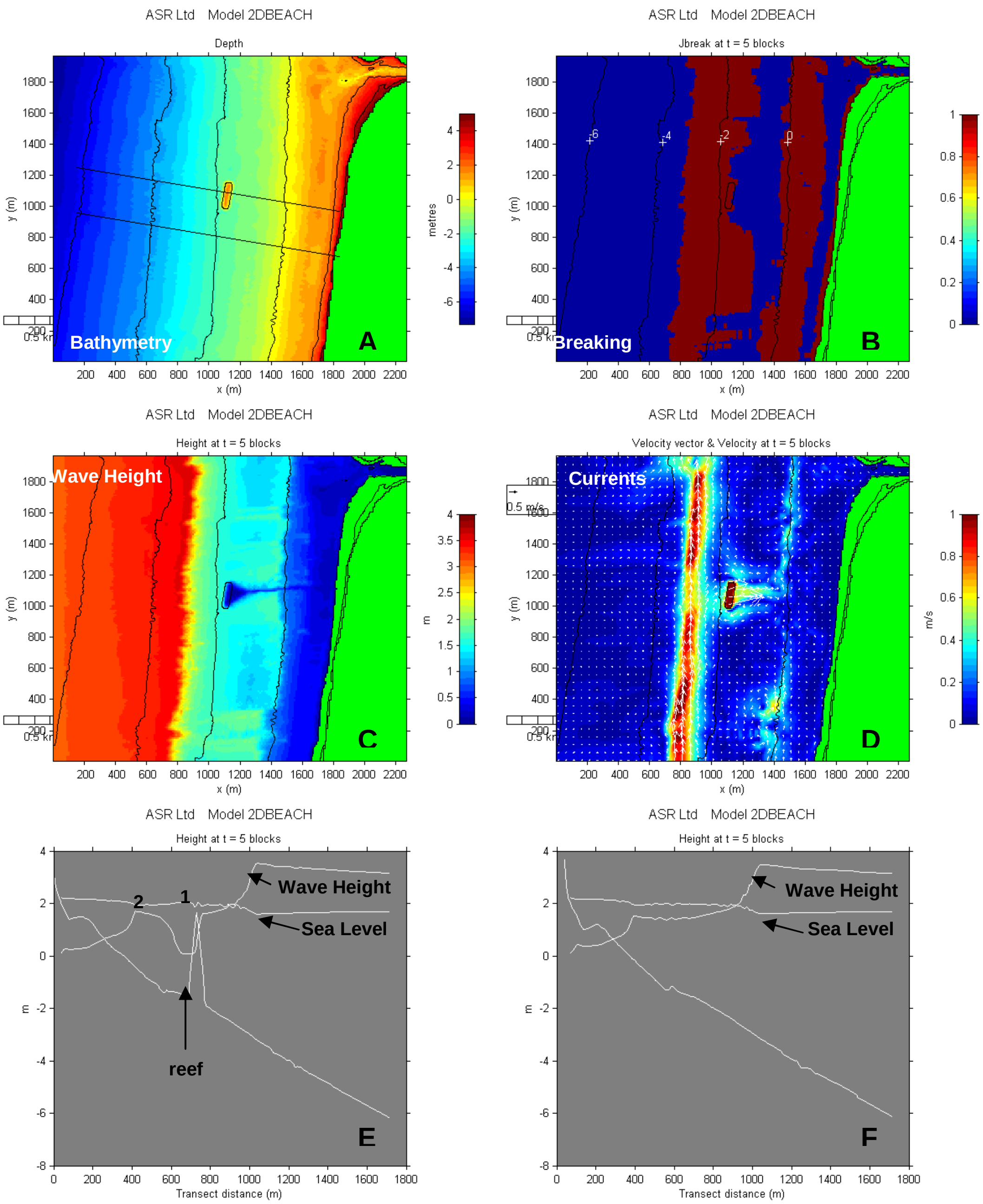


Figure 3.5. Results from the 2DBEACH hydrodynamics simulation for the case with one wide crested shore parallel breakwater with a crest height of +1.7 m. Sea level is set to low tide (MSL,); wave conditions are H = 3.1 m, T = 10 sec. Panel A is the model bathymetry. Panel B shows regions of wave breaking (rust coloured areas). Panel C is the wave height and Panel D is the velocity. Panels E and F are transects of wave height and sea level over the bathymetry. The transect locations are indicated in Panel A.

3.3.2 Current Modification and Wave Dissipation

The next series of investigations used a different approach aimed at reducing the influence of the long shore currents which transport sediment southward towards the estuary mouth where sand accumulates at the Waitemata groyne. As described in the feasibility study and in Section 2.5.3 of this report, the erosive events at Orewa are associated with larger waves approaching from a more northeasterly direction which acts to drive alongshore currents to the south, i.e. the storm waves erode sand across/offshore and the southerly directed current moves the sand towards the southern end of the beach and groyne area where it cannot naturally return to the beach. An example of this current pattern is shown in Figure 3.6 below (note, this is 'steady state' modelling with an unvarying set of boundary conditions – time series events and modulated modelling are presented in later Sections.).

In order to counteract this southerly directed flow, one common approach is to build a cross-shore structure extending into the surfzone, as has been done at many beaches worldwide in the form of jetties or groynes. Since the public sentiment at Orewa (and indeed worldwide) is clearly against the idea of a typical groyne, we attempted to achieve similar result, however with a less intrusive structure that influenced wave-driven currents rather than completely blocked sediment transport (as groynes will) to compliment the wave energy dispersive effect of submerged structures.

Again, four trial shapes for this type of structure were tested. Each of the four shapes is an obliquely oriented cross shore, submerged structure. The cross shore orientation is designed to affect the alongshore currents while the oblique angle relative to the shore aids in wave shadowing and energy dissipation, as well as wave rotation (i.e. redirection of waves to counteract wave-driven currents (Black and Mead, 2001)). For each of the designs, the structure crest height is set to 0.5 m above chart datum (equivalent to the MLWS water level). Thus, these reefs will only be exposed for a short period at the lowest of tides. The trial shapes are shown in Figure 3.7.

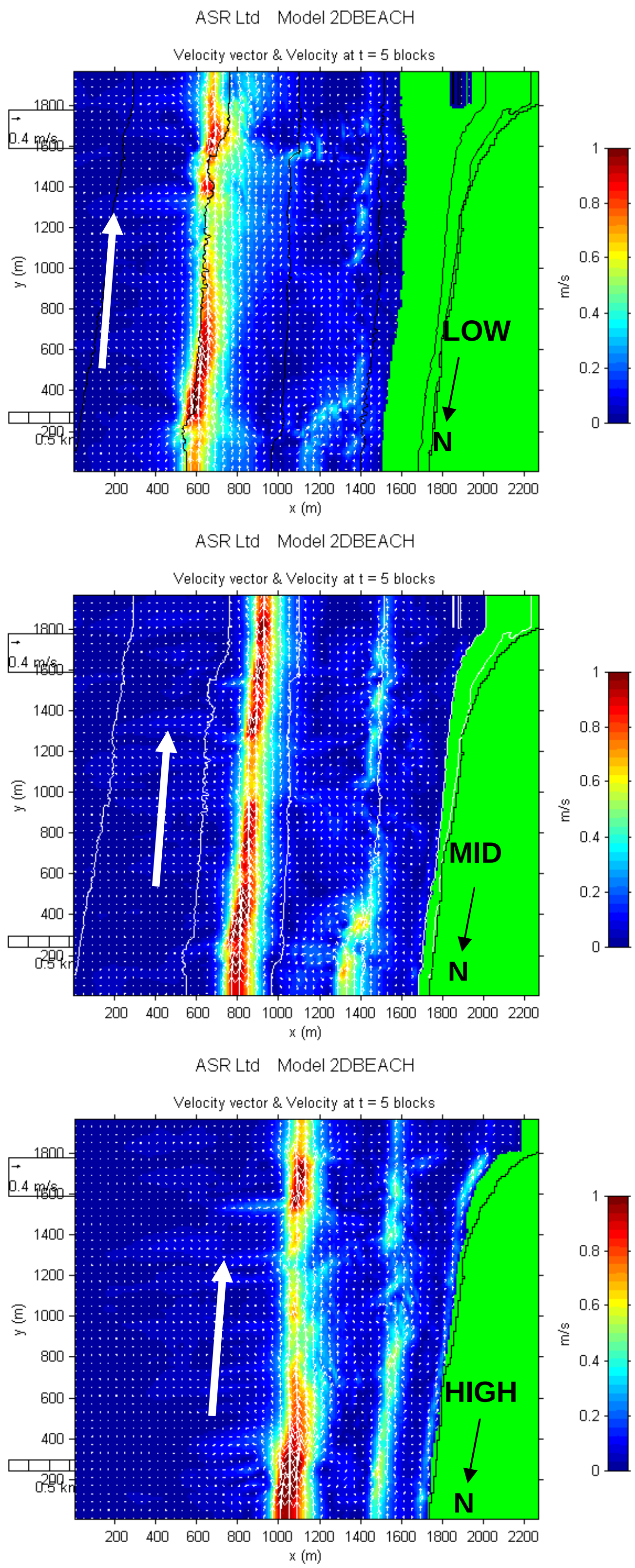


Figure 3.6. Southerly directed alongshore current generated by a 3.1 m wave at different tide levels. MLWS (+0.5 m, upper panel), MSL (+1.7 m, middle) and MHWS (+2.9 m, lower).

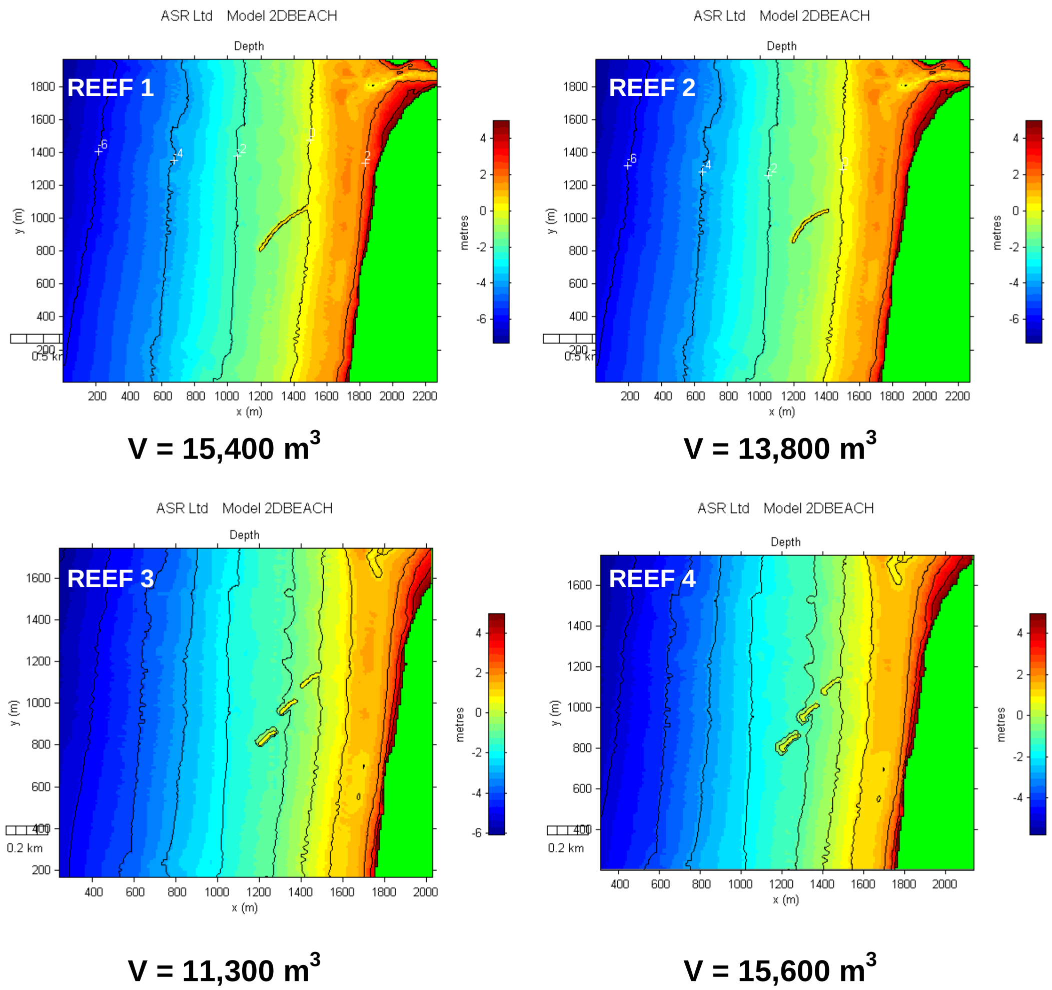


Figure 3.7. Four reef shapes tested for current modification and wave dissipation.

The hydrodynamic module of 2DBeach was first used to assess the effect on current patterns and overall velocities. The four reef shapes were tested for a 1 year wave event at mid tide and the results are summarised in Figure 3.8. For cases 1 and 2, the models suggest an acceleration of the alongshore flow between the reef and the shoreline. This effect is not seen in the segmented reef cases. Of the two segmented reef cases, Case 4 produced the best overall reduction in the velocity of the alongshore current, as well as produced low counter-rotating currents in their lee, which are conducive to good salient development (e.g. Black, 2003; Ranasinghe et al., 2006).

In terms of wave attenuation, Reef 2 and Reef 4 are compared in Figures 3.9 and 3.10. The wave shadow generated by each reef can be clearly seen in Figure 3.9 as compared to the control run. Reef 2 produces a larger, more continuous shadow area as compared Reef 4 which allows wave energy to pass through the gaps in the reefs. This is also reflected in the profile diagrams presented in Figure 3.10.

Thus, when the combination of wave dissipation and current modifications are considered, Reef 4 is the most effective.

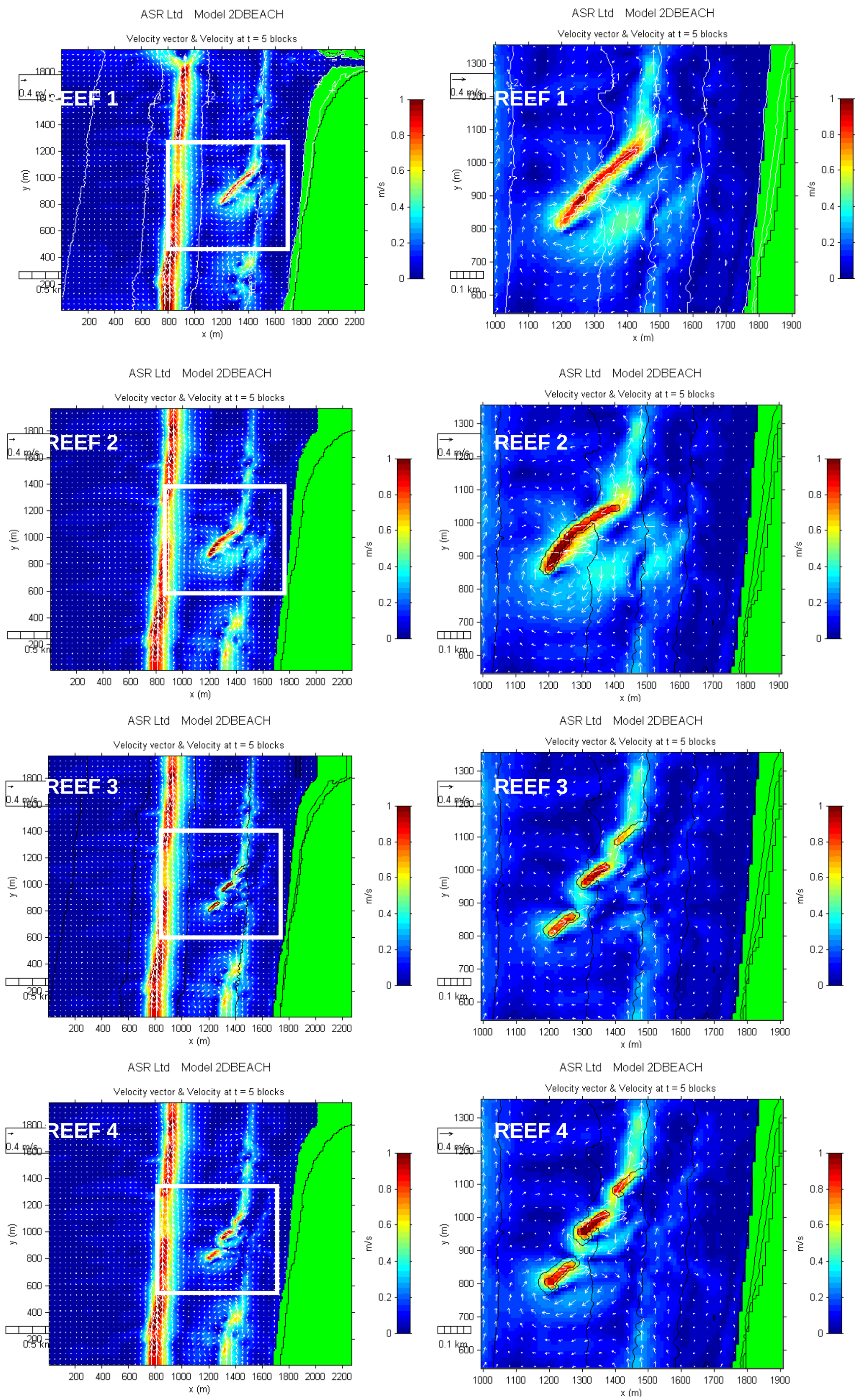


Figure 3.8. Modelled velocity patterns for 3.1 m wave heights at mid tide (+1.7 m) for the four reef shapes.

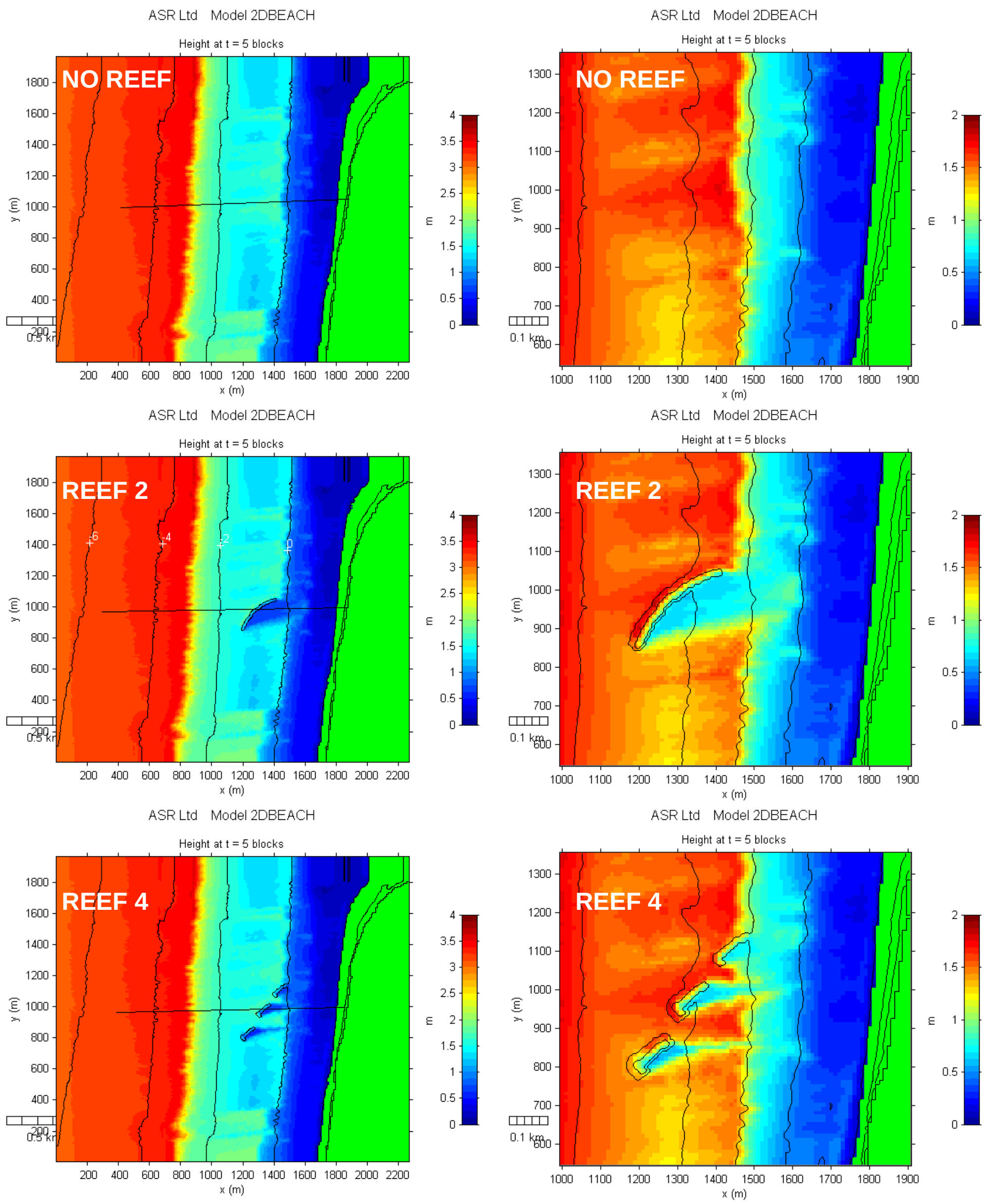
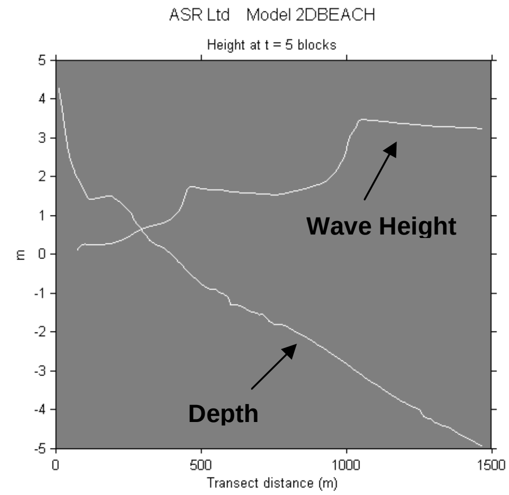
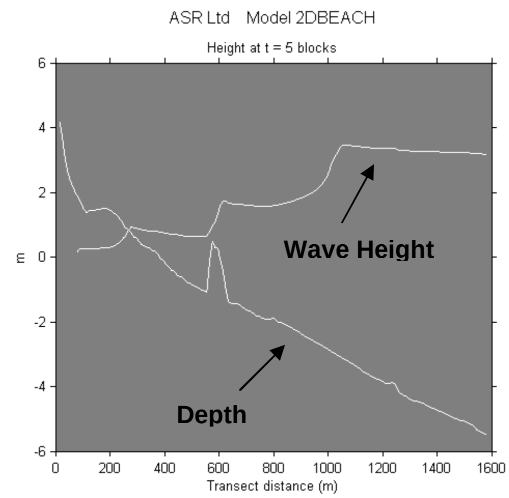


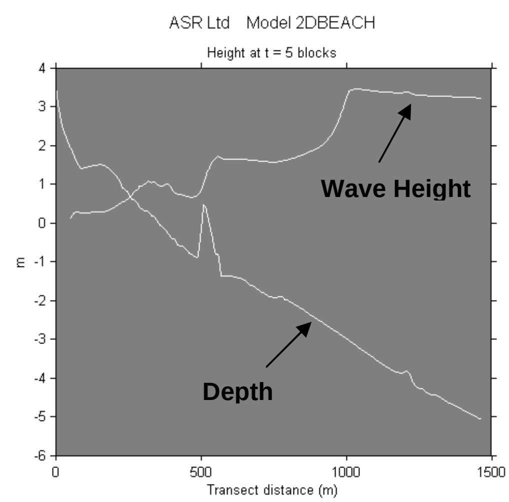
Figure 3.9. Wave dissipation for the two reef shapes versus the control using 2DBEACH. H = 3.1 m, T = 10 sec, MSL (+1.7 m). The black lines indicate the location of the profiles presented in Figure 3.10.



No Reef Mid Tide



Reef 2 Mid Tide



Reef 4 Mid Tide

Figure 3.10. Wave height profiles across the bathymetry transects indicated in Figure 3.9

3.3.3 Shoreline Response Modelling

Since Reef 2 performed the best in terms of wave dissipation and Reef 4 performed the best in terms of current modification, these two designs were selected for shoreline response modelling. For these simulations we first used the model NGENIUS, a modified/multiplied one-line model that predicts shoreline response based on wave attenuation, refraction and diffusion of suspended sediments at the break point. NGENIUS is fast model that is used to provide indicative shoreline responses for initial selection for more detailed hydrodynamic and sediment transport modelling.

For the NGENIUS simulations, a time series rather than static wave climate was used. This data set statistically replicates a full year of typical wave conditions including the large number of calm or small swell days as well as storm events, varying wave height, direction, period and tide. The wave data used for the NGENIUS simulation is shown in Table 3.2. For the NGENIUS modelling, the 1 year wave climate was used for successive times to simulate multiple years of wave activity and sand bar formation.

The results of the N-GENIUS modelling are shown in Figure 3.11. Both designs exhibit a strong shoreline response with the beach widening in the lee of either structure. Reef 4 however, shows a larger along-shore foot print. Furthermore, the hydrodynamic modelling described in the previous section suggests that the segmented design efficiently dissipates and modifies the wave-driven currents with out generating a potentially erosive return flow as seen in the curved reef case.

Table 3.2. Time dependent wave and tide climate used in NGENIUS and 2DBEACH simulations.

Time Block	H (m)	T (sec)	Dir (grid)	Dir (true)	Tide (m)
0	0.23	8	-16	78	0.67
2	0.37	7.6	-10	72	0.89
4	0.42	7.8	-2	64	1.24
6	0.36	8.2	4	58	1.62
8	0.63	9.6	6	56	1.95
10	0.36	9.2	4	58	2.31
12	0.37	8.6	-10	72	2.61
14	0.42	8.8	-2	64	2.06
16	1.65	8.7	-9	71	1.62
18	0.75	8	-14	76	1.25
20	1.12	8.1	-5	67	0.89
22	0.62	9.8	-2	64	0.8
24	0.36	10.2	4	58	1.2
26	0.36	11.2	4	58	1.4
28	0.37	9.6	-10	72	1.78
30	0.23	7	-16	78	2.4
32	1.36	8.9	7	55	2.65

34	2.04	9.7	10	52	2.28
36	0.23	9	-16	78	1.8
38	1.11	8.9	4	58	1.41
40	0.36	12.2	4	58	1.02
42	0.63	10.6	6	56	0.74
44	0.63	11.6	6	56	1.02
46	1.11	9.9	4	58	1.62
48	1.92	9.6	3	59	1.95
50	0.36	9.2	4	58	2.31
52	0.37	7.6	-10	72	2.61
54	0.42	7.8	-2	64	2.06
56	0.36	8.2	4	58	1.62
58	0.63	9.6	6	56	1.25
60	0.36	9.2	4	58	0.89
62	0.42	8.8	-2	64	0.8
64	1.65	8.7	-9	71	1.2
66	0.75	8	-14	76	1.4
68	1.12	8.1	-5	67	1.78
70	0.42	9.8	-2	64	2.4
72	0.36	10.2	4	58	0.67
74	0.36	11.2	4	58	0.89
76	0.37	9.6	-10	72	1.24
78	0.23	7	-16	78	1.62
80	1.36	8.9	7	55	1.95
82	2.04	9.7	10	52	2.31
84	0.23	9	-16	78	2.61
86	1.11	8.9	4	58	2.06
88	0.36	12.2	4	58	1.62
90	0.75	9	7	55	1.25

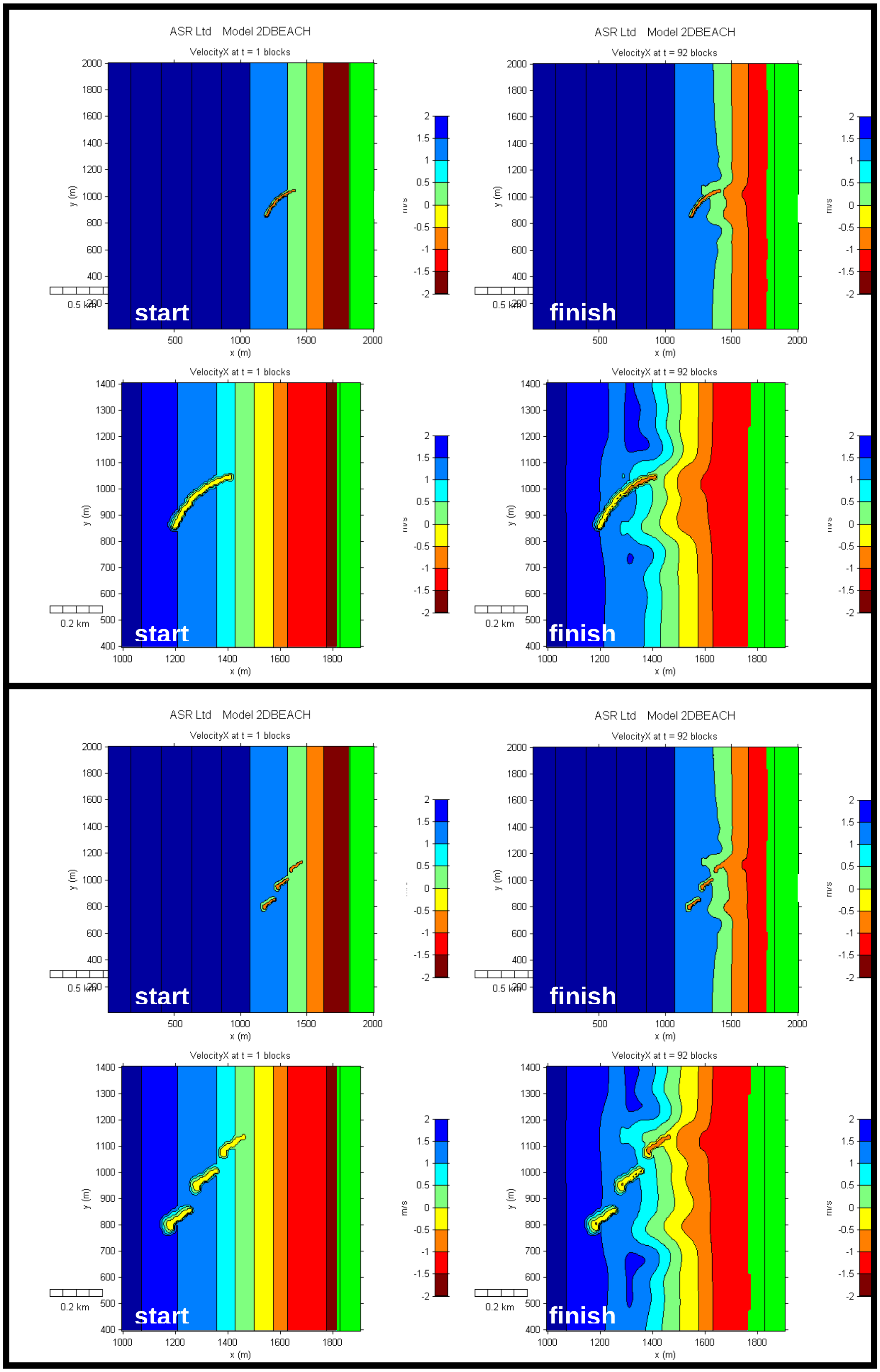


Figure 3.11. Modelled shoreline response from NGENIUS for the curved single reef (upper set) and the multiple segmented reef design (lower set). For each set, the full grid is in the top row with a close up on the reef area in the lower row (bathymetry relative to CD).

Once it was determined that a suitable shoreline response could be obtained from the NGENIUS modelling, Reef 4 was selected for detailed shoreline response modelling using the more sophisticated sediment transport module of 2DBEACH. For these simulations, the final, modified bathymetry generated by the NGENIUS simulation was used as the initial bathymetry for the 2DBEACH simulations. This was done to assess the stability of the salient that had formed as a result of the protection afforded by the reefs.

Figure 3.12 illustrates the evolution of the shoreline as a result of a large wave event attacking the beach with an existing salient in place. Panel A shows the bathymetry that was used to initialise the model. Panel B then shows the changes in bathymetry expected as a result of a 2 m wave event with a moderately high tide. Panel C shows the return to the original bathymetry. This indicates that the beach response predicted by NGENIUS will likely remain stable after the onslaught of a storm wave event. Figure 3.13 shows the magnitude of the absolute bed level change under the same wave conditions. Areas subject to erosion are coloured in blues while areas of accretion are coloured in reds.

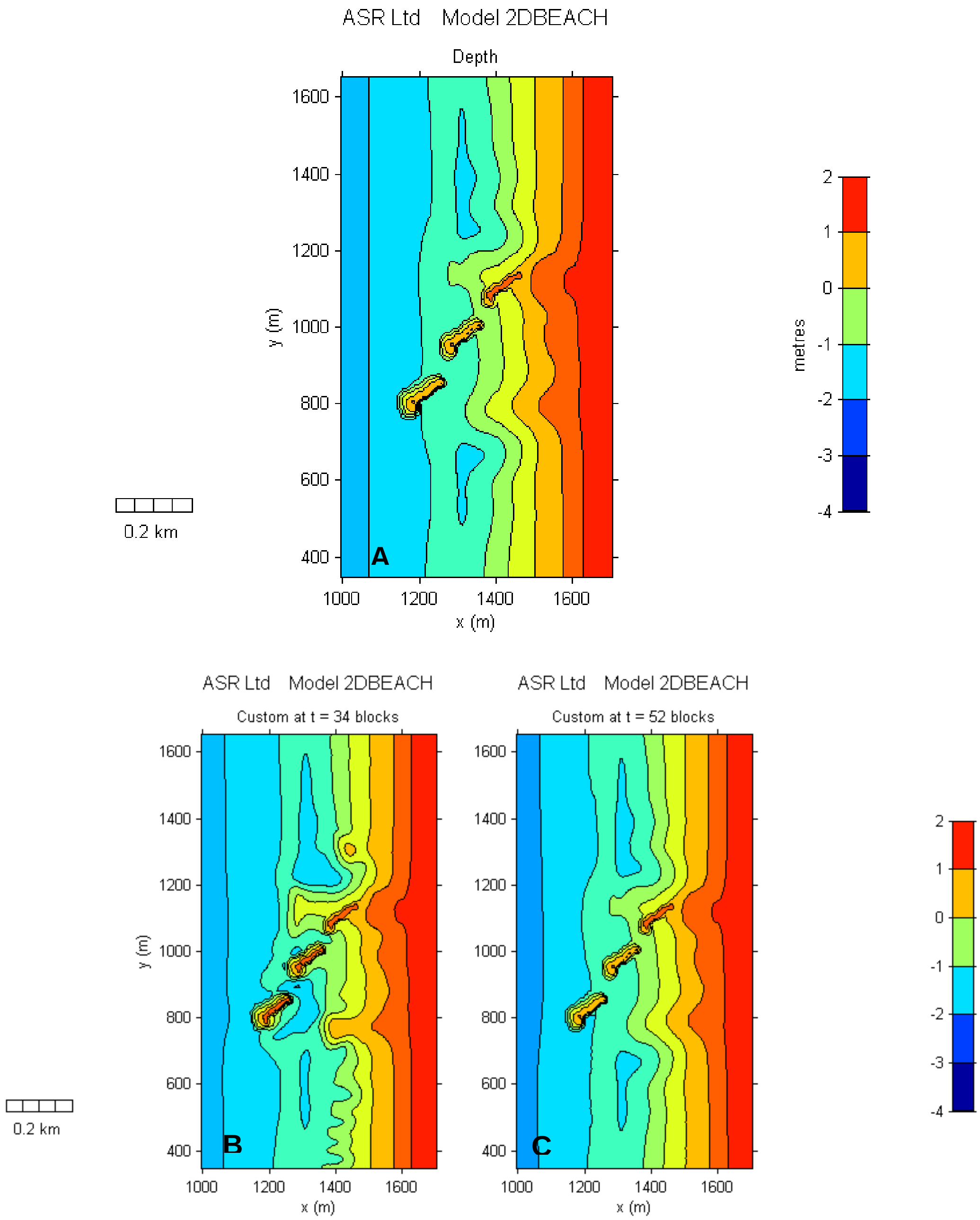


Figure 3.12. Transitional stages shoreline response during the 2DBEACH modelling. Panel A shows the initial bathymetry while Panel B shows the response after a large wave event (~2 m). Note the modified depth contours, erosion between the two reefs offshore and the deformation of the southern salient. Panel C shows the return to a steady state configuration which closely matches the initial bathymetry which suggests that the salient and shore protection afforded by the reef is stable even after large wave events. The block numbers in Panels C and B correspond to the wave conditions given in Table 3.2. (Bathymetry relative to CD).

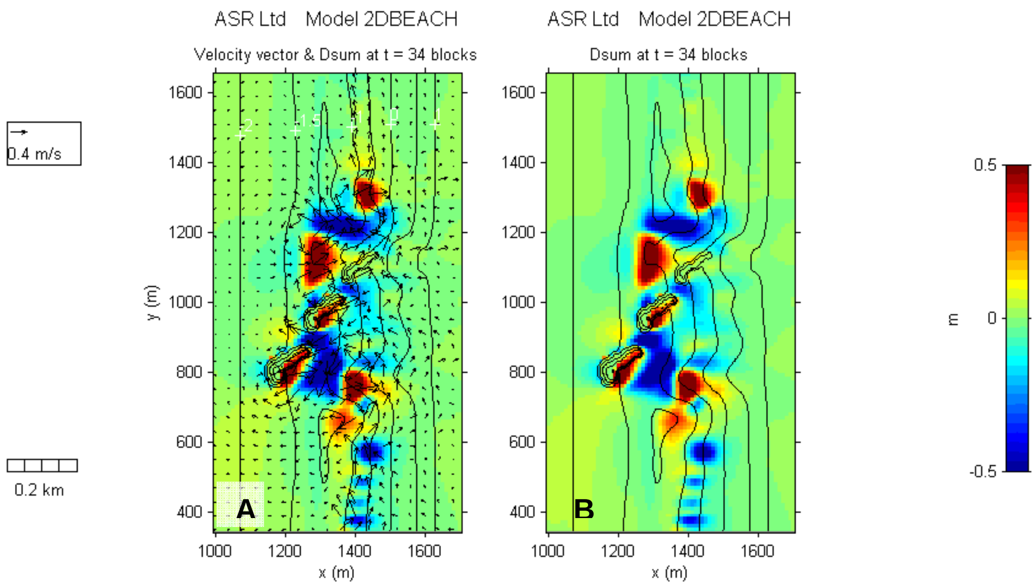


Figure 3.13. Absolute changes in bed level corresponding to Panel B in Figure 3.12. Blues indicate erosion and reds indicate accretion. Velocity vectors are superimposed on Panel A to illustrate the circulation patterns, similar to those shown in Figure 3.8.

Following this modelling, a range of additional hydrodynamic and sediment transport model scenarios were undertaken to provide further understanding of the performance and impacts of the reef system when replicated 2, 3 and 4 times along the length of Orewa beach during ‘normal’, 1, 10 and 100 year return events. However, before presenting these results, existing empirical predictive methods are assessed.

3.3.4 Empirical Predictions

The primary way that an offshore reef creates a salient is due to wave sheltering, although wave diffraction and nearshore circulation (e.g. Hsu and Silvester, 1990; Pilarczyk and Zeidler, 1996; Ranasinghe et al., 2006)) also contribute to the mechanism of salient formation, and refraction resulting in re-alignment of wave crests (wave rotation) can also play a significant role (Mead and Black, 2001). The shape of the salient that forms in the lee of an offshore reef can be predicted using empirical equations (Black and Andrews, 2001a; Andrews, 1997; Ranasinghe et al, 2006; Savolli et al, 2007). However, as demonstrated here, these simplistic assessments are of little value at Orewa Beach, mainly due to the large tidal range and low beach gradient – the empirical predictors were developed from either exposed beaches (with relatively much steeper beach gradients) or in idealized laboratory conditions.

The first empirical predictor evaluated is Black and Andrews (2001a, calculations (Eqn. 3.1) using the reef and reef location dimensions are worked through below to predict the level of coastal protection that the offshore reef would provide.

The longshore width of the reef (*B*) and the distance between the reef and the undisturbed shoreline (*S*), indicate that the reef would form a salient.

Salients form when $\frac{B}{S} < 2.00$ (3.1)

Next, by substituting the reef dimensions into the salient equations (Eqns. 3.2 and 3.3) of Andrews (1997) and Black and Andrews (2001a), the geometry of the salient can be predicted. The average salient amplitude for offshore reefs is given by,

$$\frac{X}{B} = 0.498\left(\frac{B}{S}\right)^{-1.268} \qquad (3.2)$$

where X is equal to S - Y_{off}, which is the distance between the undisturbed shoreline and the reef (S), minus the length of the shore normal between the undisturbed shoreline and offshore extremity of the salient (Y_{off}). Salient basal width is given by,

$$\frac{Y_{off}}{D_{tot}} = 0.125 \qquad (\pm 0.020) \qquad (3.3)$$

where, D_{tot} is the total length of shoreline affected.

From these equations, using the results described above, the predicted salient at Orewa can either be considered as the result of 3 individual reefs, or due to all reefs combined. For the most inshore reef of the 3 reef system, the salient has maximum dimensions cross-shore at the widest point of ~68 m, tapering down to zero accretion some 270 m in each direction longshore (Fig. 3.14). However, the alongshore length is normally reduced to allow for 10% of the across width (since it asymptotes to zero), which results in an alongshore length of approximately 400 m in this case. The width of the salient refers to the distance moved offshore by the beach isobaths and MSL.

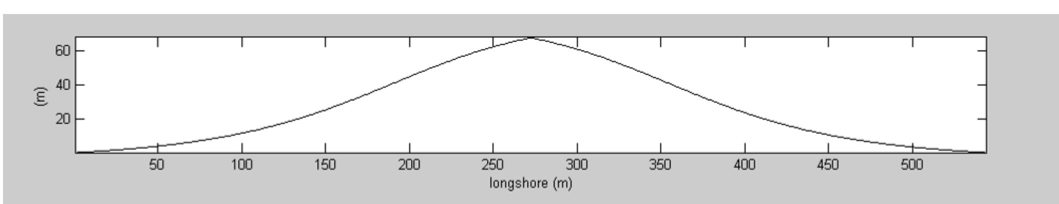


Figure 3.14. Schematic diagram of the salient formation in the lee of the most inshore reef at Orewa Beach

The middle reef results in a salient of some 80 m across shore, by 660 m alongshore, reduced to 450 m alongshore with the 10% reduction. The most offshore reef results in a salient of 87 m across shore, by 720 m alongshore, reducing to 500 m along shore. In combination, the reefs result in an undulating salient of some 700 m long, which is considered similar to the overall response of the 3 reef system found with morphological modelling, both in terms of morphology and scale.

When the 3 reefs are considered as a whole, the predicted beach response is a 150 m wide salient, some 1220 m alongshore, reduced to 900 m, which is much greater than found via numerical modelling and does not account for the often undulating nature of the salient.

Ranasinghe et al. (2006) developed an empirical predictor which indicates that a submerged structure must be outside the width of the surf zone (or SZW) to ensure an accretionary beach response (similar to the findings of Black *et al*., 2003).). This analysis is based on a series of laboratory and numerical experiments which established a relationship between the incident wave conditions and the reef geometry (Figure 3.15). The ultimate result was a set of design curves (Figure 3.16) that relate these quantities. It is important to note the Ranasinghe et al. (2006) relationships are based on a reef with a crest height set at 0.5 m below mean water level, when in the current case the crest height is set at 1.2 m below MSL.

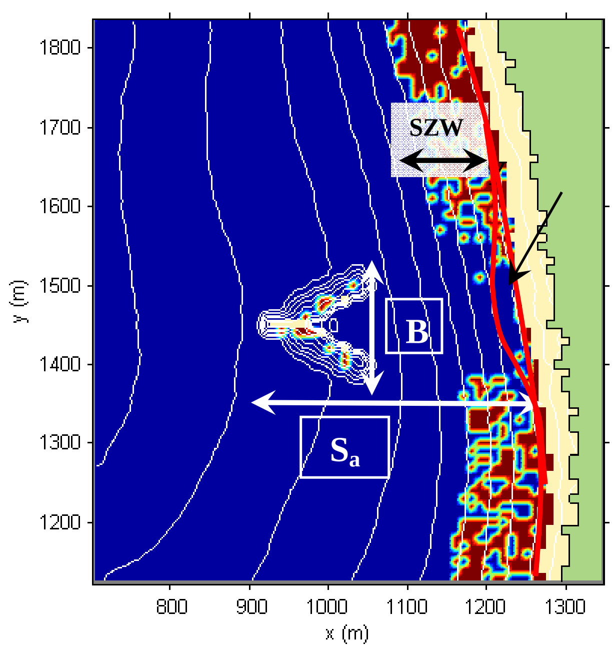


Figure 3.15. Schematic diagram for quantities used in the Ranasinghe et al. (2006) salient formation relationships.

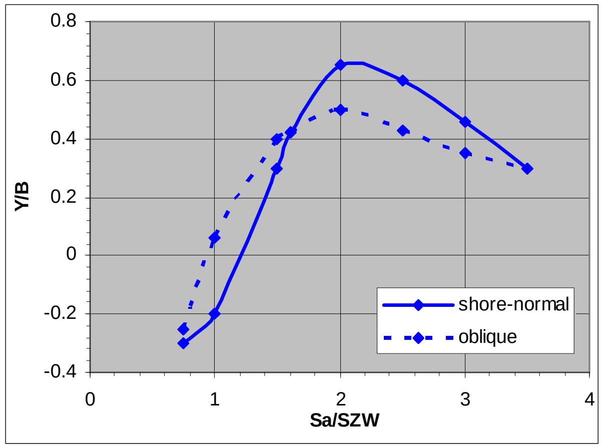


Figure 3.16. Relationship between reef geometry (Sa – offshore distance, B – cross-shore width), wave conditions (SZW – surf zone width) and salient width (Y) based on laboratory physical model experiments. Figure reproduced from Ranasinghe et al. (2006). Solid line is for normally incident waves, dashed line is for obliquely incident waves.

While this empirical method has been applied to several exposed beach situations and has proved to be a useful preliminary design tool, in the current case the wide SZW that varies greatly over a high-low tidal cycle provides little guidance. Depending on the wave event and the tidal elevation, the results range widely between accretionary and erosive. The Ranasinghe et al (2006) empirical equation is basically a steady state model, where the sea level is always 0.5 m above the crest; in the present case, the crest height varies from 0.5 m above sea level to 2.5 m below it.

A third basic empirical method is that developed by Savolli et al (2007), where the accretion coefficient is given by:

$$(\text{crest depth}/\text{depth}_{\text{total}})^{2/3} \times (\text{alongshore reef length}/\text{depth}_{\text{total}})$$

With values of <3 indicating accretion and values of >3 indicating erosion. Applying this to the Orewa reef system finds that the offshore reef causes accretion, the middle reef is also accretionary, while the inner reef is erosive. However, this is reverse to the response found with the numerical modelling, i.e. the salient is widest in the lee of the inner reef.

While empirical tools can be useful for quick assessments in environments close to those that they were developed in, in the case of Orewa Beach, the low gradient and relatively large tidal range make such tools unreliable, especially when a 3 reef system such as designed in the present case is evaluated. The empirical tools cannot deal with the interaction between the reef units or the complex wave and current modifications that they create.

3.3.5 Numerical Model Predictions

This section presents further numerical modelling investigations of physical processes associated with the implementation of multi-purpose reef systems for the development of a wider beach at Orewa were preformed. Normal conditions, as well as 1, 10 and 100 year return interval (RI) storm conditions were simulated using model 2DBEACH. Model results include both simulations undertaken prior to the reviews and additional simulations to provide further evidence of the efficacy and effects of the proposed Orewa Beach Protection Strategy.

2DBeach is a non-linear circulation model for irregular waves and contains five coupled simulations of physical processes:

1. wave height transformation;
2. wave angle refraction;
3. wave dissipation due to breaking and friction;
4. radiation stress-driven circulation and;
5. sediment transport.

Detailed descriptions of the modules have previously been elaborated by Black and Rosenberg, 1992a,b; Black and Vincent 2001; and Ranasinghe et al. 2004).

Model 2DBEACH predictions effectively bring together all the hydrodynamics occurring in response to the reef systems (wave heights, wave angles, current speed and direction, wave set-up, etc.) and provide predictions of beach response. 2DBEACH has capacity to predict features such as rip currents, sand bar movement, beach transformations, storm erosion and the build-up of beaches after storms.

Model 2DBEACH has had some additional features added to the model in the past few years and these are described here. 2DBEACH uses a unique Lagrangian scheme to transform the wave heights. This involves releasing particles at the offshore boundary and heights are carried shoreward as propagating waves by the particles. The heights respond to the processes of refraction, shoaling, friction and breaking dissipation. The model was recently refined so that the wave angles were also obtained using the Lagrangian technique, rather than solving the wave action equation. In essence, the model updates the angle carried by each particle using the initial angle of each particle as the initial condition. The final angle is obtained by calculating the angle change over the time step from the rotation caused by the depth changes along the particle path. For confirmation, a series of model tests were conducted by comparing the model to known analytical solutions on a plane beach, and for more complex cases, by comparing 2DBEACH results against those predicted by the sophisticated Boussinesq model.

The relevance of this refinement is that with multiple reefs such as at Orewa Beach, waves rotate and cross-over in the lee of the reef. By using a Lagrangian method, this complex sea state is properly simulated. For example, at any model cell in the lee of the reef, there can be waves that have passed over an 'outside' reef and the method adopted accounts for this multiple wave pattern. Specifically, the height in each cell is the average of wave heights carried by the particles. The direction in each cell is taken as the average direction of the particles, with each direction being weighted by the wave height. Thus, the larger waves have more influence on the final angle calculation. With this refinement, 2DBEACH can be used to simulate a spectral sea state with both multiple heights and directions. This new scheme has been used on an example case in northeastern New Zealand, as part of a Masters of Science project that ASR recently supervised (Spendelow, 2004). The results are very good, with the general plan shape and salient volume being predicted (Figure 3.17). RTK GPS surveys of the bathymetry and beach, along with multiple wave/current meter measurements, were used to calibrate and verify the modelling.

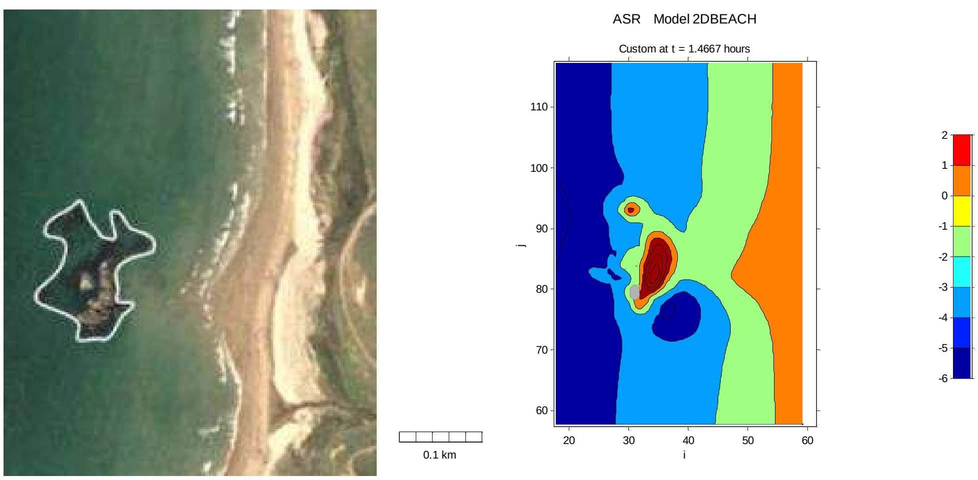


Figure 3.17. The results of salient evolution modelling of a natural reef in Opito Bay in northeastern New Zealand.

Model parameters are adopted from the detailed inshore wave climate development, which were validated during a series of calibration simulations. Boundary conditions are adopted to represent a typical (or common) wave climate as well as the 1, 10 and 100 year return interval (RI) storm conditions. A full range of wave directions, heights and periods were tested for 'normal' and storm events. Two kinds of boundary conditions were used for 2DBEACH modelling (for both hydrodynamic and sediment transport modelling. The first boundary condition was spectral, for example, the 'normal' condition boundary incorporated wave heights of 0.3-2.04 m, peak periods of 4.5-12.2 sec, and a directional spread of 28° (as found at the 7 m depth contour, e.g. Figure 4.21 of the feasibility study) and sinusoidal tidal changes. The second boundary type was modulated, which incorporates sinusoidal variations in the wave parameters centred on peak values, e.g. 1 year storm tide (mean 2.4 m, amplitude 1.0 m, phase 180, i.e. at 3.4 m incorporates storm surge and is most 'high' tide to analyse the critical period), wave height 3.5 m, direction 0° (to the grid, which corresponds with a NE storm), and a period of 8 sec.

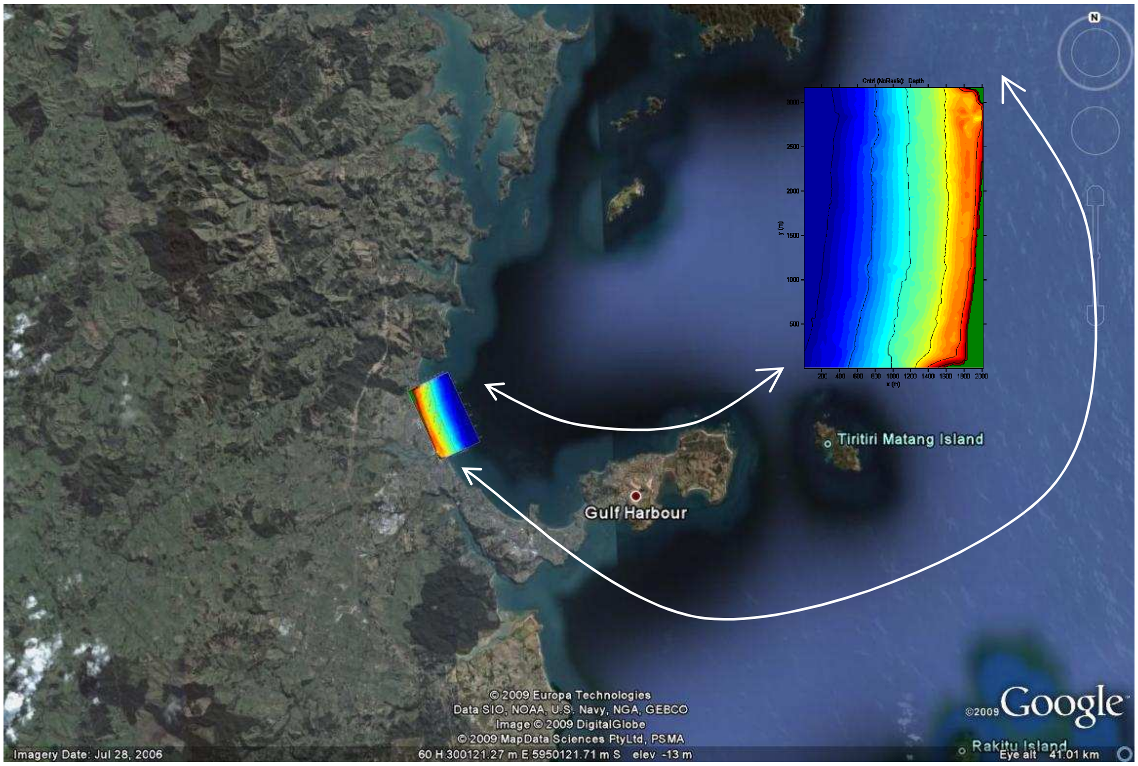


Figure 3.18. Location map of model grid showing a magnified and rotated (for modelling purposes) grid for Orewa Beach.

The sediment transport module is optional and can be included or left off. To examine hydrodynamics in the surf zone, the sediment transport is left off, but employed later to examine qualitative sedimentation trends. Because the hydrodynamics respond to bank formation, the model contains considerable feedback (Black and Mead, 2007). For this reason, sediment transport results are presented as qualitative trends as opposed to a precisely reproducible state.

Figure 3.19. Schematic of multiple reef system option at Orewa Beach.

3.3.6 Hydrodynamics in the Surfzone

Bathymetry and Scenarios

Bathymetry grids of Orewa Beach were generated to accommodate the testing of 1 to 4x 3-reef system structures. For every reef configuration tested, there is a corresponding simulation utilising natural bathymetry (i.e. no reefs systems) for comparison (Table 3.3, Figure 3.20 and Figure 3.21)

The local bathymetry at Orewa Beach has a gentle slope – on the order of 0.5% grade (or >1:100) – Figure 3.18 presents a location map of the 2DBeach modelling grid, while reef locations are shown in Figure 3.19. This implies that wave energy is typically lost multiple times as a result of depth-limited shoaling, breaking, and reorganising smaller wave fronts, i.e. a dissipative beach (Short, 2001). Wave energy is dissipated gradually over a long distance as the depth decreases along the shoreward trajectory of oncoming swell. This sort of wave energy “filtering” mechanism combined with low- to moderate-energy wave climate works to groom the beach into a homogenous planar surface. As a result, the surf zone migrates over a large distance of intertidal beach (~300 m) during a high-low tidal cycle, ‘smoothing’ out the beach response to offshore obstacles (e.g. submerged breakwaters or reefs). Thus, the 3-reef system was developed (described above), with the 3 separate units spanning some 300 m (from a depth of ~0.3 m below CD to almost 2 m below CD), to both maintain wave dissipation throughout the high-low tidal cycle and redirect alongshore currents to maintain salients in their lee and reduce the loss of beach sand during storms.

Table 3.3. Model bathymetry grids and specifics:

Grid	Cell size $dx = dy$ (m)	Dimensions $i \times j$ (cells)	Origin UTM 2000 Zone 60 (mE, mN)	Orientation (°T)
sControl	10	201 × 201	295050, 5950700	205
1x3Reefs	10	201 × 201	295050, 5950700	205
2x3Reefs	10	201 × 201	295050, 5950700	205
3x3Reefs	10	201 × 201	295050, 5950700	205
tControl	10	201 × 316	295050, 5950700	205
4x3Reefs	10	201 × 316	295050, 5950700	205

A range of boundary conditions were developed to test the range of reefs, as well as the impacts of spectral time-series modelling, mean-modulated modelling and variable period modelling. It is important to note that morphological modelling was undertaken using wind-wave spectral modelling developed from the representative long-term inshore wave climate, which was then binned into probabilities of occurrence that were represented by these weightings. For example, if the long-term probability for 0.5 m waves, at 4 sec period, from 67.5° was 20%, then 1/5th of the boundary conditions within the time series would be representative of this condition. Boundary conditions for ‘normal’ and storm conditions were represented by both time series boundaries (i.e. a 36 hour storm event would start with ‘normal’ conditions, develop into a typical storm with wave height, direction and period changes, and then drop back to ‘normal’ conditions to assess the impacts on hydrodynamics and morphological response), and modulated-modelling, where wave height and direction are varied around an average condition (with peak period). Tides were also represented as either time series from the calibrated tidal model or as a constant amplitude sinusoid (i.e. modulated). Sinusoidal boundaries are listed in Table 3.4, with an example shown in Figure 3.22.

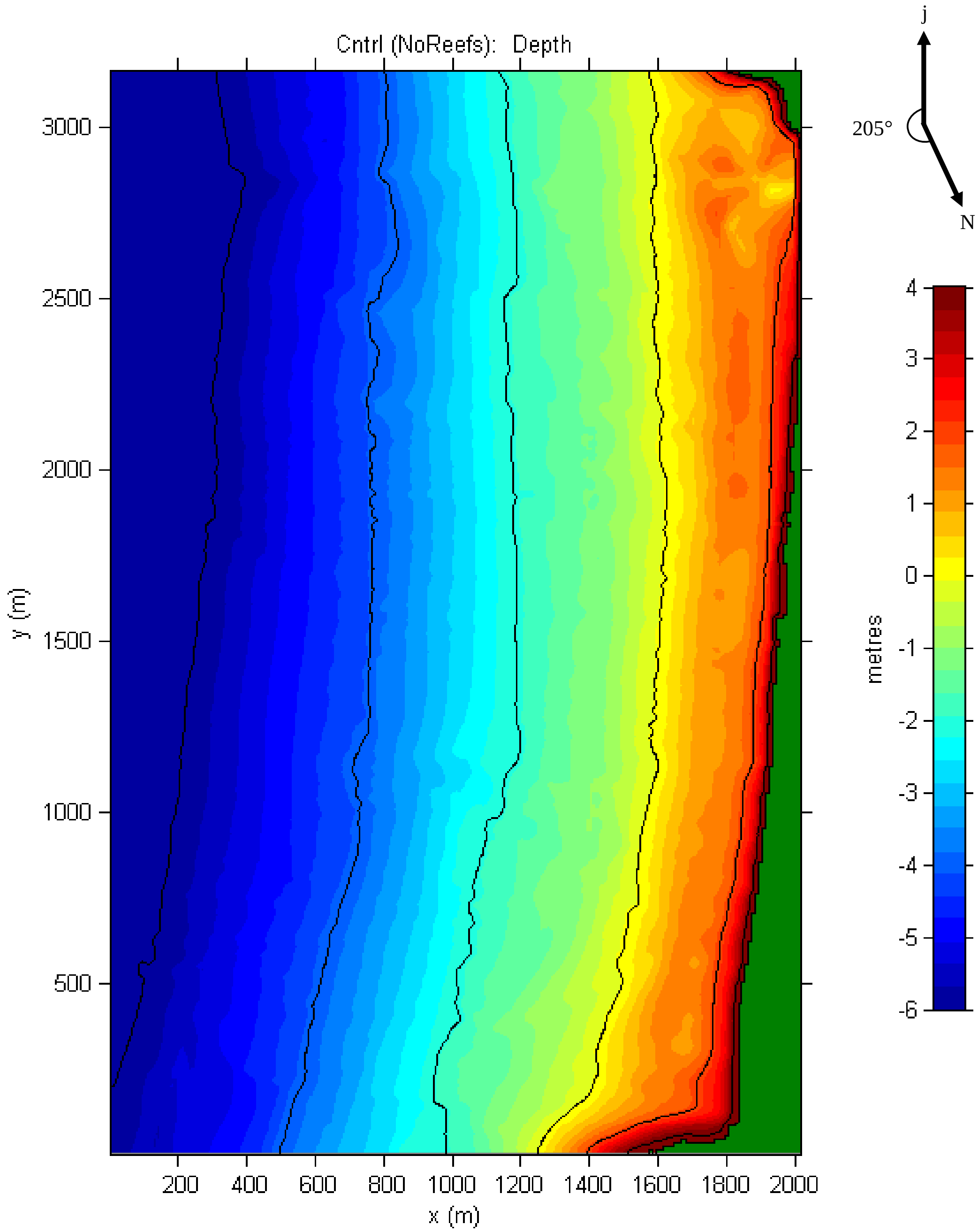
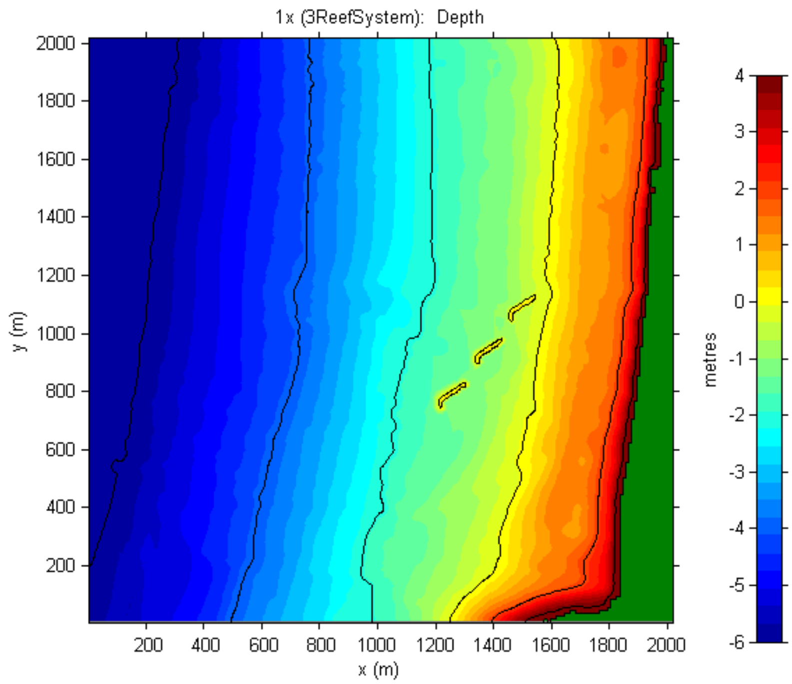
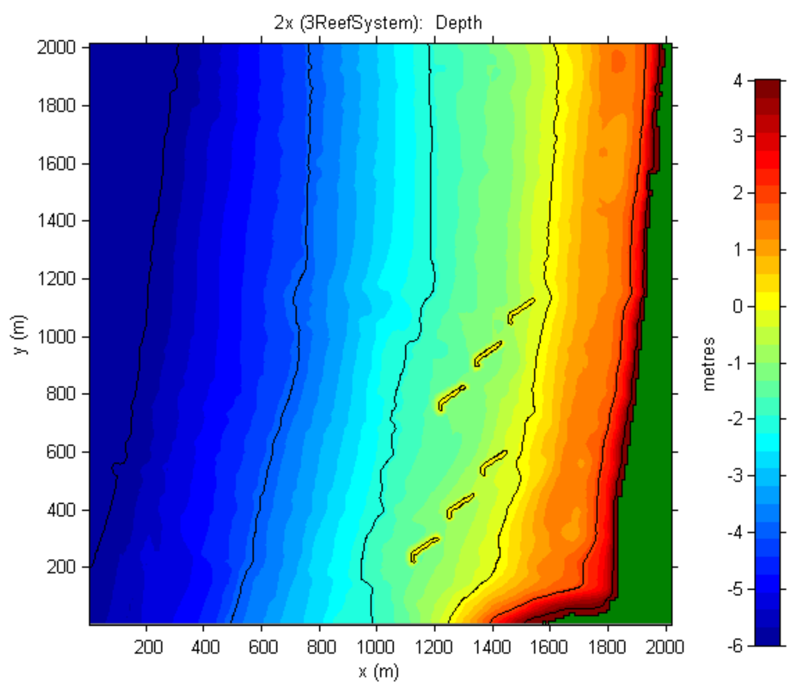


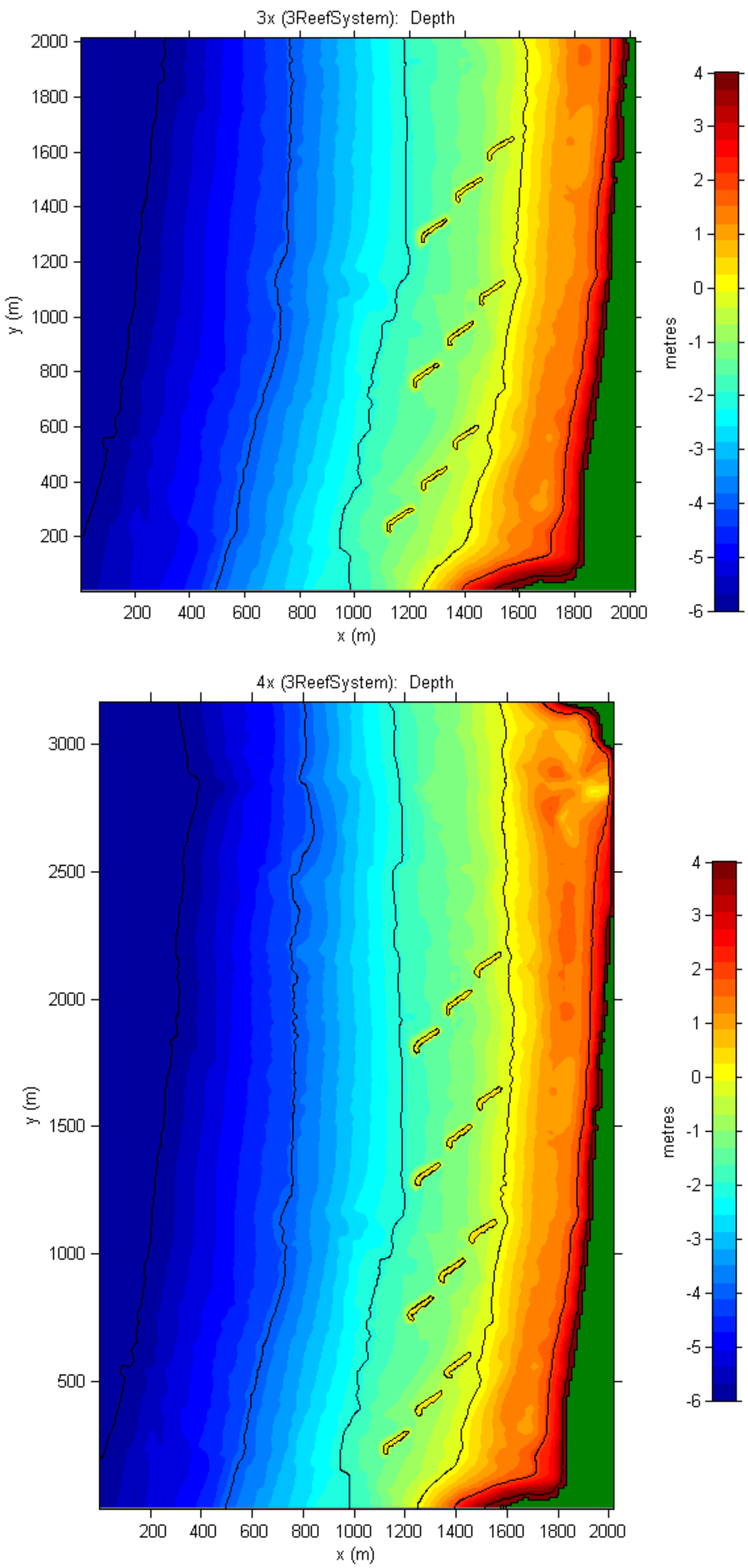
Figure 3.20 The natural ("Control") bathymetry (from Mead et al., 2004) at Orewa Beach is used as a baseline for numerical modelling comparative studies presented here. All modified bathymetry grids utilise this grid as a baseline.


1x (3ReefSystem): Depth
y (m)
x (m)
metres

a


2x (3ReefSystem): Depth
y (m)
x (m)
metres

b



c

d

Figure 3.21 Bathymetry grids including (a) 1x, (b) 2x, (c) 3x and (d) 4x 3Reef system structures.

Table 3.4. Sinusoidal coefficients applied as boundary conditions

Conditions	Component	Mean	Amplitude	Period (sec)	Phase (deg)
Common	Wave Height (m)	0.55	0.25	3600	180
	Wave Angle (° rel. *lhs*)	-3	8	7200	180
	Wave Period (s)	7.5	4	3600	180
	Tide (m)	1.7	1	25000	180
1 yr RI Storm	Wave Height (m)	3.5	0	*n/a*	*n/a*
	Wave Angle (° rel. *lhs*)	0	0	*n/a*	*n/a*
	Wave Period (s)	8	0	*n/a*	*n/a*
	Tide (m)	2.4	1	25000	180
10 yr RI Storm	Wave Height (m)	5	0	*n/a*	*n/a*
	Wave Angle (° rel. *lhs*)	0	0	*n/a*	*n/a*
	Wave Period (s)	8	0	*n/a*	*n/a*
	Tide (m)	2.4	1	25000	180
100 yr RI Storm	Wave Height (m)	7	0	*n/a*	*n/a*
	Wave Angle (° rel. *lhs*)	0	0	*n/a*	*n/a*
	Wave Period (s)	8	0	*n/a*	*n/a*
	Tide (m)	2.4	1	25000	180

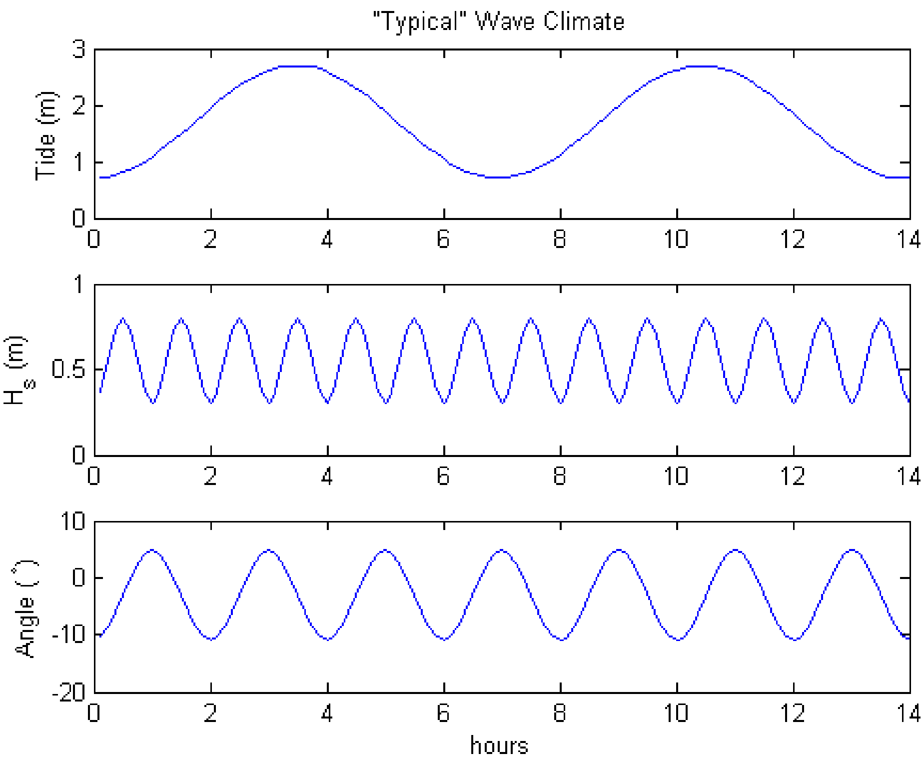


Figure 3.22. “Common” conditions applied to the LeftHandSide(LHS) of the model domain as described by sinusoids in Table 3.4. Sinusoidal coefficients applied as boundary conditions.

Single System

The following plots show the *mean* wave height over a simulation of typical wave conditions (Hs = 0.3 - 0.8 m including tides) at Orewa (without and with 1xReef system) (Figure 3.23 and Figure 3.24).

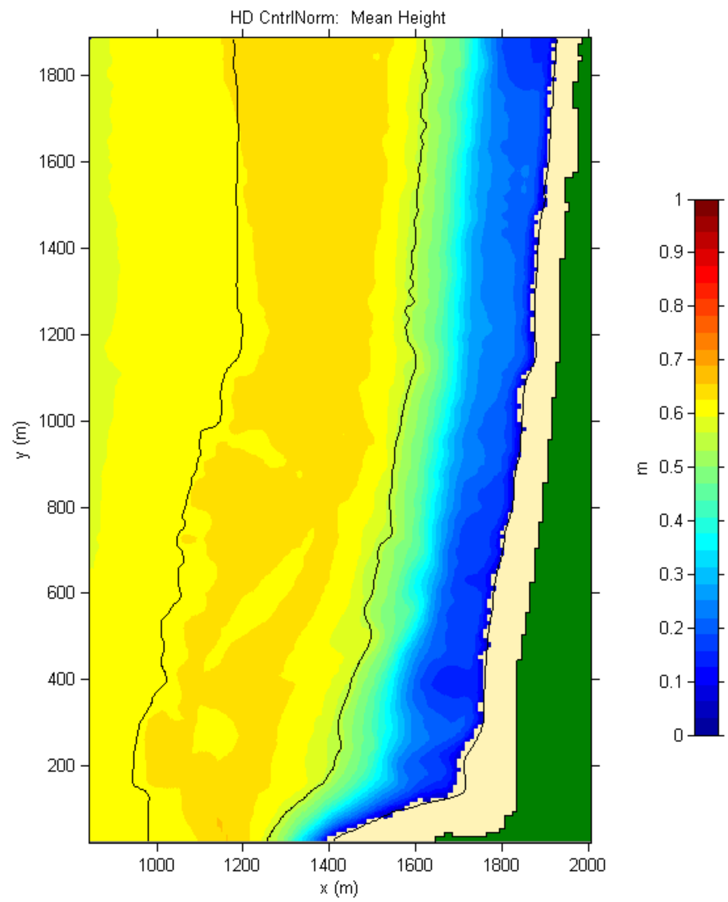


Figure 3.23. The mean wave height through a series of tidal cycles and typical wave conditions over the natural bathymetry grid of Orewa Beach.

Waves approach shore relatively uniformly. Wave energy is dissipated on the gradually sloping beach. As the bathymetry slowly gets shallower, wave heights are slowly diminished due to depth-limited shoaling. As the gradient of the nearshore bathymetry is so gradual, waves are typically reduced offshore, reformed with less energy into a young yet organised swell and propagate further inshore until it also is depth-limited, shoals, breaks and reforms. As a result, the surf zone at Orewa Beach is relatively wide.

With tidal modulation, the depth felt by approaching swell affects the distance from shore that the wave energy can reach without full dissipation. For example, at high tide, waves may break some 200 to 400 metres further inshore than at low tide – purely because of the increased water depth which enables shoreward propagation of wave energy before dissipation by depth-limitation.

Waves of higher period contain more energy, and on a gentle-sloped beach like Orewa, wave breaking begins as the ratio of breaker height to water depth at breaking is proportional to the square of the breaker period (i.e. H_b/d_b is proportional to T^2). As the quantity of wave energy increases, the distance over which the gradually sloping beach dissipates energy is increased. For example, for waves of a constant height, by increasing the peak spectral period, wave energy can penetrate much further inshore.

By encouraging early and localised wave dissipation offshore, a shadow is created where wave energy is reduced in the direction of propagation. This equates to reduced orbital velocities at the seabed at the inshore shallows in the lee of the reef(s). It should be noted that the induced breaking of waves at the reef structure will increase the local velocities. However, this effect is isolated to a neighbourhood bounded by the wave energy and the local rate of dissipation, i.e. by increasing energy dissipation on the reefs, less energy is available in the adjacent areas of the reefs influence.

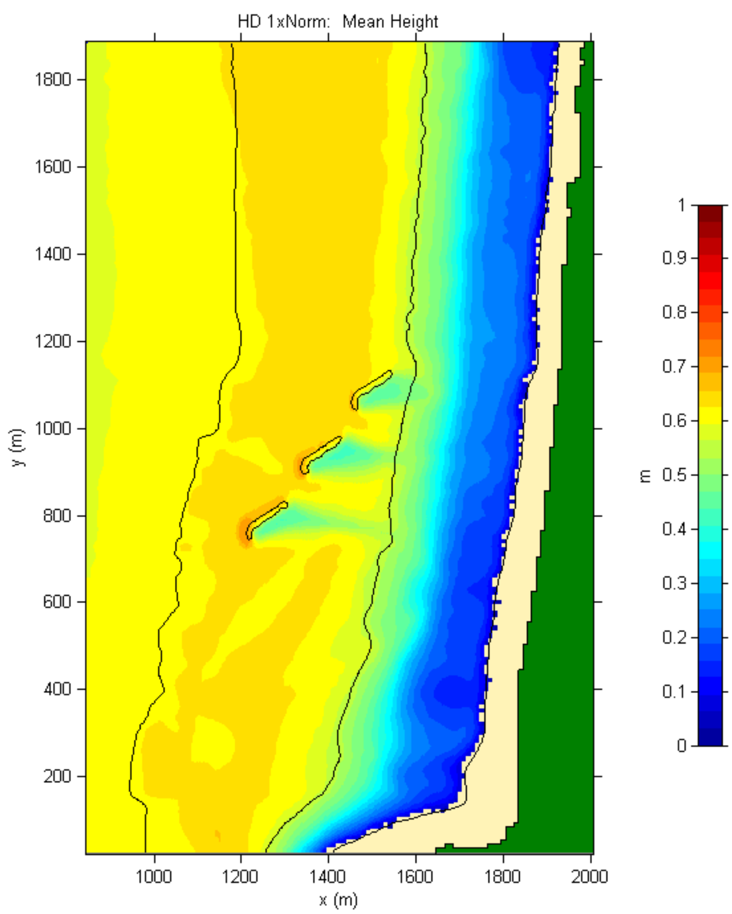


Figure 3.24. The mean wave height through a series of tidal cycles and typical wave conditions over a single set of 3 reefs (1x3Reef Structure) at Orewa Beach.

The most notably obvious aspect of the inclusion of a 3-reef structure is the offshore breaking of waves. Also there is a substantial wave shadow in lee of the reef components of the structure. Wave energy is permitted to pass between the individual components of the 3-reef structure, but the individual wave fronts are unsupported and will dissipate due to radiation stress. This intermittent shadow/transmission of wave energy acts to break up the connectedness of nearshore swash zone processes traditionally responsible for reducing bathymetric modifications in the nearshore, i.e. the 'smearing' of the beach response due to the wide surf zone.

Looking closer at wave-driven currents around the reef structure, the additional function of the reef system (i.e. in addition to wave energy dissipation) of redirecting currents can be analysed (Figures 3.25-3.29). The reefs encourage breaking of waves and drive water currents shoreward in lee of the reefs. This process initially results in scour in the lee of the structure (with scour bags and combi-grid used to counter undermining of the structure and ensuring stability). Return flow is visible to the sides of this action, resulting in dynamic circulation cells around the reefs. Inshore directed currents are dominant in the lee of the reefs. Furthermore, as the reef system is composed of staggered reef components, the furthest offshore reef component acts to direct water to the next (shoreward) reef component which is in-turn fed into the breaking wave action of the next reef and so on. The process is like a conveyor belt redirecting and breaking wave energy while directing offshore sediments further inshore and maintain sand in the beach system rather than. It is worth noted, that under these 'normal' conditions, currents are relatively low apart from on the reef crest and directly in the lee of reef units, i.e. <0.15 m/s, with 0.3 m/s being the threshold of sand movement. Thus, the main mechanism for moving sand in these conditions is wave action 'lifting' sand, while the residual (or vector-averaged) currents move sand along.

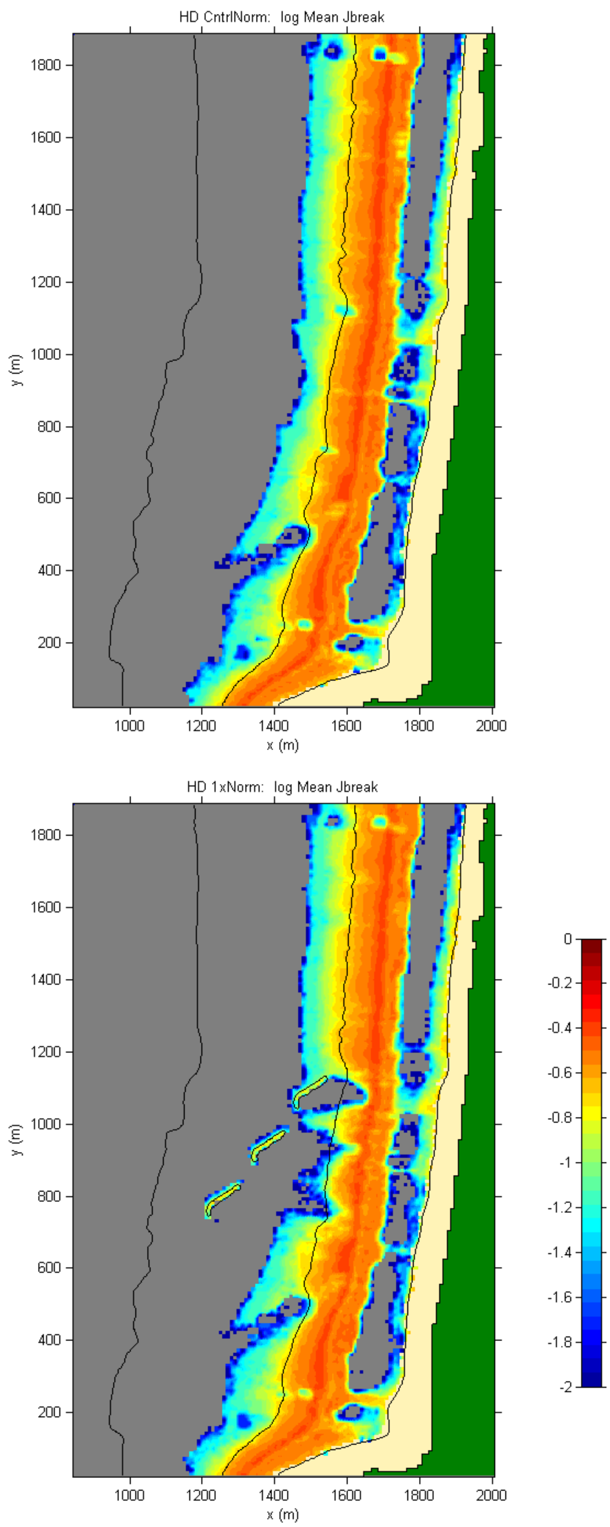
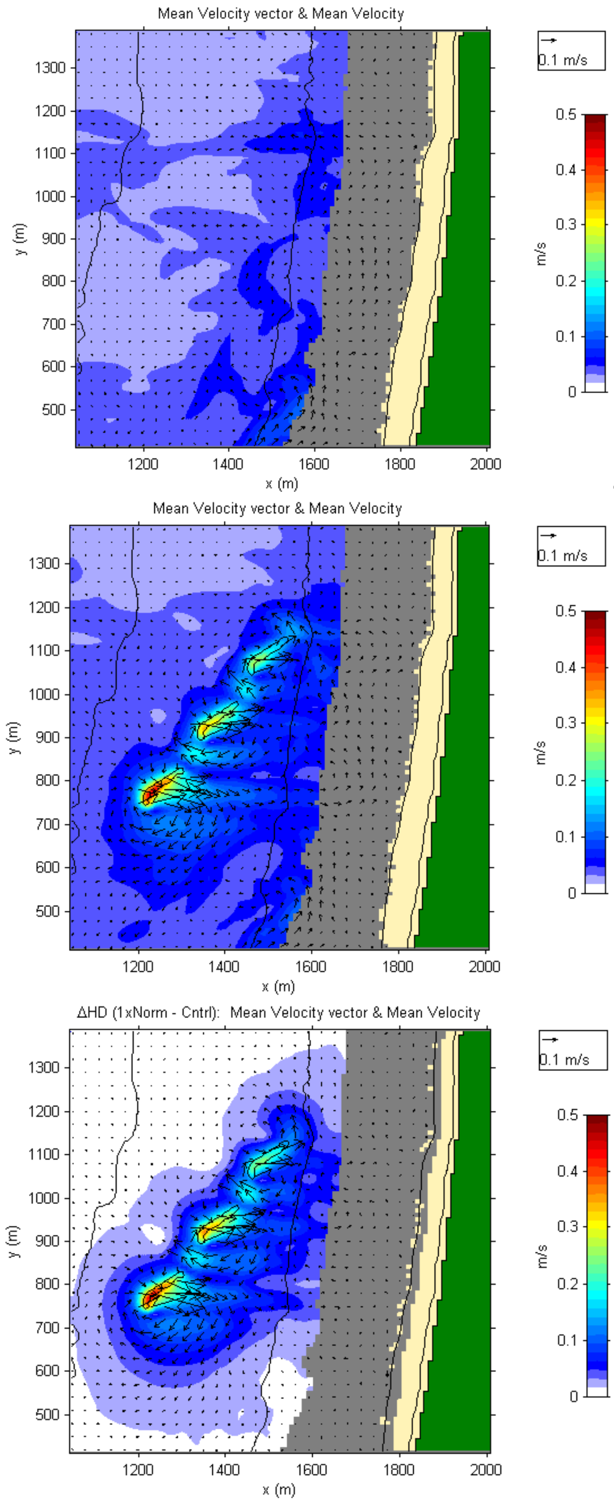


Figure 3.25 Mean Breakpoints through the simulation for Control and 1xReefSystem and high tide The Reefs are shown to encourage offshore breaking of waves, reducing the inshore wave energy and breaking-up the longshore coherence of wave-associated turbulence.

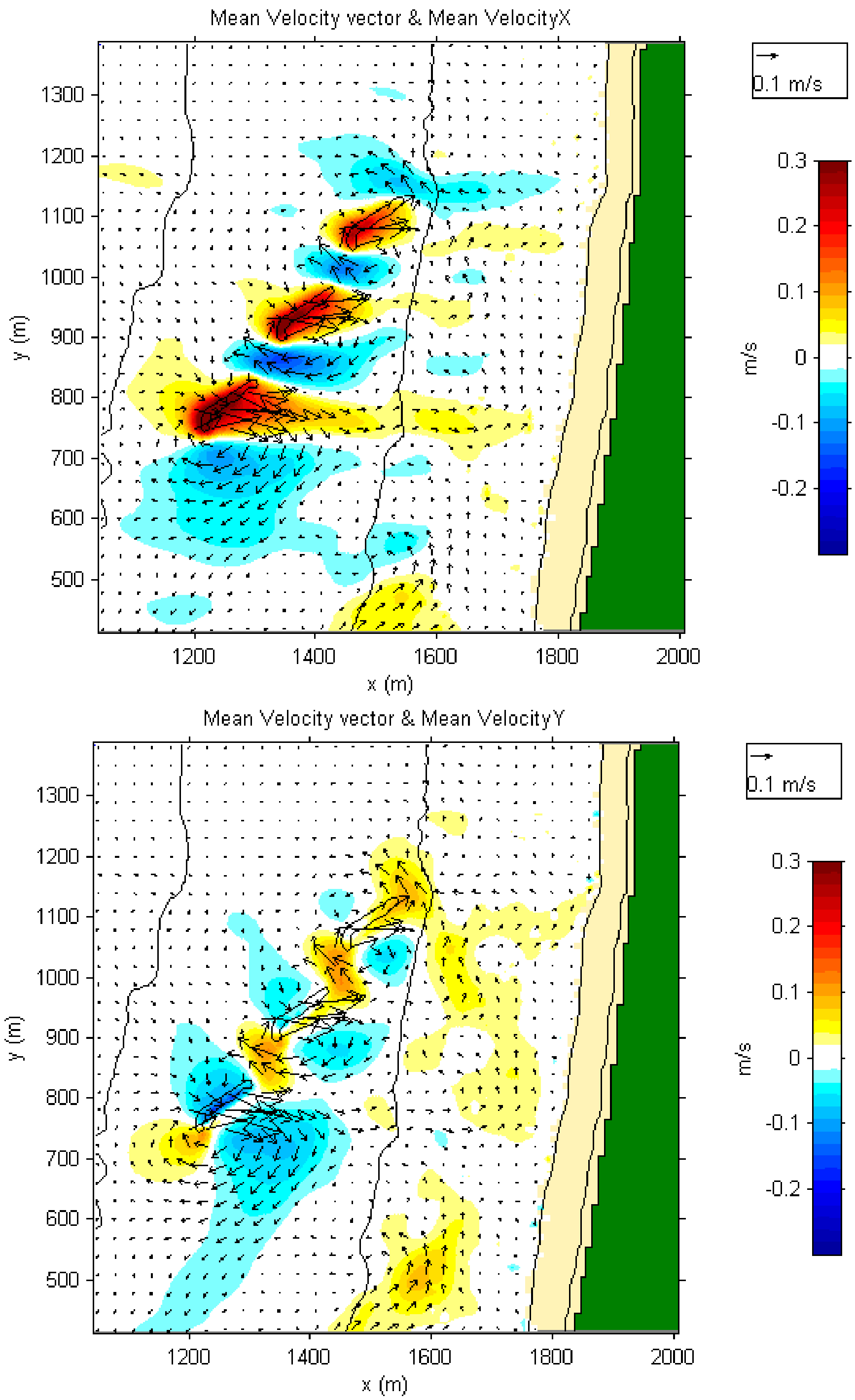


a

b

c

Figure 3.26 Vector-averaged velocity of wave-driven currents around the 3reef system during the ‘normal’ condition simulation (a) without reefs, (b) with reefs and, (c) as a difference from the control run (c). Current velocity is stronger on and around the reef system due to the shoaling and breaking of waves. Wave breaking over the reef redirects currents in the onshore direction, while return flow is seen between and around the individual reef components of the system. Changes in currents are isolated to a local region around the system. Currents are mostly low, below 0.15 m/s.



a

b

Figure 3.27. Vector-averaged velocity of wave-driven currents around the 3-reef system during the simulation decomposed into (a) cross-shore, and (b) longshore components.

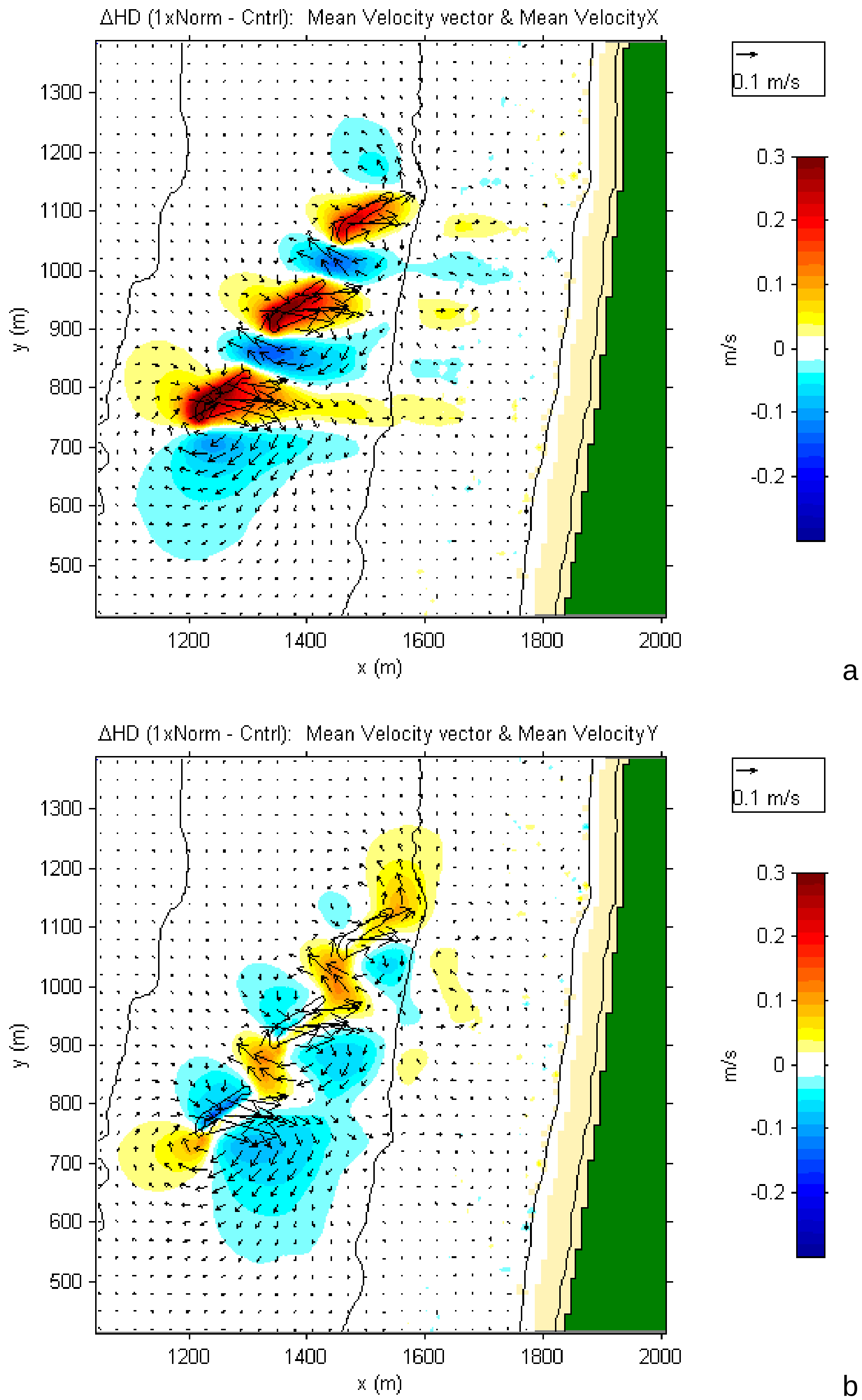


Figure 3.28. The change in mean current velocity in terms of (a) cross-shore (VelocityX), and (b) longshore (VelocityY) components.

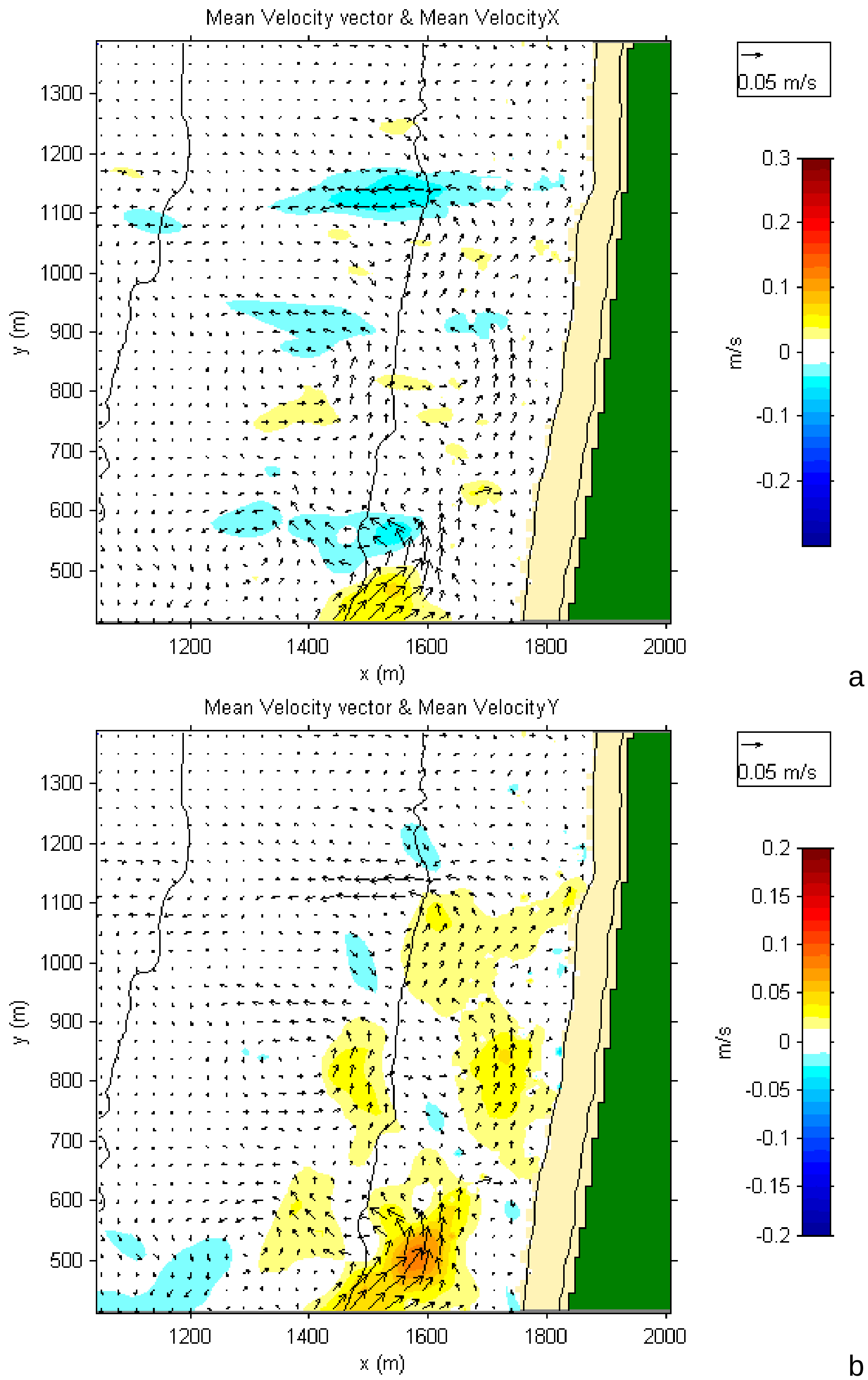


Figure 2.29. Vector-averaged velocity of wave-driven currents without reef structures during the simulation decomposed into (a) cross-shore, and (b) longshore components. Quiver size is increased to aid in visualisation.

Another noteworthy aspect is the alternating direction of cross-shore currents along the length of the beach in lee of the reef system. The existence of this feature implies the break-up of coherent organised turbulence which is the main driver of sediments to the south (up the page given the rotation for modelling) during the most common storm scenarios (i.e. from a northeasterly direction). That is to say that the wide sweeping swash zone currents lose their upstream influence as they enter the region in the lee of the reef.

For the cross-shore component, the reef structure can be seen to increase the shore-ward currents (Figure 3.26-3.29). Highest increases occur directly on the reef, as the reef induces wave breaking while providing static support at the seabed. Local sea level gradients drive return flow at the nearest channel or bathymetric depression – in this case between the reefs. Also, for any number of reef structures *N*, there exist *N+1* gaps around the structures for return flow. As volume is conserved, it can be seen that the forward velocities are significantly stronger than the offshore directed return flows. This will aid in the shoreward delivery of suspended sediments.

The change in the mean longshore velocity component with the inclusion of the reef structure is shown in Figure 3.28. The magnitude of change in the longshore velocity component is smaller than that of cross-shore velocity. One notable feature is the increase in northward (page down) longshore current in the offshore extents of the structure (comparing Figure 3.29 to Figures 3.26-28). Alternatively the longshore currents trend southward (page up) in the nearshore without the reefs is seen to significantly diminish with the presence of the reefs (comparing 3.29 and 3.27).

While Figures 3.24 to 3.29 illustrate the mean wave height transformation and mean current patterns characteristic during ‘normal’ conditions, it is important to acknowledge the actual performance of the structure during particular conditions.

Performance during Storms

Typically, Orewa experiences a low-energy wave climate. However, it is occasionally subjected to storm conditions.

While the reef systems are designed to develop and maintain a wider beach during ‘normal’ conditions, it is important to examine how they will perform during storms. During storms, waves can get quite large at Orewa. Table describes the waves applied during the storm simulations (note, these wave heights are at the 7 m deep + high tide/storm surge offshore boundary (Table 3.5).

Table 3.5 Definition of storm return interval conditions

Storm RI	H_s (m)	T_p (s)	Angle (° rel LHS)	Storm Surge (+ m)
1 yr	3.5	8	6°	0.50
10 yr	5	8	6°	0.50
100 yr	7	8	6°	0.50

Again, due to the gradually sloping bathymetry at Orewa, waves of significant size are first broken relatively far offshore. The ‘messy’ storm seas have been reportedly seen to break and reform several times along the shoreward propagation (pers. comm./obs.).

Also, during storms at Orewa, large scale wind-driven set-up and low barometric pressure can increase the local water level in what is termed storm surge. Large wave events accompanied by large storm surge can be particularly erosive because if they are timed to correspond with an astrological high tide, water levels can exceed MHWS levels and encroach higher up the beach. As the water level is increased, so is the depth and associated wave energy penetration. Storm scenario modelling includes storm surge 0.5 m (as recommended as in Coastal Hazards and Climate Change: a Guidance Manual for local government in New Zealand (NZCCO and MfE, 2004) during a high spring tide (i.e. 3.4 m tidal elevation), while wave set-up and run-up are simulated by the model.

Mean wave heights over the 1 yr RI storm simulation are shown in Figure 3.30. Large waves are seen to break well offshore – offshore of the 4m (below LAT) isobath. However, wave energy still exists and continues to move shoreward, dissipating as effective depth decreases. Regardless, substantial wave shadows exist in lee of the reef system – indicating their effectiveness at reducing wave energy during a storm.

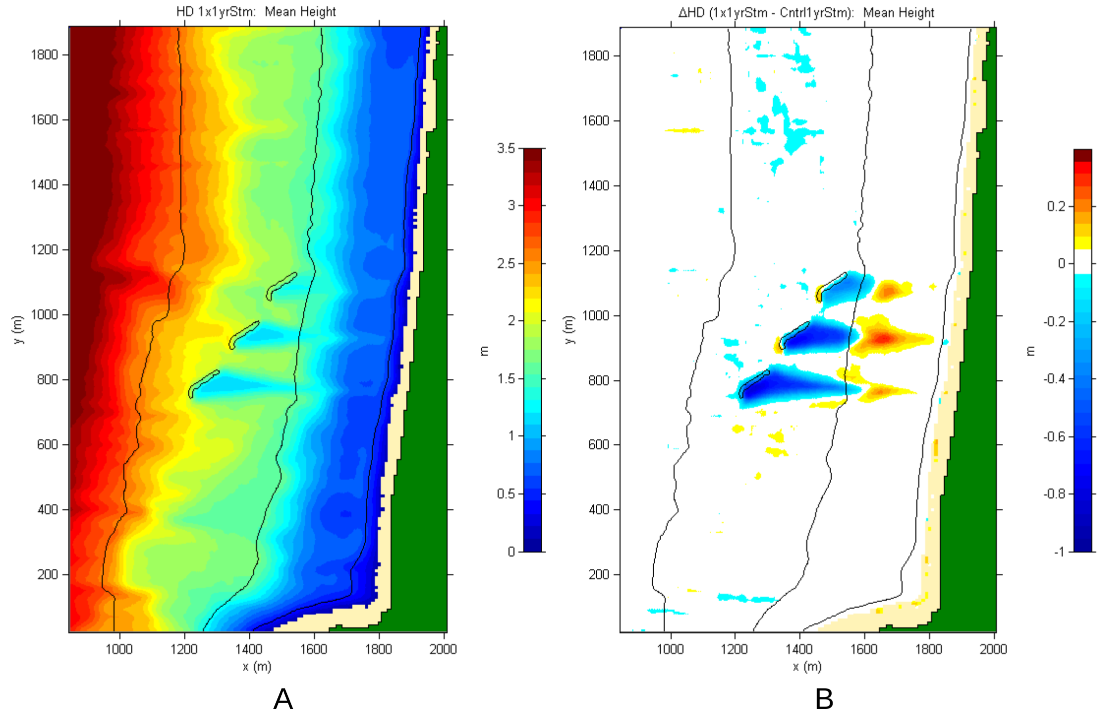


Figure 3.30. Mean Wave Height during a 1 yr RI storm simulation (a), and the difference in mean wave height with inclusion of the reef system structure (b). Depth contours indicate the position of the reef system, and substantial wave shadowing is evident in lee of the structure. On average during the simulation, wave height is substantially reduced in lee of the reef structure.

Of course, with tidal modulation wave energy is allowed further inshore before dissipation and during a storm tide even further. Figure 3.31 shows the effectiveness of the system during a full storm tide.

Differencing the storm simulations (i.e. Reef – Control) reveals the actual change in wave heights between the two simulations. Figures 3.31 to 3.33 show that the structures reduce the wave height substantially and create a shadow of wave energy. Each reef component has a small wave focus at the shoreline. This is an artifact of wave refraction/diffraction behind the structures and is relatively small with respect to both initial wave height and height reduction. Also, wave reduction is seen laterally from the nose of the system. This is due to the loss of energy within a wave front as the reef system causes wave breaking.

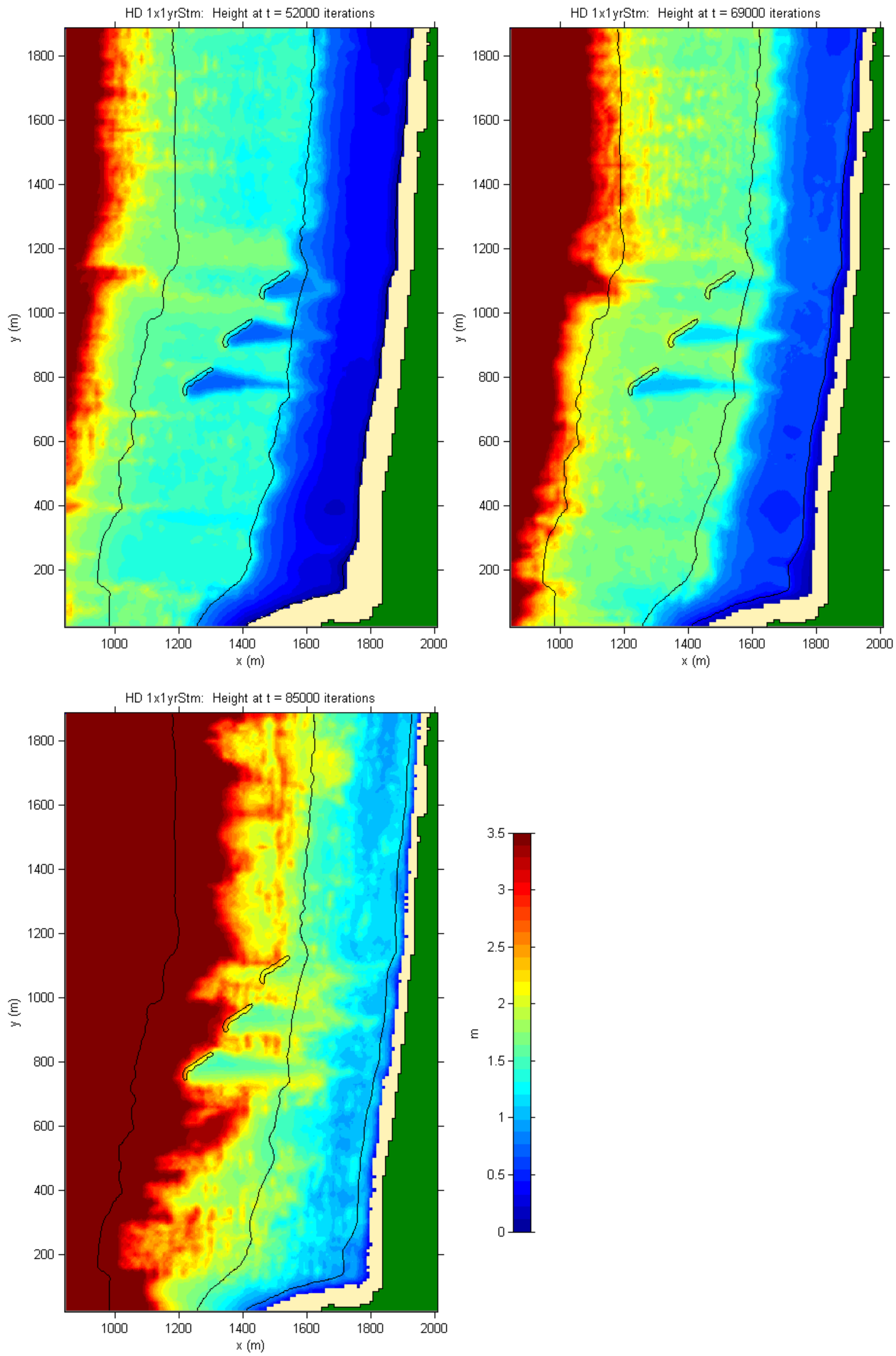


Figure 3.31 Wave heights during a 1 year RI storm event (Hs = 3.5m, + 50 cm storm surge) during low, mid and high tides.

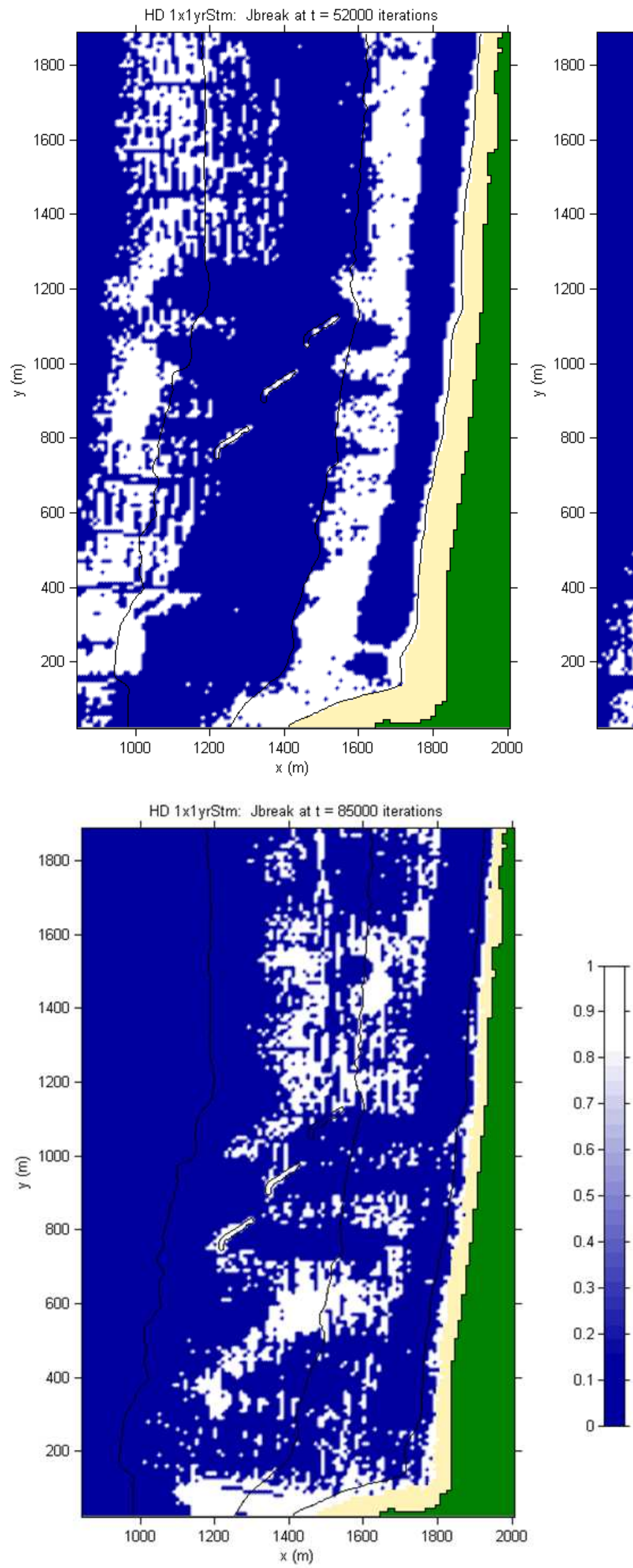


Figure 3.32. Breaking waves during low, mid and high tide during the 1 yr RI storm simulation.

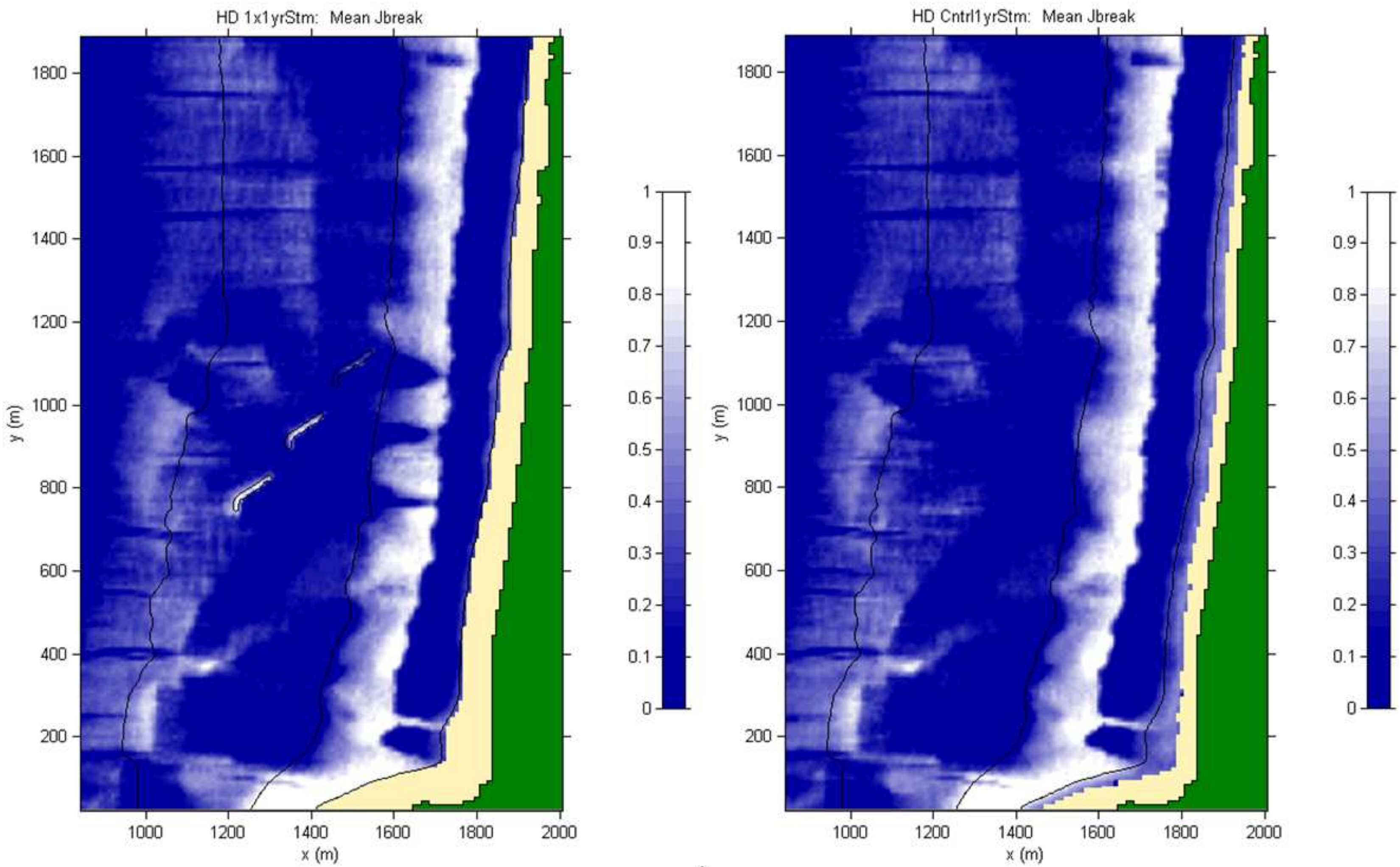


Figure 3.33. Mean Jbreak indicates where waves are breaking during the simulation as a fraction of time. Notice the reduction of breaking at the shoreline in comparison to the control (no reef) situation.

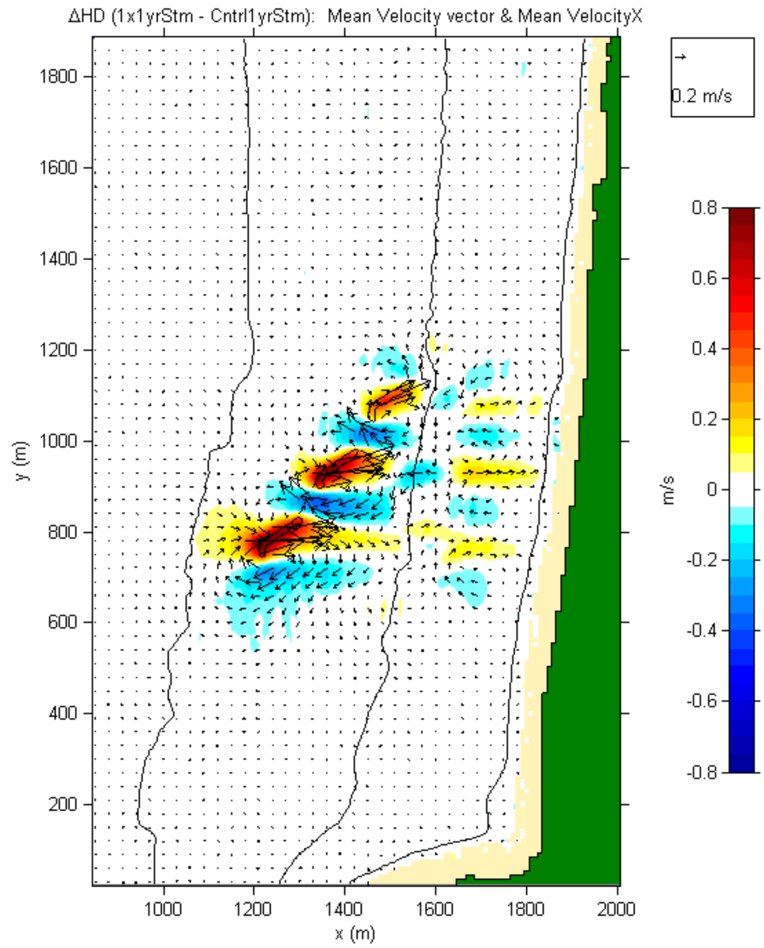


Figure 3.34. The change in mean current velocity in terms of the cross-shore (VelocityX) component.

As for the ‘normal’ conditions, for 1 year return interval storm events wave-driven currents are stronger in the direction of the shoreline and act to break up coherent swash movement in along the shoreline (Figure 3.34).

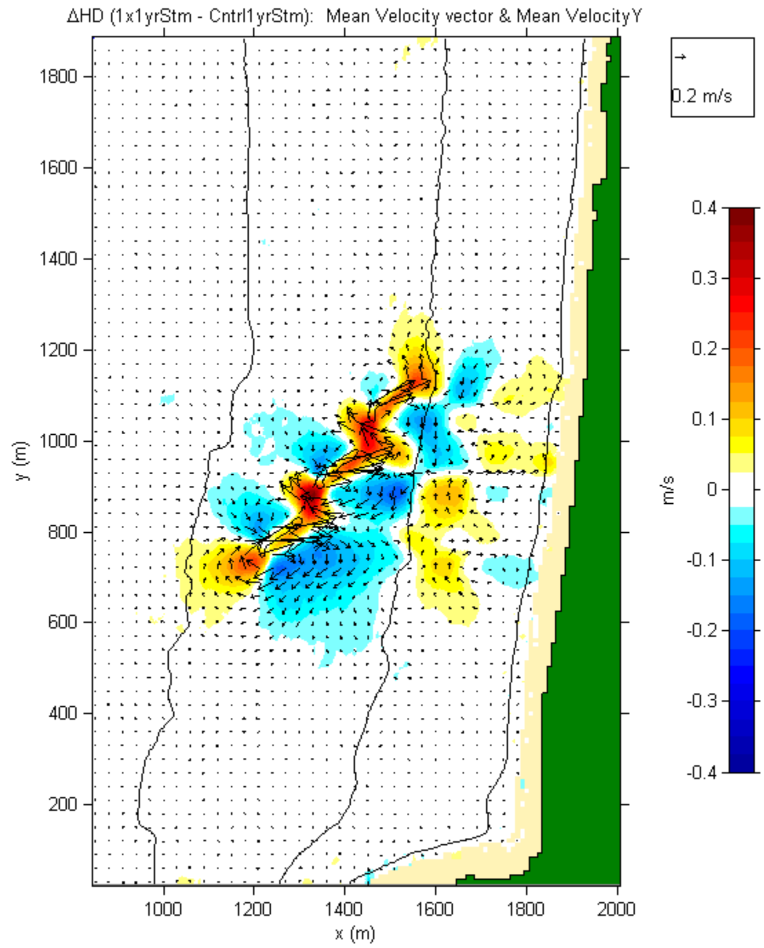


Figure 3.35. The change in mean current velocity in terms of the along-shore (VelocityY) component.

During a storm, the longshore velocity seems to build a semi-connected mass moving northward (page down) in the lee of the reef system and a lesser southward (page up) directed current further inshore (Figure 3.35). At the appropriate isobath, where the two currents meet, there will be a convergence of sedimentary processes, aiding the development of a salient or wider beach. In comparison, the no-reef control displays the dominant southwards (up page) currents responsible for removing renourishment material without coastal protection structures (e.g. T&T, 1994) (Figure 3.36).

Similar modifications to the currents (i.e. a change to dominant inshore and additional northward currents) are seen when 1, 10 and 100 year RI events are considered (Figures 3.36-3.38 and 3.39-3.41). Due to the low gradient beach and depth limited breaking, the currents are not significantly stronger in higher wave events. However, storm surge and wind/wave set-up is likely to increase the relative sealevel, allowing waves to penetrate higher up the shore and so cause more damage during these events.

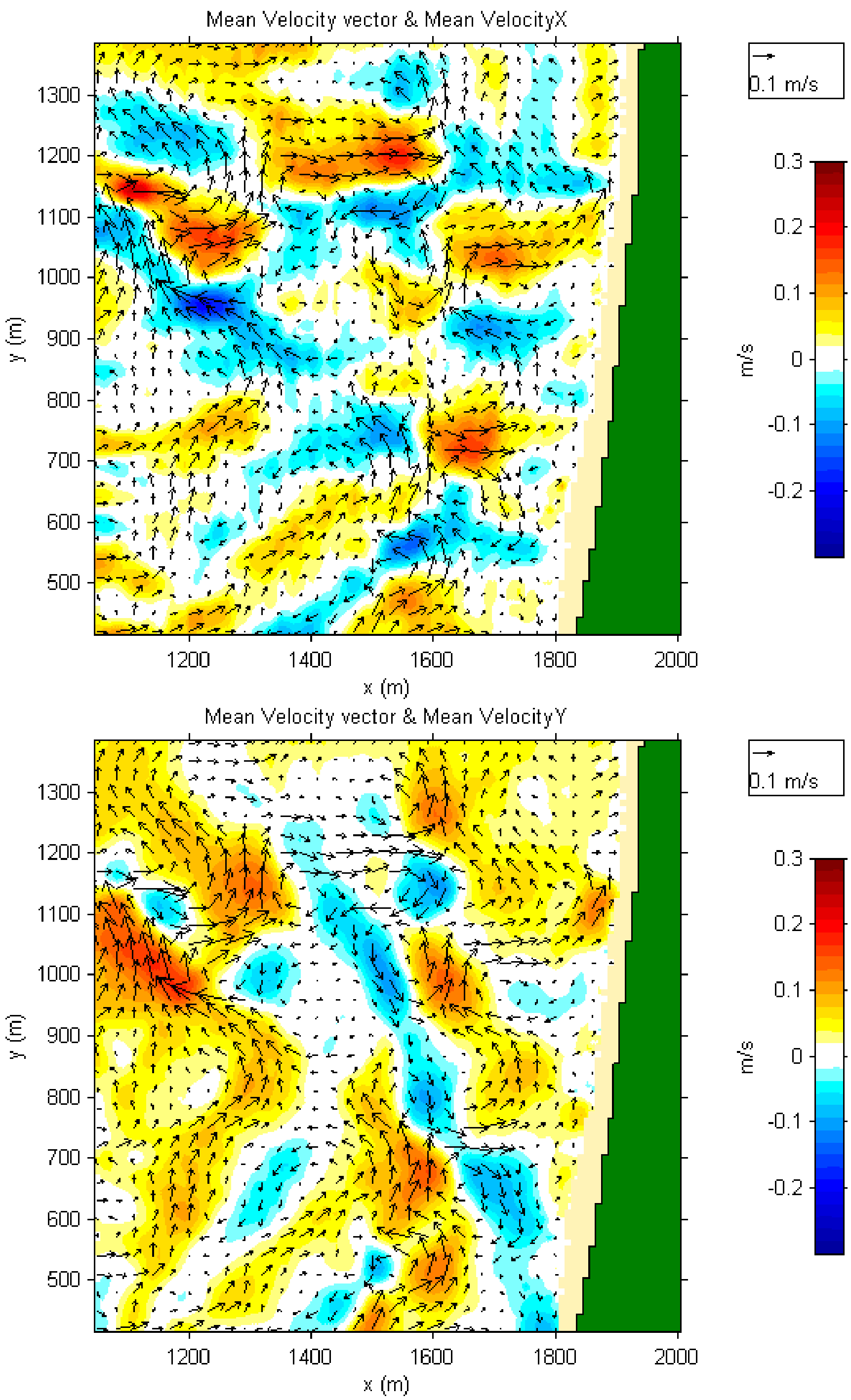


Figure 3.36. Velocity components without reefs during 1 yr RI storm conditions

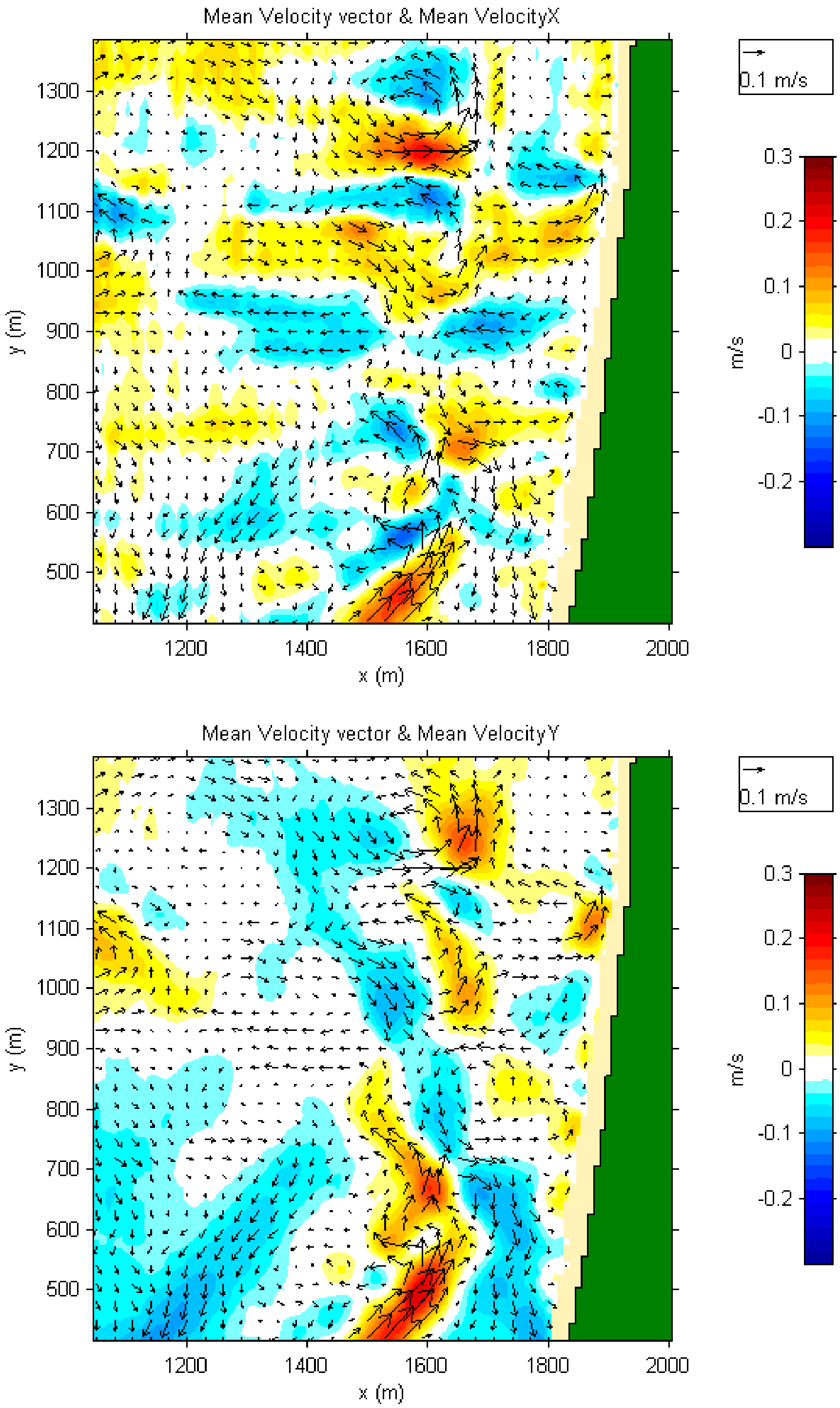


Figure 3.37. Velocity components without reefs during 10 yr RI storm conditions

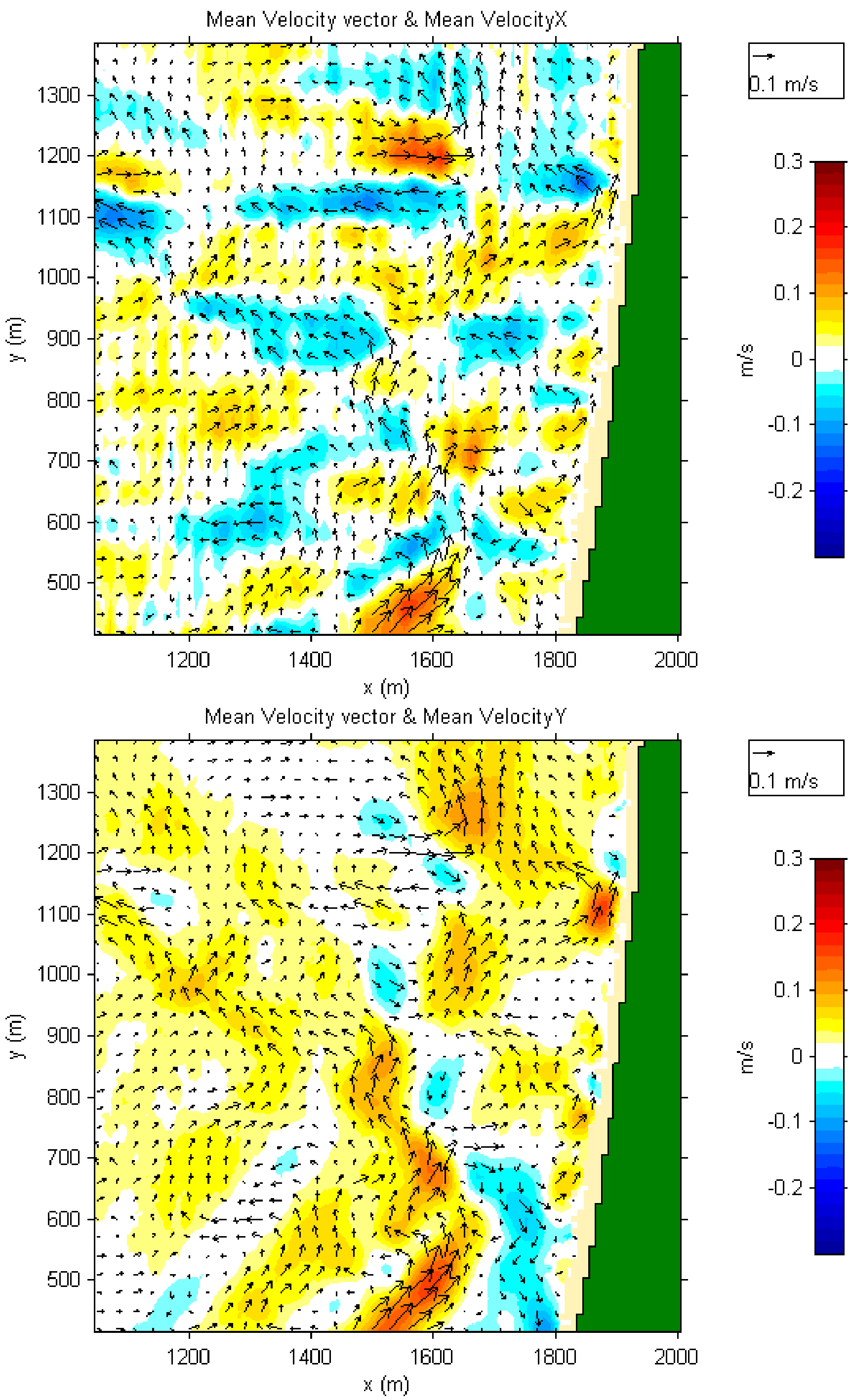


Figure 3.38. Velocity components without reefs during 100 yr RI storm conditions.

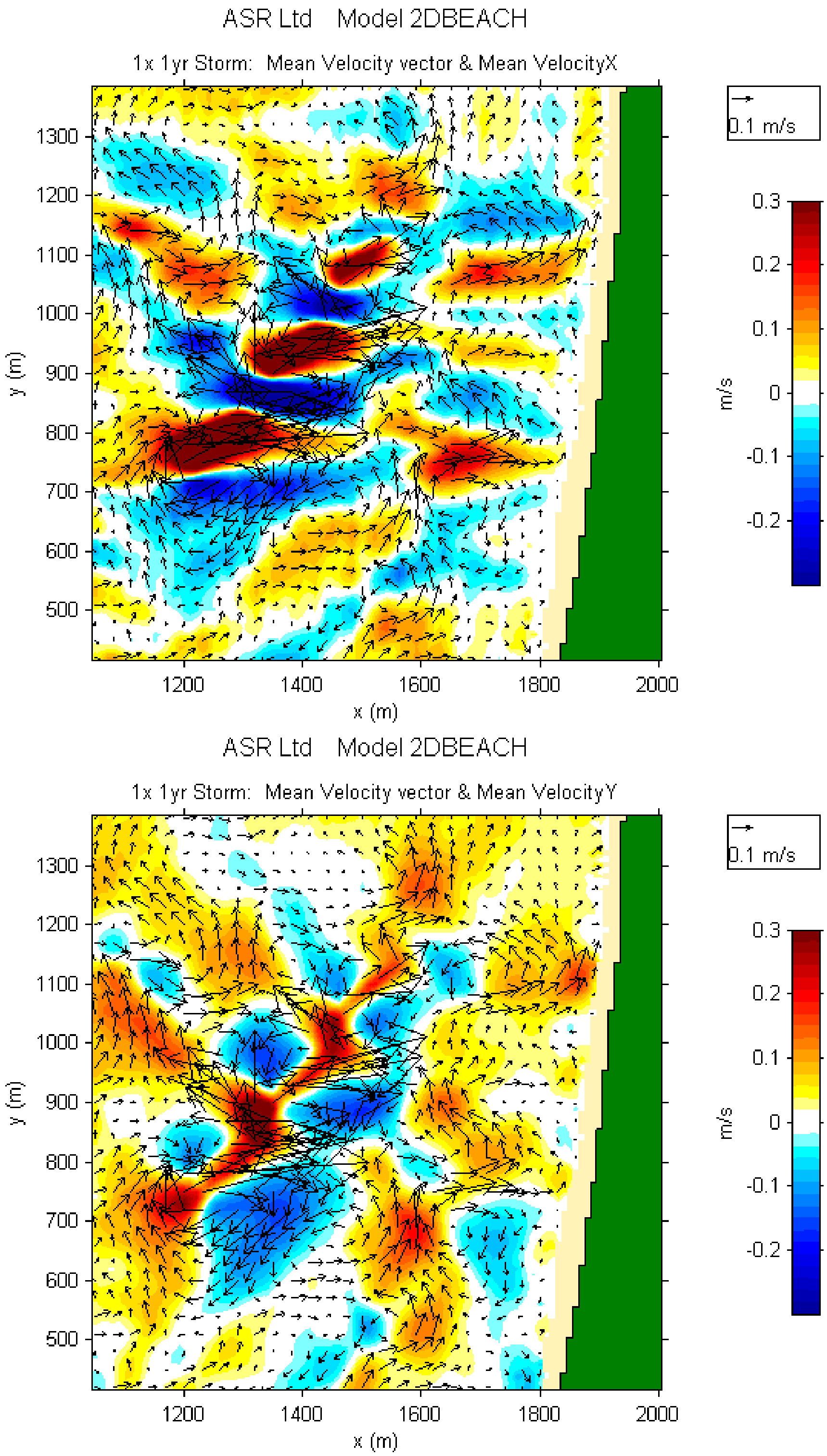


Figure 3.39. Velocity components without reefs during 1 yr RI storm conditions with a reef system.

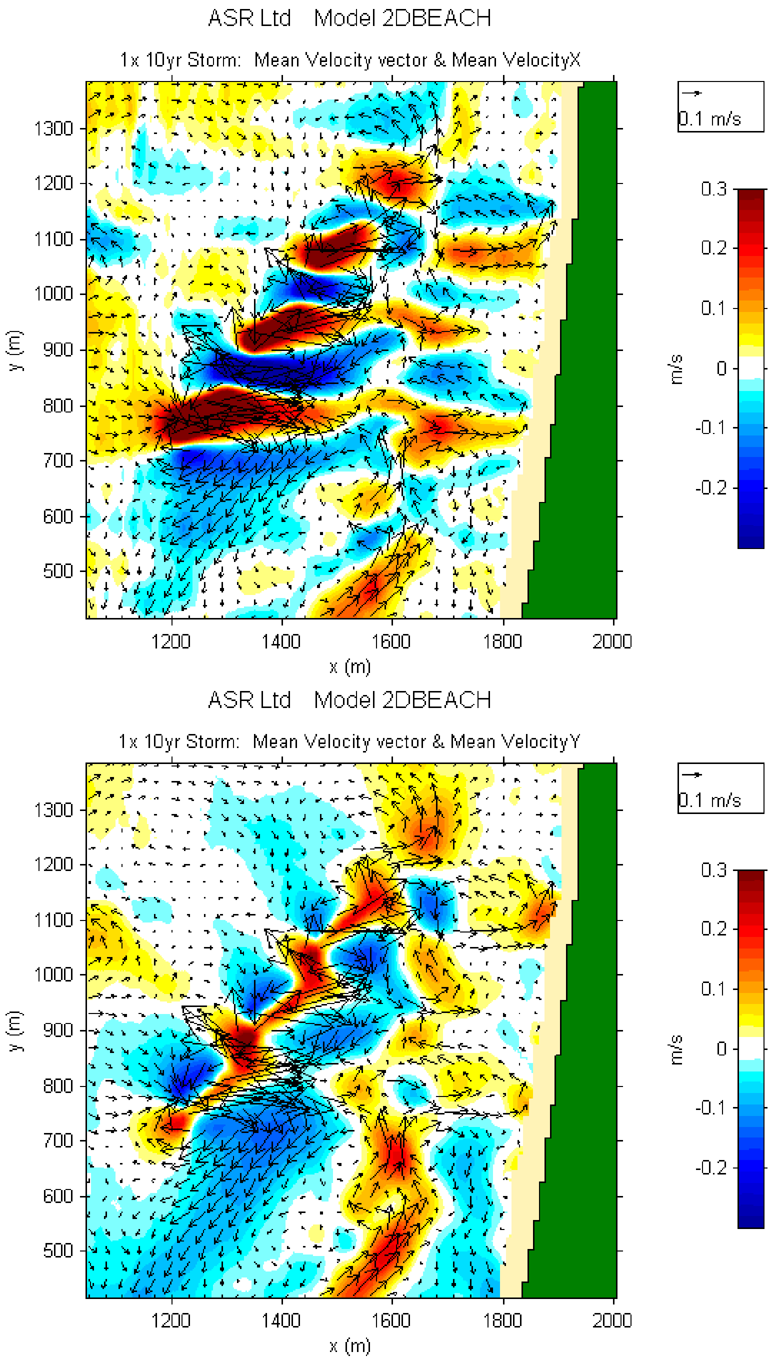


Figure 3.40. Velocity components without reefs during 10 yr RI storm conditions with a reef system.

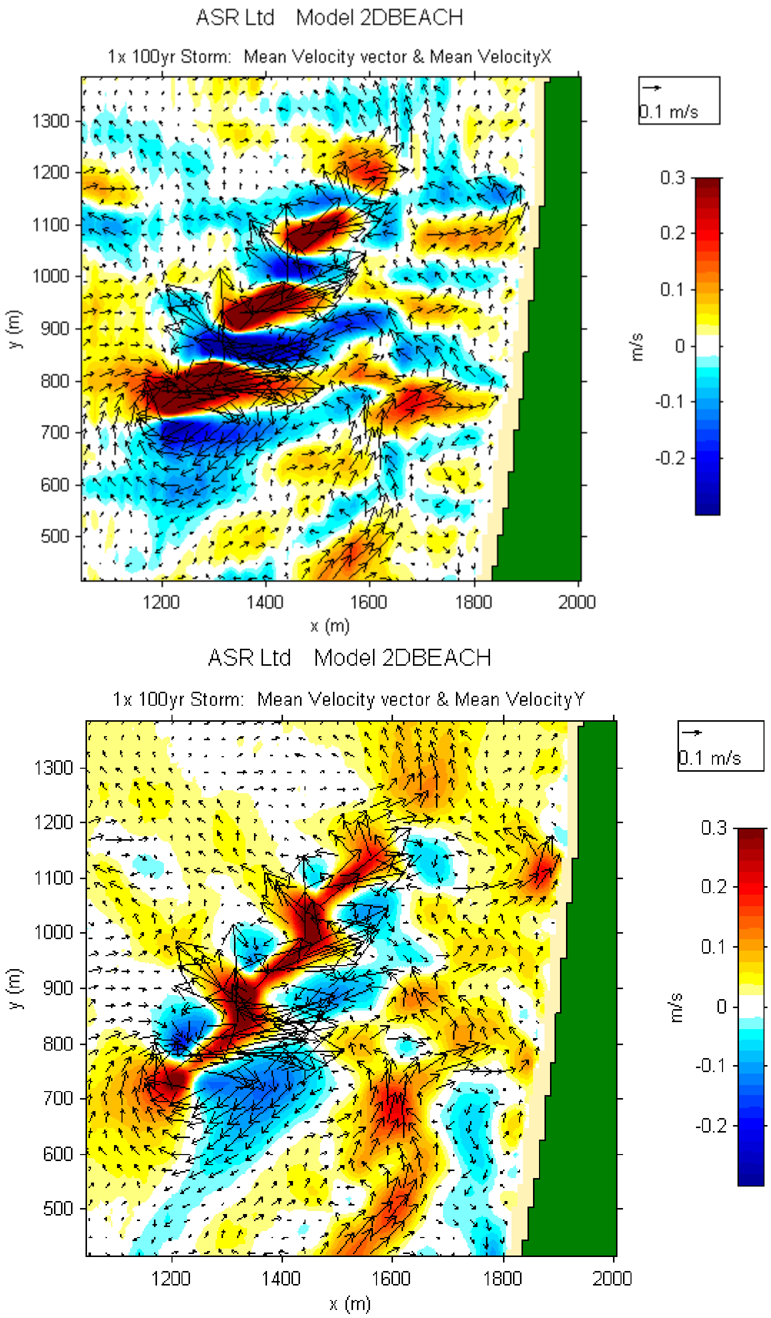


Figure 3.41. Velocity components without reefs during 100 yr RI storm conditions with a reef system.

Multiple Systems

While it can be seen that the influence of a reef system in terms of current patterns is relatively localised to the immediate vicinity of the structures (e.g. Figure 3.26c indicates that changes in current velocities of >0.15 m/s are restricted to within 50 m of the structures), interactions between multiple reef systems were considered. It is noted that due to feedback between currents and morphological changes to the bathymetry, the extent of seabed modifications can be of orders of 100's of metres away from the reef structures (e.g. inshore salient development, small changes in overall seabed levels (Scarfe, 2008); these effects are explored in a later Section.

One method of visualizing Interactions between reef systems is to plot the stream-plots, which show the paths of currents (Figures 3.42 and 3.43). These plots show the interactions and connectivity of multiple reef systems, with reef units further offshore 'feeding' into inner reef units (as described above). The net effect of this is feedback that leads to 2 reef systems being more effective at retaining beach sand and developing salients than the total of simply multiplying the response of an individual reef system by 2.

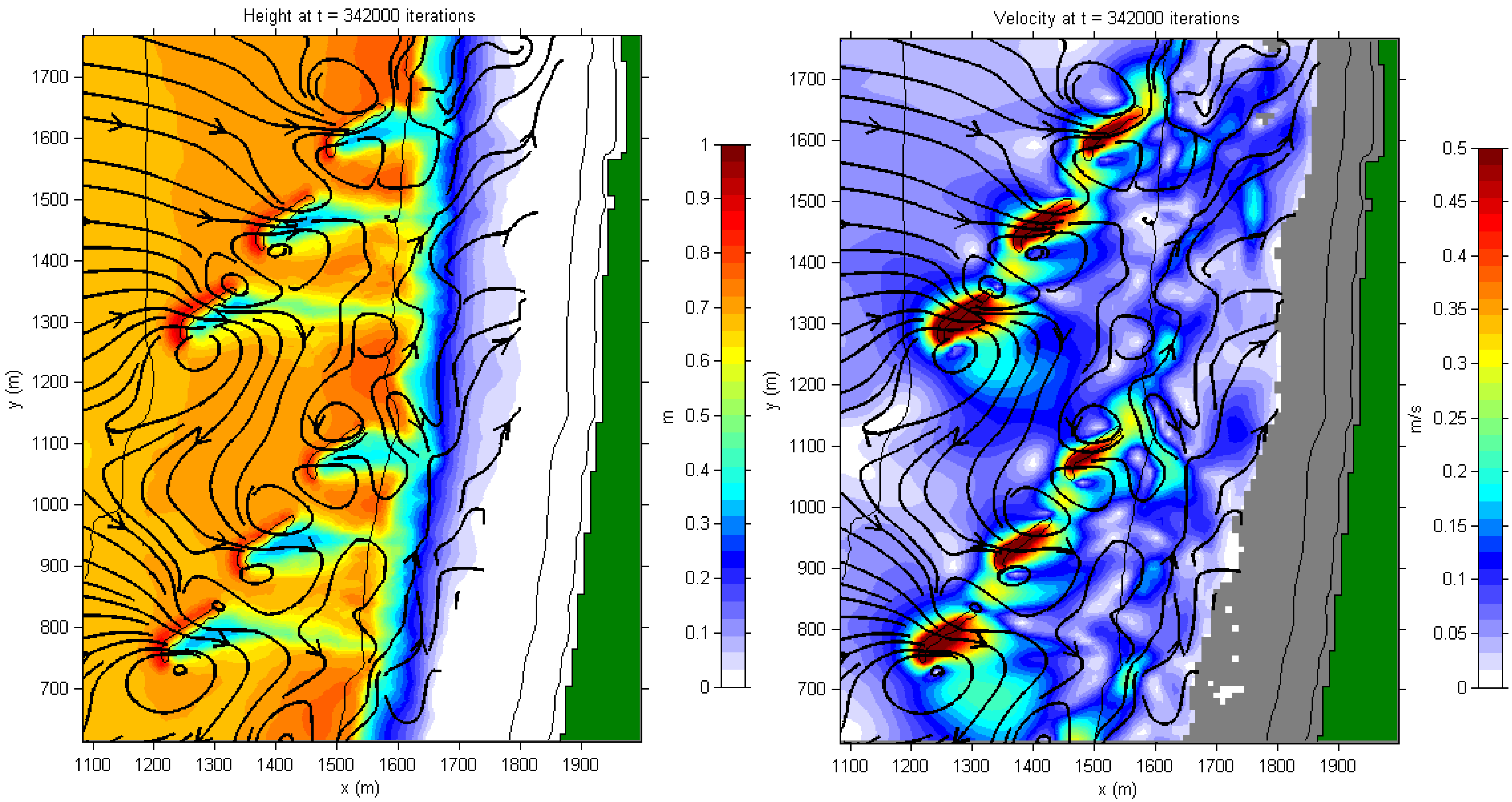


Figure 3.42. Instantaneous wave height and velocity with velocity streamlines highlighting the connectivity of multiple reef systems.

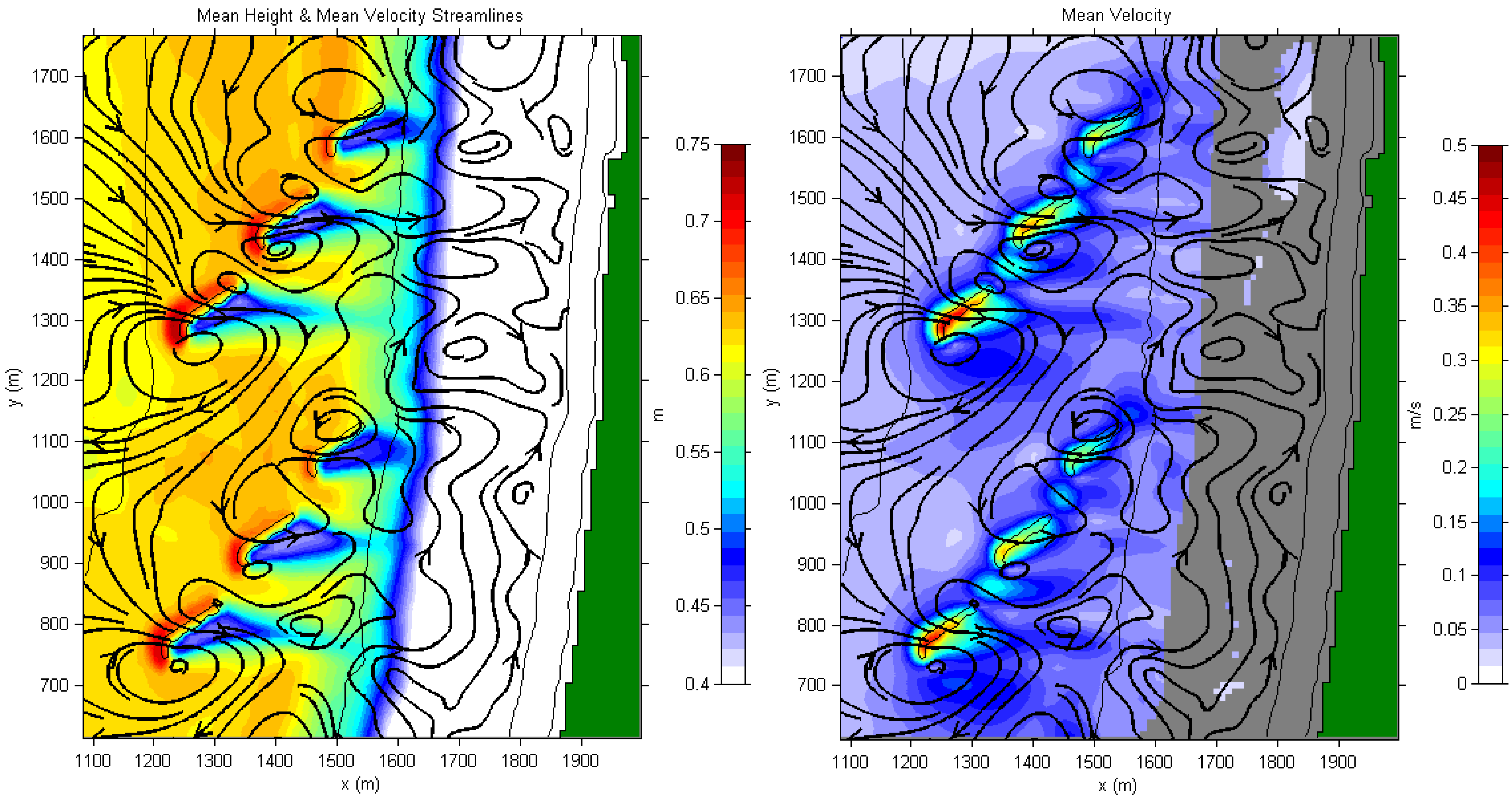


Figure 3.43. Mean wave height and mean velocity with mean velocity streamlines highlighting the connectivity between multiple reef systems.

Sediment Response

To investigate the morphological response to the reef system, the 'sediment transport' module of 2DBEACH was applied, along with 'rocks' files to protect non-erodible parts of the bathymetry (i.e. the reefs, the rocky headlands, etc). Variable boundary conditions were applied with wave heights of 0.3-2.04 m, peak periods of 4.5-12.2 sec, and a directional spread of 28°, with a sindusiodal tidal range of 0.3 to 2.7 m.

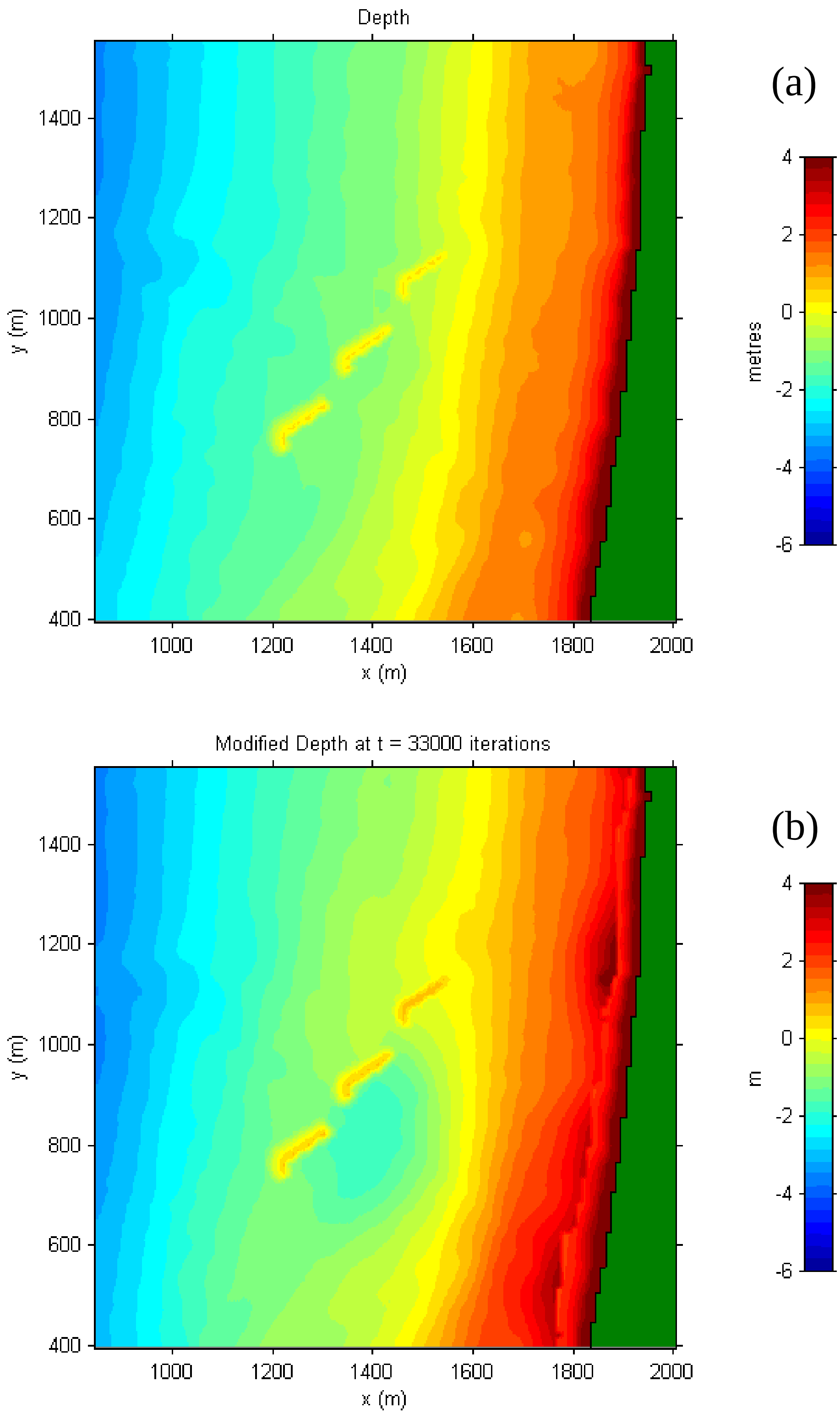
The results shown in Figure 3.44 are extracted at different iterations to demonstrate how the salient response first builds and then varies about an equilibrium (i.e. reaches a dynamic equilibrium). The most significant features are the double-horned salient response (which is very similar to the earlier simulations using NGENIUS), which results in the offshore migration of the 0.0 m contour of ~100 m in the lee of the inner reef unit and to the north (down page) of the outer reef unit, with the latter as would be expected due to the predominantly southerly sediment movement. On the beach and intertidal zone it can be seen that the +2.0 m contour also migrations offshore some 100 m and that significant sand builds up at the high tide mark[7]. Also notable is the deepening of the seabed in the lee of the offshore reef unit. Similar to analysis of the Mount Reef bathymetry surveys, this feature may be up to 1 m deeper than the pre-reef situation during some wave conditions, and then reduce during other periods (e.g. Scarfe, 2008; ASR unpub. Data). The modelling indicates that during higher wave events, the depression behind the reef increases as does the accumulation of sand on the beach (Figure 3.44b). During following lower wave periods, the depression becomes shallower (Figure 3.44c). Time series monitoring will provide further quantitative understanding of the magnitude and processes of beach and seabed effects.

Thus, the reef system is influencing a large area of the seabed and beach, similar to other coastal protection structures. At the Gold Coast, the Narrowneck reef creates an asymmetric salient that stretches several kilometres to the south due to the predominantly northern sediment transport (it was originally estimated through numerical modelling that, although undetectable given the average daily change in the high tide mark of 18 m, the salient impacts would reach as far south as Burleigh Heads (some 12 km away). Similar modifications to the seabed have been observed at the Mount, where depending on the recent wave events, the seabed can be slightly elevated on either the northwestern or southeastern side of the reef (e.g. Scarfe, 2008; ASR unpub. Data). Groynes also have wide-ranging effects, with accretion and erosion patterns extending considerable distances (in some cases kilometres) up and down the beach (e.g. Basco and Pope, 2004).

1, 10 and 100 year storm events were modelled with 'sediment transport' incorporated. However, due to the low beach gradient and wide surf zone, these model runs did not remain stable. Thus, we refer to the hydrodynamics

[7] Note, renourishment of this area in the lee of the reef system is part of the strategy, and that this has not been 'added' to the system for this modeling. As a result, the salient evolution above the high tide mark is not incorporated into the simulation (since the hydrodynamics are only operating up to around 2.7 m) and the upper beach response is considered conservative.

to consider the response to sediment transport, since the wave-driven currents are by far the dominant process determining sediment transport during storm activity. These results (presented above) indicate that the current patterns are similar to those seen during higher wave events during the 'normal' period simulations, with shoreward and northward sediment transport dominating, which is supported by the increased beach response seen during higher wave events in this model simulation (i.e. Figure 3.44). This occurs because of the wide surf zone, i.e. wave heights have diminished due to depth limited breaking by the time they are at the depth of the reefs system. It is important to note that these morphological response model simulations are undertaken without the input of 20,000-25,000 m3 of sand in the lee of the structures. Construction includes the addition of beach material to ensure that salient formation does not have negative impacts on other parts of the beach, which is nowadays best practise engineering.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metres
Modified Depth at t = 33000 iterations
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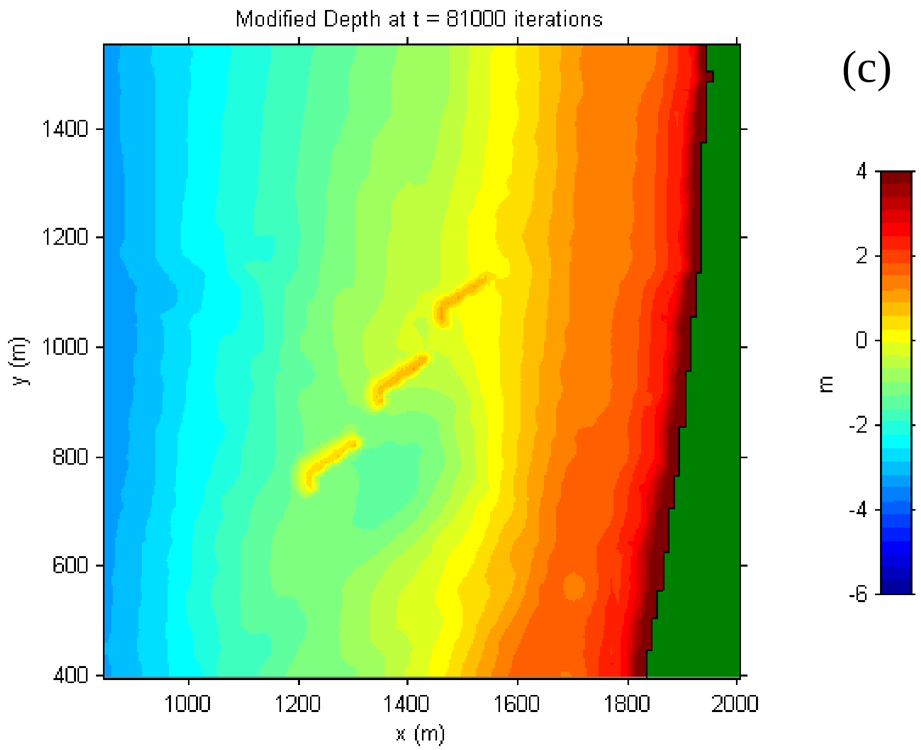
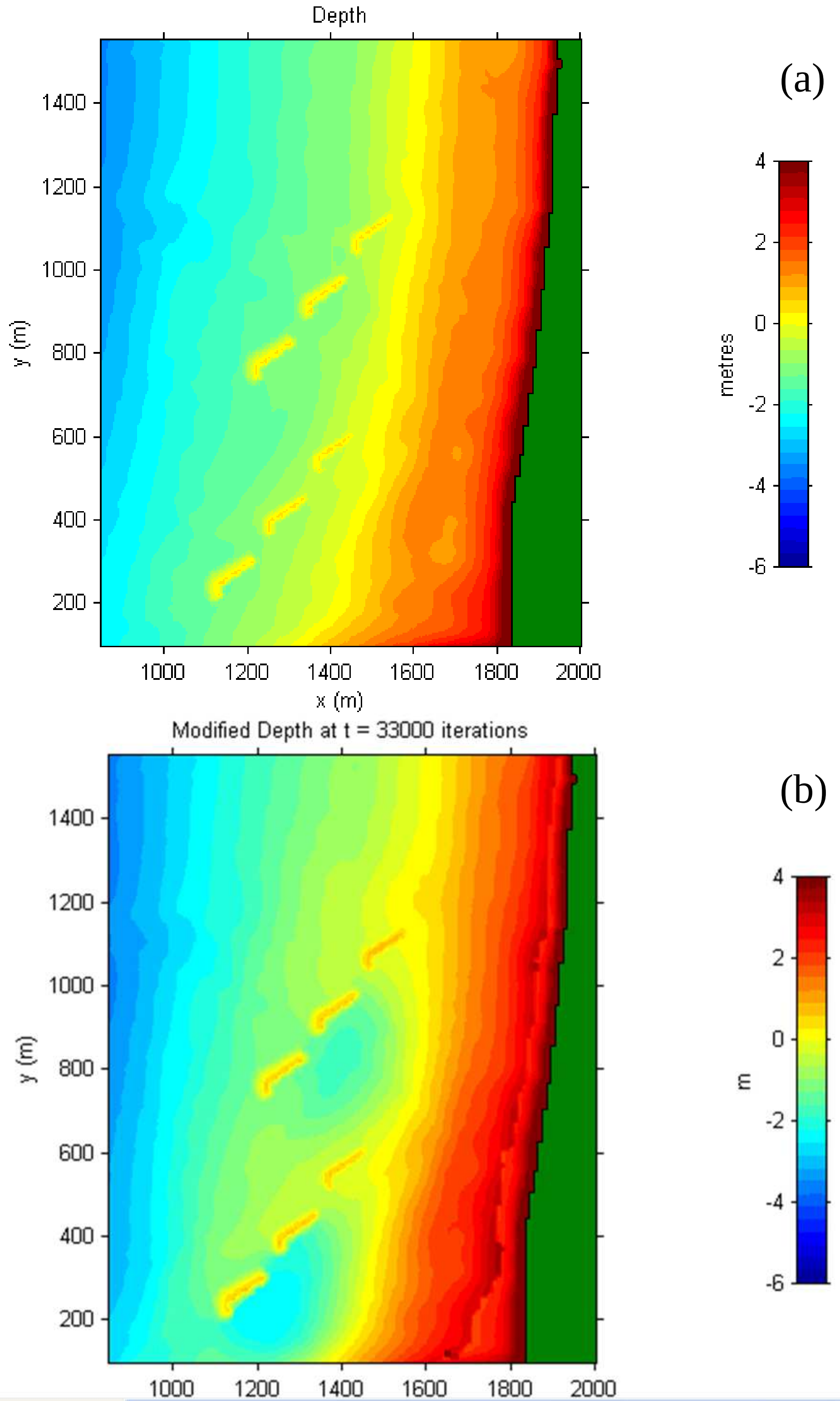


Figure 3.44. "Initial" bathymetry near the single 3-reef system (a), modified bathymetry at different stages of beach morphology (b-c) under 'normal' conditions.

Following the morphological modeling of a single reef system, multiple systems were modeled to determine connectivity between the structures in terms of sediment transport and beach response.

Similar results as those found with a single reef, i.e. offshore migration of depth contours (sediment build up) to form a double-horned salient, increased beach response during larger wave events, the depression in the lee of the outer reef unit and dynamic fluctuations around the equilibrium (Figure 3.45). However, one notable difference is the increase in beach response of the southerly (up page) of the reef systems. This indicates feedback between the systems and that 2 reef systems are more effective than 1. This is again evident when 3 reef systems are considered (Figure 3.46) – 1, 2 and 3 reefs system beach responses are compared side-by-side in Figure 3.47.



Depth
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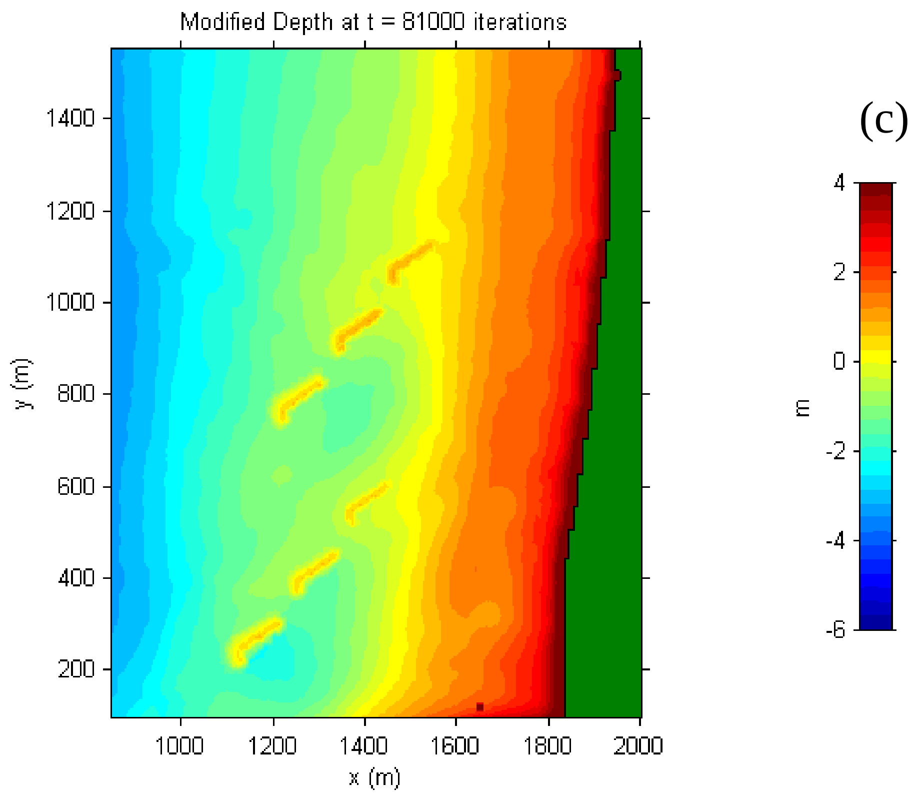
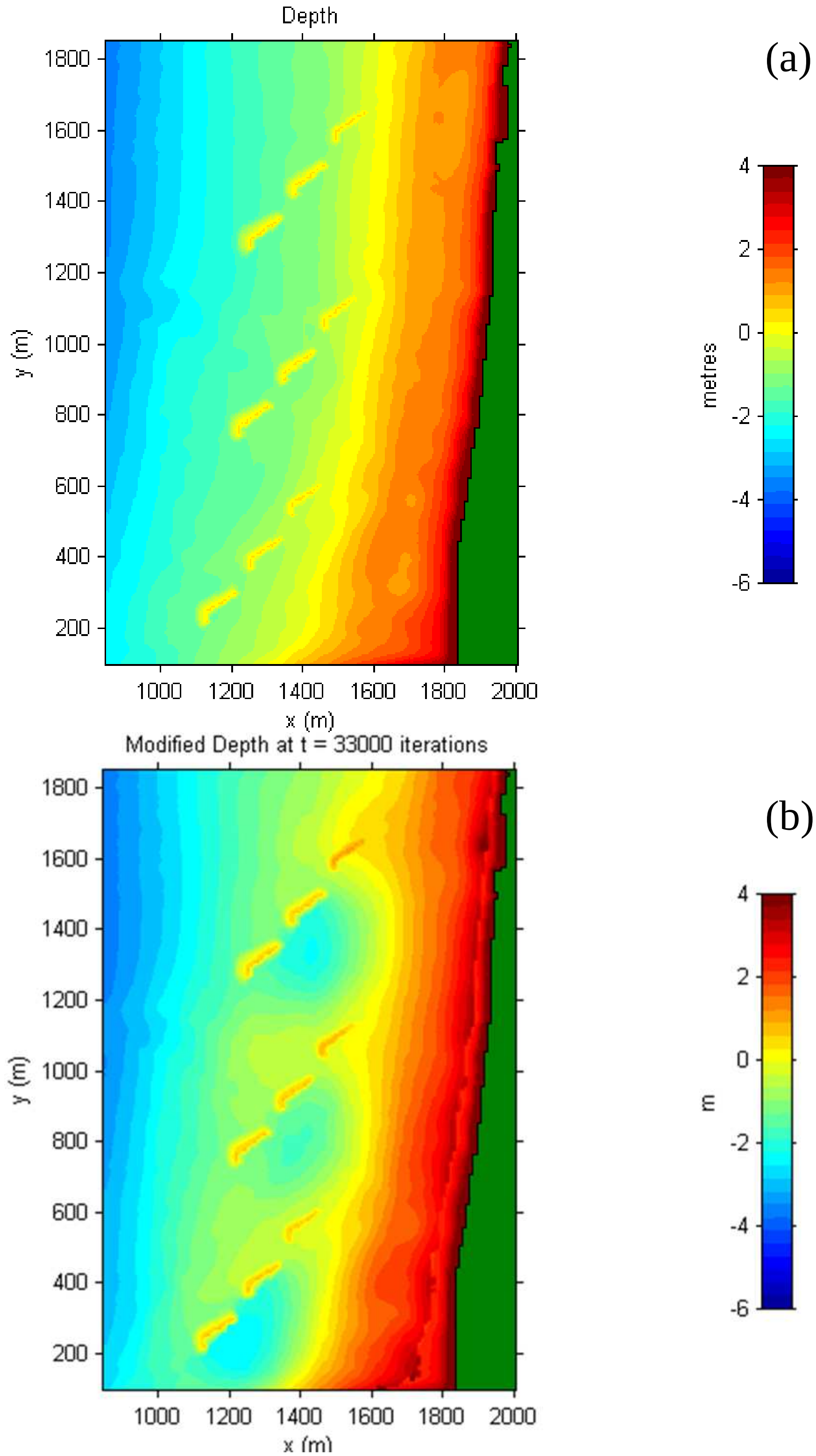


Figure 3.45. “Initial” bathymetry near the 2x 3reef systems (a), modified bathymetry at different stages of beach morphology (b-c).



Depth
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Modified Depth at t = 33000 iterations
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y (m)
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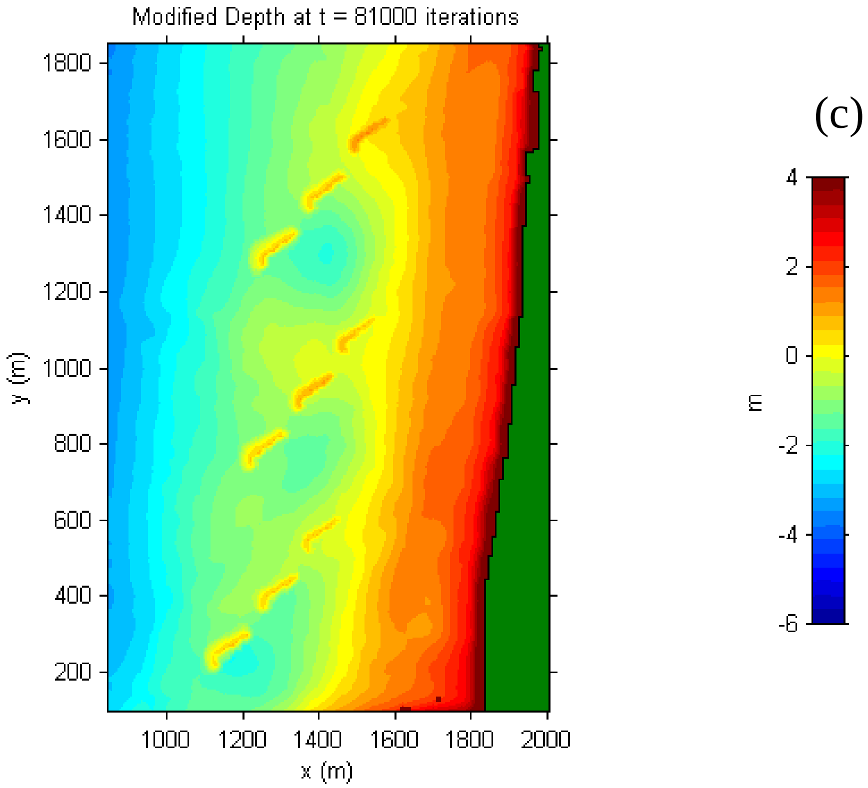

Figure 3.46. "Initial" bathymetry near the single 3reef system (a), modified bathymetry at different stages of beach morphology (b-d).

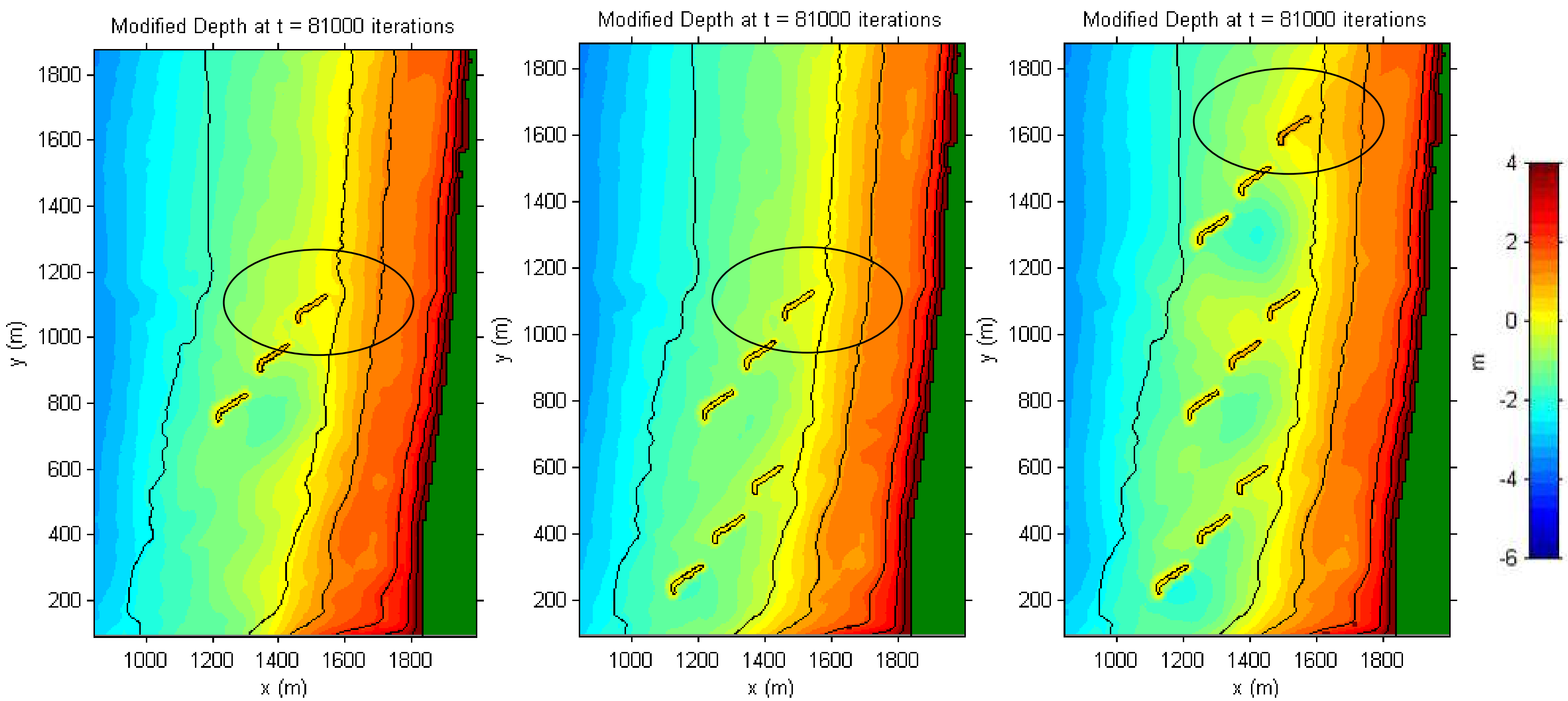

Figure 3.47. Interactions between multiple reef systems seems to feed sediment downstream leading to a greater salient response.

3.3.7 Wider Bay Impacts

In order to consider impacts on the wider bay (i.e. beyond Orewa Beach), hydrodynamic simulations of normal, 1, 10 and 100 years were undertaken using the boussinesq model 3DD (Figure 3.48). Simulations were undertaken with and without reefs, and difference plots of wave heights, and more importantly, hydrodynamics were generated.

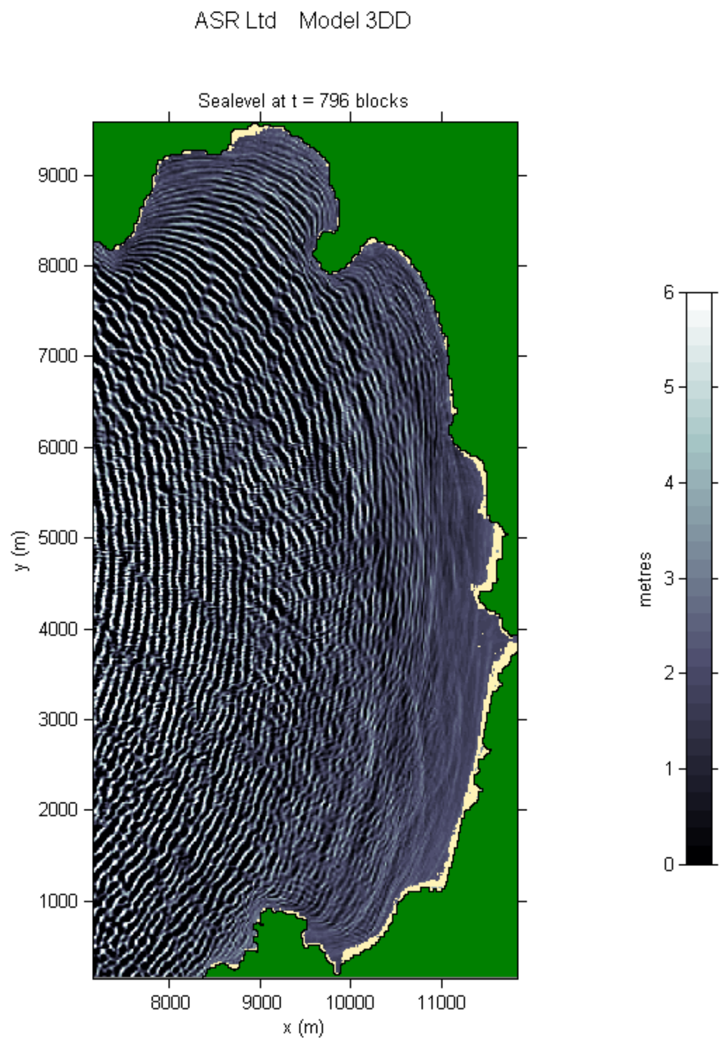

Figure 3.48. 3DD boussinesq simulation of a 100 year wave event.

As would be expected, the largest differences were found during a 100 year RI event. However, the results show that changes to wave and current patterns are restricted to the Orewa Beach area, and other than in the immediate vicinity of the reefs, is of low magnitude (Figures 3.49 and 3.50).

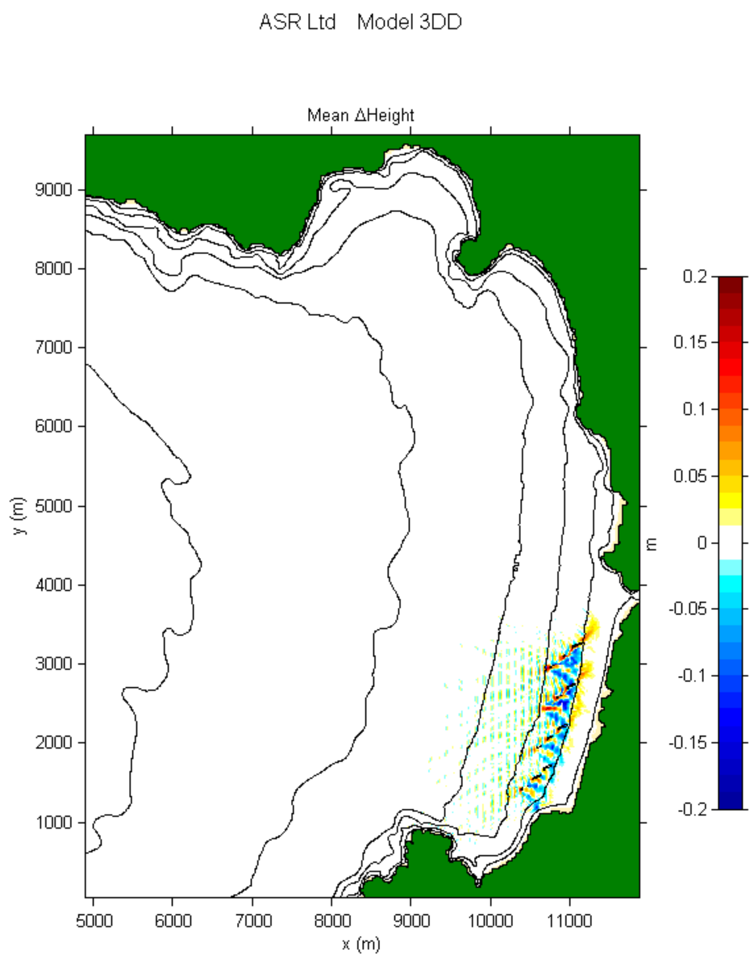

Figure 3.49. Difference plot of wave heights for 3DD boussinesq simulation of a 100 year wave event (with and without 4 reef systems is compared). Note, the light green marks (i.e. tiny magnitude) are an artifact of reflection interactions with wave crests during the averaging process.

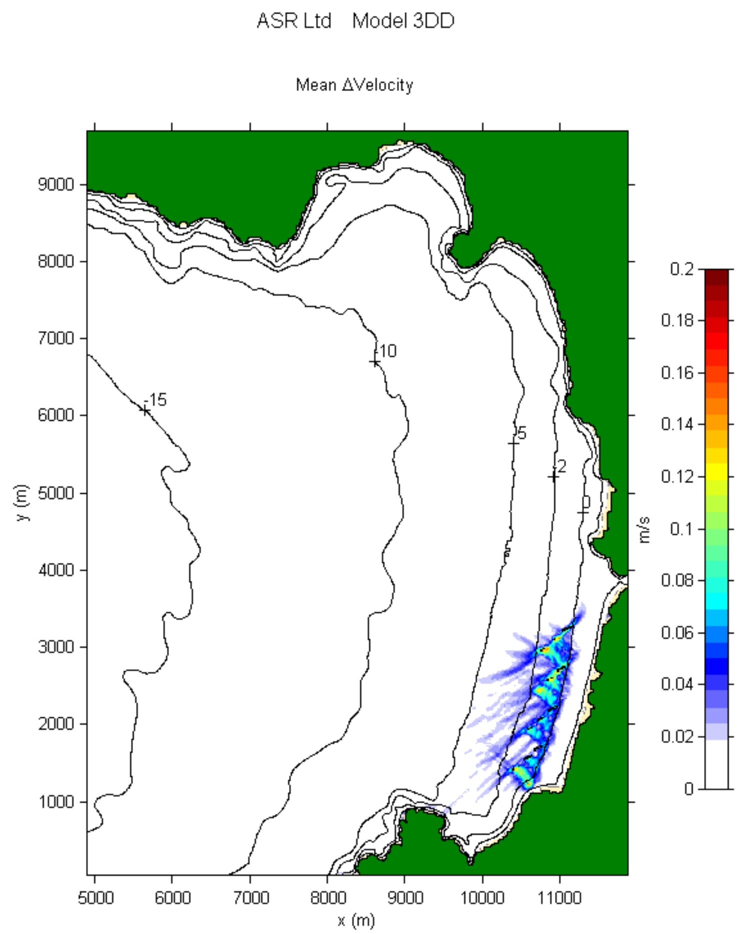

Figure 3.50. Difference plot of current for 3DD boussinesq simulation of a 100 year wave event (with and without 4 reef systems is compared).

3.3.8 Design Recommendation

The calibrated numerical modelling study presented here has shown that a shore protection system using submerged offshore structures will be effective at Orewa Beach. The overall design of such a system would call for a series of reef units to be placed offshore in an orientation that will reduce incident wave energy as well as disrupt erosive alongshore currents.

The design presented in Figure 3.44 is comprised of three separate reef units arranged along a line extending obliquely towards the north from the shoreline Orewa Beach. Each reef segment has a crest height of +0.5 m relative to Chart Datum, approximately the level of Mean Low Water Spring tide. The reefs are positioned with the innermost segment extending seaward from the from water line at lowest astronomical tide (LAT). The next segment seaward extends from approximately 0.75 m depth to 1.25 m depth with the segment furthest offshore extending to approximately 1.5 m depth.

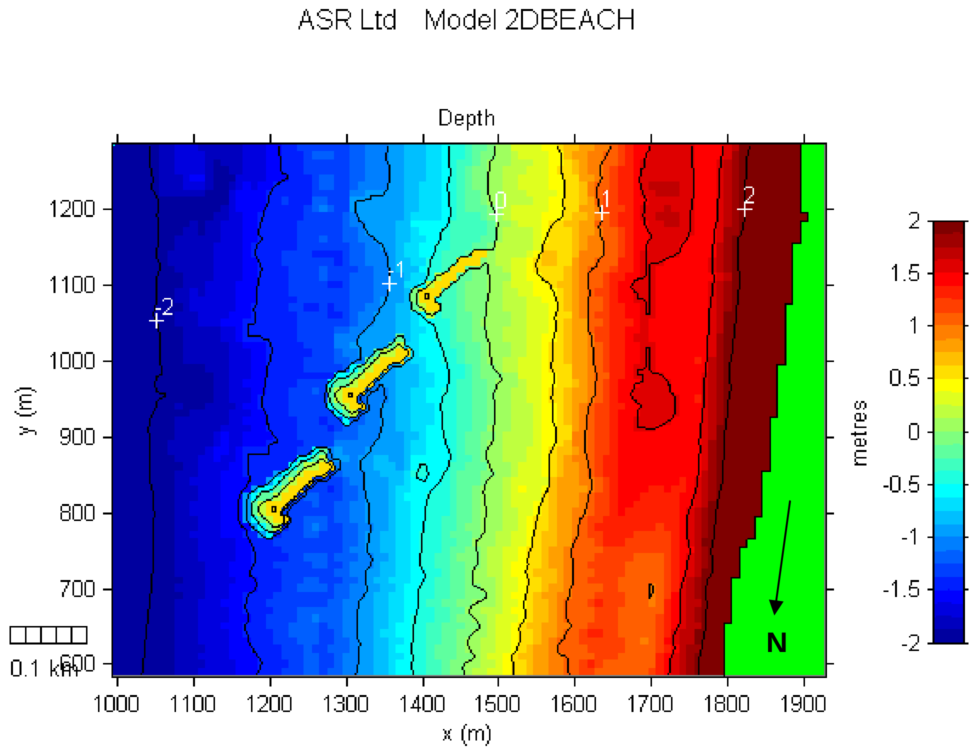

Figure 3.44. Configuration of the preferred design for shore protection reef system at Orewa Beach.

Schematic presentation of the proposed reef structures is shown in Figure 3.45 (A – D). Panels A and B depict the first of four proposed reef systems located directly in front of the Surf Lifesaving Club. Because each reef system is designed to protect and enhance ~600 m of beach front, four reef systems will be required to protect and enhance the full 2.4 km of Orewa Beach. The reef systems should be built in a phased approach over a period of several years, with monitoring results being used to assess the efficacy and determine following reef system construction. A dimensioned plan of the first reef system is provided in Appendix 3, along with geographic coordinates pertaining to the centre of each of the 12 reef units.

It is important to realize that the modelling results have indicated that there is some feedback between reef systems, i.e. 2 reef systems are more effective at retaining sand on the beach that 1 and 3 reef systems are more effective at retaining sand on the beach than 2. This is because even though the influence of the structures is mostly localized in terms of hydrodynamic, the net reduction and redirection of energy in the beach system is reduced with each additional reef system. A staged approach is planned and recommended, with the first system of reefs located a northerly section of the beach in that is considered the highest priority for beach protection by the RDC.

A

B

C

Figure 3.45. Schematic presentation of the proposed reef structures at just below MSL (i.e. 1.0 m tide height). Panel A depicts the first of four proposed reef systems located in the northerly section of the beach. Panels C and D show all 4 reef systems. The beach response has been extracted from the model output - – the transition from grey to sand in panel A is the approximate high tide mark.

An important component of the beach protection strategy is dune management through the establishment of a vegetated foredune system to reduce wind-blown sand transport and provide a robust buffer zone during storm events. The proposed solution to enhance the beach amenity at Orewa (namely, provide a wider dry beach) fits very well with the proposed esplanade and beach enhancement plan. While it is noted that the esplanade enhancement strategy is still in the proposal stage, it is important to note that the types of beach planting proposed in the strategy are a crucial component of this proposal – with a wider dry beach it will be imperative to reduce aeolian sand transport (i.e. sand movement (loss) due to wind) with the planting of appropriate native beach vegetation. Indeed, during the fieldwork undertaken during stormy conditions in February, the large volumes of sand being blown landward in the area without dune vegetation adjacent (just south) of the SLSC was very evident (pers. obs.) Dunes grow as plants trap wind driven sand, keeping sand in the beach system. The foredune acts as a “buffer zone” to the erosive storm waves, not by countering wave erosion, rather it allows sand to be moved offshore during storm events providing protection by dissipating wave energy in the surf zone (Dahm *et al*., 2005). The dune then re-builds and repairs during accretionary periods of low or long-period wave action. The self-repairing capacity of natural dune systems is therefore a very important component for the mitigation of coastal erosion. Such measures have proven successful in the southern area of the beach (Figure 3.46) and in many parts of North Island New Zealand (Dahm *et al*., 2005), and it is strongly recommended that they are applied to the upper beach areas created by the 'managed advance' that will be achieved by the propose reef systems. It is envisaged that plantings and protection of these new areas will be undertaken by project partners RDC.

Figure 3.46. Dune stabilisation at the southern end of Orewa Beach.

When sealevel rise is considered, while the modular nature of the structures allow for height increases, the best measure for addressing associated impacts is the formation of a healthy foredune system. With reference to restoring dunes through planting of native sand-binding species, De Lange and Jenks (2007) state that:

> "With respect to coastal hazards, the restored dunes provide improved protection from tsunami, storm surge inundation, and coastal erosion. During 5-10 years of dune restoration over the past decade, all of the restored sites show a significant trend of accretion, despite climatic conditions favouring erosion. The measured rates of accretion are an order of magnitude larger than worst sea level rise predicted by the fourth assessment report of the IPPC."

As the results of the past renourishment scheme at Orewa have indicated, sand placed on the beach does not remain for long, and so, planting of the main beach areas where there is presently no dry beach will not be successful. However, with the 'managed advance' of the beach response due to the presence of the reef systems, dune restoration becomes a feasibility coastal hazard mitigation measure.

3.4 Summary

Four numerical models from the 3DD Suite of Coupled Models were used for reef design and assessment of the functional performance (primarily sand retention/coastal protection) – 3DD, WBEND, NGENIUS and 2DBEACH. Several factors were assessed to determine the best location for the placement of a multi-purpose reef including the distance from the beach that a reef should be placed, effective dissipation of waves, wave rotation/attenuation (i.e. modifying the waves without breaking them), and shoreline response. The combination of numerical modelling results provides very good evidence that a system of offshore multi-purpose reefs can be used to retain sand at Orewa Beach in the form of a salient, i.e. achieve 'managed advance' of the beach.

From the modelling investigation, the recommended reef design calls for a system of three reefs arranged 'en echelon' extending obliquely seaward from the 0 m (CD) depth contour. The depth of the reef will be between 0.3 and 1.5 m (below chart datum (CD)). The crest height has been set at +0.5 m (to CD), which is equal to mean low water spring (MLWS) and 1.2 m below mean sea level (MSL). The reef system would have a volume of 15,000-17,000 m^3 above the seabed.

Because each reef system is designed to protect and enhance ~600 m of beach front, it is likely that four reef systems will be required to protect and enhance the full 2.4 km of Orewa Beach. This scheme is designed to be a whole beach solution. The reef systems should be built in a phased approach in over a period of several years. The first reef system is proposed along a northerly section of the beach in that is considered the highest priority for beach protection by the RDC. Monitoring will be utilized to identify the efficacy of the first reef system and construction of following reefs. An important component of the beach protection strategy is dune management through the establishment of a vegetated foredune system to reduce wind-blown sand transport and provide a robust buffer zone during storm events. It is strongly recommended these techniques are applied to the upper beach areas created by the 'managed advance' that will be achieved by the propose reef systems.

CHAPTER 4 – CONSTRUCTION METHODS AND MATERIALS

4.1 Introduction

This section describes the construction materials and methodology (and alternatives) considered for the Orewa Multipurpose Reefs construction based on the recommendations from the preceding chapters. In is noted that a great deal of information on construction materials, methodologies, case-studies, etc, is provided in the feasibility study Appendices.

4.2 Materials

While shore-parallel submerged breakwaters have been used for coastal protection for many decades, multi-purpose reefs are a relatively new and innovative form of coastal structure with specifications over and above those of most marine construction. Such reefs must fulfil a large number of requirements that are standard to all coastal structures such as:

- Durability
- Environmental impacts
- Stability
- Economic feasibility
- Workability of construction methods
- Maintenance

In addition, two further unique specifications are required to ensure the designed wave transformations occur for coastal protection (e.g. rotation of wave crests), and especially if surfing amenity is to be incorporated into the structure, as in the present case, these are accuracy and safety.

Accuracy is required to ensure the designed wave transformations occur for coastal protection and to ensure a high quality surfing wave during the applicable conditions. The reef profile must be constructed to achieve fine tolerances and be free from large steps and irregularities to maximize the functional performance of the reef (e.g. Button, 1991). In terms of safety, the exposed surface of the reef must be as soft as possible to minimize the injury risk to surfers using the reef. The reef surface should also be free from sharp or rough edges and small holes capable of trapping swimmers or surfers, features that are common on natural reefs, rocky headlands or rubble mound/armour layer structures.

To achieve the requirements outlined above, geotextiles are considered as the best material. Geotextiles are a family of synthetic materials including polyester and polypropylene that are formed into flexible, permeable and durable sheet fabrics resistant to tension and tear. Large containers are prefabricated and then filled with sand (sand-filled containers (SFC's)) to form the reef structure.

For structures that do not require a high degree of accuracy in final constructed shape, i.e. breakwater segments, other construction materials can be used, such as rubble mound with a stone armour layer. Some preliminary calculations and recommendations for this type of structure are presented below and compared to a similar analysis for SFC's.

A typical reef section that incorporates surfing amenity is shown in Figure 4.1.

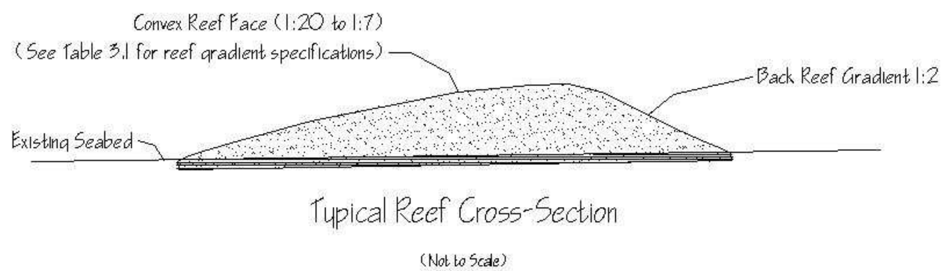

Figure 4.1. A typical reef cross-section incorporating a convex reef profile.

4.3 Construction Methodology

This section describes the suggested methodology for installing and filling large, sand filled, geotextile containers (SFC's). The unique requirements in terms of precision and construction tolerances necessary for creating high quality surfing waves necessitates an innovative approach to reef construction. Previous construction of submerged geotextile reefs (i.e. the Narrowneck Reef at the Gold Coast in Australia) was accomplished by dropping the pre-filled SFC's from a split-hull dredge or barge (Figure 4.2). However, this method of construction has several limitations. Firstly, the requirement for a plant that can handle very large and heavy (300-400 tonne) containers is, in many areas, not feasible due to the large costs associated with mobilizing large vessels or the proximity to large ports – this is the case with the present location. Secondly, the draft on vessels of this size is usually too big to be used in water depths shallower than 3-4 m. Thirdly, in order to incorporate amenity such as surfing, the tolerances of the reef have to be small, i.e. large humps and hollows in the reef deform the wave face and greatly reduce the quality of the waves. It is difficult to place large containers in precise positions by dropping them from a dredge or barge. Therefore a construction method is required that minimizes costs, while ensuring accurate reproduction of the structure.

Figure 4.2. SFC's being filled and placed during construction of the Gold Coast reef.

4.4 Bag Deployment

To address the construction issues described above, ASR developed the R.A.D. (Rapid Accurate Deployment) method for submerged reef construction. This method was demonstrated during the construction of the Mount Maunganui surfing reef, with successful section deployment in less than 4 hours. The RAD method aims to:

- Eliminate gaps between bags by accurately placing them relative to each other;
- Minimize cost and construction downtime by undertaking much of the construction on land, and;
- Provide a smooth surface to the reef face to optimize surfing wave quality.

The RAD method can be summarized as follows (Figure 4.3):

- A "web" of high tensile strapping with a 12 tonne breaking strain is constructed;
- The webbing is stretched out on land and the reef geobags are tied onto the web to create the reef section;
- Anchors are placed on the seabed at accurately surveyed pre-determined positions;
- The reef is floated out to its position, pulled down onto the seabed at each anchor and secured by shackles, and;
- The bags are pumped full of sand through inlet nozzles in the geobags.

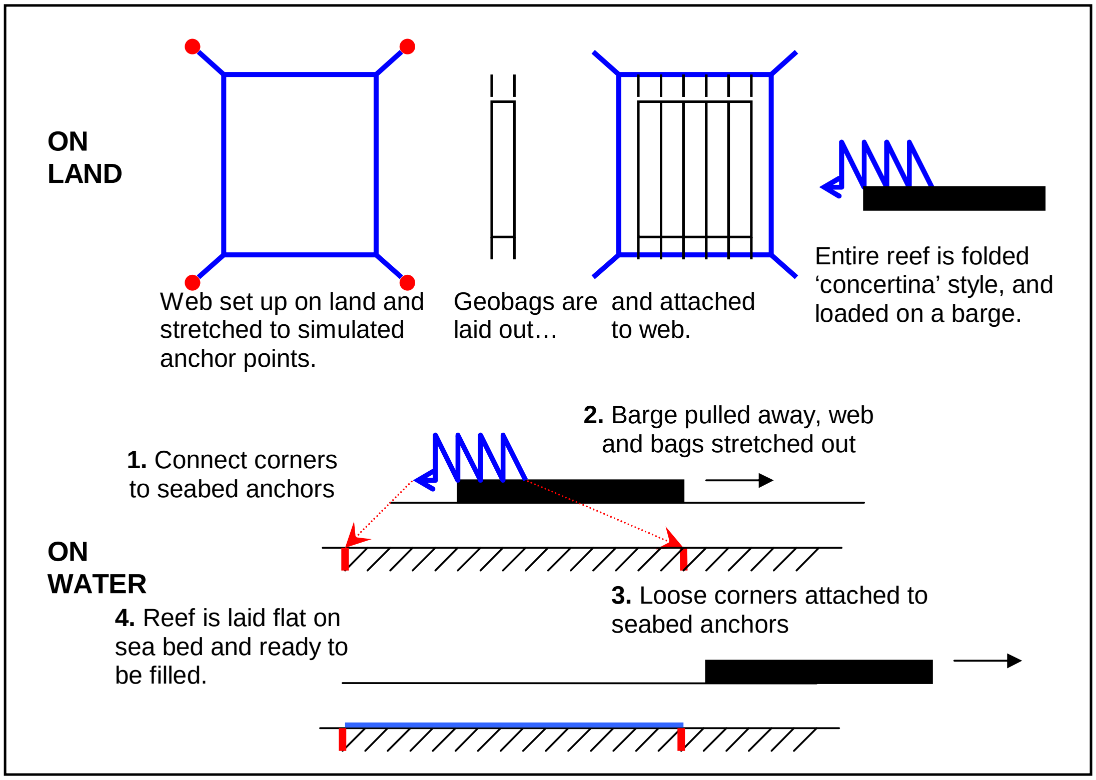

Figure 4.3. A simplified schematic of the RAD method for deploying geobags.

In the case of the Mount Maunganui Artificial Reef, the reef was built in two sections, with each section deployed independently. Photos from of the construction of the Mount Reef are shown in Figure 4.5. Once in position, the SFC's are filled *in situ*, either from a barge or from the beach. These options are discussed in the following section. Recently, a combigrid foundation material has been adopted for reef construction. This material incorporates a tensile grid and a non-woven geo-fabric. Details and specifications are included in Appendix 6.

4.5 Filling of Geobags

Depending on the situation and the relative costs for each option, the reef bag units can be filled either by a sea based or a land based pumping system. A conceptual schematic of each pumping system is show in Figure 4.4. In both cases, a slurry of sand and water is pushed by a large pump, through a pipeline to a nozzle that is inserted into the filling tube of a geobag. A sea-based system was used during construction of the Mount Maunganui Reef (Figures 4.5 through 4.7) while a land based system is currently being used to build the Opunake reef (Figure 4.8). The beach/land-based system is the preferred methodology for Orewa Beach, with sand for both the reef system and salient stock-piled on the beach during the construction process.

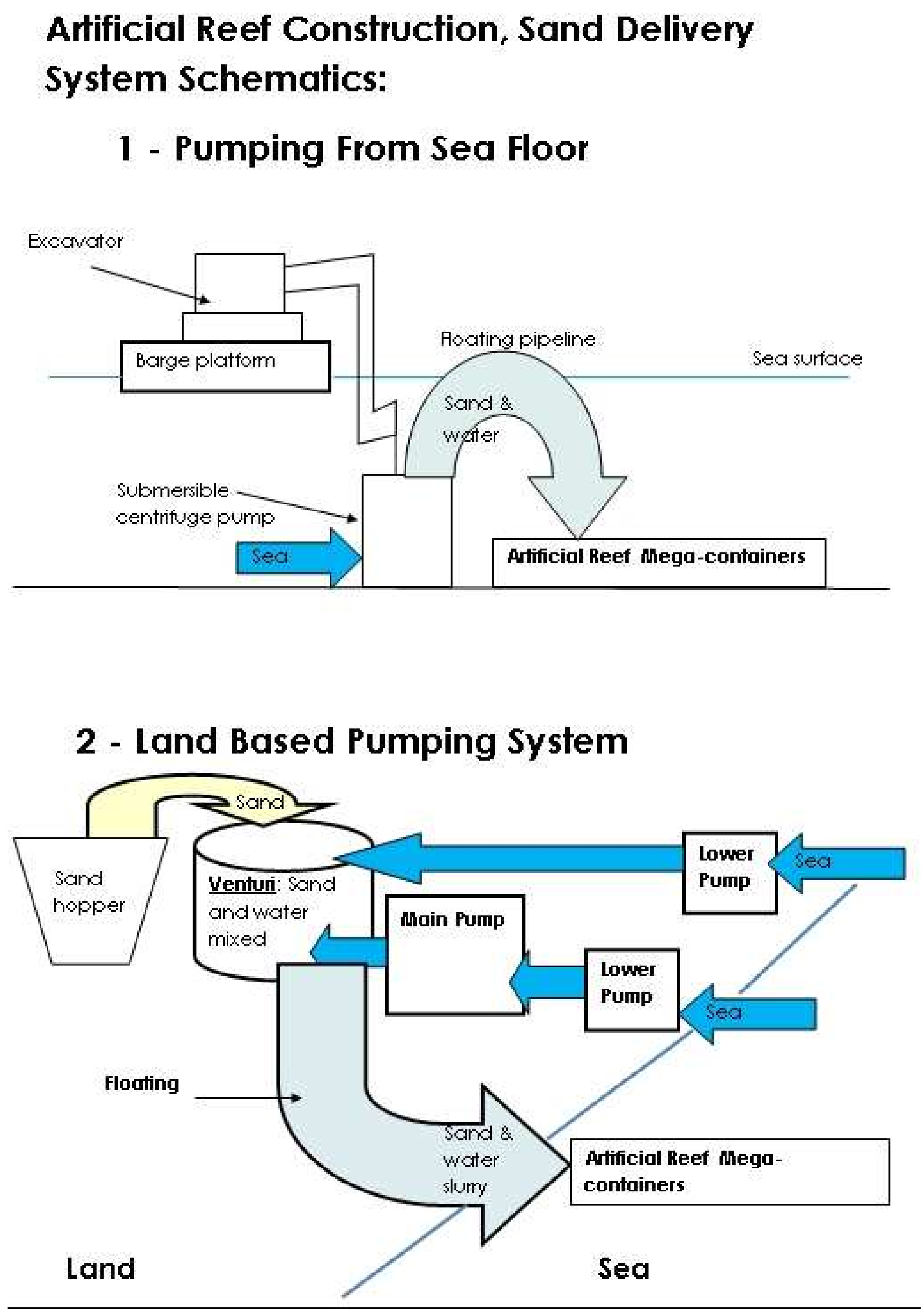

Figure 4.4. Overview schematic for two methods of filling large Sand Filled Containers (SFC's).

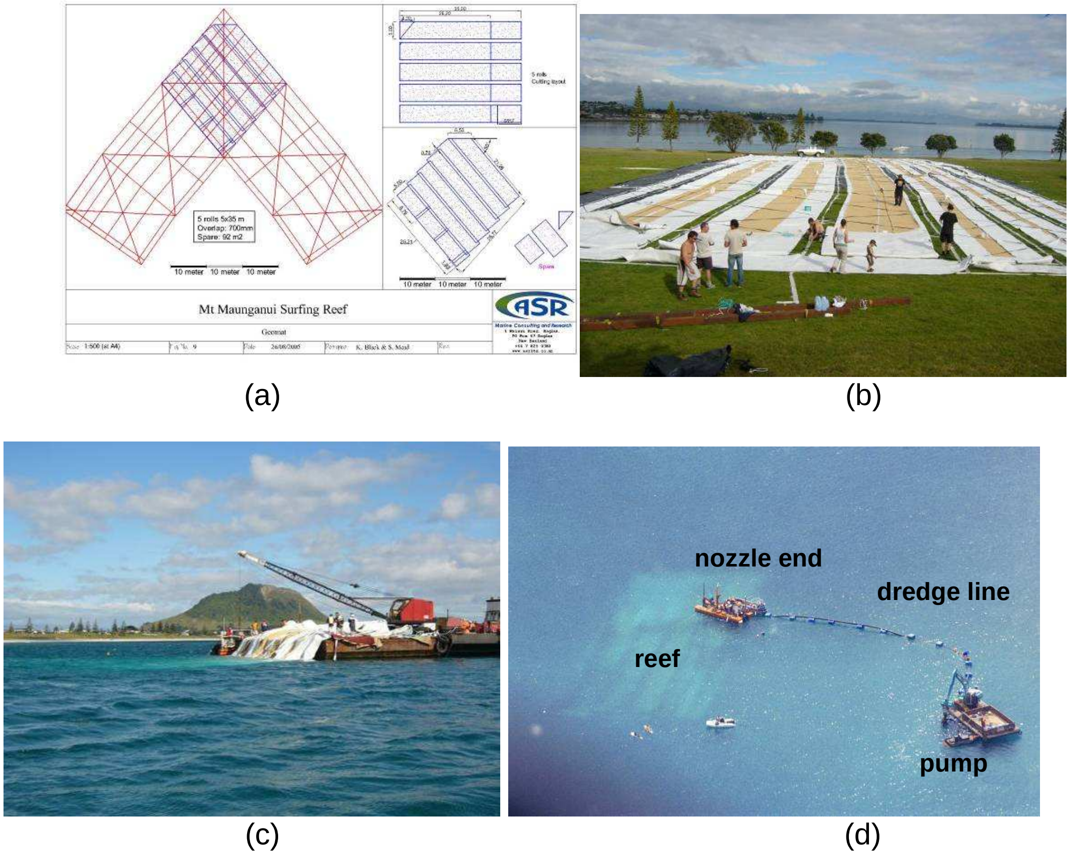

Figure 4.5. (a) The web design for the Mount reef. (b) Attaching SFC's to the web. (c) Winching the reef to the seabed. (d) Filling the SFC's.

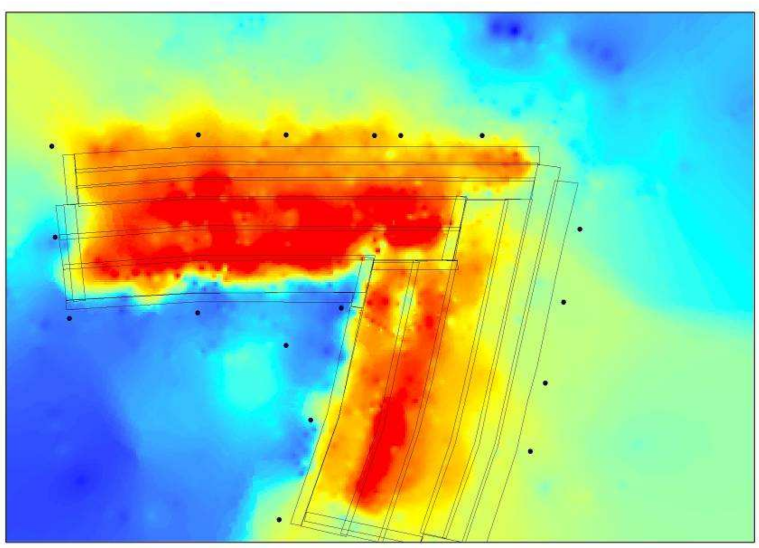

Figure 4.6. Bathymetry survey of the Mount Reef. Only one half of the reef was complete at the time the survey was conducted.

Figure 4.7. Images of waves breaking on the Mount Reef in New Zealand.

Common factors between the two operations are that both systems are pumping a sand slurry and both require extensive use of divers and dive contractors. With respect to sand slurry pumping, the contractor must be aware of the consequences of blocking the delivery pipeline. A blockage can result if the flow velocity in the dredge line falls below a threshold allowing sand to fall out of suspension and cause a blockage. This can be caused by insufficient pump power, extremely long pumping distances, very coarse grain sizes, improperly shutting down the pumping system or a combination of factors. If the pump is not powerful enough, the flow velocity inside the pipeline may become too slow near the outlet initiating a blockage. Pumping efficiency is also reduced over very long pipeline runs. Grain size is also a factor in that larger grains take more pumping power to move through the pipe. When shutting down the pumping system it is important to stop sand delivery and allow the system to pump clean water for a long enough time to clear sand from the line. If the slurry feed is stopped abruptly, the sand in suspension in the pipeline will settle out and cause a blockage when the system is restarted. Whatever system is to be used, the goal should be for a delivery of a minimum of 120 to 200 cubic meters per hour of solids. By assuming that the solids should make up 10 to 30% of the slurry volume, the total flow volume necessary can be determined.

In addition to the pumping issues mentioned above, both systems require extensive use of divers and dive contractors. Divers are required to set the anchors, assist with reef section deployment, move the dredge line and to insert and secure the filling nozzle into the bag. In practice this can be a very time consuming and delicate task which requires two divers to work together. In situations with low visibility and strong currents or surges, it is even more difficult. Quick couplings on container inlets and diver 'rehearsal' of the nozzle placement on land to minimise potential mistakes are some of the ways that diver time can be minimized and construction can be undertaken efficiently as possible. A full construction methodology and diver protocol will be developed with the construction contractors for each different project – plant, depths, weather conditions, etc, differ in each case and need to be understood and addressed in turn.

4.5.1 Sea-Based Filling

Figure 4.8 (a-d) shows an over view of the sea-based filling operation at Mount Maunganui. A dredge pump is located at the end of a hydraulically controlled digger arm (the digger also acts as the hydraulic power pack for the pump) and is lowered to the sea bed. Water jets on the pump agitate the sand beneath the pump, causing it to go into suspension where it can be sucked into the pipeline and pumped to the reef. Issues associated with the sea based operation include rough seas and barge stability. For example, it is advised to use a 4-point mooring system for the barge for stability and precise manoeuvring. While this system is available for Orewa, it is not the recommended methodology.

4.5.2 Land-Based Filling

The land-based filling option, proposed for Orewa, does not have the problems associated with working on the water, but does present other challenges that must be addressed. Primarily the need for large amounts of water to be mixed with the sand to form a slurry that can be pushed down the dredge line. Figure 4.8a shows the water intake lines that are connected to 6-inch diesel powered water pumps. The water is drawn up pipes running along the boat ramp (Figure 4.8b). The sand and water are mixed in the sump shown in Figure 4.8c. The sand is delivered by means of a variable speed conveyor belt visible in Figure 4.8c. The mixture is agitated by the thrust of the incoming water supply. The slurry pump is sitting inside the sump and pushes the sand/water slurry out through the pipeline shown in Figure 4.8d. The pumping distance in this example is over 350 m – pump and pipeline specifications must be met to pump specific sand sizes specified distances at that required volumes for efficient construction.

(a) (b)

(c) (d)

Figure 4.8. The land-based pumping system at Opunake. Maximum pumping distances are 350-400 m in this case.

Figure 4.9 a shows the design recommendation for the first reef system situated offshore of the northern high priority section of the beach, as described in Section 3.3.8. Based on work undertaken on the recently completed Boscombe Reef, for a beach/land based pumping system a work area of approximately 50 m x 25 m would be required.

Initially, 20,000-25,000 m^3 of sand would be placed within the 600 m stretch of beach with approximately 15,000 m3 in a stockpile and the remainder used to elevate the local beach to a level some 4.0 m above CD. This area would be fenced off and the pumping system (in 2 containers) would be placed adjacent to the stockpile, with a digger used to ‘feed’ the slurry system (Figure 4.9).

Two pipelines (one for water intake and the other for sand delivery) would also be required. The delivery and water intake lines would be between 450 and 650 m long, extending out from the shoreline. The pipelines can be secured to the sea bed with moorings (e.g. 2 tonne concrete blocks) or sand anchors. A similar system would be set up for each subsequent reef group.

The proposed filling system is a land-based filling system, where sand is stockpiled on the beach for pumping offshore and filling the predicted salient on the beach using the Waitemata Groyne sand reserves and resource consent. However, if the sand source is barged into Orewa Beach for later reef systems, it may need to be placed in an offshore location and pumped into adjacent reef units and onto the beach to form the predicted salients. Thus, both methods could be utilised at Orewa and should be considered for Resource Consent application.

When a land based pumping system is used, an on land area would be necessary to stockpile the source sand material. Figure 4.9 shows a schematic for a proposed land based construction and pumping system for the Orewa Reef. The 50 x 25 m construction area noted in Figure 4.9a would include space for a sand stockpile such as shown in Figure 4.9b.

A

b

Figure 4.9. Schematic overview of the work area required for construction of the Orewa Multipurpose Reefs (A). The lower photo shows the typical beach/land construction set-up, with the sand stockpile, containers with 550 hp pumping system and construction fencing to keep out the public during construction (Boscombe, England).

4.6 Structure Stability.

Analyzing the stability for submerged structures made from large, sand-filled geotextile containers (SFC's) differs somewhat from traditional analyses for stability of submerged structures. The differences in both the size and shape of the structure overall, as well as the dimensions of the individual units, requires different levels of analysis to quantify stability. Failure can be seen from a number of perspectives including but not limited to; displacement of individual units, settling of the structure, scour or self-burial of the structure, as well as tearing of the geotextile fabric.

4.6.1 Overall Structure Stability

Calculation for overall forces on the structure from wave uplift and dynamic pressure changes (Natal'chishin, 1974) suggest that the overall mass of a typical multi-purpose reef structure is too large to be moved by wave pressure forces alone.

4.6.2 Individual Unit Stability

Individual unit stability is normally determined through the use of either the Hudson or Van de Meer formulas which give a required rock diameter or weight of an armour unit required to ensure stability of the armour layer. These equations are not directly applicable to the situation where large SFC's are used as the primary construction element in a structure.

Taveira-Pinto (2005) compared formulas for stability of armour stone in the construction of submerged breakwaters. He found that for a typical submerged, shore parallel breakwater with steep slopes under large wave conditions (5-7 m) with a 10 or 15 second period, the recommended armour unit weight is between 6 and 20 tons. When compared to the weight of one typical SFC which weighs on the order of 100 to 200 tons (and up to 1600 tons), the SFC units should be sufficiently stable. However this study is specifically for stone armour units and may not be entirely applicable to SFC construction.

A series of laboratory tests were performed at the University of New South Wales, Water Research Laboratory (Hudson and Cox, 2001) to assess the stability of SFC's. In their experiments they tested a range of wave conditions on model-scale SFC's (scaled 20 m long by 5 m diameter SFC's). They defined three types of bag motion related to stability; skin movement where the geotextile fabric is moved by the action of waves or currents, initial movement where an entire unit moves by rocking or pulsing and bag displacement where an entire bag was moved by half a bag dimension.

The results of their experiments produced a linear relationship between the modified spectral stability number (N_s') and the level of submergence (y/B_h), where y is the depth of water above the crest and B_h is the bag height. N_s' is defined by the following equation:

$$N_s' = (H_c^{2/3} + L_p^{1/3}) / (\Delta B_h)$$

Where, H_c is the critical deep water wave height, L_p is the Airy deep water wavelength and ρ is the submerged density equal to $(\rho_b - \rho_w)/\rho_w$. This formulation is taken from Van de Meer (1991) however modified by using the bag height (B_h) instead of D_{50} (a nominal size of the rock armour units).

The recommended relationship between Ns` and Bh is given as:

$$N_s' = 5(y/ B_h) + 25$$

while the following relationship:

$$N_s' = 5(y/ B_h) + 20$$

could be used for a more conservative estimate.

Using these relationships, the authors determined that the required bag height (B_h) for a deep water wave height of 6 m, period of 12 sec, reef crest at 1 m submergence, with a relative submerged density of 0.4, should be no less than 1.6 m or 2.0 m depending on whether the recommended or conservative relationship is used.

By assuming a more extreme scenario where y = 0.1 m, the resulting B_h is approximately 1.8 m or 2.2 m. If, however, the submerged density is increased to 0.8 (a more typical value for multipurpose reef deployments) then the stable bag heights decrease to 0.7 and 0.85 m for the first case (y = 1 m) and 0.9 and 1.1 m for the second case (y = 0.1 m).

It should be noted that Hudson and Cox (2001) state that their tests were only conducted for y/B_h values between 1 and 4, i.e. where the depth above the reef is at least as high as the reef unit itself. However, in many cases for artificial reefs, this ratio could be quite small, i.e. 0.5 m of water over a 3 m diameter geobag, implying a y/B_h ratio of 0.17. Even in the example provided by the authors, they calculate a stable bag height of 1.6 m for a depth of 1 m, or a y/B_h ratio of 0.625, which is also out of the range of the study.

Overall, the study suggests that large SFC's with heights of 1 m or greater are quite stable even under extremely large wave loading. ***The bag units used in construction of multipurpose reefs to date have been cylindrically shaped on the order of 1.0 – 5.0 m tall and 30 to 50 m long.*** Bags of these dimensions are very stable when the analysis of Hudson and Cox (2001) is applied.

Physical modelling tests of SFC stability under a range of wave conditions were undertaken at the University of Sydney in 1998. Tests were performed on SFC's equivalent to the Terrafix 1200R containers described above (120-300 Tonnes). These tests showed that SFC's stacked upon each other would remain in position even after a prolonged period of extreme high sea conditions (Couriel *et al.,* 1998). Part of reasons for the good stability of SFC's can be attributed to the bags allowing water to move through them and the use of marine sands to fill them. Unlike concrete and some types of rock, which have air trapped within their mass causing a dramatic weight loss once they are placed underwater due to buoyancy (e.g. concrete weighs approximately half its land weight underwater), water can enter the geotextile bags and the mainly quartz sand is not porous. The stability of the units under wave action is also ensured due to the large weight of each individual container and the friction between adjacent units. The stability of SFC's was confirmed on the Gold Coast in April 2001, when a 1 in 50 year wave event hit in the coast (cyclone Sose). Comparison of bathymetric surveys of the Narrowneck reef on the Gold Coast 1 week before and 1 week after the event failed to identify any structural changes (unpub. data). In March 2004, an extreme wave event occurred, and waves peaked at 14 m on the Gold Coast without damaging the SFC constructed Narrowneck reef (J. McGrath, pers. comm.).

4.6.3 Wave Impact

In the study by Hudson and Cox (2001), they noted the phenomenon where the bag itself was not moved, but the geotextile fabric surrounding the sand filling pulsed with the passing of a wave crest. They called this 'skin movement' and noted its occurrence relative to partial or total movement of the bag it self. While skin movement was observed on nearly all test cases, it was also noted that the model bag units were not filled extremely tightly and that the geotextile used may have different permeability and elasticity properties than prototype bags. This is an important consideration, geotextile reefs that have been built so far have had the individual bag units filled to capacity resulting in a very tight and firm surface where wave impact may occur. This would minimize the deformation associated with each wave and increase longevity.

Technical information provided by the geotextile manufacturer claim that wave impact will not result in significant deformation of the geobag's shape. The sand inside the bags does generally become redistributed to some extent, however, this is generally not significant. Jackson *et al.,* (2002) report deformations on the order of 200 - 300 mm on the bag in the wave impact zone.

Wave impact loads would also cause abrasion to the geotextile fabric. All materials in the dynamic near-shore environment will suffer from abrasion (whether it is treated timber, concrete, steel or geotextiles). Recently, extensive tests on a number of geotextile materials (including Terrafix 1200R and 1209RP) have been undertaken in Australia to assess their abrasion resistance. The laboratory test undertaken was an accelerated test, which runs for 80,000 revolutions, and used sharp angular crushed basalt rock as the abrasion material. The results showed that there was a large variation in the performance of different geotextiles with some types failing completely within the test period, while others showed minimal strength loss. Terrafix 1200R and the related Terrafix 1209RP were the best performing materials, with very little loss of strength and no reduction in mass (woven and continuous filament materials were reduced during the tests). While it is not unreasonable to expect some reduction in mass with extended testing, these results demonstrate the high resistance and strength of the material and explain the manufacturer's willingness to offer extended warrantee for the product. The oldest Terrafix 1200R structures in the marine environment has been in place for more than 15 years and are still performing well to date.

Modern geotextiles are durable materials, commonly supplied with a guaranteed service life of 20-30 years and a postulated life of up to 100 years even in a challenging marine environment. Tests by Naue Fasertechnik (a German geotextile company) and others have shown that the material is resistant to chemical and biological influences and can be effectively protected from ultra violet degradation by applying UV stabilized products. With respect to submerged structures, UV is not an issue – the UV component of sunlight does not penetrate more than a few millimetres into seawater. Experience gained on previous, successfully completed projects also shows that geotextiles are puncture and abrasion resistant and with correct care, remain undamaged by the construction process (Naue Fasertechnik). Some concerns have been expressed about the resistance of geotextiles to determined vandalism and while it is true that some risk exists, previous experience on projects around the world suggest that the risks are not significant and that if necessary repairs can be made easily with patches (marine glues, nylon-screwed and Velcro have all proven effective).

4.6.4 Settlement and Scour

The foundation design of the reef is important to limit the effects of overall and differential settlement of the seabed beneath the reef structure, which could significantly affect the accuracy of the final reef profile. Jackson et al. (2002) reported controlled field investigations that investigated the settlement of individual cylindrical units with a 3 m diameter and a length of 20 m. The initial settlement was observed to be on the order of 0.1 m. However, other reports suggest that the overall crest height of the Narrowneck Reef settled some 2 m one year after construction. Whether this is due to the large mass of the overlying bags or to an overall change in the seabed level (Jackson *et al.,* 2002) or to the lack of a geotextile foundation or scour mat, is unclear. The beach at Narrowneck was nourished with 1.2 million cubic meters (Turner *et al.,* 2001) of sand before reef construction, which affected the profile over which the reef was built. In other cases, where foundation webbing and matting have been used, settlement has been minimal and only occurred after initial filling (i.e. Mount Maunganui (comparison of 12 time series bathymetry surveys over 3 years), Boscombe England (comparison of 7 time series bathymetry surveys of base layer over 7 months) Opunake (comparison of 12 time series bathymetry surveys over 3 years), unpublished data), and in line with measurements of previous similar projects, as described in Appendix 4 accompanying the application (i.e. the feasibility study), ~0.3-0.5 m. However, previous construction of structures on sandy seabeds indicates that a foundation material is an effective means of reducing settling (mass or differential) and so it has been incorporated into the Orewa Reef design.

To counteract settlement, a foundation layer of structural geotextile (e.g. CombiGrid) can be placed on the seabed over the entire reef footprint. This layer of base reinforcement acts to span localized settlements and zones of weak soil. Using a structural geotextile as basal reinforcement is a commonly used technique and has the added benefits that it will help to improve the overall stability of the structure by resisting lateral spreading of the reef units, and will improve the bearing capacity of the foundation. This layer of CombiGrid (see Appendix 6 for specifications) can either have containers attached during the land-based phase of construction (Section 4.4), or the CombiGrid foundation can be deployed and secured using 'scour bags' around the perimeter ready for the SFC's to be attached and filled. The latter method is the preferred option at Orewa. Scour is around the structure is prevented by scour-bags, or T0.5 Softrock containers. These containers sit along the edges of the reef and following scour (which will mainly occur along the inshore edges of the structure), will settle into the scour holes and prevent undermining of the structure. Similar measures are used world-wide and have proven successful along the inshore end of the Mount Reef mega-containers..

As well as limiting the differential settlement of the reef, a geotextile foundation layer will act to separate the reef from the underlying strata providing scour protection. Scouring of the seabed around the edge of a coastal structure is a common cause of failure. Increased wave and current velocities close to a structure cause local erosion of the seabed, creating a scour hole. In some cases the effects of scour can undercut a structure and lead to catastrophic failure. Extending the foundation geotextile beyond the edge of the reef (1-2 m) prevents erosion of the seabed in that area, as has been shown with many similar constructions world-wide. In addition, ‘scour bags’ (1 m diameter SFC’s) can be incorporated around the edge of the structure (these were incorporated into the Mount Maunganui reef – analysis of reef surveys since October 2005 indicate that there has been no settlement of this reef). Scour bags can be considered somewhat sacrificial in that they are located to ‘slump’ into scoured areas along the base of the structure to prevent further undermining.

With respect to geotechnical viability/strength, sand-filled containers represent sand resting on top of sand, with the bearing capacity of packed sand (as existing at Orewa Beach) being relatively high. Cores have not been taken over the entire subtidal area of Orewa Beach at reef locations, and based on the available information, they are not considered necessary. Anecdotal evidence during field work and the deployment of 3.0 m long 4” galvanised pipes for mooring attachment, using a 4” trash-pump to jet them into the seabed, indicates sand and shell fragments to depths of 3.0 m and 300 m offshore (from CD). International literature review (as attached in the feasibility study) shows that such structures have the capacity to settle between 0.3 and 0.5 m when placed on sand. Analysis of 13 bathymetry surveys of Mount Maunganui reef and 7 of the lower layer of Boscombe reef indicate that following initial settlement of 0.2-0.3 m, containers have remained at the same elevations over almost 4 years of monitoring.

4.7 Sand Sources

Two potential sources of sand are under consideration for the Orewa Multipurpose Reef construction and beach restoration project. The first sand source is the supply of sand which has collected on the northern (updrift) side of the Waitemata Groyne (Figure 4.10). This sand has been used repeatedly in recent years as source for the periodic renourishment of Orewa Beach (see Table 2.7) with volumes of up to 50,000 m^3 extracted from the beach adjacent to the groyne and redistributed further north along the beach on an annual basis. Discussions with the RDC indicate that this source of sand can be utilized for the construction and salient placement (renourishment) of the first reef system (some 40,000-45,000 m^3). However, due to the closed nature of the Orewa Beach system, outside sources of sand will be required for the future reef system construction and salient placement. In total, it is estimated that 100,000-120,000 m^3 of sand will be required for renourishing the beach in the form of the predicted salients (25-30,000 m^3 per reef system), with the first 25-30,000 m^3 being extracted from the Waitemata Groyne.

The second sand source would likely be derived from offshore deposits located offshore of Pakiri Beach, approximately 50 km north of Orewa. The sand extraction permits for this area have been granted to McCallum Brothers Ltd. Ltd., a commercial sand extraction organization with extensive experience in supplying marine derived aggregates to the New Zealand construction industry. The extraction area for this source is in an 8 km by 1 km area located approximately 2 km offshore of Pakiri Beach (Figure 4.10) in 30 to 40 m water depths. This source has previously been used to renourish Mission Bay with >50,000 m^3 of sediment in recent years.

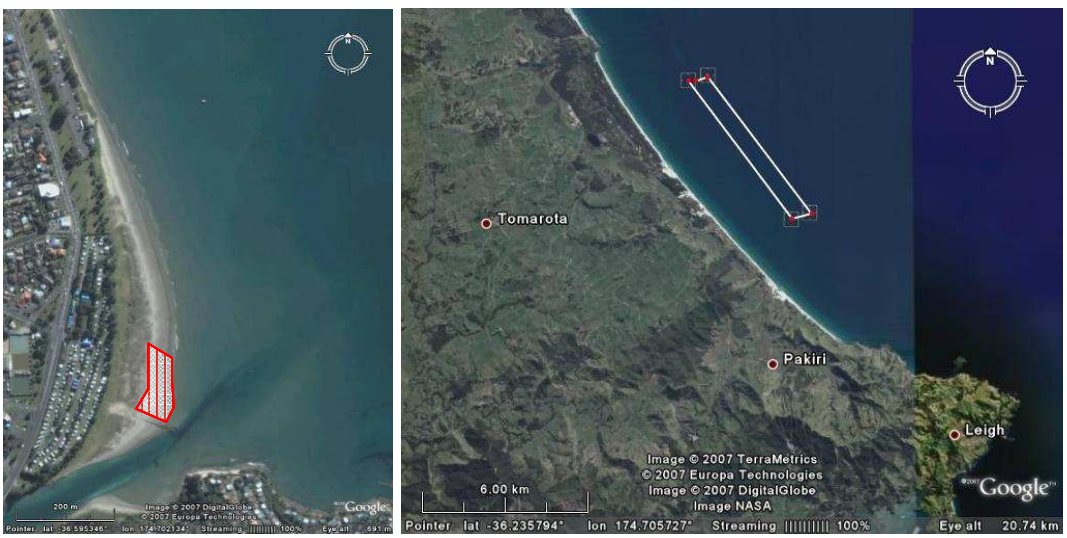

Figure 4.10. Possible sand source location for the construction of the Orewa Multipurpose reefs. The Waitemata Groyne on the left and the offshore Pakiri sand deposits on the right.

With respect to the first sand source (i.e. the southern groyne), a resource consent was granted in 2006 for the continued use of the accumulated material to renourish the central section of Orewa Beach in response to severe erosion events. Specific conditions for this consent require that extraction for renourishment only take place when the beach levels fall below a threshold level based on existing survey marks. The conditions further require that the amount of material extracted would leave a minimum of 60,000 m^3 of material in the beach system at the groyne location.

For the offshore sand source, extensive pre-dredging assessments of this area were carried out as part of the specific resource consent application process to extract the material (Healy and Immenga, 2003, Mead et al., 2003). The studies indicated that the offshore area is characterized by shore parallel wave generated ripples indicative of medium grained sands. A biological assessment of the dredge site showed that the area was populated with benthic fauna, but did not reveal the presence of important or sensitive habitats or species. This sand source has been used in the recent past for beach nourishment projects in the Auckland area including 35,000 m^3 delivered to St. Helliers Beach in Auckland between May and October 2006. The offshore sands are quartz-feldspathic in composition with approximately 91% of the particles lying between 0.15 to 0.6 mm diameters. For particles outside this range, the majority (~8% of the total) are between 0.6 and1.2 mm. Thus the material is quite fine and ideal for use in the reef construction as it is more efficient to pump finer sands over longer distances, and in addition is similar to the grain sizes presently found on Orewa Beach (i.e. mean of 0.12 mm).

Regardless of the sand source, the fill material will require transport to the construction site and staging area by some mechanical means for delivery to the offshore reef units. For the groyne source, this would likely be accomplished by loader and truck as specified in the 2006 Resource Consent Application (Beca, 2006). The sand would be stockpiled at the appropriate location where it could be easily accessed and tipped into the pumping mechanism. This sand source would be the most economical source per cubic meter.

For the offshore derived sands, the material could be delivered by two methods – by ship transport or by truck transport. For the ship based transport option, the material would be brought in on a 52 m vessel able to transport approximately 460 m^3 per load. The vessel would moor offshore and either pump directly into the reef units, or pump the material onto the beach for stockpiling and later delivery to the reef units. This method has the advantage of a quicker delivery time for the total necessary volumes as well as minimal impact on traffic congestion due to on land truck movements. Sand delivered by this method would have a lower overall cost than if delivered by truck but would be more expensive than the sand moved from the groyne.

The second delivery option would involve trucking sand from the Auckland wharves to Orewa. With each truck load able to accommodate approximately 17 m^3 of sand, it would require 27 truckloads to equal one ship load per day. This method of sand delivery is the most expensive as it would require 2 hours of road travel time per load as well as additional charges for wharfage fees as the material is offloaded from the dredge. The advantage of this delivery option is that it would not be constrained by weather and could represent a viable option for sand source continuity during the construction process.

In terms of construction staging and scheduling, it would be ideal to have a large quantity of sand delivered via the offshore barge method. This will form the initial stockpile of nourishment sand and provide a source for starting the reef construction. Once a suitable stockpile has been placed, reef construction would begin while sand deliveries continue until the project phase has been completed and the require volume of sand has been placed on the beach.

4.8 Summary

Construction of the proposed Orewa Multipurpose reefs will be accomplished using very large sand filled geotextile containers (SFC's). This general method has been used successfully in four separate multipurpose reef projects as well as a very large number other coastal and maritime construction projects. Because of the very large size and mass, structures made from large SFC's have shown to be very stable and resistant to damage from wave attack. Additionally, the geotextile material is resistant to bio fouling, ultra violet light and vandalism

To build an SFC structure, bag units are deployed from a barge and fixed to the sea bed using winches and anchors. This is followed by pumping the reef units full of sand with either a land based or barge based pumping system. Both methods have been used successfully in the construction of multi-purpose reefs.

In the land based system a large pump is situated on land. Suction pumps bring seawater to this pump where sand is mixed in and the resulting slurry is pumped back out to the reef bags. This method has the advantage of being somewhat independent of the swell conditions, however the disadvantages are the large pumping distances requiring a large land based pump. This methodology is the recommended methodology for the first reef system at Orewa

In the water-based approach, the pumping equipment is situated on a barge located near the reef site. The material for the reefs is excavated directly of the sea bed nearby and pumped directly into the bag. This method has the advantage of shorter pumping distance and smaller overall hardware requirements, but is extremely dependent on weather and swell conditions. While land based pumping is preferred, given the uncertainty with the source of sand for following reef system construction, it is recommended that this system is also considered for Resource Consent application.

If subsequent beach nourishment or reef construction is to take place at Orewa, sand will have to be brought in from outside sources since there is not enough sand available in the natural system. Such sand can either be truck onto the beach to accommodate a land based system or brought in by barge to accommodate a sea based-system. Based on cost information provided by sand suppliers, the barge option is cheaper and logistically much simpler than the truck delivery option.

CHAPTER 5 – ASSESMENT OF EFFECTS ON THE ENVIRONEMENT

5.1 Coastal Landscape, Noise and Traffic

With respect to the coastal landscape, the Orewa Beach multipurpose Reef project will have a minor and temporary impact during construction. During construction of the reefs, the physical plant necessary for construction will be contained within a fenced off work area of approximately 25 x 50 m. Additionally two pipelines for water intake and sand delivery (up to 9” diameter) will be required during construction, which will run across the intertidal zone. The construction process for each reef system is expected to be on the order of 12 – 16 weeks (including mob/demob of site). After this time the construction equipment will be removed and the area returned to its normal pre-construction state, albeit with significantly more sand (renourishment).

Additional temporary impacts include increased vehicular traffic during the construction process and noise due to the construction equipment (pumps, earthmovers, etc). This noise, however, will be mostly limited to normal working hours (8 am – 6 pm, possibly later if construction proceeds during summer months), the site is located in front of residential properties, similar to the situation in Boscombe, England (Figure 4.9b). It is noted that Orewa’s main road was until very recently the route for heavy vehicle traffic (i.e. it is a noisy road throughout the day and night). The slurry pump, water pumps and front-end loader will be operating during times that sand is being pumped. The machinery noise is minimized by having the main pump in a container, while water pumps will be placed at the seaward edge of the construction area. Construction material (sand) sourced from the Waitemata Groyne will be delivered to the construction site via earthmoving equipment, as has previously been the case. However, if material is brought in from other sources (see Section 4.7) then there will be either daily barge traffic offshore or additional truck traffic to deliver the sand to the construction staging area – these latter effects are not expected for the construction of the first reef system, which will utilize sand from the Waitemata Groyne.

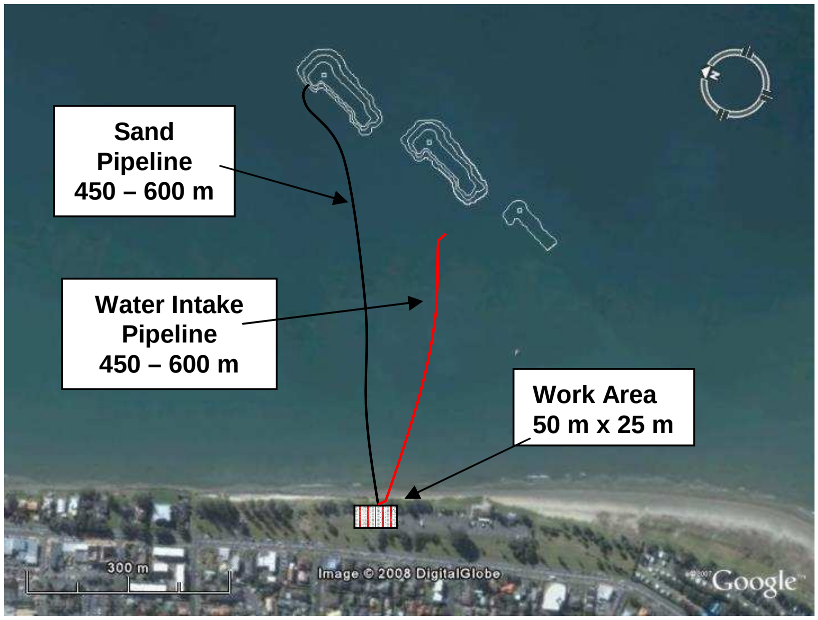

Figure 5.1. Approximate scale of construction area and extent of pipelines necessary for reef construction.

The permanent effects of the reef on the coastal landscape will also be very slight – minor and insignificant. The reefs themselves are designed to have their crest height at +0.5 m. This is equivalent to the Mean Low Water Spring (MLWS) water level. Thus, the reefs will be fully submerged at all but the lowest tides (<1% of the time). Therefore the reefs will have a minor effect on the visual landscape. Indeed, they may be considered an enhancement by some, since waves will be seen to break offshore on the structures. The geotextile material of the reef is of a similar colour to the sand on which it will rest, although the fast colonization (see ecological AEE report) will result in the structures darkening to the appearance of natural reefs. On clear calm days the dark outline of the reef may be visible from the top of the beach and higher vantage points. In small swell conditions, the crest of larger waves will peak up on the reef at low tide and in bigger swells, waves will break and peel from left to right when looking out to sea. Wave breaking is dependent on water depth, therefore waves would be visibly breaking on the reef at different times, depending on tidal cycles and swell size and duration. This kind of visual rhythm may be considered an aesthetic enhancement that compliments the ebb and flood of the tide and the ever-changing moods of the sea (i.e. the swell conditions).

The most noticeable impact of the proposal will be an increased area of dry beach above the high tide mark. In terms of natural character, this impact may be viewed as an enhancement to Orewa Beach, which would be complimented by the planting of native sand-binding plants (Section 3.3.4).

5.2 Waves, Tides, Sea Levels and Currents

In terms of sea levels and tides, the reefs themselves will have no impact. Tidal currents at the reef site are very low and will not be influenced by these comparatively small structures. Similarly, overall sea levels will in no way be influenced by the presence of the reef.

Relative to waves and currents, the reefs are specifically designed to modify wave and current patterns, thus, there will be a permanent and beneficial change in the wave and currents in the immediate vicinity of the site. Figure 5.2 shows the current patterns around the proposed reef structures during a 1 year RI storm event. As described in the modelling Section, the main changes in the current patterns are the redirection of currents inshore due to waves breaking over the reefs, and a northerly rather than southerly dominance.

Figure 5.3 then shows the change in the current velocities that is expected in the vicinity of each reef system. These plots take the difference between the currents for the case with reefs versus the case with no reefs (i.e. the existing conditions). It is clear that the reef effects on the currents are localised to the immediate area around each reef component, which are driven by waves breaking over the reef system and redirection of currents

In comparison to existing current patterns on the beach, the currents around the reef are far more regular and predictable due to the presence of permanent non-erodible structures.

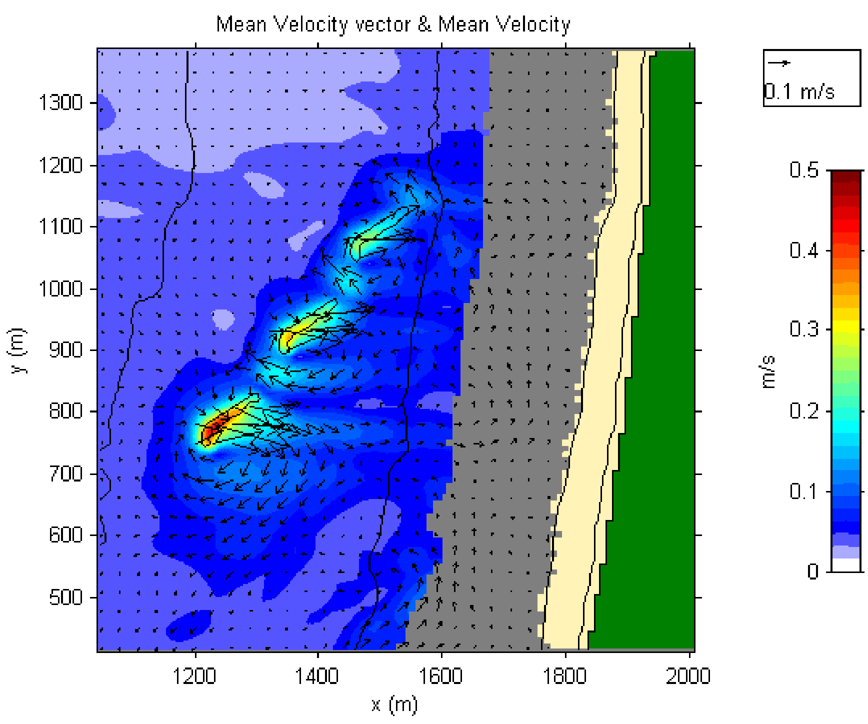

Figure 5.2. Modelled current patterns around the final reef design during normal conditions.

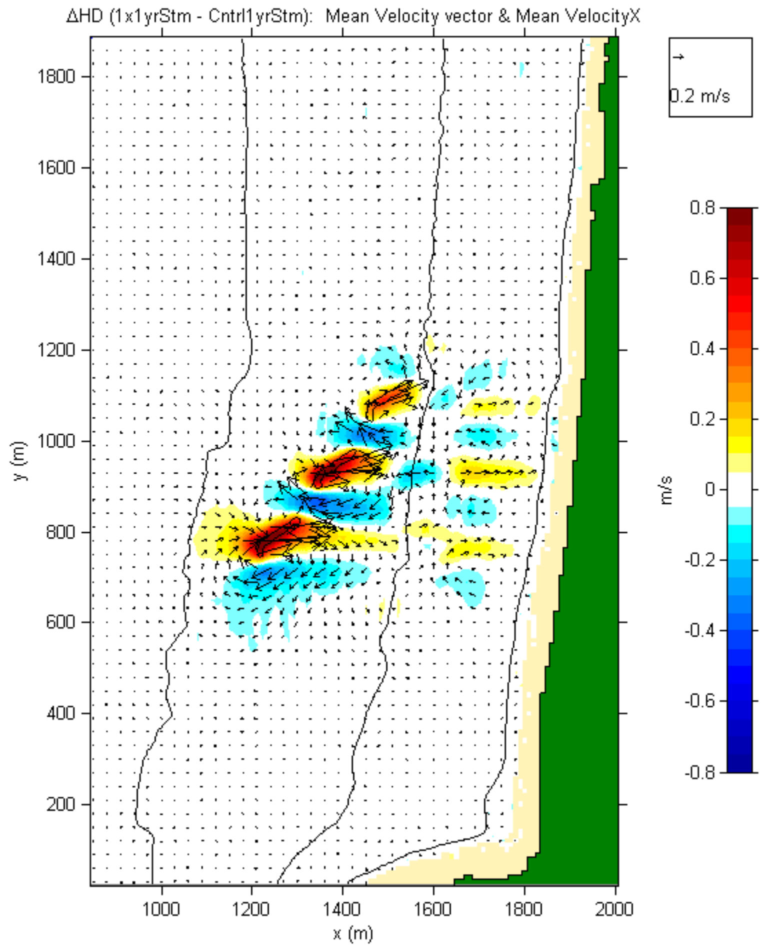

Figure 5.3. Modelled difference plot of current patterns around the final reef design during a 1 year RI event.

Relative to wave conditions, the reefs are designed to cause the waves to break further offshore and therefore dissipate energy. This is manifested as a reduction of overall wave height in the shadow of each reef structure. This reduction in wave height is central to the functionality of the reef (less energy reaching the beach results in less erosion potential and the settling of sand in the lee of the structures), thus, the effects are considered to be minor yet permanent.

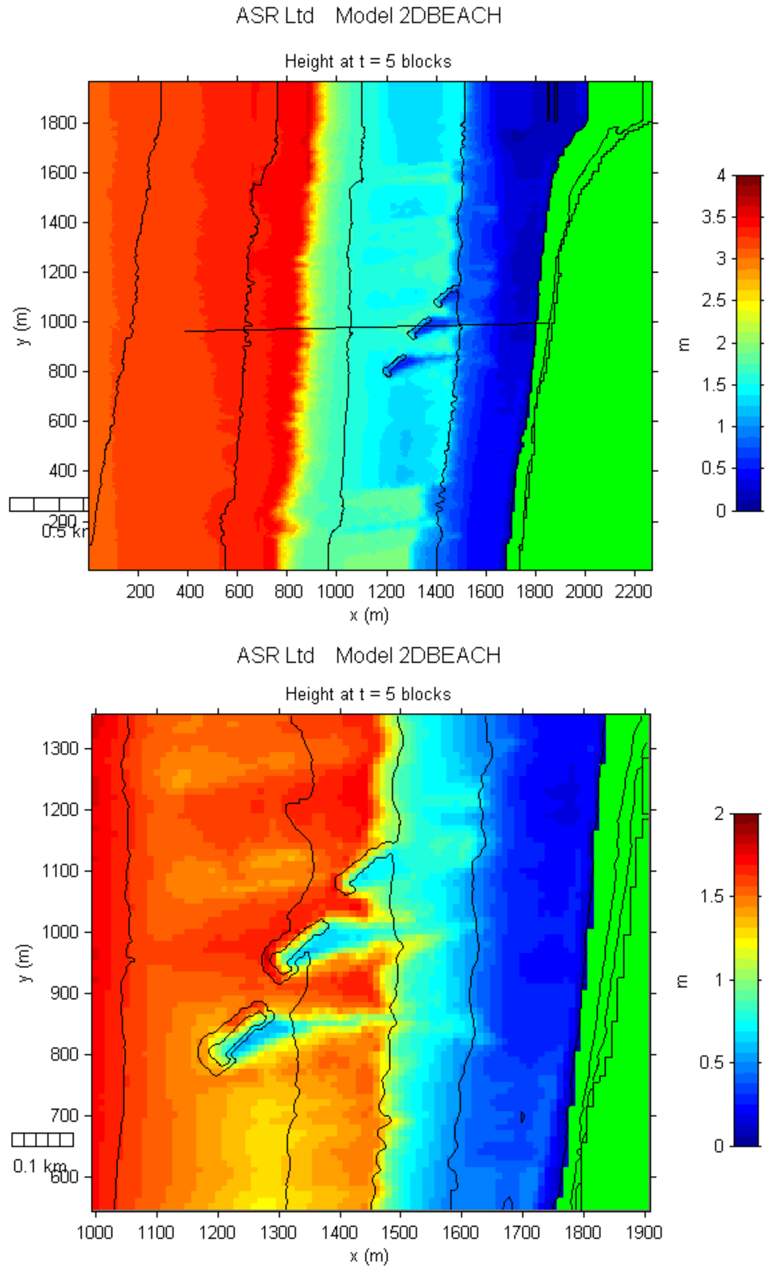

Figure 5.3. Wave dissipation for the final proposed reef shape versus the control using 2DBEACH. H = 3.1 m, T = 10 sec. The wave shadowing effect of the reef structures can be clearly seen.

5.3 Shoreline Stability

The primary function of the Orewa multi-purpose reef project is to increase and enhance shoreline stability. As described in Section 3.3.3, a net widening of the dry beach is expected as a result of the combined effects of the reef structures and the ongoing renourishment programme (the latter albeit greatly reduced due to the presence of the reef systems). Figure 5.4 shows the expected seabed and shoreline response due to the presence of the reef. Note the shift in the contours seaward due to the sheltering effect of the reef and the depression in the lee of the outer most reef unit – these are the main morphological responses which occur due to the changes in hydrodynamics (waves and currents) due to the presence of the reefs).

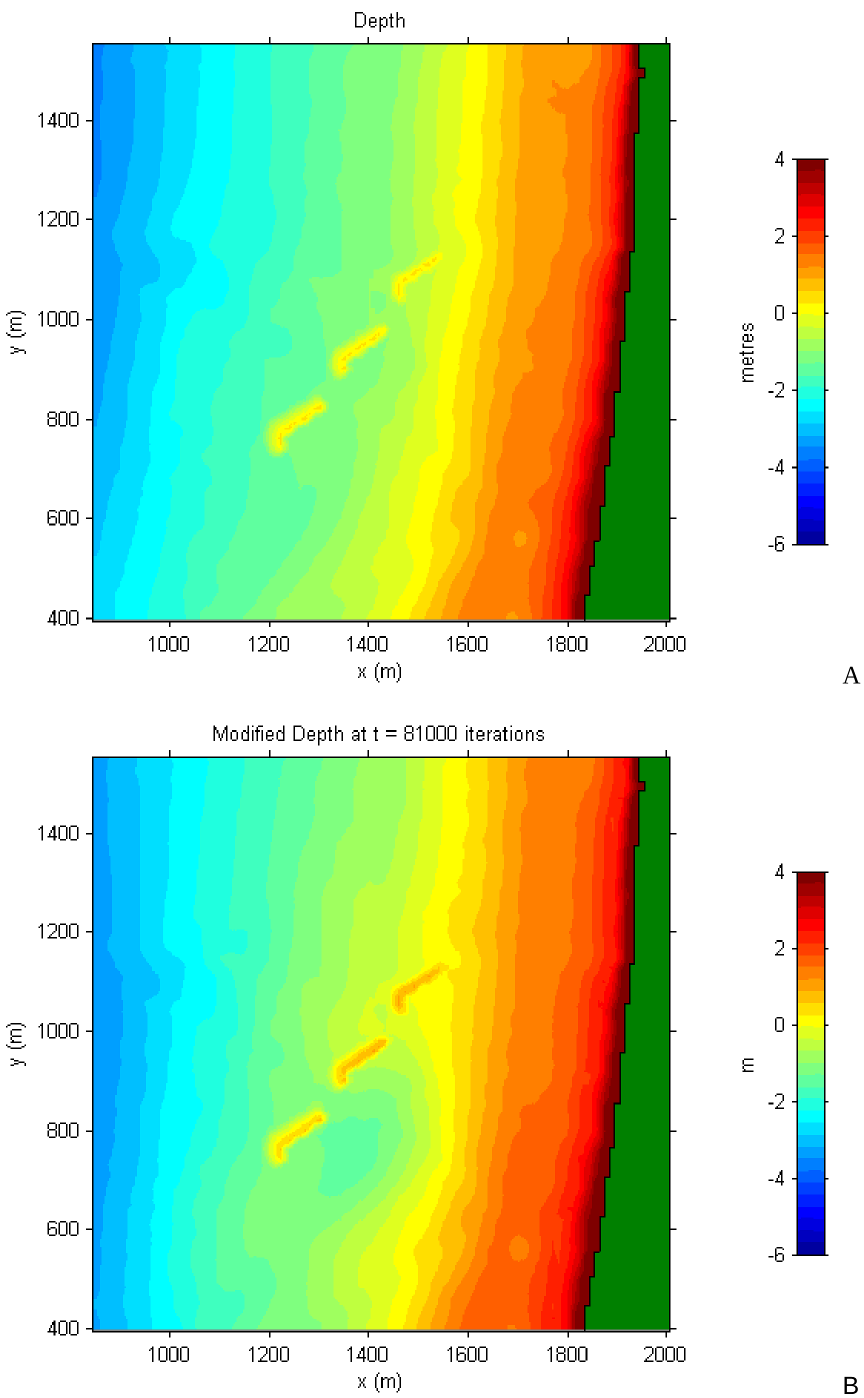

Figure 5.4. Modelled shoreline response from 2DBEACH for a single reef system, a) before, and b) after.

The most significant features are the double-horned salient response, which results in the offshore migration of the 0.0 m contour of ~100 m in the lee of the inner reef unit and to the north (down page) of the outer reef unit, with the latter as would be expected due to the predominantly southerly sediment movement. On the beach and intertidal zone it can be seen that the +2.0 m contour also migrations offshore some 100 m and that significant sand builds up at the high tide mark. Also notable is the deepening of the seabed in the lee of the offshore reef unit. Similar to analysis of the Mount Reef bathymetry surveys, this feature may be up to 1 m deeper than the pre-reef situation during some wave conditions, and then reduce during other periods (e.g. Scarfe, 2008; ASR unpub. Data). The modelling indicates that during higher wave events, the depression behind the reef increases as does the accumulation of sand on the beach. During following lower wave periods, the depression becomes shallower. Time series monitoring will provide further quantitative understanding of the magnitude and processes of beach and seabed effects.

Thus, the reef system is influencing a large area of the seabed and beach, similar to other coastal protection structures. At the Gold Coast, the Narrowneck reef creates an asymmetric salient that stretches several kilometres to the south due to the predominantly northern sediment transport (it was originally estimated through numerical modelling that, although undetectable given the average daily change in the high tide mark of 18 m, the salient impacts would reach as far south as Burleigh Heads (some 12 km away). Similar modifications to the seabed have been observed at the Mount, where depending on the recent wave events, the seabed can be slightly elevated on either the northwestern or southeastern side of the reef (e.g. Scarfe, 2008; ASR unpub. Data). Groynes also have wide-ranging effects, with accretion and erosion patterns extending considerable distances (in some cases kilometres) up and down the beach (e.g. Basco and Pope, 2004)

Further tests indicate that the induced beach plan shape is likely to remain stable under consistent and/or large wave attack. Thus, the overall effect of the multi-purpose reef systems are the desired widening of the beach that is not detrimental to the physical processes of the beach system, i.e. it is a new dynamic equilibrium produced by the construction of new beach control points.

When 2, 3 and 4 sets of reefs were modelled, although hydrodynamic impacts are relatively confined to the vicinity of the reefs, 2 reef systems retain more sediment than 2x the volume retained by 1 reef system, 3 reef systems retain more sediment than 1.5 times 2 reef systems and 3 reef systems retain more sand than 1.33 time 3 reef systems.

In summary, the reef systems have a significant impact on the beach and seabed morphology, modifying wave and current patterns which in turn influence the seabed and beach morphology, creating a wider beach as to achieve the objectives of the project.

5.4 Surfing, Recreation and Economics

The Orewa Beach Reef Project is expected to have an overall positive impact on recreational amenities and local economics (TRI, 2004). The primary recreational amenity addressed is surfing and surfing wave quality. Other recreational amenities that stand to benefit from the presence of the reefs include kite boarding, windsurfing, fishing, snorkelling, SCUBA diving and sea kayak riding.

The effects on surfing on the surfing amenity along Orewa Beach are expected to be generally positive. Directly over the newly constructed reefs, the steeper seabed gradient caused by the reef will cause waves to break with greater intensity than over the normal seabed – this is an enhancement, as the waves at Orewa are generally considered to by 'fat', or with low intensity. The presence of the reef will also create a defined take off spot and a increase in the wave peel angle (defined in Figure 5.5, i.e. slower peeling waves, rather then close-out conditions), which is generally considered to be an improvement in surfing wave quality when compared to the more random and closed-out (small peel angle) nature of most beach breaks.

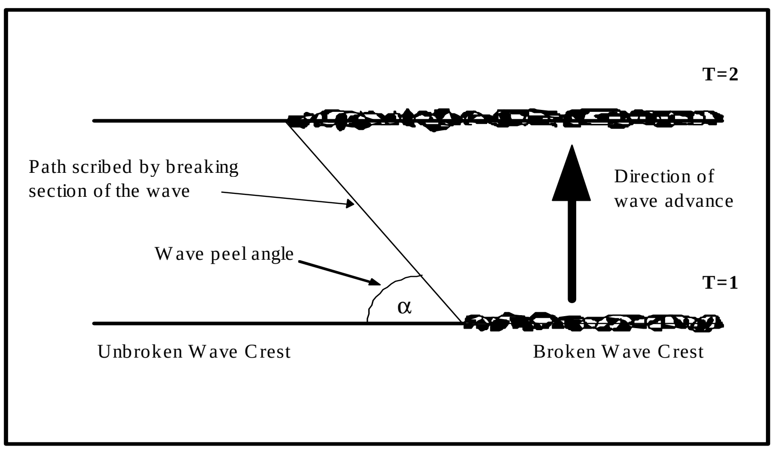

Figure 5.5, Schematic diagram of the wave peel angle showing movement of the breakpoint during an increment of time. Large peel angles equate to slower breaking waves while small peel angles are fast breaking waves.(Source – Hutt, 1997).

While Orewa Beach does occasionally get good quality surfing waves, as shown in Figure 5.6, the narrow swell window and sheltered nature of the beach and wave climate means such waves are relatively infrequent. It is expected that the presence of the reef will enhance surfing by providing more organization and focus to choppy wind driven swells that frequently affect Orewa. While a reef can have only a minimal effect on the size of the waves locally focusing and shoaling them), as that is governed by offshore weather patterns, it can affect the way the waves break. This is described diagrammatically in Figure 5.7. The red box covers the range of wave heights and peel angles currently experienced at Orewa. When the waves are good for surfing, there are slow breaking soft waves, indicated by peel angles between up to 60 degrees. During choppy or larger wave conditions, the peel angle can be much smaller, approaching zero (i.e. perfectly closed out). In the presence of the reef, the same size waves will break with manageable peel angles between 45 and 70 degrees (shown by the green box) which will provide a consistent and manageable ride of up to 100 m on each section.

Figure 5.6. Photos from a very good day of surfing at Orewa. Wave heights are on the order of 1 m and peel angles appear to be between 50 and 70 degrees.

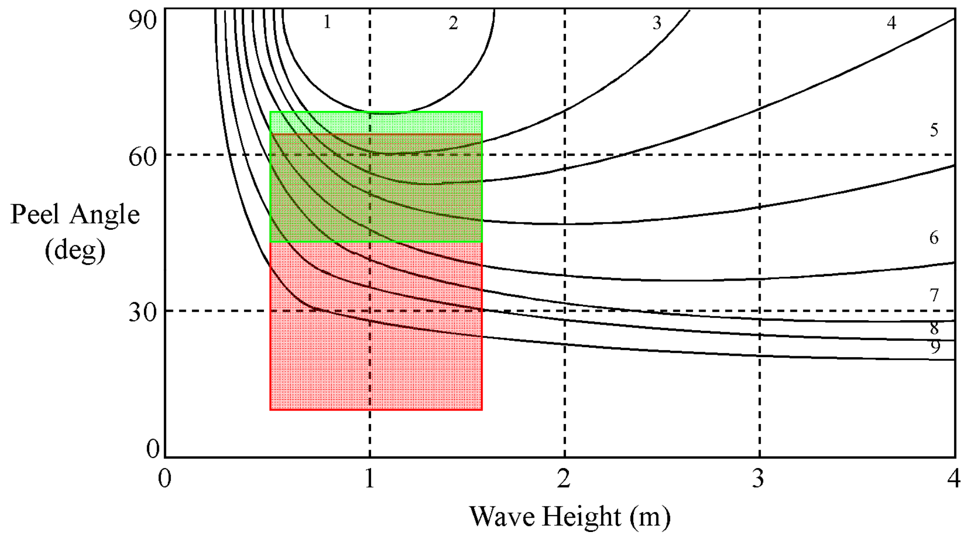

Figure 5.7, Classification of surfing skill (contours, labelled 1-9) rated against peel angle (y-axis) and wave height. (x-axis). The green box indicates expected surfing conditions on The Orewa Reefs while the red box covers the range of wave conditions currently experienced. The design wave heights for the reef are between 0.5 and 1.5 m with peel angles between 45 and 70 deg.

For other water sports such as windsurfing and kite-boarding (also called kite-surfing), the reef will provide a useable obstacle for either wave jumping or wave riding by participants of either of these sports. While this may lead to some user conflicts (i.e. between pure board surfers and wind-riders) these issues are generally solved by the sea state or environmental conditions of the day. The conditions that are ideal for each of the activities are mutually exclusive, when it is windy enough for wind sports, it will be too windy and choppy for surfing whereas when the conditions are right for surfing – clean swell and light wind – the wind sports will not be an option. Furthermore, if the project is brought to completion with all 4 reef units installed, certain reef group could be designated for certain uses. This issue however can be resolved when the time comes and monitoring of the reefs determines which parts of the beach are ideal for each user group. The following is an article from the local newspaper in Boscombe, England, describing the positive impacts of the surfing reef recently completed for the windsurfing/kite-boarders:

Windsurfing champion puts Boscombe surf reef to the test

8:15am Friday 17th July 2009

ALTHOUGH Europe's first artificial reef isn't due for completion until the autumn, it didn't deter windsurfing ace Guy Cribb from being first to put it to the test.

As stormy seas lashed the South Coast, the 13 times UK champion from Poole made surfing history.

Guy, 39, who has windsurfed all over the world, told the Daily Echo: "I've ridden waves in Hawaii and Australia but it was great to score some good surf in Bournemouth in July.

"The reef is already showing huge potential as a great wave spot."

Bournemouth council stressed that the reef was still a construction site and unsafe for surfers.

But Rex Pollock, contractor ASR Ltd's marine construction manager, said: "It's great to see the reef producing decent surfing conditions before it's even finished. We don't, however, encourage people to use the reef.

"The base layer was |completed last year, the flat layer on top is now complete and we are now working on the ramp which pushes up the waves. Construction is on schedule; 11 out of 18 huge |geo-textile bags are laid and filled.

"One they are all laid we still need several weeks to fine tune the structure. If weather conditions are consistently good we anticipate an autumn finish date."

The reef will be about the size of a football pitch and contain 26,000 tonnes of sand, sitting about four metres from the seabed and 250 metres off shore.

Mr Pollock added: "Already our team of expert divers have seen marine life growing on the reef which is covered in fish eggs and home to spider crabs."

Figure 5.8, Champion wind-surfer Guy Crib jumping waves on the Boscombe Reef, England.

During calm periods with clear water, SCUBA divers and snorkelers may be interested in viewing the sea life which has colonised the reef. This has been the case at the Narrowneck Reef on Australia's Gold Coast where the reef has become part of a local tour operator's 'Dive Trail'. The reefs will be of interest to sea kayakers and provide a point of interest for participants of that activity. Fishing is also another possible recreation that would be enhanced, although it is likely that the reefs would be no take and fishermen would benefit from the spill-over effect created by these new habitats. In general the reefs will provide a variety of recreational opportunities for several different user groups depending on the conditions present at any given time.

Additionally, the amenity benefits associated with multi-purpose reefs are becoming increasingly more apparent, especially because of the economic spin-offs that are associated with them. A recent paper on the economic value of beaches in the USA (Houston, 2002) focused on tourism and highlighted some important points relevant to the application of multi-purpose reefs in California. Beaches are the leading tourist destination in the US, receiving 85% of all tourist related revenue, over $260 billion annually (Houston, 2002). Over 500,000,000 tourists visit the Californian beaches, with Californian State beaches receiving 72% of the visitors, even though they represent only 2.7% of the State parks (Houston, 2002). Similar to the Gold Coast in Australia, erosion is the number one concern that people have about beaches, but relatively very little has been spent on addressing erosion problems in California (Houston, 2002). In comparison, Miami's beach restoration experience has shown that the presence of wide sandy beaches is valued at a benefit/cost ratio of 500:1 (Houston, 2002). Houston (2002) advocates the need for "… a paradigm shift in attitudes toward the economic significance of travel and tourism and necessary infrastructure investment to maintain and restore beaches …". The current project is an example of the current thrust to develop novel erosion control methodologies with low negative environmental impacts while providing opportunities to enhance amenity value along the developed coastal areas.

Studies into the economic benefits of artificial reefs are now becoming increasing common. Indeed, a recent socio-economic study of reefs in southeast Florida demonstrates the huge economic contribution of reef related expenditures (boating, fishing, SCUBA diving, snorkelling) that artificial reefs make to the region (Johns *et al.*, 2001) – this area of the US is a world-leader in habitat enhancement. Incorporating wide beaches and quality surfing conditions into artificial reefs adds more to this equation. For example, events associated with the beach and surf can be of considerable economic importance. A recent festival at Noosa in Australia to celebrate the restoration of the beach attracted an estimated 20-30,000 visitors over the weekend and therefore at least A$1M into the economy if visitor spending was only A$50/ visitor. Surfing competitions are now heavily promoted and publicized - for instance, a single international level surfing event (short board or longboard or bodyboard, etc) can bring hundreds of thousands of dollars into the local economy. On the Gold Coast, it is estimated that a single high profile surfing event is worth AU$2.2M (Raybould and Mules, 1998). In Cornwall, England, it is estimated that direct spend by surfers in the local economy is in the region of £21M each year (Ove Arup & Partners International, 2001).

Studies of benefits associated with the construction of multi-purpose reefs at various locations around the world have all shown significant benefit/cost ratios. The lowest being approximately 20:1 for a small reef in Bournemouth, UK (Black *et al*., 2000), to over 60:1 for the Narrowneck reef on the Gold Coast, Australia (Raybould and Mules, 1998) (detailed in Section 1.8 below). A recent report for a multi-purpose reef in Wellington, estimated a "very conservative benefit:cost ratio of 24:1" (Baily and Lyons, 2003). Table 5.1 summarises the findings of the studies described above.

Table 5.1. Summary of the economic benefits that multi-purpose reefs can provide

Site	Cost:Benefit Ratio	Annual Spend/Value	Surfing Competitions	Reference
*Gold Coast, Australia	1:70	-	AU$2.2M	Raybould and Mules, 1998; McGrath, 2002
†Mount Maunganui, New Zealand	-	NZ$0.5M	-	Gough, 1998
‡Cornwall, England	-	£21M	-	Ove Arup & Partners International, 2001
†Noosa Beach, Australia	-	-	AU$1M	Jackson *et al,* 1999
‡Florida, USA	-	US$84.63M		Johns *et al.,* 2001
†Lyall Bay, New Zealand	1:24	-	-	Baily and Lyons, 2003
†Bournemouth, UK	1:20	-	-	Black *et al*., 2000
**Miami Beach, USA	1:500	-	-	Houston, 2002
§Californian's Beaches	-	US$5.5B	-	King *et al*., 2001

*Based on the 'beach' amenity and associated businesses
†Based on additional income from attracting surfers
‡Based on revenue from all sources associated (e.g. hospitality, boat sales, equipment rental, etc.)
**This figure relates to the economic benefits of beach renourishment in Miami (i.e. is not associated with artificial reefs, although they can be used to greatly increase the success of renourishment).
§This is not an economic impact estimate of artificial reefs, but rather an estimate of the loss of GNP if beaches are not maintained in California, i.e. the present economic value of beaches in California.

5.5 Navigation

The presence of the reef systems will obviously impact on nearshore navigation. However, it is expected that these effects can be effectively mitigated through public education and awareness and appropriate marking. Since the reefs will be very shallow to nearly exposed at low tides, they can become a navigational hazard on calm days when boaters may be present. Therefore, the reefs should be marked with adequate lighted buoys to prevent collisions between water craft and the reefs. This will include notices at local boat ramps/marinas and gazetting of the reefs to the nautical charts. Similar measures have been applied at the Mount and in Boscombe.

Local knowledge suggests that the nearshore area of Orewa Beach is not highly utilised by boat traffic (pers. comm.). Because Orewa Beach is used by both water-skiers and Personal Water Craft (PWC, i.e. 'jet ski' or 'wet bike') enthusiasts, a 'ski lane' approximately 200 m wide and marked with buoys can be established between reef groups. This will provide adequate access to the beach for picking up and dropping off skiers and boat riders. In fact, the existing ski lane is already located between two reef systems in the centre of Orewa Beach, as shown by the white lines in Figure 5.9. Alternatively two access lanes can be established (Figure 5.9).

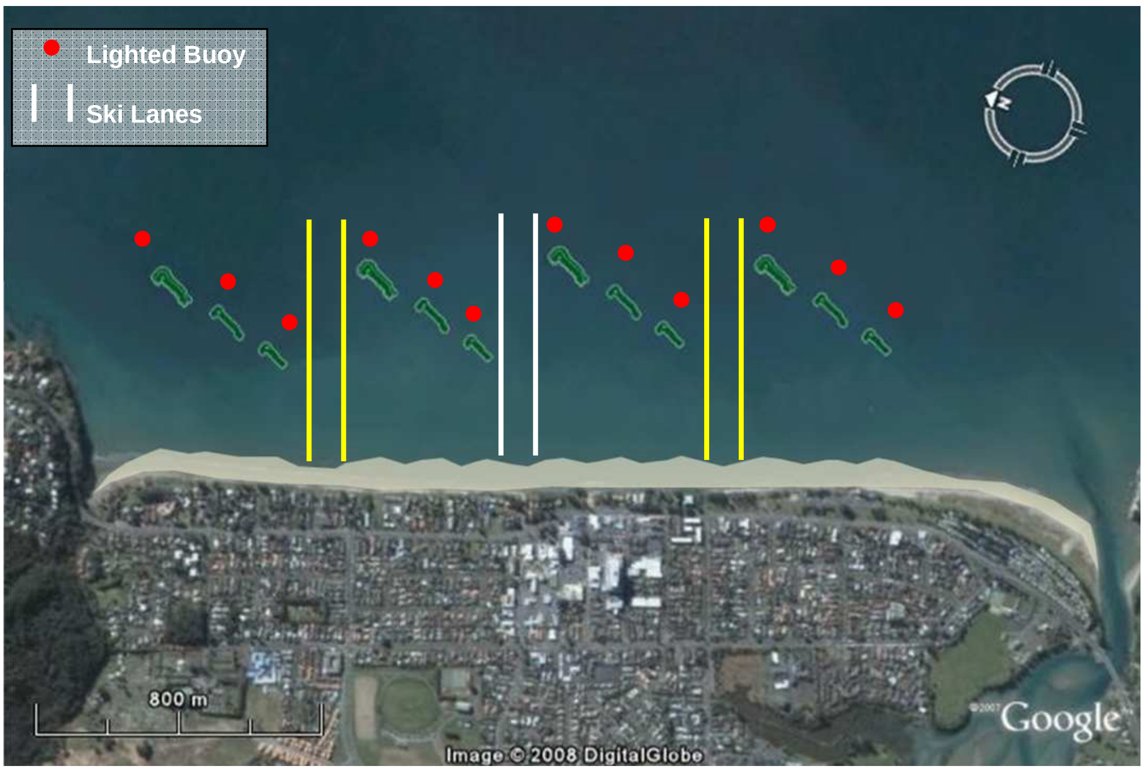

Figure 5.9, Possible configuration of marker buoys and beach access lanes for boats and PWC's. The white lines indicate the location of the existing ski lane, whiler double lanes (yellow) could be established.

To date, discussions have been undertaken with the Harbour Master and RDC, and similar measures have been undertaken as those applied at Lyall Bay, where user groups include 2x SLSC's, an existing surfing break (The Corner), and a 200 m wide wetbike/jetski lane within the <1 km long bay. Lighted buoys and notification (via signage at local boat launching sites) and addition of the reefs to the navigation charts are proposed. The location of the existing ski lane is between reef systems 2 and 3, i.e. there is no change to this, and there is space available for more, noting that this predisposes the outcome of monitoring of the first reef system. Previously, issues raised following Resource Consent notification have been successfully addressed through pre-hearing meetings with the effected parties, the Harbour Master, the applicants and the planners. Thus, while mitigation measures have been proposed in this technical report, it is expected that following notification any effected parties will have the opportunity to respond and be heard or have issues addressed prior to the hearing.

Of importance with respect to swimmer safety is the offshore directed current flows between units of the reef system during storm activity. While these currents are relatively small during normal conditions (<0.3 m/s), during larger wave events (i.e. greater than 2 m), these currents increase in intensity (Figure 5.3). Like the wider beach in general, which develops series of rip-cells during storm events, these areas should be avoided during storms, unless you are an experienced surfer or wind-surfer/kite-boarder.

5.6 Project Performance Monitoring Programme

In order to assess the performance of the Orewa Reef and beach nourishment project, a robust and comprehensive monitoring programme is proposed. The reef and beach nourishment will be monitored for performance in terms of coastal protection, biological responses, and recreational amenities. The monitoring will commence with the construction of the first reef system and should continue up to three years past the completion of construction. This monitoring will not only provide a solid basis for decisions regarding future works at Orewa, but will also provide valuable information for continued research into shore protection strategies involving submerged offshore reefs.

5.6.1 Seabed and Beach Monitoring

The bathymetry of the area and the beach profile of the seashore and foredunes have been, and are continuing to be, measured monitored. These surveys, as well as surveys of the reef shape, should continue for a minimum of 3 years beyond the final reef construction. The time intervals between surveys and further survey details are set out below. These monitoring data will provide information on both the effects of the reef system(s) and an indication of the requirements for future renourishment. As is the case world-wide, beach nourishment (which is designed to fail, i.e. put sand on the beach and once it's been washed away, replace it) are being coupled with structures and dune stabilization to greatly lengthen the duration of the nourish material. At Orewa Beach, nourish material currently has a relatively short residence time, especially when the wave climate is taken into consideration. The presence of the reef systems and appropriate plantings will greatly increase the residence time of any nourishment material, making the solution more sustainable in the long-term, both environmentally and economically.

MINIMUM ANNUAL MONITORING REQUIREMENTS

- 2 full bathymetry surveys at approximately 6 month intervals, depending on local sea conditions.
- Quarterly (4x/yr) beach profiles at established profile sites + 5 new sites established along the beach in the lee of the reef system (i.e. at approximately 100 m intervals) Additional profiles should be measured after significant storm events.
- Reef diving inspections for stability and local scour will be carried out during bathymetry surveys, as well as after exceptional storm events.
- SEO or LEO observations as often as possible (daily to once weekly) or during any time when there are a significant number of recreational users.

5.6.2 Biological Monitoring

Since the biological impacts are considered mainly positive, biological monitoring is not recommended as a condition of resource consent, but would be useful for educational purposes. Using the biological survey described in this report and in the report “ASSESSMENT OF ECOLOGICAL EFFECTS OF MULTI-PURPOSE REEFS FOR BEACH SAND RETENTION AT OREWA BEACH” as a baseline, annual surveys could be conducted at the Orewa Reefs – ideally at the same time of year as the initial survey. Surveys would cover the area previously surveyed and follow a similar methodology. Additional sites on and near the reef will also be incorporated into the annual surveys to investigate the settlement of organisms onto the reef and into the sediment around the artificial reef.

5.6.3 Reef Performance in Terms of Recreational Amenities

The proposed Orewa Reefs are called ‘Multipurpose Reefs’ because they are designed to provide multiple benefits after construction, This multipurpose designation extends to the recreational aspect of the reef. In addition to surfers, other groups who could potentially benefit from the recreational amenities provided by the reefs include swimmers, kayakers, snorkelers, kite surfers or windsurfers.

For the quantitative assessment of surfing wave quality, the best measure is based on visual observations from other surfers. For this reason it is important to establish a core group of volunteer observers who are willing to give daily reports on the wave quality and number of users on the reef structures. The surf quality monitoring programmes employed at previously constructed surfing reefs (i.e. Cables Station near Perth, West Australia, Narrowneck Reef on Australia's Gold Coast and Pratte's Reef in Los Angeles) have each used different methods to assess the surfing wave quality. Both Cables and Narrowneck (and more recently Mount) have employed automated videographic techniques to quantify the number of days that the reef have caused waves to break (see Jackson et al., 2007 and Pattiaratchi, 2007). While this does have some value it is generally a poor indicator of surfing wave quality. Borrero and Nelsen (2003) used a monitoring program based on direct observation by volunteers. For this method, the Surf Environment Observation (SEO) form was used (Figure 5.10). This direct observation method used at the reef site and at a control site (with no reef) would likely provide the best information to quantify the surfing wave quality at the proposed reef.

SURFING ENVIRONMENT OBSERAVATION FORM

OBSERVER

STATION ID | YEAR | MONTH | DATE | TIME (24 Hour Syst.)

WIND SPEED (Please Circle): Calm Medium Hard

WIND DIRECTION: Onshore Offshore From South From North

DROGUE DISTANCE (Feet)

DIRECTION OF DROGUE TRAVEL: To Right To Left Offshore Inshore

NUMBER OF SURFERS

TIME FOR 11 WAVE CRESTS TO PASS

		PEEL DIRECTION	
		RIGHT	LEFT
WAVE HEIGHT (FT)	INSIDE		
	OUTSIDE		
WAVE TYPE	INSIDE	SPILLING PLUNGING COLLAPSING SURGING	SPILLING PLUNGING COLLAPSING SURGING
	OUTSIDE	SPILLING PLUNGING COLLAPSING SURGING	SPILLING PLUNGING COLLAPSING SURGING
RIDE LENGTH	INSIDE		
	OUTSIDE		
RIDE TIME	INSIDE		
	OUTSIDE		

COMMENTS

Figure 5.10, The Surf Environment Observation (SEO) form.

To assess the reef performance for other recreational users, similar methods can be established. However this will require a committed effort by volunteers to provide observational data at every opportunity.

Monitoring reports and data will be submitted to the consenting authority for evaluation.

5.7 Summary

The overall effects of the Orewa Reef on the environment are described in the preceding chapter. This includes effects on the coastal landscape, waves, currents, tides and sea levels, seabed and shoreline stability, recreational amenities, the local economy, and nearshore navigation. A comprehensive monitoring programme to assess the effectiveness of the reefs is also described. This includes the recommended interval for offshore surveys, beach profiling and observational surveys. These monitoring data will provide information on both the effects of the reef system(s) and an indication of the requirements for future renourishment. In general the effects are believed to be largely beneficial – after all that is the intent of the project. These will be manifested in the form of a wider beach which requires less beach nourishment at greater time intervals to remain stable. Because there will be more dry beach space as a result of the nourishment programme, economic studies suggest a substantial return on investment for this newly placed sand due to increased beach usage. Recreationally, the reefs will provide additional amenities in terms of improved surfing under the right swell, wind and tide conditions as well as an additional amenity for kite boarders and wind surfers. SCUBA divers, snorkelers and ocean kayakers are also expected to benefit due to the presence of several focus points (i.e. the reefs) which can be included in various outings related to those sports. A list of the various activities and the overall impact is included in Table 5.2.

Table 5.2. The expected effect of the Orewa Reefs on a variety of environmental, social and recreational factors.

Category	Effect	Duration	Comments
Coastal Landscape	Positive	Long term or permanent	Long term effect is to create a wider, more stable beach
Coastal Landscape	Negative	Short term	Short term, temporary negative effects during construction period related to visual effects, increased vehicular traffic and noise.
Waves	Positive	Long term or permanent	Wave heights are reduced in the shadow of the reef structures. This is a necessary effect for the functionality of the structures and the development of a wider beach
Currents	Positive	Long-term or permanent	Currents are effected in the close proximity of the reefs, where they are redirected shoreward (the existing condition is strong alongshore currents) and help to ensure sand is retained at the beach. As aspect of currents is the offshore directed flows between reefs during storm events; like rip-cells that occur on the beach during storm events, these can be dangerous to swimmers, however, they are at a known and permanent location, which can be managed through signage and education.
Tides and Sea levels	None	N/A	The reefs will have no effect on tides, tide currents or general sea levels.
Shoreline Stability	Positive	Long term or permanent	The purpose of the reef structures is to stabilise the beach through wave energy dissipation and modification of longshore currents.
Recreation (overall)	Positive	Long term or permanent	The reefs will provide a variety of recreational outlets and opportunities depending on the weather and swell conditions of the day.
Surfing	Positive	Long term or permanent	The reefs were designed to improve surfing conditions under typical swell conditions. Smaller waves will break with more intensity and in a consistent location. Peel angles will be increased providing a more rideable wave during larger swells.
Kite and Windsurfing	Positive	Long term or permanent	The reefs an provide a wave jumping or wave riding obstacle for participants of wind driven sports.
Snorkelling/Swimming/SCUBA	Positive	Long term or permanent	The reefs themselves will provide a point of interest for snokelers, swimmers or SCUBA divers. Sea life attracted to the reef will also be present and of interest to recreational users.
Sea Kayaking	Positive	Long term or permanent	The reefs will forma a point of interest and a destination for kayakers.
Navigation	Negative	Permanent	The reefs will become a permanent fixture in the nearshore environment presenting the same hazards as any submerged natural reef. Education and awareness campaigns and inclusion in navigational charts will be required to mitigate this hazard. Lighted buoys should be installed and properly maintained to mark the location of the reefs. Ski lanes can be established to allow boat and PWC traffic to approach the beach for passenger pick up and drop off..

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APPENDIX 1

Previous Orewa Beach Reports, (1980 – 2008) In Chronological Order:

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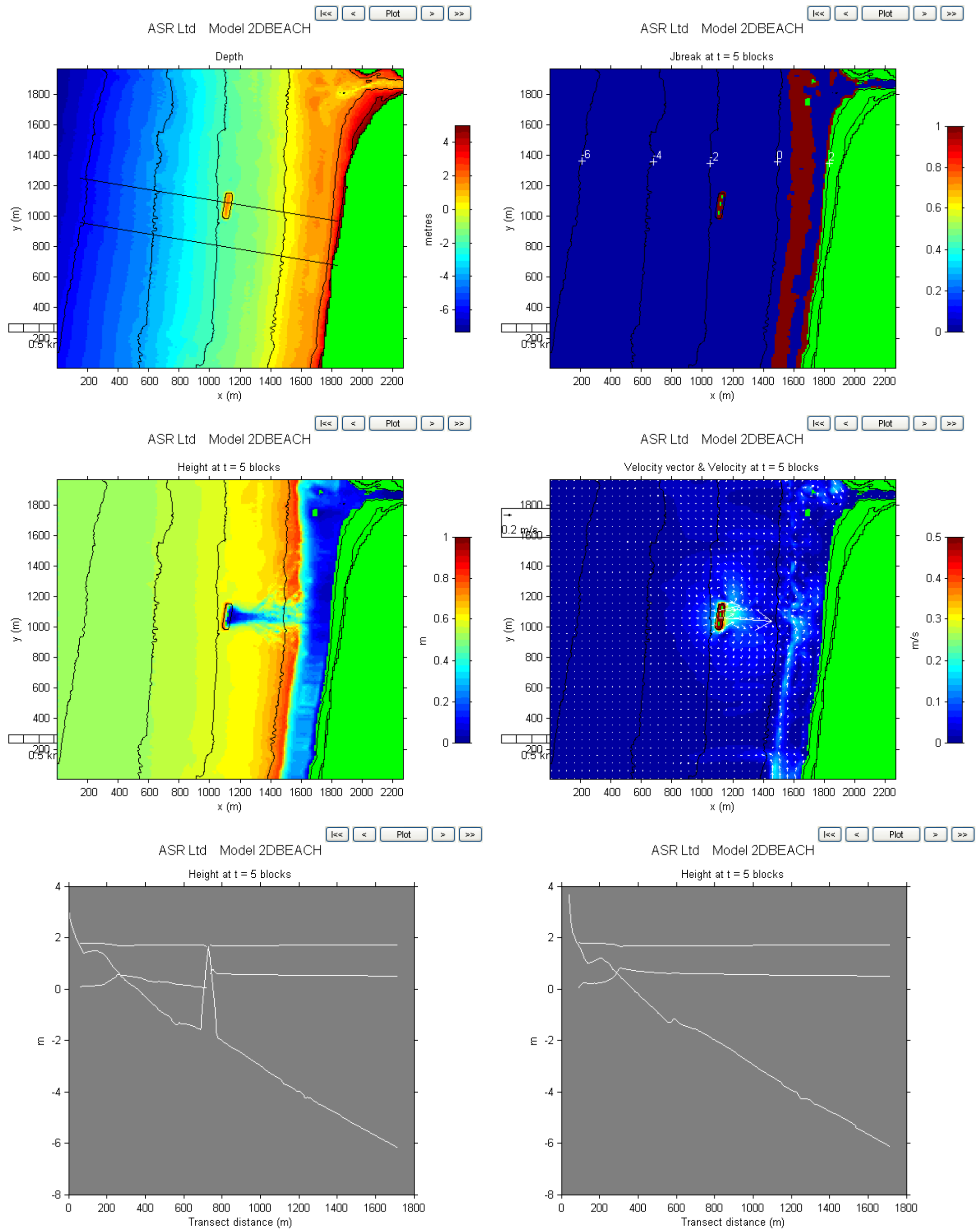
May 2006: Beca Carter Hollings and Ferner Ltd., Application for Resource Consents & Assessment of Effects on the Environment, Orewa Groyne and Beach Nourishment.

September 2006: Coastline Consultants Ltd., September 2006 Orewa Beach Storm Event: Impacts and Implications.

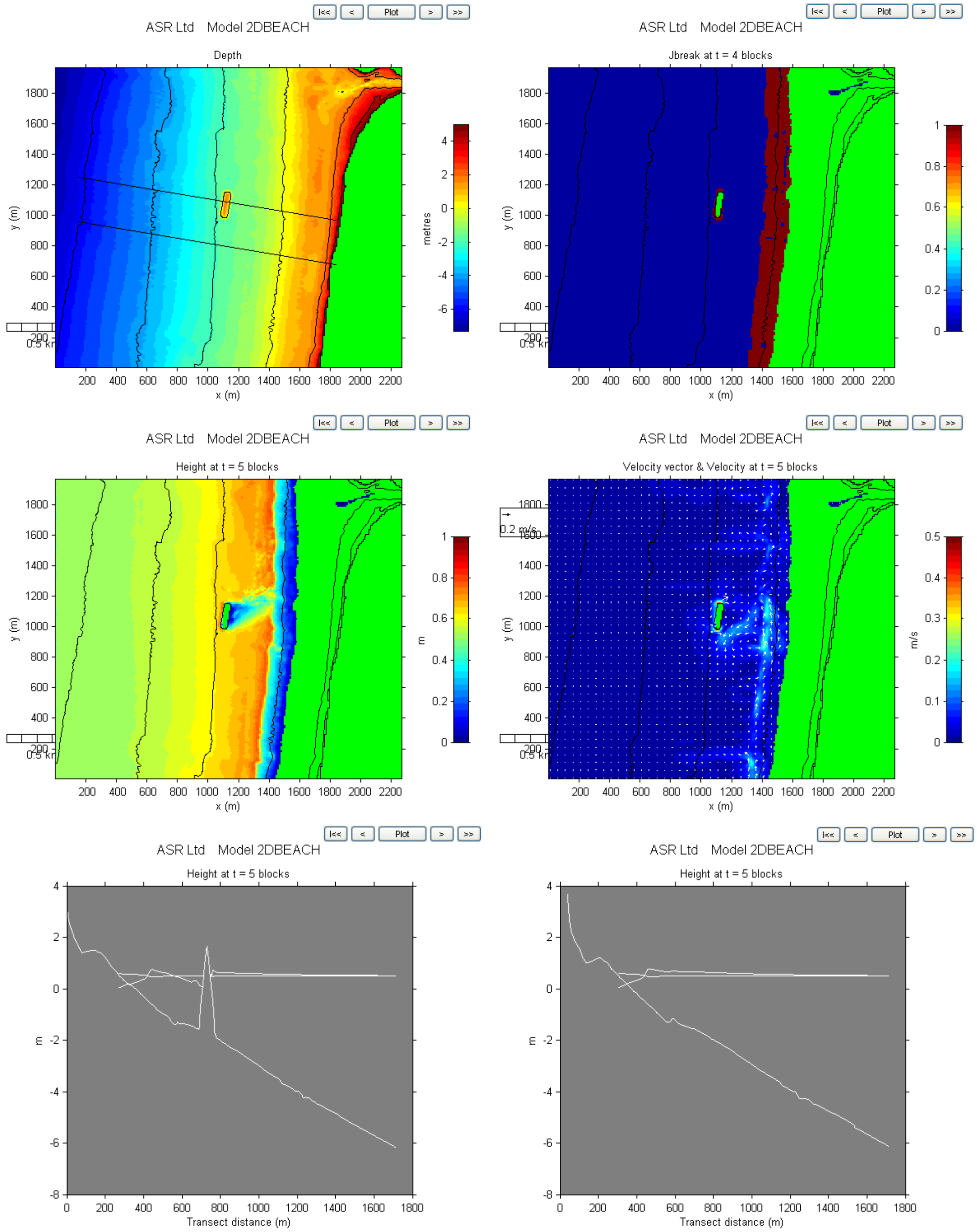
March 2008: ASR Limited: Preliminary Design for Multipurpose Reefs at Orewa. Report Prepared for Orewa Beach Reef Charitable Trust and the Rodney District Council.

APPENDIX 2

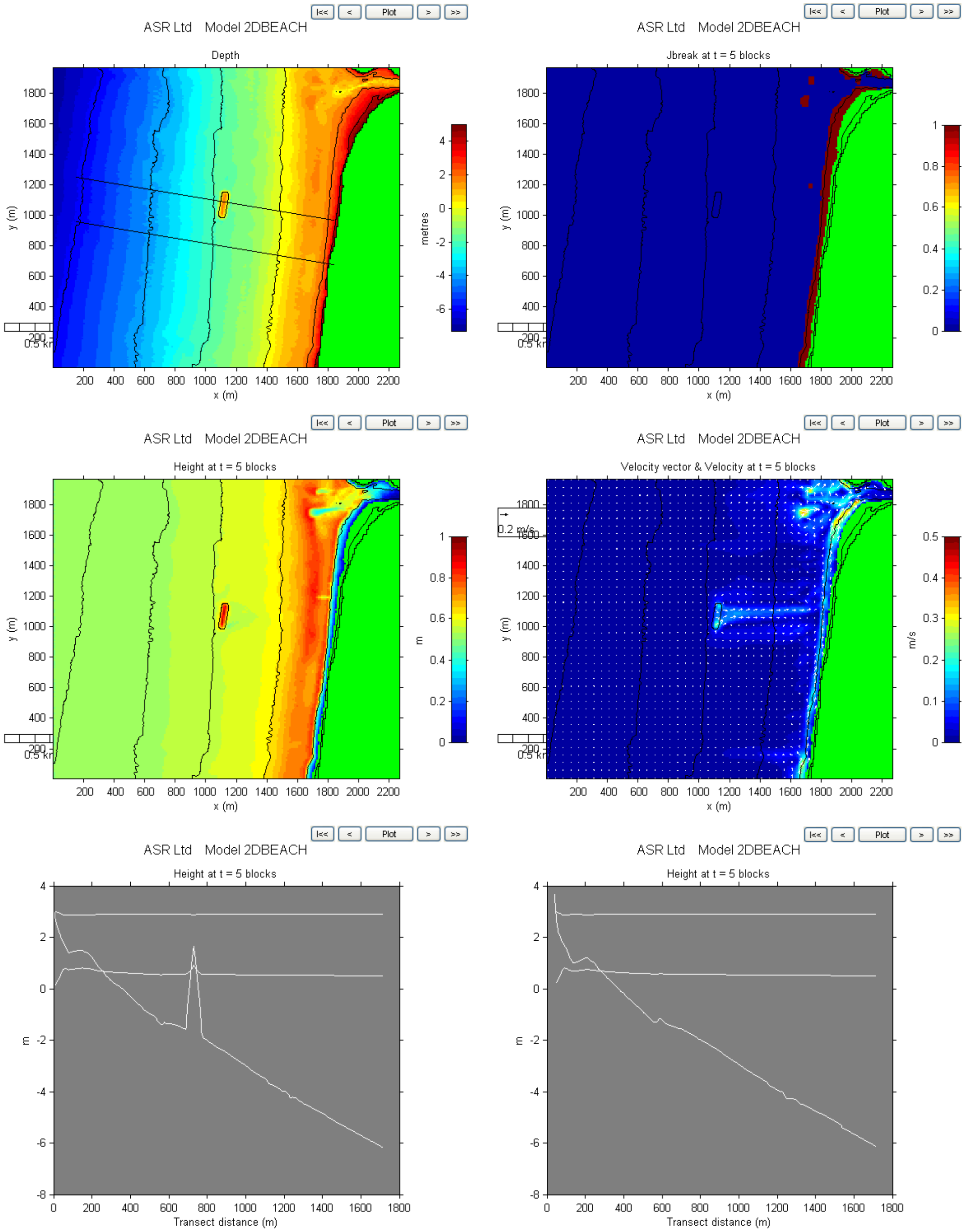
Reef 1, Case 1: H = 0.5 m, Tide = 1.7 m (MSL)



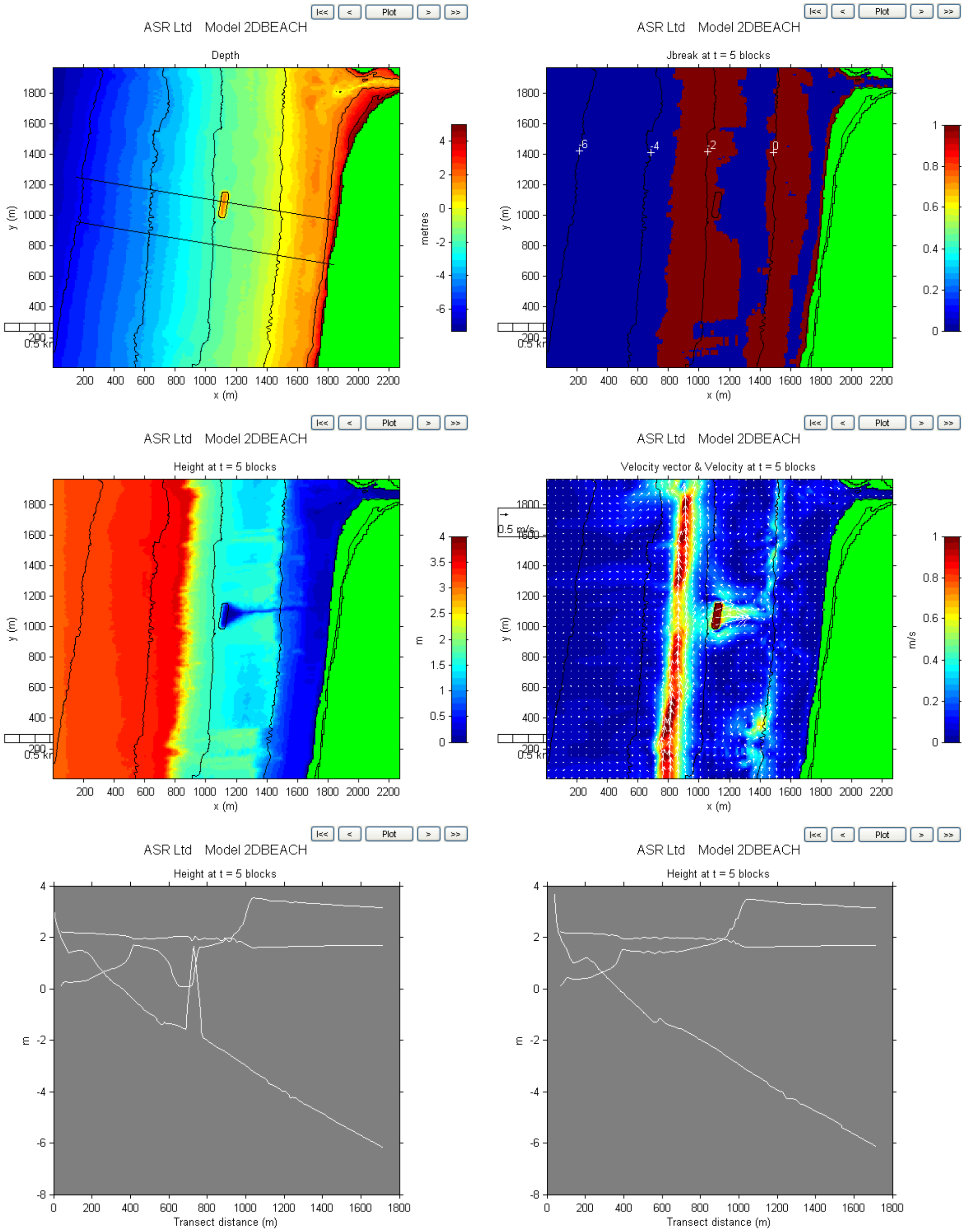
Reef 1, Case 2: H = 0.5 m, Tide = 0.5 m (MLWS)



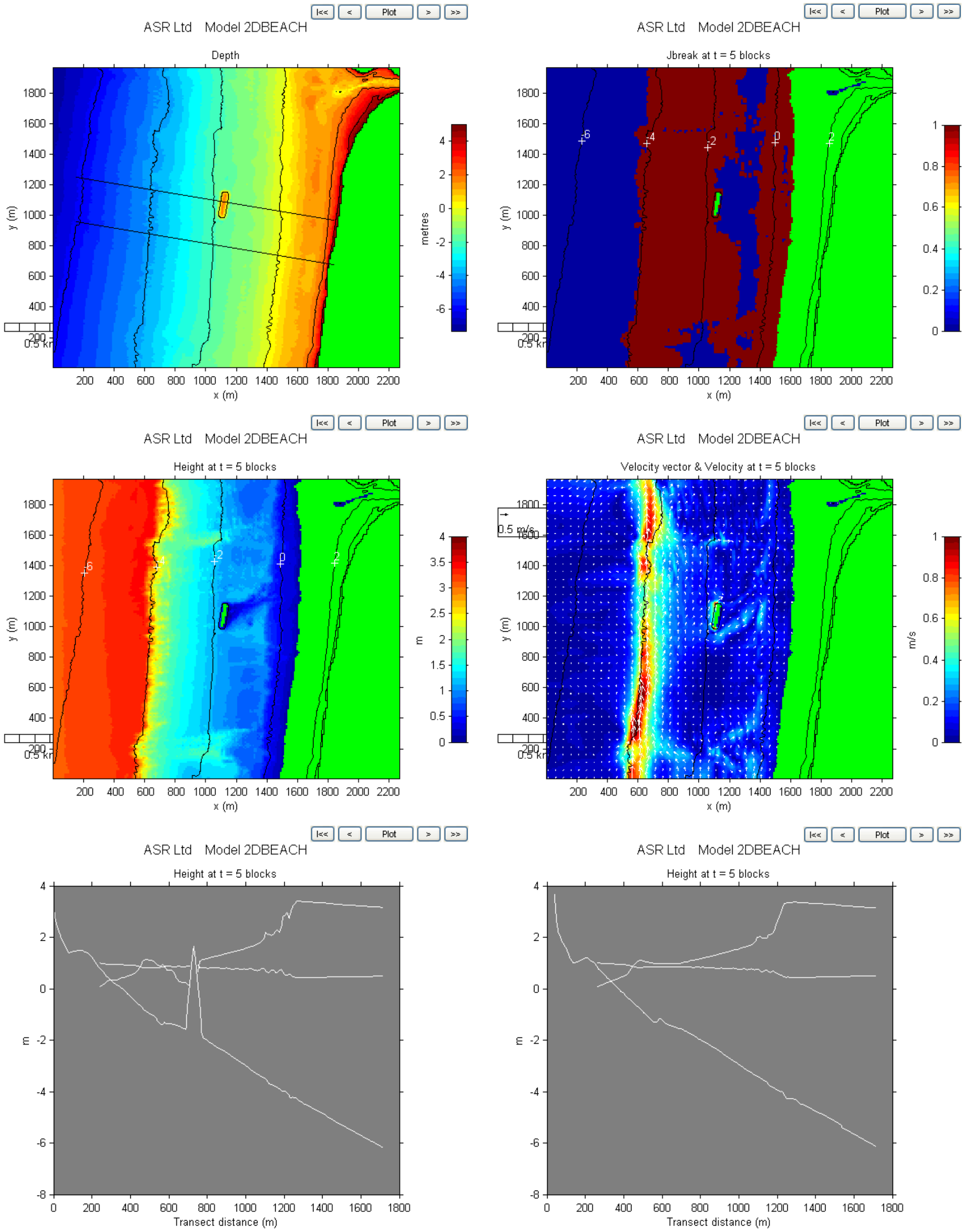
Reef 1, Case 3: H = 0.5 m, Tide = 2.9 m (MHWS)



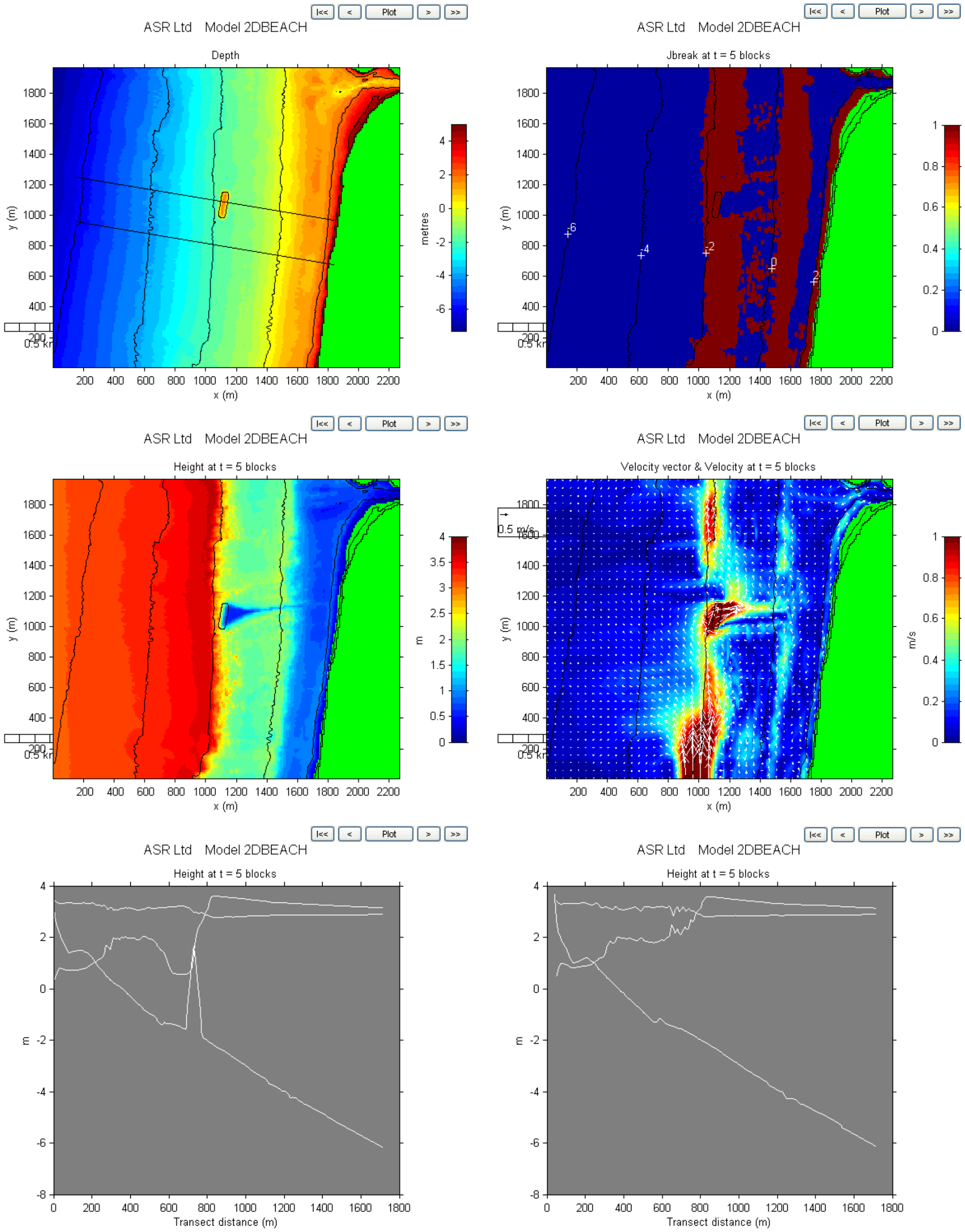
Reef 1, Case 4: H = 3.1 m, Tide = 1.7 m (MSL)



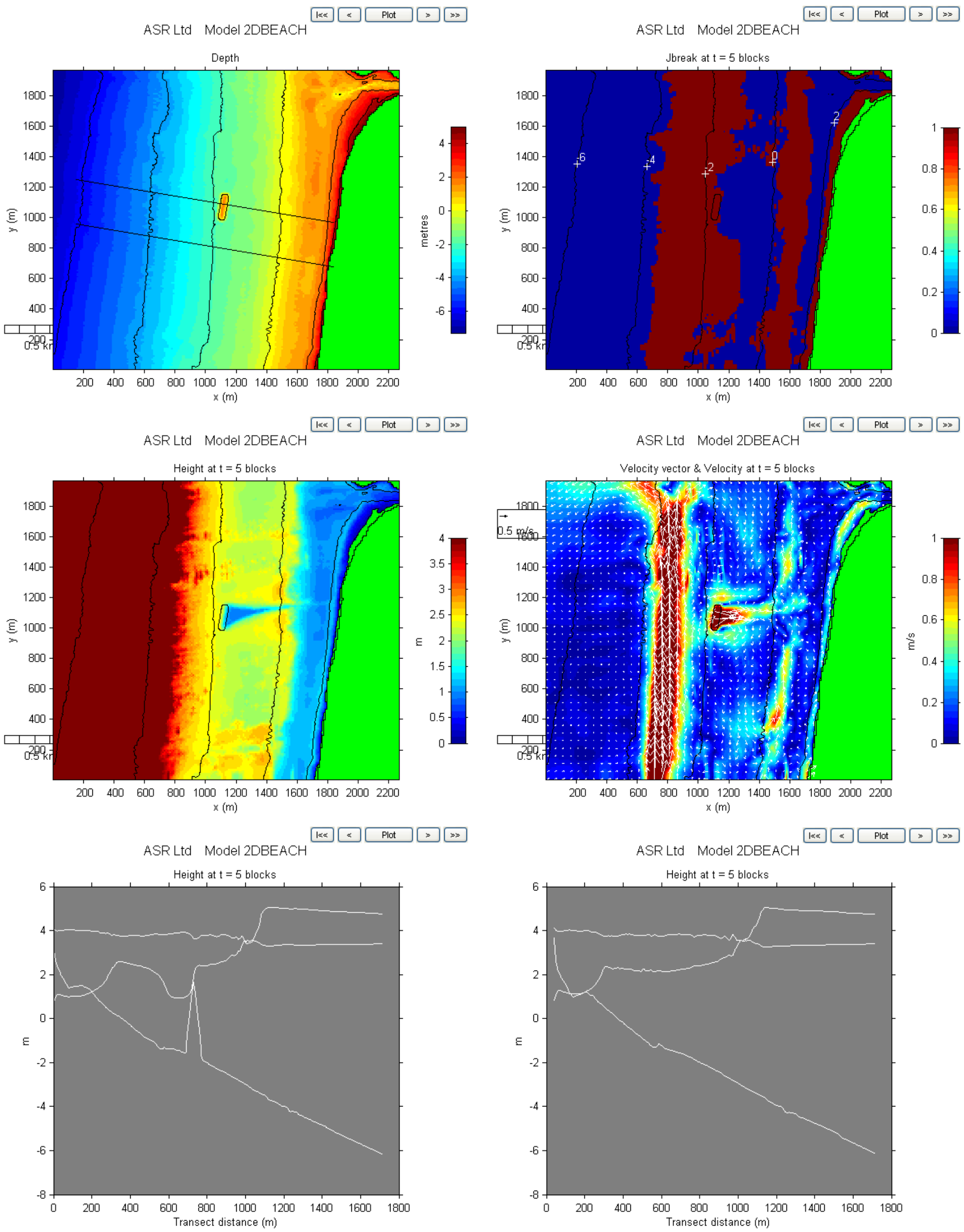
Reef 1, Case 5: H = 3.1 m, Tide = 0.5 m (MLWS)



Reef 1, Case 6: H = 3.1 m, Tide = 2.9 m (MHWS)

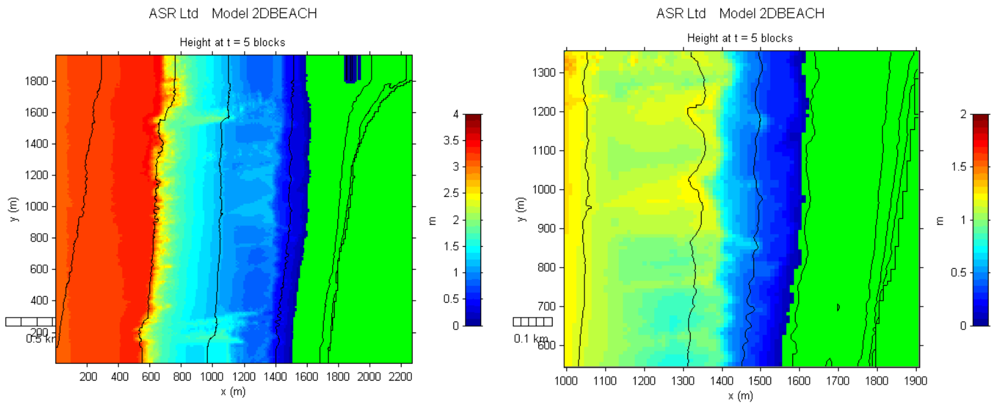


Reef 1, Case 7: H = 4.7 m, Tide = 3.4 m (MHWS+ 0.5 m Storm Surge)



APPENDIX 3

Wave Height Attenuation
No Reef, H = 3.1 m, T = 8 sec



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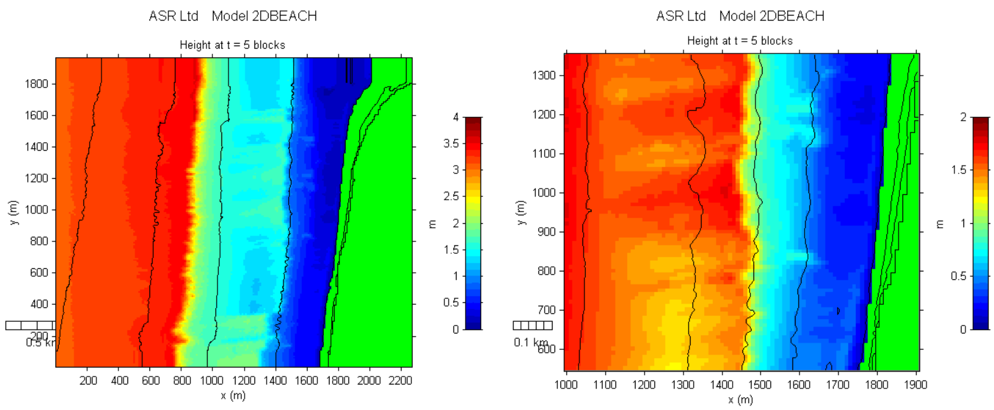

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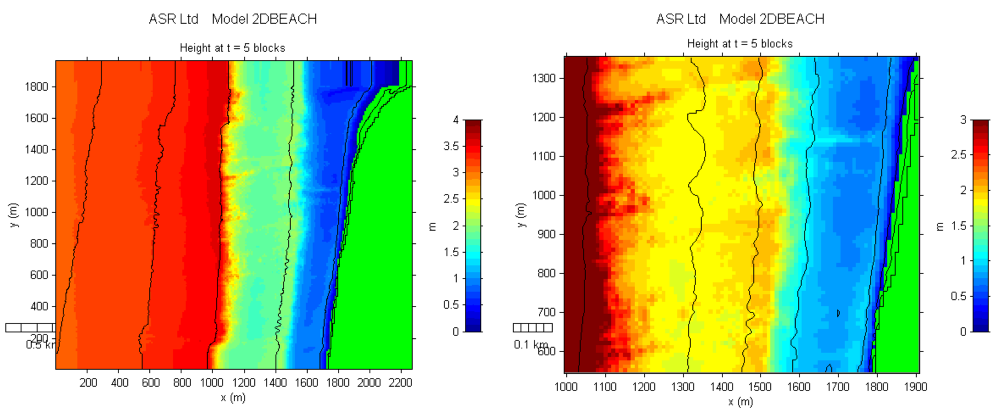
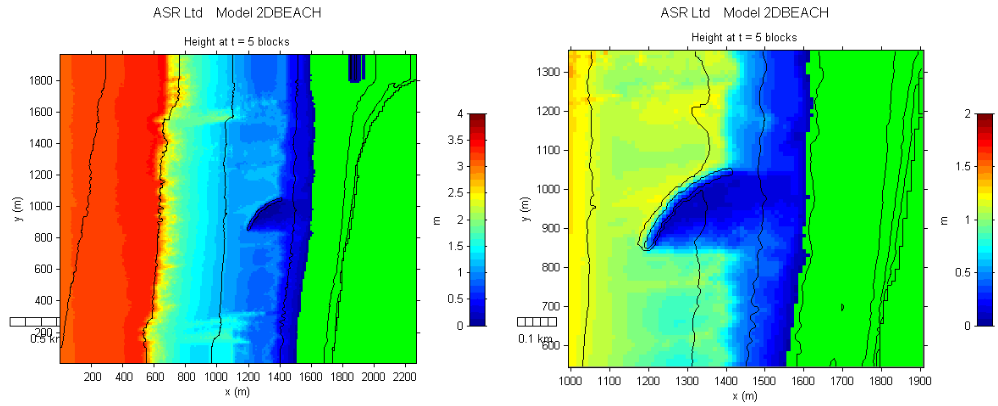


HI

Wave Height Attenuation
Option 2, H = 3.1 m, T = 8 sec



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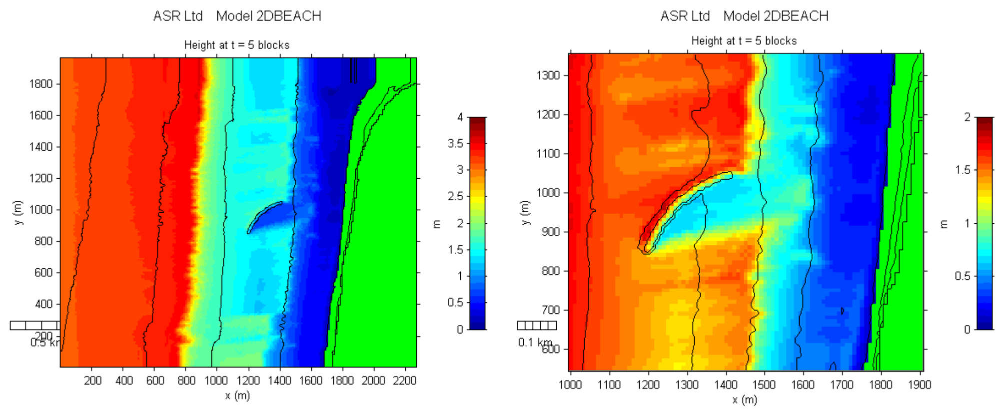

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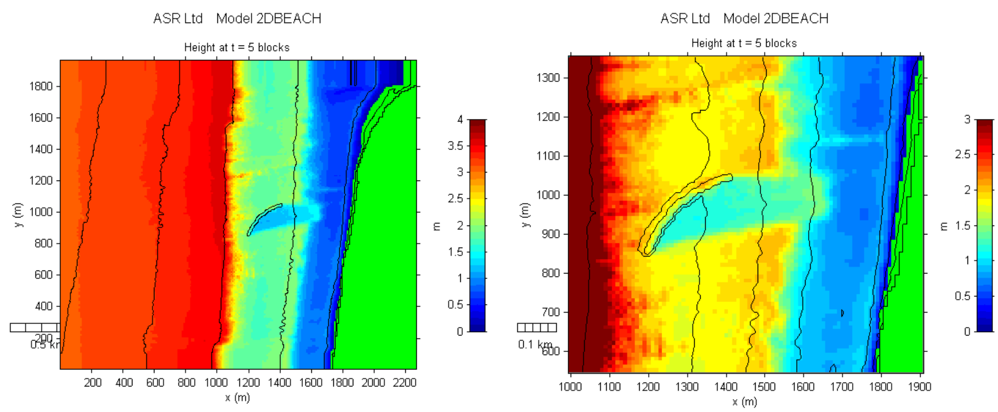
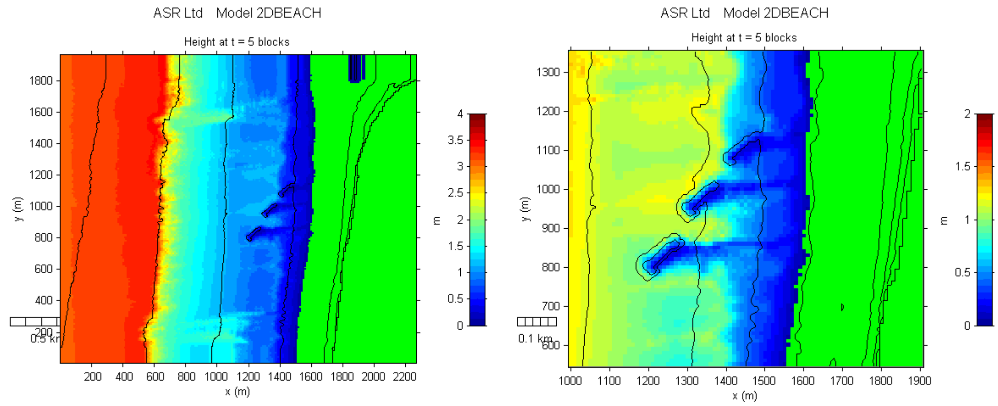


HI

Wave Height Attenuation
Option 4, H = 3.1 m, T = 8 sec



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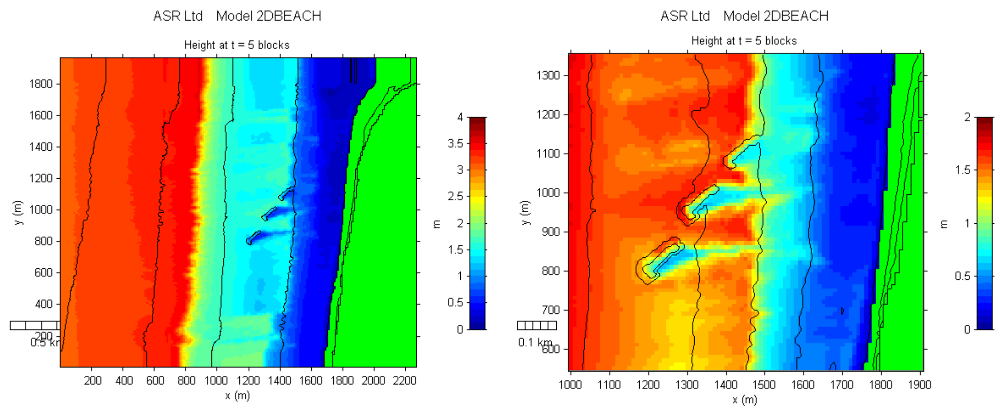

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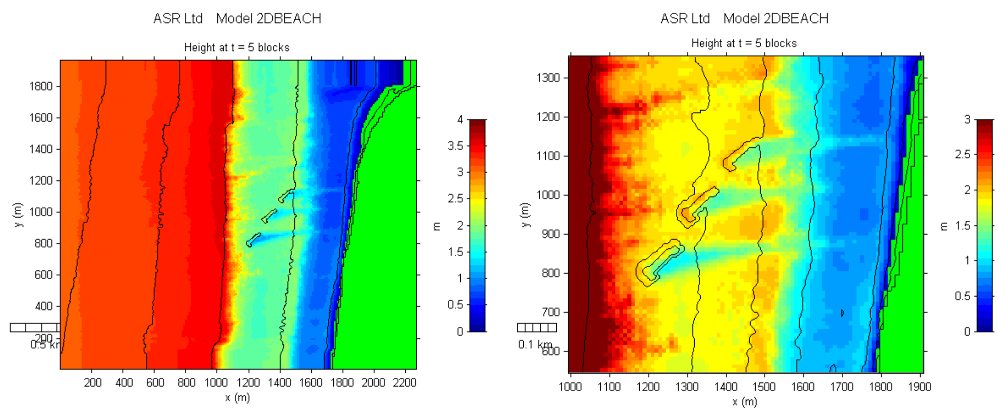
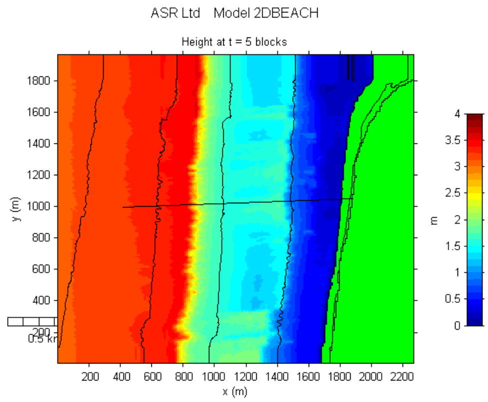
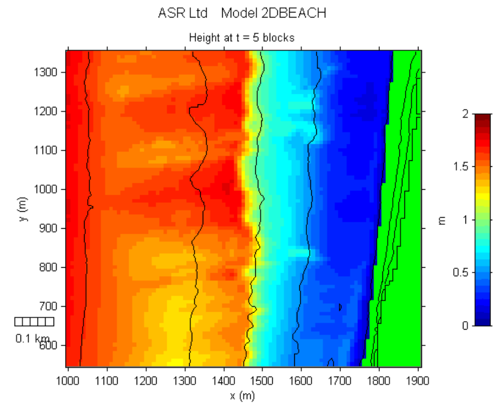


HI

Wave Height Attenuation, H = 3.1 m, T = 8 sec

No Reef





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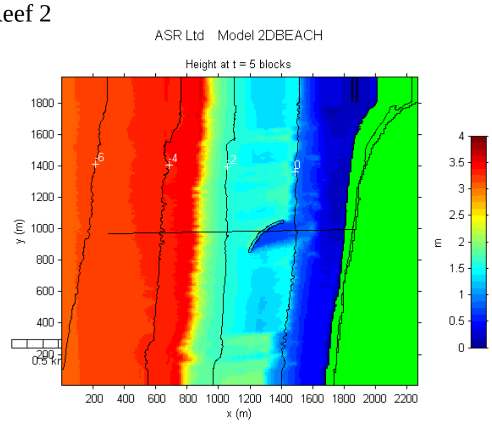
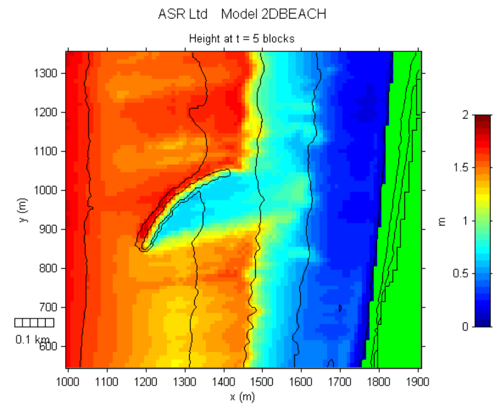
Reef 2





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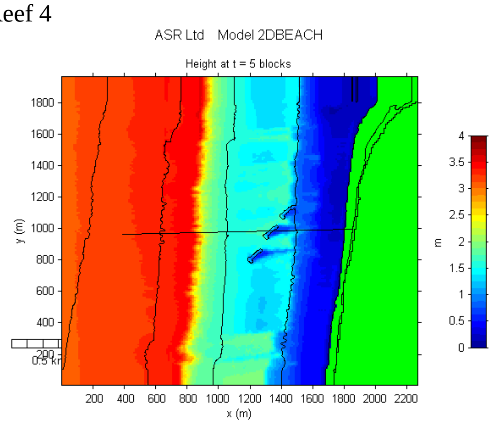
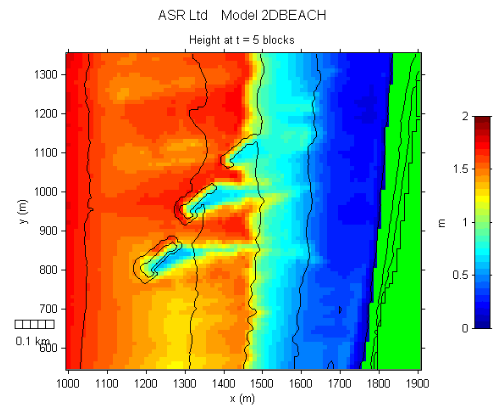
Reef 4





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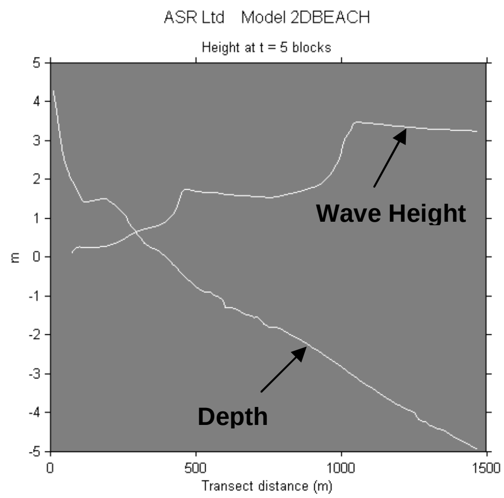
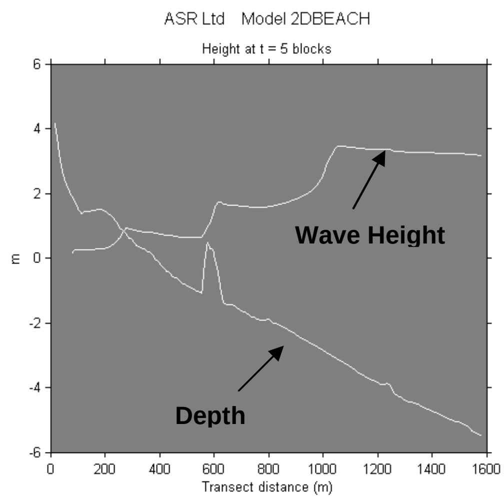


No Reef MID



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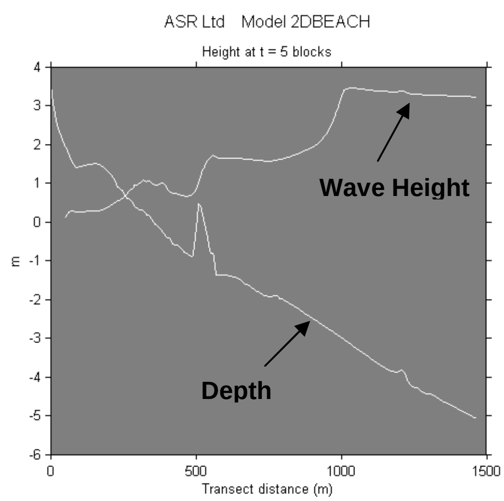
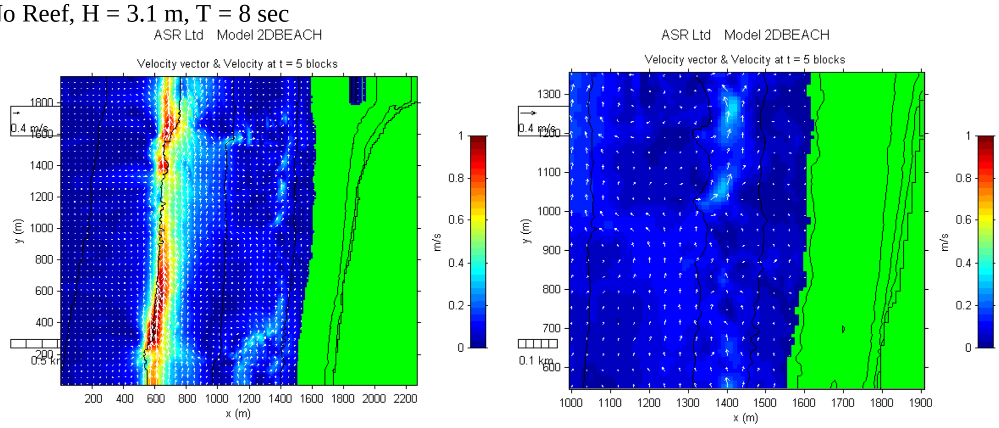


Reef 4 MID

Velocity Patterns
No Reef, H = 3.1 m, T = 8 sec



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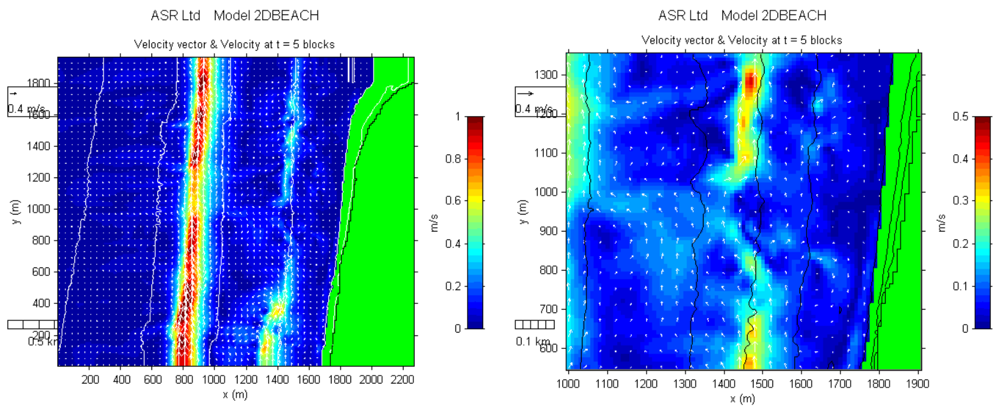

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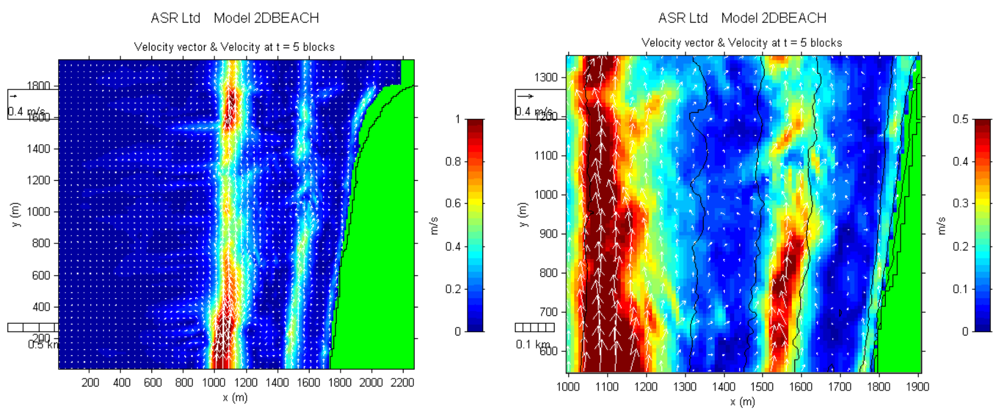
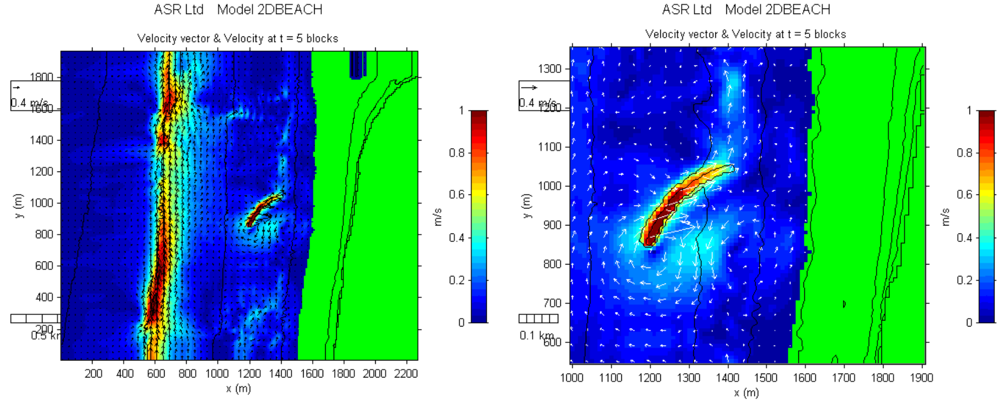


HI

Velocity Patterns
Reef 2, H = 3.1 m, T = 8 sec



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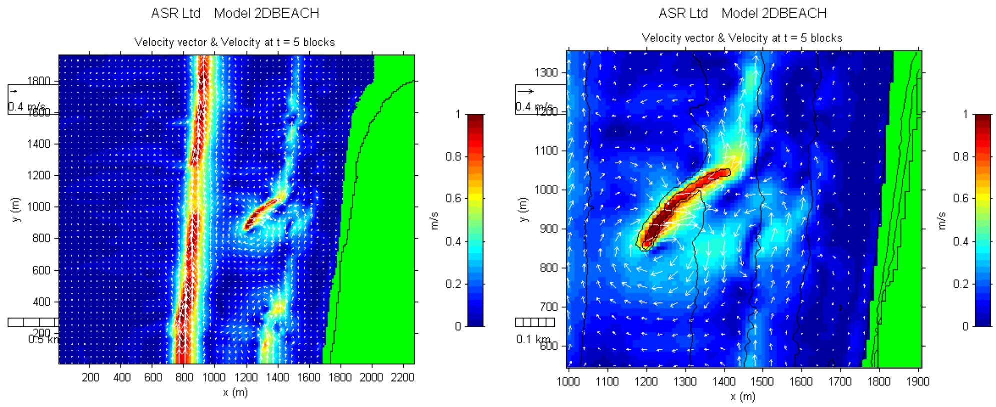

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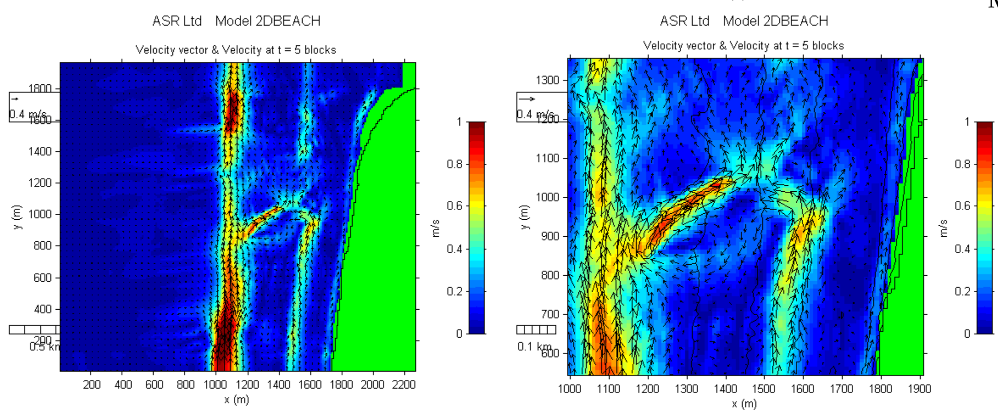

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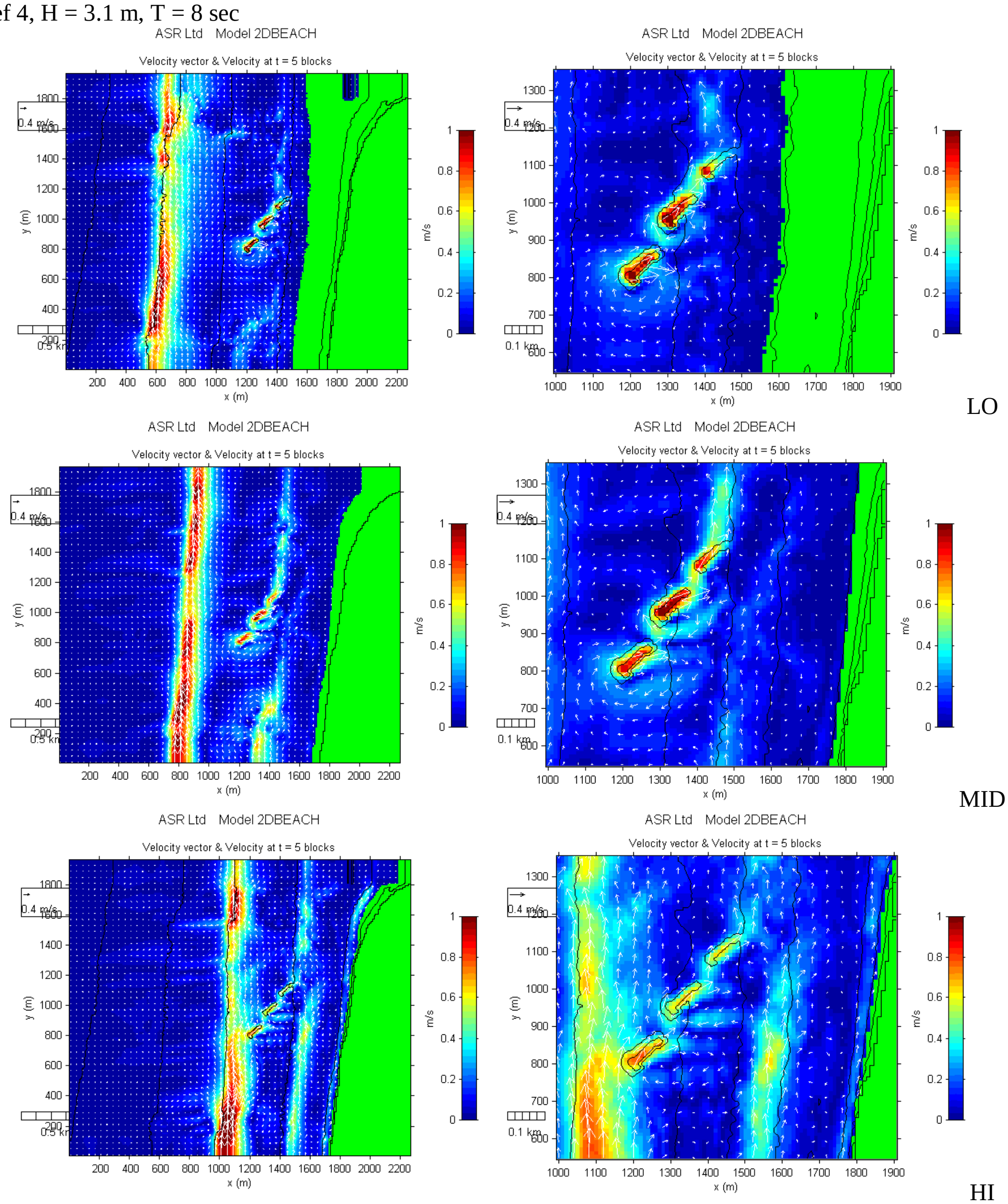
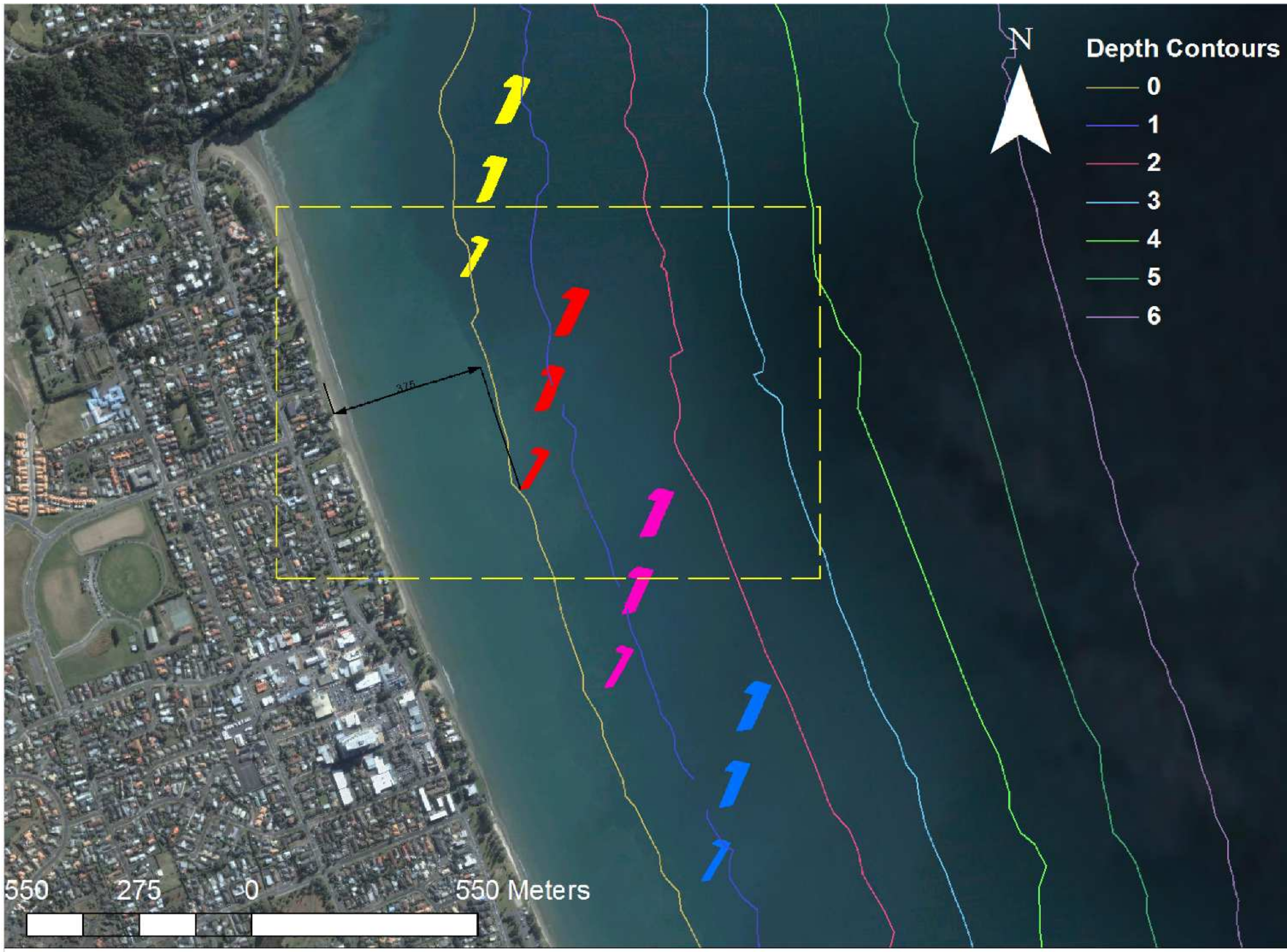
Velocity Patterns
Reef 4, H = 3.1 m, T = 8 sec



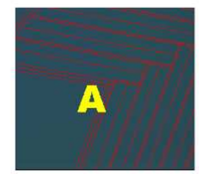
APPENDIX 4 – REEF SYSTEM LAYOUT PLAN



LEGEND

1. Datum LAT (Lowest Astronomical Tide)
2. Refer to bag layout drawings for individual reef dimensions
3. Table of reef

Easting	Northing	Reef
294133	5950019	North Big
294082	5949826	North Middle
294052	5949640	North Small
294277	5949503	Center North Big
294224	5949317	Center North Middle
294199	5949123	Center North Small
294484	5949013	Center South Big
294436	5948824	Center South Middle
294404	5948641	Center South Small
294720	5948543	South Big
294673	5948352	South Middle
294642	5948169	South Small



Reef locations refer to the apex on the inside of the reef (A).

Eastings and Northings in Universal Transverse Mercator (UTM) zone 60 south

Orewa Reefs

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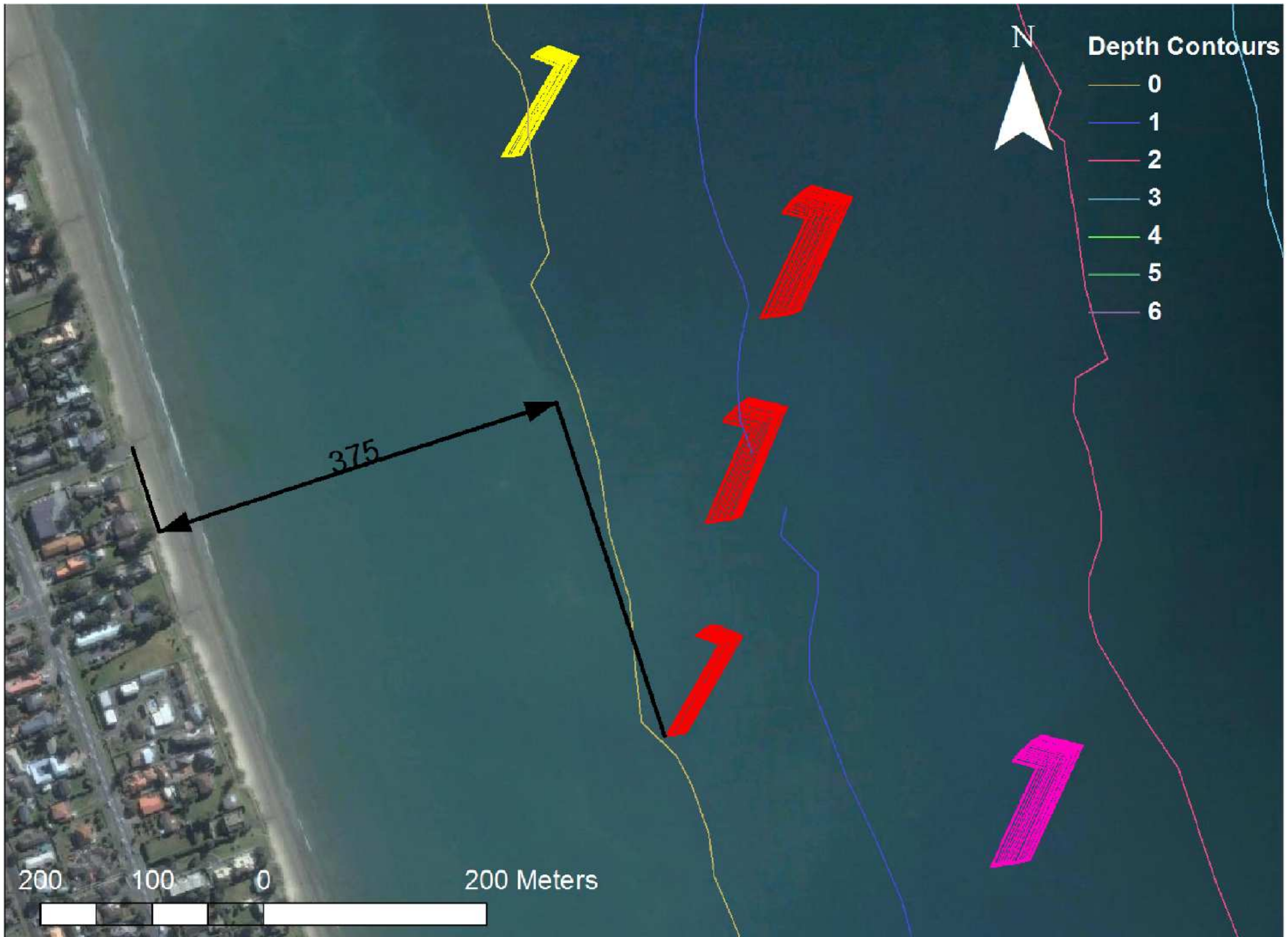
Locality map of the four reef systems for orewa beach

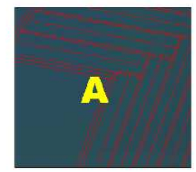
Draw No.	Date	Designed
Orewa/Loc/1.00	17/07/2009	Jose Borrero and Shaw Mead



LEGEND

1. Datum LAT (Lowest Astronomical Tide)
2. Refer to bag layout drawings for individual reef dimensions
3. Table of reef locations

Easting	Northing	Reef
294277	5949503	North Center Big
294224	5949317	North Center Middle
294199	5949123	North Center Small



Reef locations refer to the apex on the inside of the reef (A).

Eastings and Northings in Universal Transverse Mercator (UTM) zone 60 south

Center North Orewa Reefs

Locality map of first reef system to be construted

Draw No.	Date	Designed
Orewa/Loc/2.00	17/07/2009	Jose Borrero and Shaw Mead

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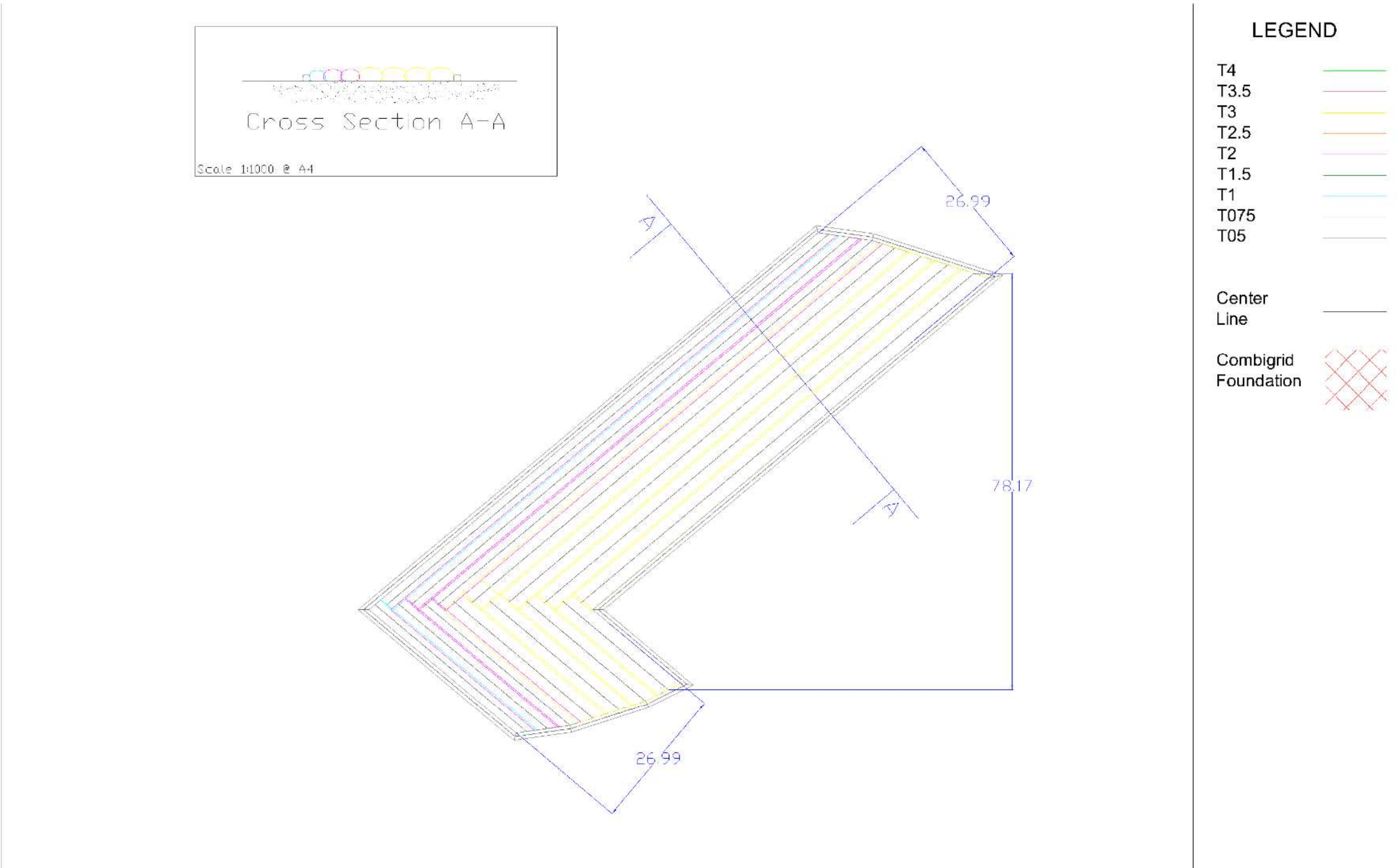

Orewa Large Reef

Bag Layout Plan

Scale 1:1000 @ A4	Draw. No. Orewa/LrPlan/1.00	Date 17/06/2009	Designed Jose Borrero and Shaw Mead

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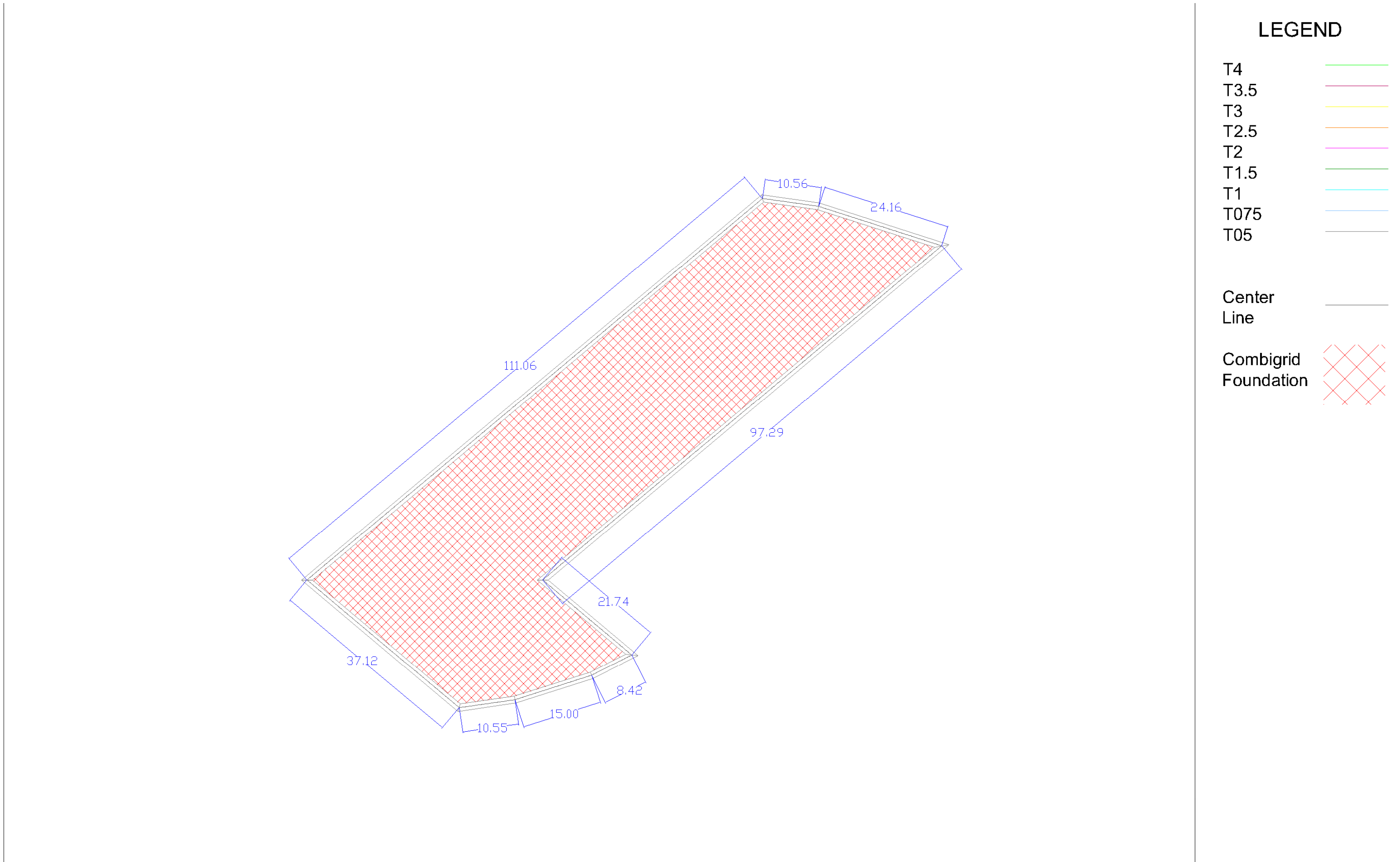

Orewa Large Reef

Foundations & Scour Bag Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/LrBase/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead	

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Orewa Large Reef

T1 Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/LrT1/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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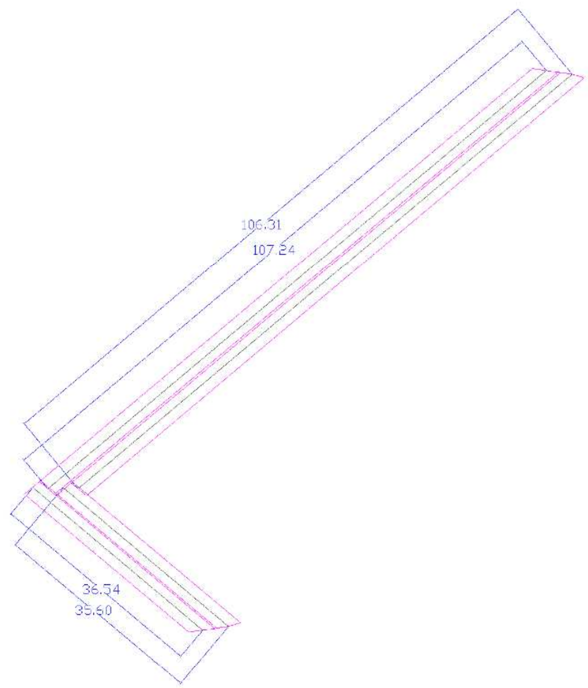

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Center Line

Combigrid Foundation

Orewa Large Reef

T2 Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/LrT2/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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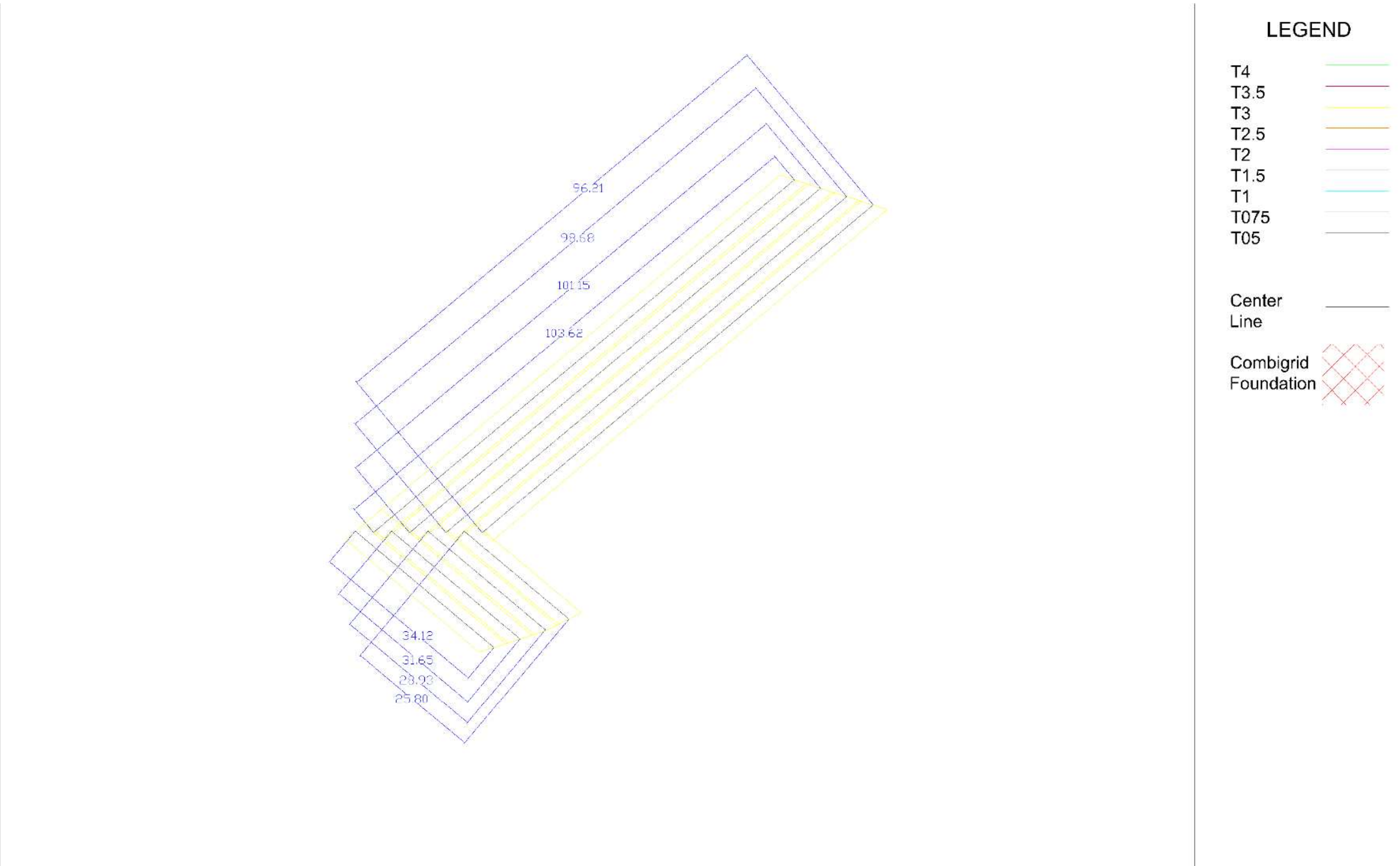

Orewa Large Reef

T3 Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/LrT3/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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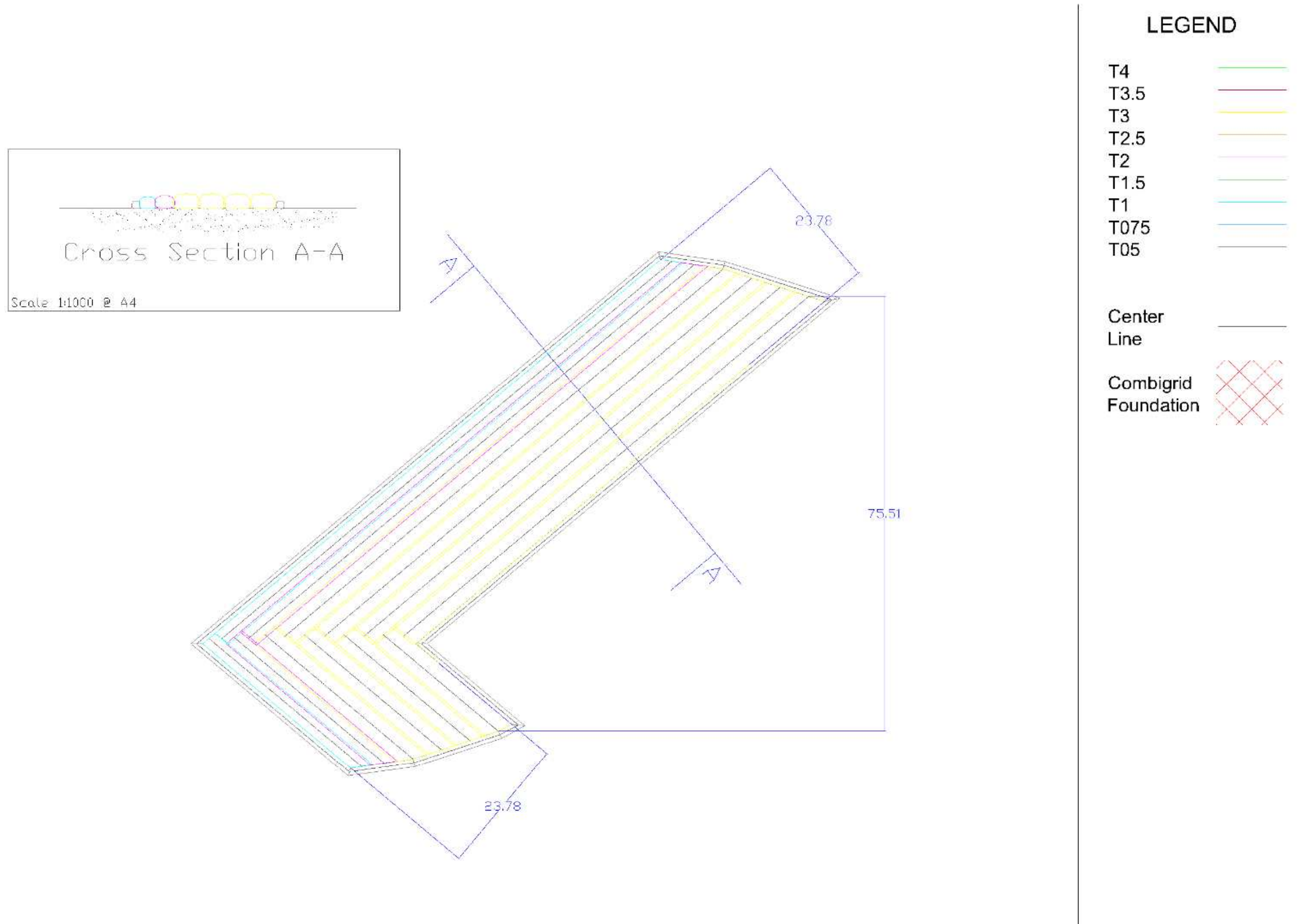

Orewa Middle Reef

Bag Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/MidPlan/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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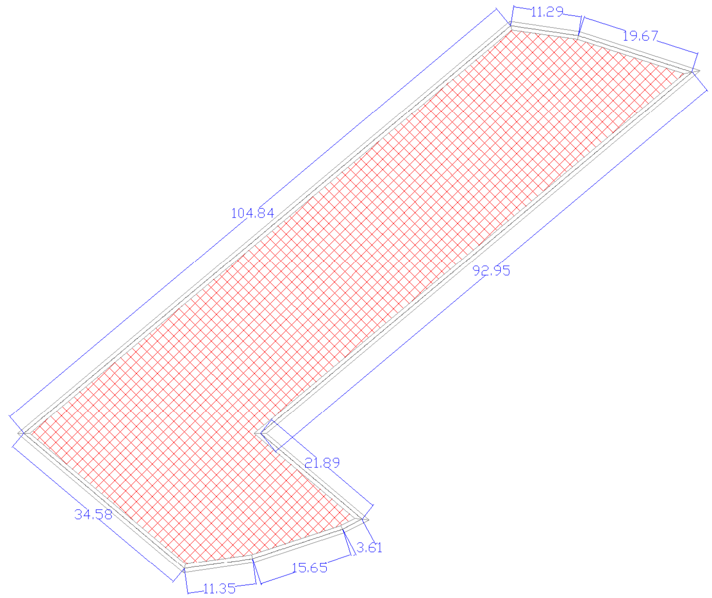

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Center Line

Combigrid Foundation

Orewa Middle Reef

Foundation & Scour Bag Layout Plan

	Draw No.	Date	Designed
Scale 1:1000 @ A4	Orewa/MidBase/1.00	17/07/2009	Jose Borrero and Shaw Mead

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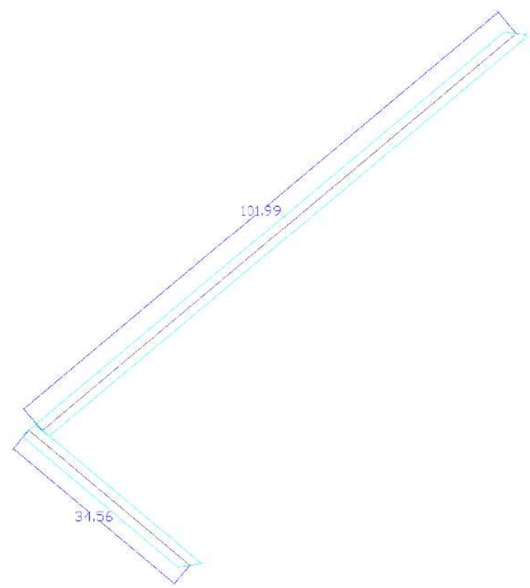

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Center Line

Combigrid Foundation

Orewa Middle Reef

T1 Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/MidT1/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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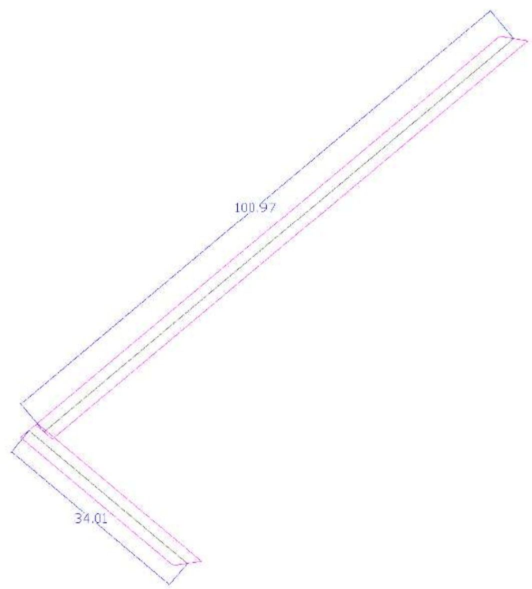

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T05

Center Line

Combigrid Foundation

Orewa Middle Reef

T2 Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/MidT2/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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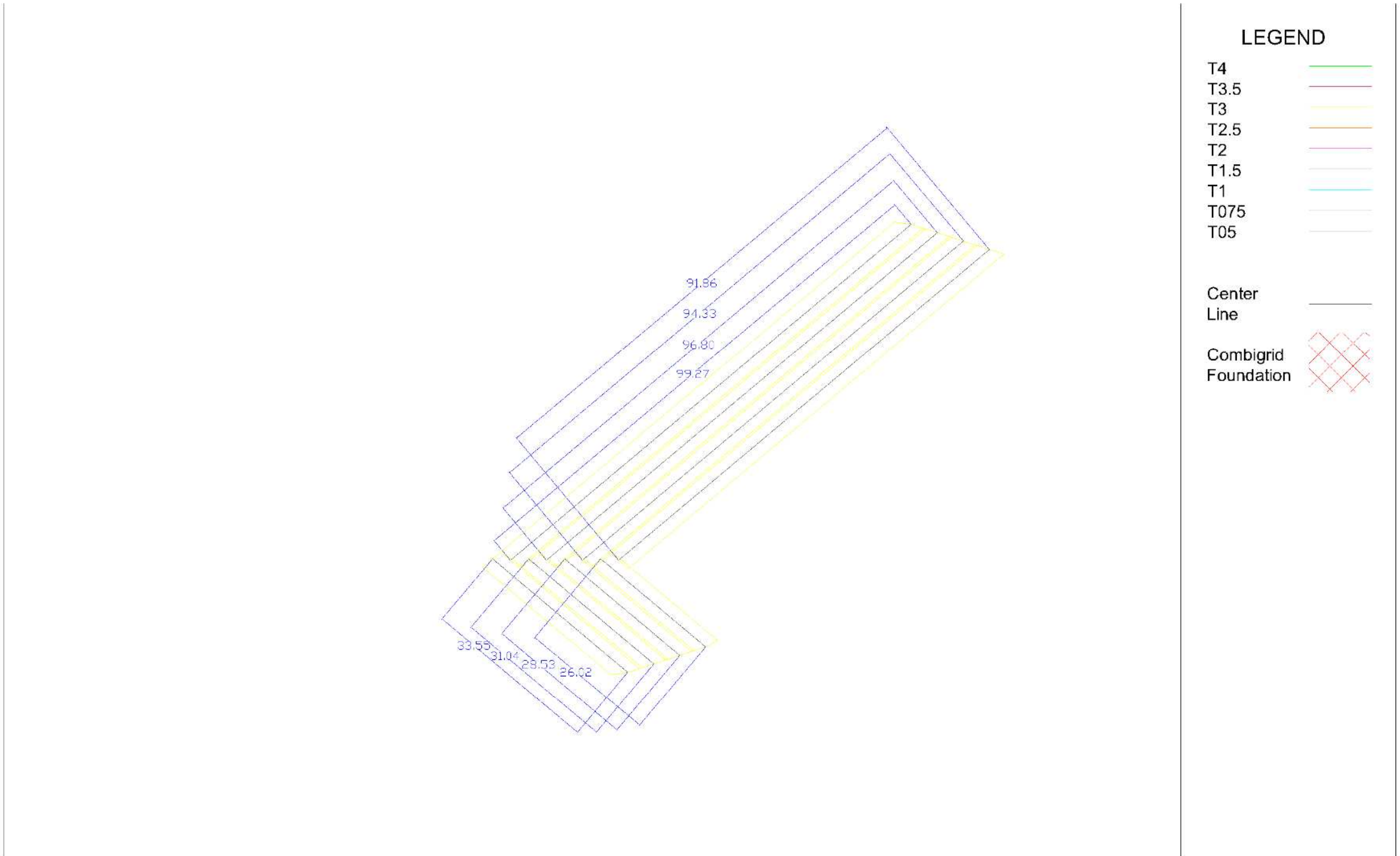

Orewa Middle Reef

T3 Layout Plan

Scale 1:1000 @ A4	Fig. No. Orewa/MidT3/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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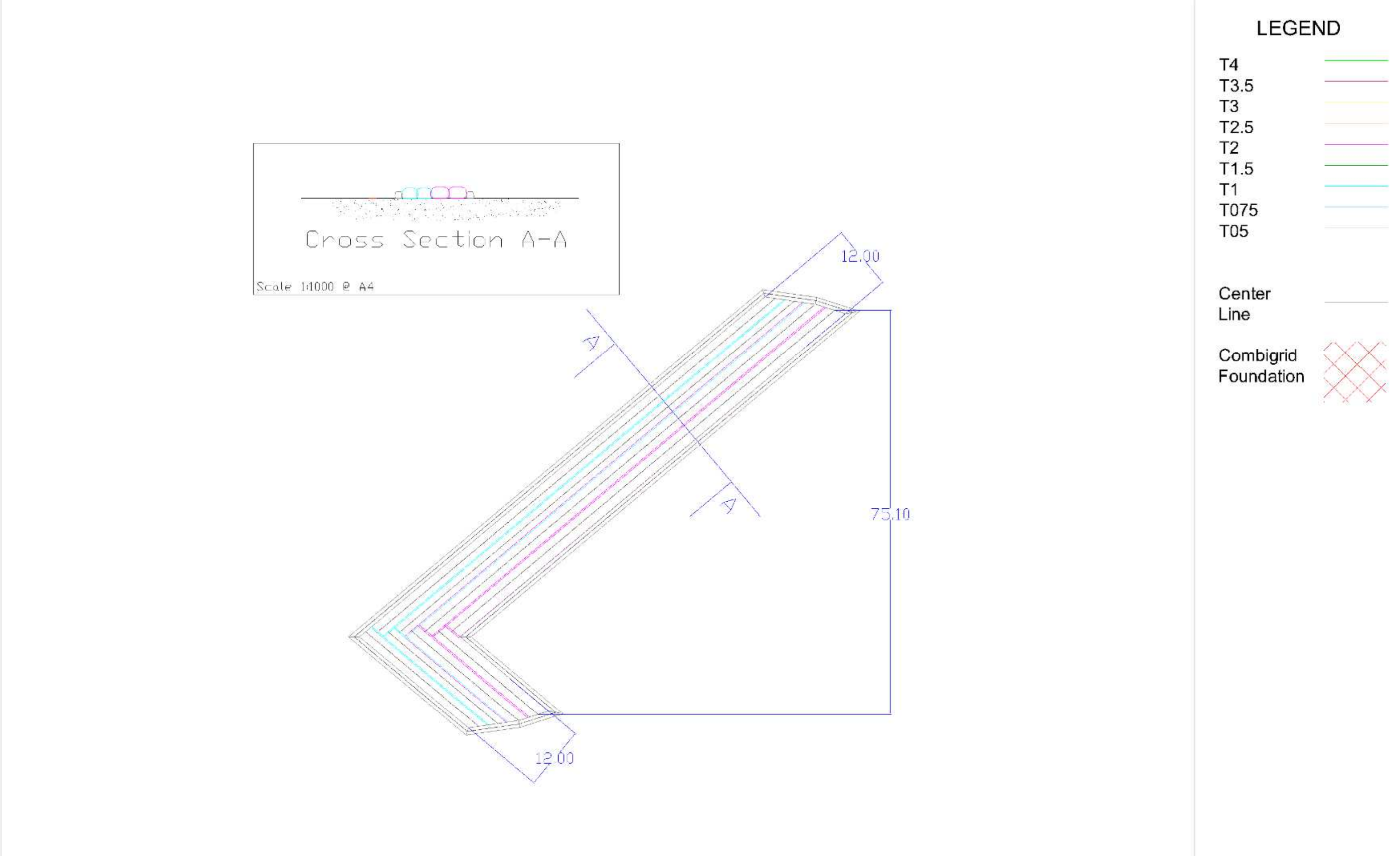

Orewa Small Reef

Bag Layout Plan

Scale	Draw No.	Date	Designed
Scale 1:1000 @ A4	Orewa/SmlPlan/1.00	17/07/2009	Jose Borrero and Shaw Mead

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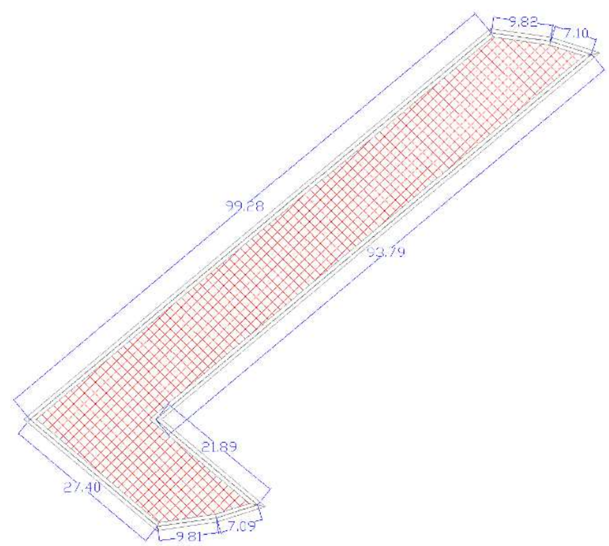

LEGEND

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T05

Center Line

Combigrid Foundation

Orewa Small Reef

Foundation & Scour Bag Layout Plan

Scale 1:1000 @ A4	Draw No. Orewa/SmlBase/1.00	Date 17/07/2009	Designed Jose Borrero and Shaw Mead

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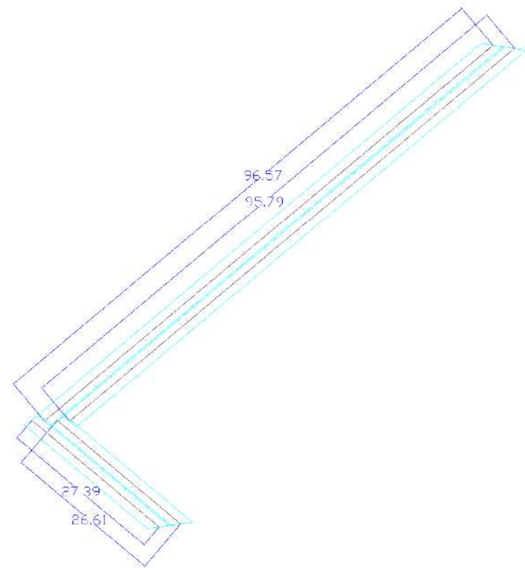

LEGEND

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Center Line

Combigrid Foundation

Orewa Small Reef

T1 Layout Plan

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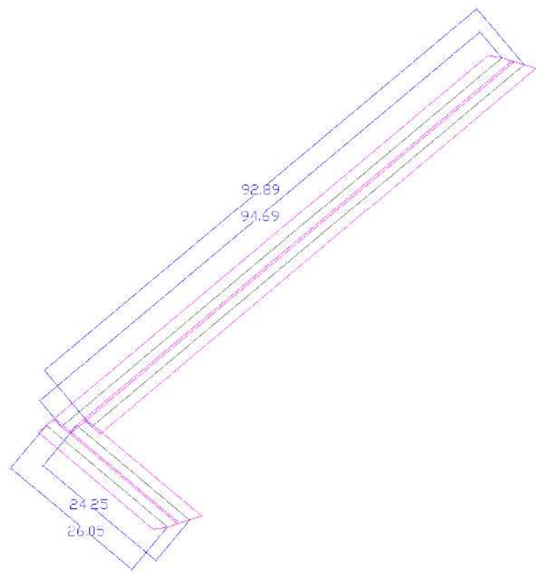

LEGEND

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Center Line

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Orewa Small Reef

T2 Layout Plan

Scale	Draw No.	Date	Designed
Scale 1:1000 @ A4	Orewa/SmlT2/1.00	17/07/2009	Jose Borrero and Shaw Mead

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APPENDIX 5

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29 July 2009

Zane Taylor
Chairman
Orewa Beach Reef Charitable Trust

Dear Zane

Re: Responses to Technical Review of the Design/AEE Report in Support of the Application for the Multi-Purpose Reefs at Orewa Beach.

This letter briefly outlines the responses to the reviewers (Mr. Richard Reinen-Hamill and Dr. Brad Scarfe) of our technical report developed in support of the resource consent application by OBRCT and the RDC. Information has been directly added into an updated version of the technical report (attached), however, this document provides direct feedback to the reviewers and interested parties (e.g. The Trust, the RDC, Andrew Benson and Paul Klinac at the ARC, the reviewers, etc) to ensure that all items raised have been addressed, and also incorporates the notes taken during our workshop on 28 November 2008. Reviewers comments have been copied from the pdf format letter dated 21 November 2008, and response are provided in blue text for clarity. Note, the items address are in reference to the 2 tables provided by the reviewers, since these tables repeat the general text of the reviewer's document and also provide additional detail with respect to their queries.

The pertinent parts of following has been appended to the Technical Report (Appendix 5) as the responses provide clarification to questions that may arise during the notification process and are not necessarily incorporated into the technical reports:

Appendix A: T&T assessment and commentary table

1. Pg 5, 4th para

"..the beach is currently stable (or in dynamic equilibrium), except during major storm events

This is the project definition and is repeated through the document. Options presumably to ensure no damage during a major storm event?

This refers to the main application document, rather than the technical reports. The technical report defines the project as follows in the first paragraph of the Executive Summary:

"The aim of this project is to provide a workable design for a system of partially to fully submerged offshore structures ('multipurpose reefs') which will modify and dissipate incident wave energy resulting in a reduction of erosive currents and ultimately cause the formation of beachfront salients resulting in a wider beach that is able to withstand the onslaught of typical storm events, i.e. provide a dry breach where presently one does not exist. The beach enhancement scheme presented here is part of a whole beach solution that would be applied in stages."

2. S1.3, p6

"to find a long term ..solution to the erosion problems.."

Conflict with the above

Also in the main application – it is agreed that the beach is currently dynamically stable (as is evident by beach profile data), however, due to the lack of a dry beach/foreshore or dune system, property along the foreshore is either eroded (e.g. the reserve area) or threatened leading to historical placements of rock revetments and other seawall structures along the majority of the beach. These existing structures impact the visual and amenity value of the beach and also have the capacity to exacerbate erosion. The multi-purpose reef project is a component of a larger holistic Orewa Beach Esplanade Enhance Project (OBEEP) being developed and implemented by Rodney District Council (RDC).   OBEEP includes ongoing beach nourishment, upgrading some of the existing erosion protection structures in specific locations, and planting of the sand-binding native vegetation.   A full evaluation of OBEEP is currently being prepared by RDC as support for applications for resource consent for the proposed upgrade of existing erosion protection structures.

3 S1.4 & Appendix 1

Inadequate description of the proposal. Plans are lacking detail and there is no provision of cross-sections, long sections, typical details and land based works described in the text.

This section and Appendix 1 identifies what is being proposed. However, it is significantly lacking detail. There is no cross-section, long section or typical detail. Bathymetry is not shown in all drawings and there is no indication on these plans if beach nourishment is included. Presumably if it is not shown on the drawings it is not included. No details of dune planting, areas where that will be applied and provision for public access and controls.

A comprehensive set of drawings has been developed, which include container layouts x-sections, etc. Beach planting, access, etc, is part of the RDC's Esplanade Enhancement Plan (OBEEP) as described above, and other than general 'areas' shown on our plans, these details need to be worked through with the RDC's officers so that they fit in with the overall plan, which does presently include conceptual plans of plantings, beach access, walkways, etc. Indeed, access ways are already present at Orewa Beach; this project does not change that, only puts a wider dry beach in front of them. It would be expected that similar measures as in place at the southern end of the beach would be applied to protect sand binding plants, as is seen around many northern NZ beaches.

4 S1.5

This section identifies deposition of sand is required for maintenance purposes, but this is not described in the plans or in Section1.4 or Appendix 1.

Additional renourishment is likely to be required prior to all 3-4 systems going into place, since the unprotected areas of the beach will continue to function as always (i.e. transporting additional sand to the Waitemata Groyne). This is also demonstrated in the modelling with consecutively more systems in place, where it is seen that feedback causes more sediment to be retained in each case (included in updated technical report). As is the current policy, renourishment would be expected to be required as needed. . It is noted that between 25,000 and 50,000 m^3 of sand was extracted from the Waitemata groyne and placed on the beach annually from 1994 to 2001 (a total of 235,000 m^3 in less than an 8 year periods) – Table 2.15 Appendix 2. However, in the case of the first reef system in priority zone 1, sand from the Waitemata Groyne will be utilized for reef construction and renourishment, and until comprehensive monitoring has confirmed the efficacy of the first reef system, no others will be constructed. Future reef systems and renourishment will use sand of a similar grain size as that on Orewa Beach brought in from an external source (e.g. the offshore subtidal Mangawhai-Pakiri sand resource consent).

5 P11, Table 3 and Figure 6

Statement that wave data at 30 m contour inconsistent with Figure 2.5, Appendix 2 that shows it at 27.5 m. Hincast represents swell conditions better than sea state, so may under represent sea conditions.

Figure 2.5 has been modified

The hindcast data represents peak Hs, peak Tp and peak Dir for each 3 hour record. The comments referring to spectral effects and modelling the full spectrum to ensure that all conditions are covered and that there are not impacts being missed due to ‘spectral’ effects. There are several important issues with this approach, which are described throughout; even so, we have undertaken ‘sensitivity testing’ by modelling a range of different wave periods to assess the differences between them, which has been included in the updated technical report – this clearly shows that by using lower ‘mean’ periods a great deal of wave energy is not accounted for. The following applies to several of the following comments from the reviewers:

- The peak statistics are where the majority of the energy is in the wave spectrum, which why is used for modelling purposes (this is the standard worldwide), i.e. the peak statistics are the most representative of the event.
- SWAN and 2DBEACH do not monochromatically model the wave and wind inputs, SWAN is a third-generation spectral wind-wave model, while 2DBEACH uses a unique Lagrangian scheme to effectively bring together all the hydrodynamics occurring in response to a reef (wave heights, wave angles, current speed and direction, wave set-up, etc.) and provides predictions of beach response.
- Modelled/hindcast/forecast data and measured wave events have been validated against one another and also have calibrated well in the models (included in the update technical report).
- The 20 year hindcast was validated with 8 months of wave –rider data off of Tiri Tiri Matangi Island, has been validated via TOPEX/Poseiden and ERS1&2 satellite data
- Development of the inshore wave climate is comprehensively described in the feasibility study (a 24 page section of the report), and further validation of this inshore wave climate has been provided by comparison to the archived Marineweather.co.nz data (which utilizes data from the same type of worldwide wave model as was used to develop the 20 year wave hindcast) and on-site measurements on October 2007 and February 2008 – the modelled and measured data match up very in Hs, T_{peak} and Dir_{peak} (this is included in the updated technical report)

Wave data taken from numerical model at 30 m depth contour. Average wave direction is 236°, with a standard deviation of 22°. Waves then transformed to 7 m. Average wave direction at 7 m depth is 242°, but storm waves are from 235° (Figure 2.21, Appendix 2).

The mean direction change of 6° is expected when transforming waves some 10 km into the bay, and the Figure shows that higher wave events more often occur from some 7° more north than on average – this is part of the information required to model storm events from the north, which in the past have been the main cause why renourishment has not remained where it was placed on Orewa Beach of any length of time, i.e. it was pushed south down the beach and back to the groyne and estuary area.

6 P13, Figure 7, Appendix 2

Conclusion on Page ~~13~~23, taken from wave analysis (Appendix 2) is that "local seas generated by wind within the Hauraki Gulf account for the majority of waves at Orewa and come from a more easterly direction than storm events that are from a more northerly source". States that "larger, long period wave more likely to reach the beach at a more oblique angle, resulting in stronger alongshore currents". As the hindcast model is largely swell, this is a difficult conclusion to justify, although I support the first statement. I am less convinced that larger, longer period swell waves reach the beach at more oblique angles, as they will begin refracting earlier than the local sea state due to their length.

We agree, this is misrepresented in the text – longer period waves will indeed have less of an oblique angle when they arrive at the beach because they will have been refracted in earlier. This should refer to larger wave heights (from a more north easterly quarter) more likely to reach the beach at an oblique angle. The text has been updated

This table shows max sig wave height of 7.19 m. Table 2.1, Appendix 2 shows 1 part per thousand wave height of 4.5 – 5.0 m.

The maximum event derived from the 20 year data set, which is 1 part per 7,305 on a daily basis, so would be expected to be greater that 1 part per thousand, or once every 3 years. These data match with the r.p. calcs on page 17 of the technical report

7 Table 2.5, Appendix 2

Final wave height seems unrealistic due to depth limiting conditions and wave periods do not seem credible (too long). This suggests possibility of unrealistic boundary conditions to model. I also note the table shows a period of 7.9 s to 8.8 s for a Hs of 1.5 m and 7 to 8 seconds for an 0.5 m wave. This is significantly longer periods than observed. Ref Table 2.9 shows a Tp of around 5.5 for a 1.3 m high wave. I also note Table 2.5 shows a number of wave bins with different periods. These seems to be a rationalization that may under predict the average wave height used in subsequent modelling

As described above, a great deal of investigation was undertaken to develop the inshore wave data. I'm unsure of which 'observed' data with respect to wave periods and wave heights is being referred to (Leigh Marine Lab observations?), however, the best evidence that we have with respect to wave observations are the onsite instrument data, which corresponds very well to the modelled data, which is the same source used to develop the inshore wave climate. The Tiri wave-buoy data included mean period, which is significantly smaller than peak period (i.e. where the majority of the wave energy exists), which is also shown in the modelled output. For example, on 10 October 2007, the measured periods were similar to the modelled, and sometimes slightly higher, rather than the opposite way around, which is the concern of the reviewer.

In addition, the measured mean period from the inshore aquadopp data in October 2007 is similar in terms of the difference to the peak period as that shown between the modelled peak period and the measured mean period at Tiri Island, Great Barrier Island (Tab 4.2 Feasibility study). Note, Direction bins are 10° in the model output for ease of plotting to the website (www.marineweather.co.nz).

10-Oct-07					
Hs		Tp		Dir	
Modelled	Measured	Modelled	Measured	Modelled	Measured
0.7	0.54	3.5	3.23	55	55.57
0.5	0.5	8.1	3.58	75	67.23
0.5	0.43	8.1	8.71	35	40.29
0.6	0.43	8.1	8.73	65	57.11
0.5	0.42	8.1	8.63	65	59.58
0.5	0.45	8.1	9.39	65	59.28
0.4	0.4	8.1	9.32	75	57.71

21-Feb-08					
Hs		Tp		Dir	
Modelled	Measured	Modelled	Measured	Modelled	Measured
0.5	0.67	4	3.87	75	70.16
0.5	0.64	4	4.08	75	68.4
0.6	0.71	4	4.35	75	67.6
0.6	0.57	4.2	3.67	75	68.1

Similar close correspondence between measured and modelled periods are also evident at higher wave events and shorter periods:

22-Feb-08					
Hs		Tp		Dir	
Modelled	Measured	Modelled	Measured	Modelled	Measured
1.2	1.34	6.1	4.82	65	70.12
1.1	1.26	6.1	4.66	65	70.99
1.2	1.32	6.1	5.51	65	71.18
1.3	1.19	6.1	5.65	65	67.78

From Aqaudopp 10/11/07		Extract from Table 4.2			
Mean Period	Peak Period	Mean Period Whangaparaoa	Peak Period Whangaparaoa	Mean Period Waverider	Peak Period Great Barrier
3.36	8.71	5.9	8.6	3.1	10.1
3.74	8.66	5.6	9.0	2.6	9.4

3.82	8.64	6.5	8.9	3.6	8.8
3.77	8.8	5.8	8.7	3.2	8.5
3.87	8.75	4.7	7.2	2.9	8.8
4.16	8.38	6.1	8.8	3.1	8.7
4.51	8.65	6.4	8.9	3.3	9.1
4.22	8.41	6.0	8.2	3.4	8.0
4.22	8.63	5.8	8.8	2.8	9.8
3.99	8.73	5.7	8.3	3.2	8.7
4.09	8.66	6.2	8.6	3.4	8.6

The consequences of using shorter period waves for modeling purposes are described in the following demonstration.

Due to Orewa's (relatively) sheltered location it is subjected to a wide spectrum of wave frequency. The many fringe islands of the Hauraki Gulf block long period "older" swell. To gauge the sensitivity of both the structure's wave height reduction and sedimentation abilities simulations were ran differing the period (rather than wave direction and height, noting all 3 are combined and varied in morphological modelling) since the period is the most important variable with respect to wave energy at any given wave height..

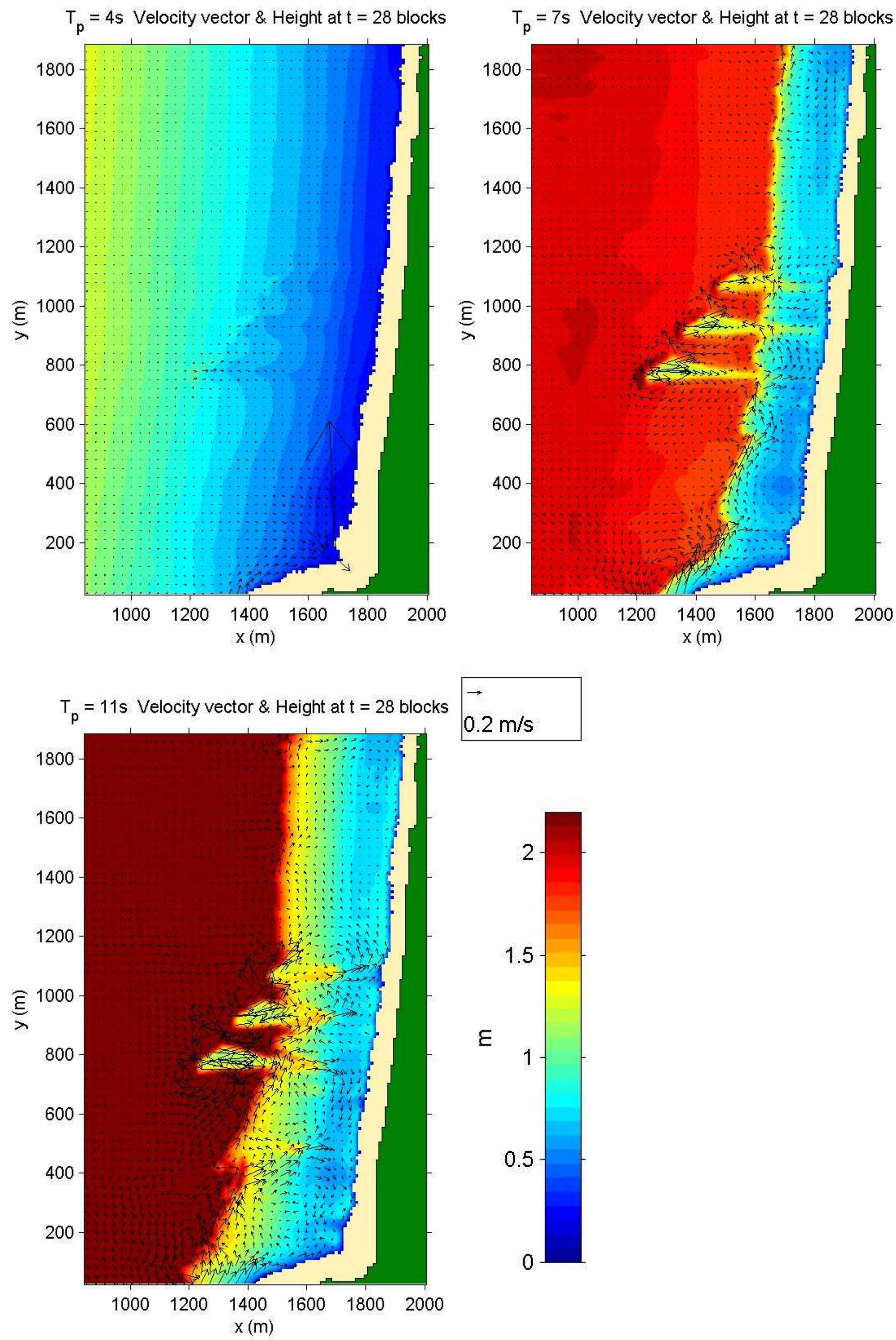


Figure 1 Wave heights and wave-driven currents due to varied wave periods (4, 7 and 11 sec) with H_s = 2m (high-tide).

The results are as expected. It is evident that waves of higher period propagate further inshore before wave heights are reduced by shallow bathymetry (Figure 1-3). This is due to the nonlinearity of wave energy in terms of period. Alternatively for incipient wave breaking, the ratio of breaker height to water depth at breaking is proportional to the square of the breaker period (i.e. H_b/d_b is proportional to T^2) (USACE, 2003). These simulations support the use of the peak period of wave statistics rather than mean wave statistics, e.g. mean period is regularly in the 3-5 second range at Orewa, while peak periods are 7-9 seconds; modelling of mean periods would underestimate the wave event's energy.

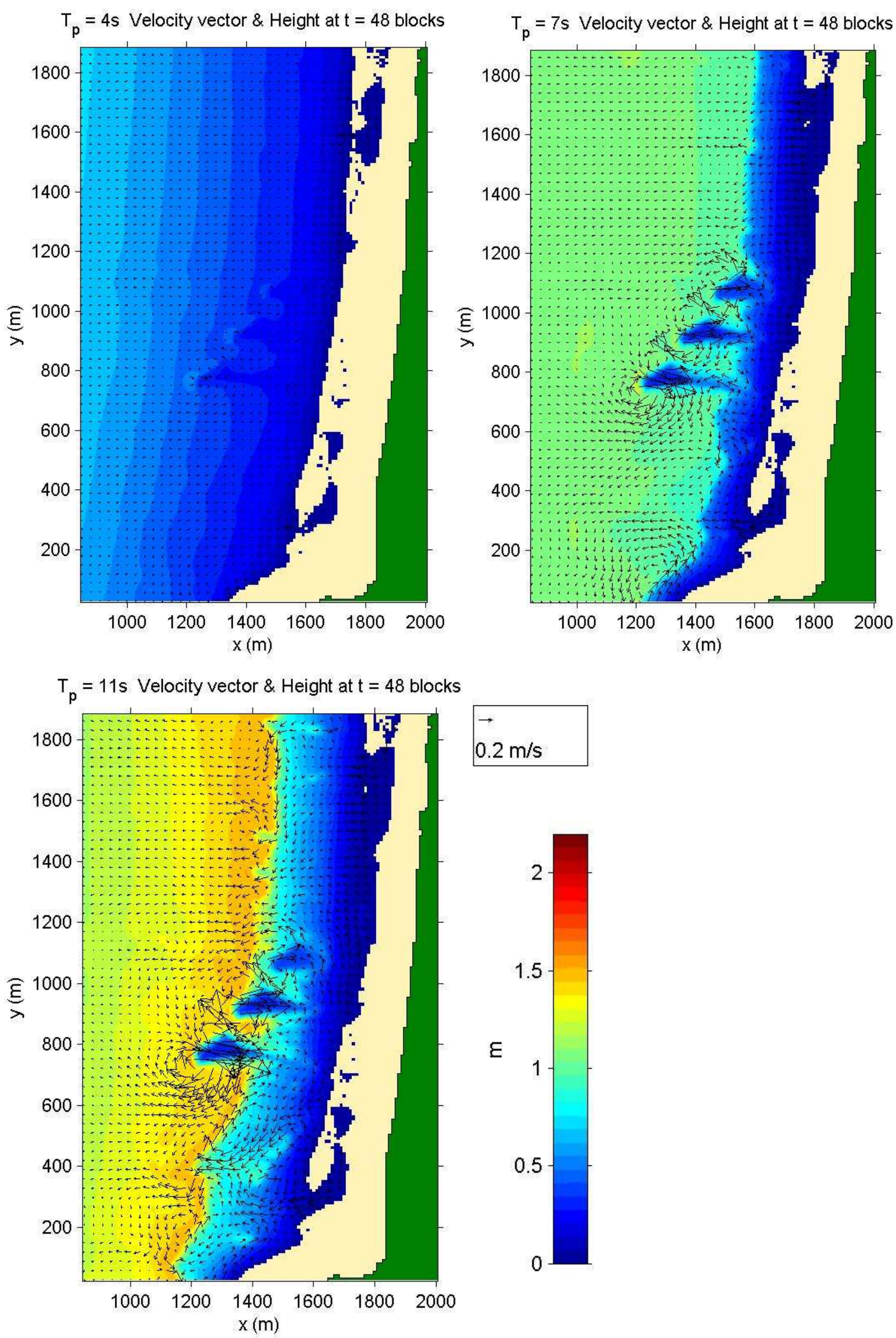


Figure 2 Wave heights and wave-driven currents due to varied wave periods with H_s = 1m (low-tide).

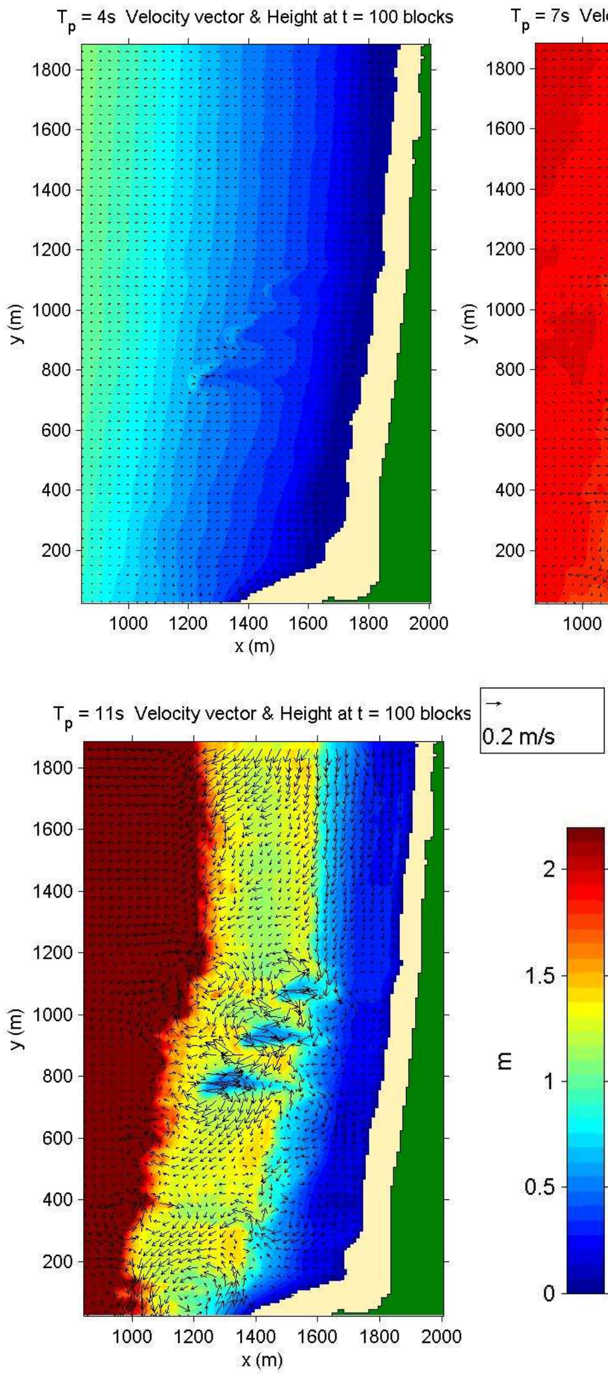


Figure 3 Wave heights and wave-driven currents due to varied wave periods with H_s = 2m (mid-tide).

This demonstration was also applied to morphological response (the boundary conditions included Hs = 1.5 ± 0.5m, Tp = 7 ± 4s, Dp = -3 ± 8° relative to grid orientation). Three runs, identical except for wave period (which was centered on 4, 7 and 11 sec) are compared. All three cases show an increased salient in lee of the reef structures. As would be expected, the beach responds with greater magnitude in response to the high energy conditions (Figures 4-6).

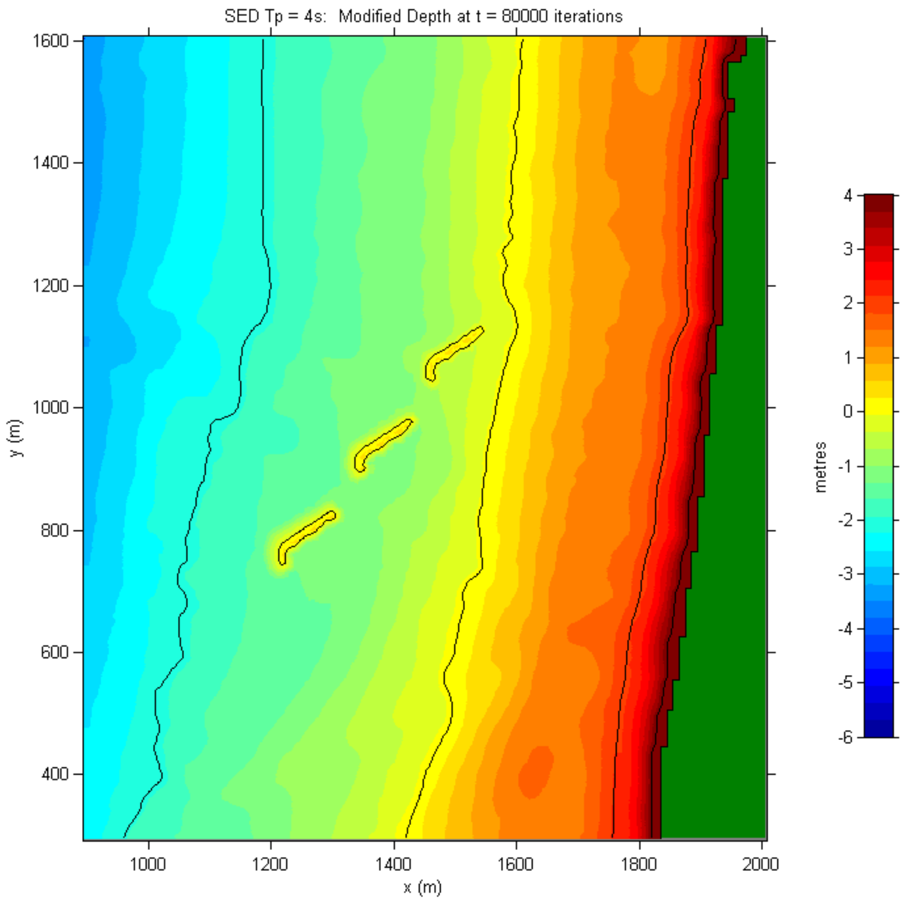


Figure 4 Modified bathymetry under a 4 sec period swell.

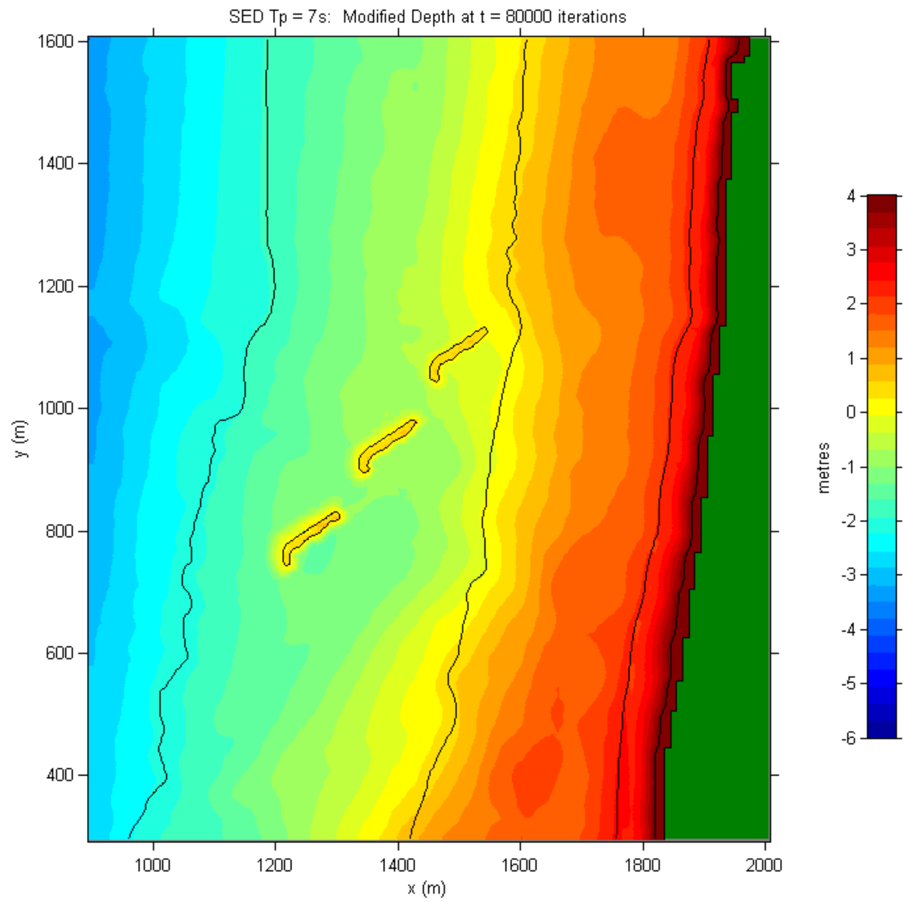


Figure 5 Modified bathymetry under a 7 sec period swell.

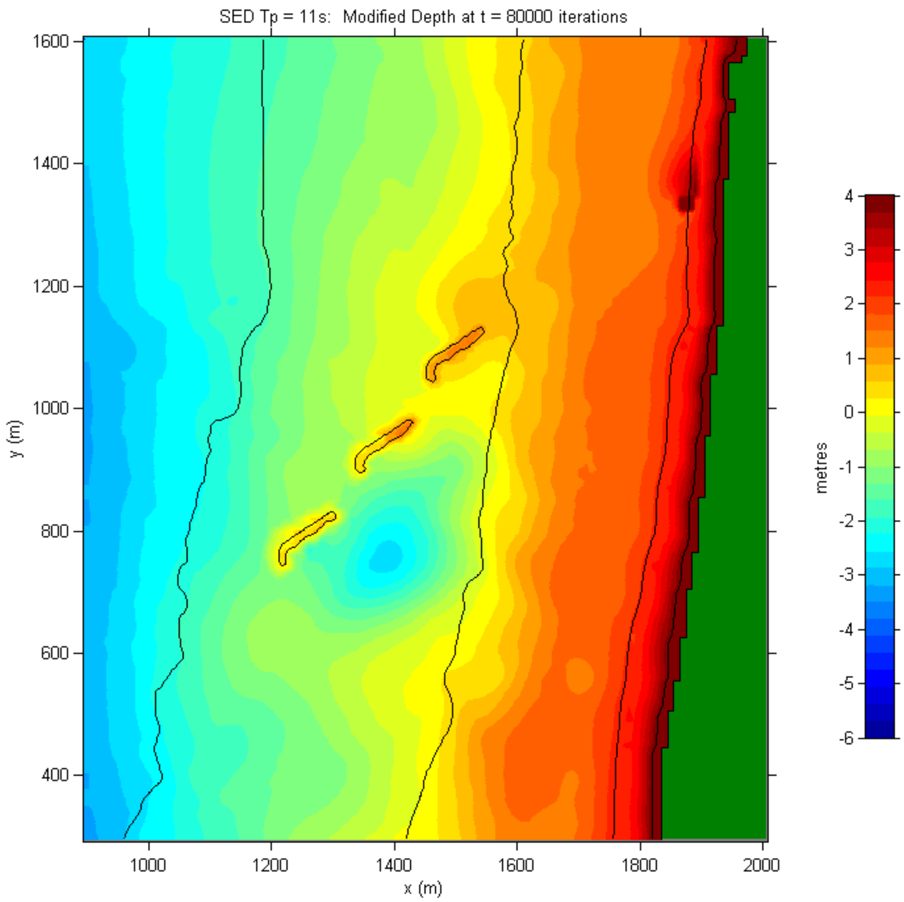


Figure 6. Modified bathymetry under a 11 sec period swell.

Thus, using peak statistics, as is the normal process in modelling, is very important with respect to energetic and consequential currents and sediment transport.

Maximum Hs of 6.5 m and a period of 12.55 and a direction of 229.5 (49.5) is shown as a 0.01% frequency in 7 m water depth. This is well outside wave heights able to be sustained by solitary wave theory (typically 0.78 times the water depth) and significantly greater than typical sea states (0.55 to 0.65 times the water depth (i.e. 3.9 m to 4.6 m)

Transformation modelling was done at 3 tidal levels, 0.7, 1.7 and 3.0 (e.g. Figure 2.17), with the 3.0 m tide reflecting the conditions when the beach is susceptible to erosive events, i.e. when it is 10 m deep at the inshore transformation site, or 10.5 m when the MFE coastal hazard standard 0.5 m of storm surge is also added to the assessment. At 0.78 breaking criteria, 8.33 m of water is required, at 0.65 breaking criteria, 10 m water is required for 6.5 m wave heights. While the typical 'steep' wave sea state reduces the breaking criteria, the very low gradient works in the opposite direction (i.e. gently shoaling waves break in deeper water than steeply shoaling waves), both of which are accounted for in the 'equivalent height enhanced breaking' scheme incorporated into model WBEND which was used for the transformation modelling.

8 Figure 2.25, 2.26, Appendix 2

These plots are not that similar, as noted by ASR, Figure 2.26 shows a significant increase in energy from the 60° sector, but also less energy from the 110° sector. If the 1 year data is used as representative, this would result in a more north-easterly, rather than east-north-east average and affect sediment transport predictions.

With respect to 1 years versus 10 years and annual variability, these data sets are very similar, with the 'significant' energy increase from the 60° sector being relatively minor at <3% and at a greater probability than the 10 year data set providing a more conservative or worst case scenario, which is useful and quite often applied for modelling purposes.

9 Figure 2.29, Appendix 2

This figure shows two "holes" in the nearshore off Orewa Beach and a central "tombolo". This is not discussed and may be relevant. What data was used to construct bathymetry?

While the original Figure could not be located, the tidal model grid doesn't have this strange feature (it has now been replaced with this in the report), and we have confidence in the inshore bathymetry due to a combination of bathymetry surveys (including multi-beam conducted by Dr. Scarfe), digitised charts and beach profiles.

10 Figure 2.33, 2.34 Appendix 2

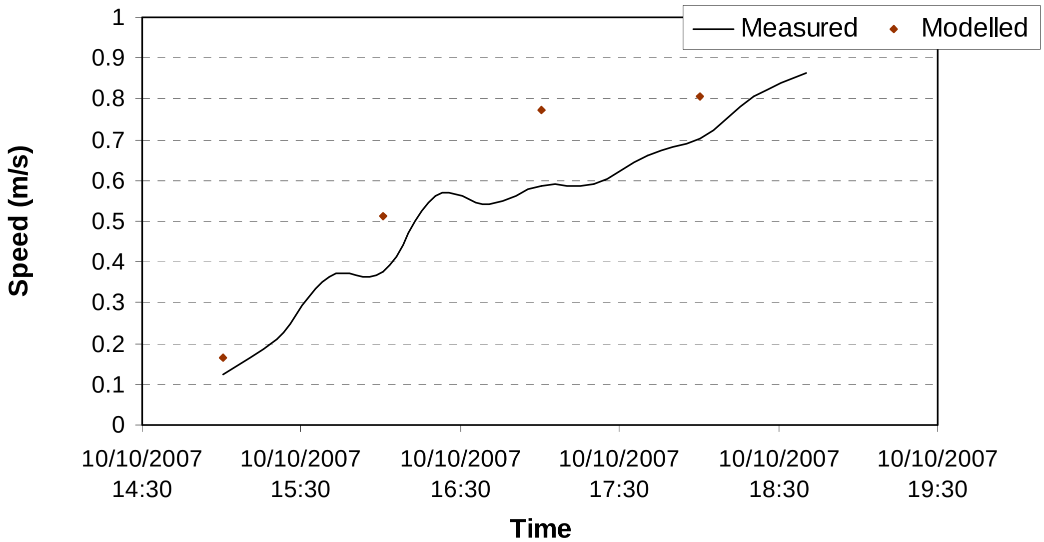
Calibration plot of current velocity in the ebb delta and in front of the SLSC bears no resemblance to measured data.

Water level appears well calibrated, but currents not well calibrated, apart from within the narrow channel to the estuary.

At between 1 and 4 cm/s, these currents measured at the SLSC are extremely low and with the modelled and measured data falling between these 3 cms of current fluctuation and picking up some of the overall trends, this is a satisfactory calibration. The model with no other forcing factors that water level boundaries, shows a clear 6:17 hr tidal signature, while the measured data shows the general trend (more in the latter part of the data set, since a small swell was present during the first half) even though waves and winds are also interacting. It is important to note that sealevels are used to drive the model from offshore boundaries, and in all cases, the sealevels of measured and modelled are very close. It is also important to note that currents below 30 cm/s are insufficient to move sand, while currents below 10 cm/s are insufficient to move fine silt – at the location of the most southerly set of reef systems (i.e. the SLSC) tidal currents are too low to move sand, while even in the ebb jet, only fine silts can be moved by the tidal currents. The very low tidal currents are consistent with previous tidal models of the Hauraki Gulf and Orewa (e.g. Black et al, 2000), and although are incorporated in wave and sediment transport modelling, have only a small residual impact on results, i.e. wave and wind-driven currents are orders of magnitude greater.

> From the updated technical report: "Good model calibration of the Orewa estuary numerical model was achieved. The predicted current speeds from the model reasonably match the measurements. Pressures recorded by the ADV were converted to water levels and corrected to the depth of the measurement. Figures 2.31 to Figure 2.36 illustrate the calibration at measurement sites. There is a

variation between the current speeds modelled and those predicted at the ebb delta site (Fig. 2.33), which is due to the presence of the unstable eddy formed during the outgoing tide, i.e. the lateral movement of this eddy during the out-going tide, combined with the 3-dimensional complexity of this kind of feature means that the velocity changes as the eddy 'wobbles' over the delta – which is not well reflected at a single point (cell) in a 2D model, and at <2 m deep in this location, is not easily modelled in 3-dimensions. However, the peak and mean velocities, as well as the overall trend in velocities during the out-going tide, are represented in the model output and are of similar magnitudes. The 'shedding' of eddies in the region of the ebb tidal instrument deployment is visible in Figure 2.37.



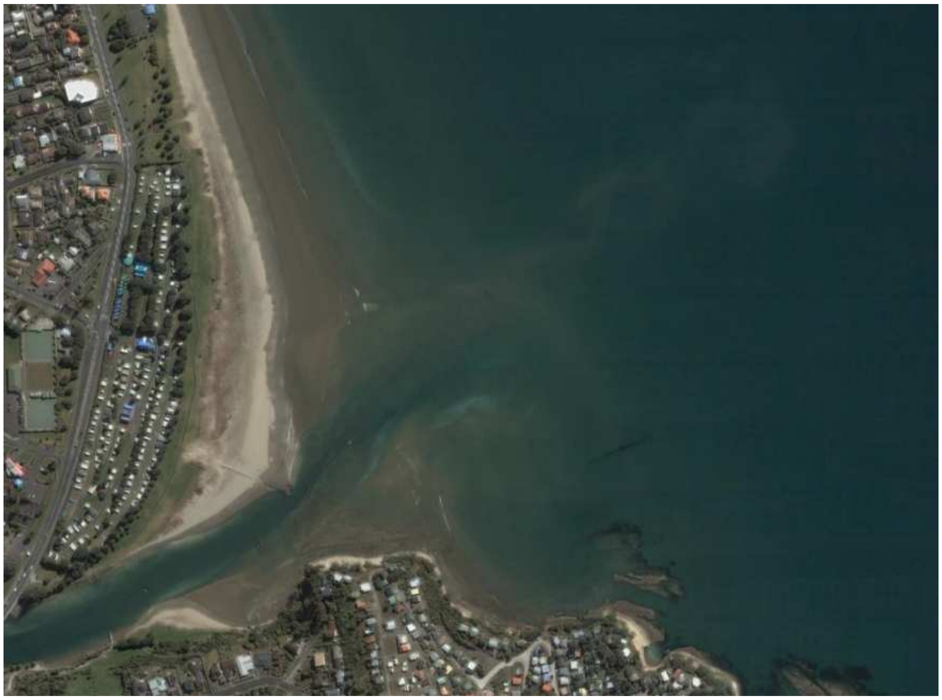


Figure 2.37. Google Earth image showing the variable eddies moving out over the ebb tidal delta (31 August 2004).

Assumptions and explanations of the graphs have been brought out in the updated technical report.

11 Figure 2.39, Appendix 2 and Figure 7 in AEE

This shows strong northerly currents
can be generated by local seas, different
to Figure 7 in AEE.

Figure 7 refers to the *net results* of the wave data analysis as detailed in both App 2 and 4, as is described in the caption "***Common wave directions*** **at Orewa showing potential effect on sediment transport",** while Figure 2.39 in Appendix 2 it shows the currents generated by a *particular storm* on February 22 2008, that generated northerly directed currents since it was generated with waves coming from a direction slightly south of shore normal.

It is unclear where the waves
heights were measured. At each location the sled was stopped for 20 mins, as described on page 24
However, they show waves from
245° and 250° providing Fig 2.39 shows waves from 65-70deg generating the currents?
moderate to strong northerly
currents capable of sediment
transport. Winds from the east.

12 Figure 2.40, Appendix 2

It is very unclear if calibration is
successful. Plots of vector results at the
output locations at the same tide stage
should be included in a mark up of
Figure 2.39

Without additional information
on the calibration it is difficult to
determine the suitability of the
modelling.

Figure 2.39 has been marked up with arrows attributed to modelling (to compare to those measured) and more text has been added to better describe the results of the calibration

13

Crest design tested at 0.5 m and 1.7 m
above Chart Datum (note 1.7 is around
Mean Sea Level). 0.5 m used as the
design level.

That's correct. Originally we would have preferred to have fully submerged structures, with crests 0.5 m below LAT. However, the relatively large tidal range in comparison to the wave climate, as well as the gentle beach gradient (which encourages wave reformation) required a higher crest to ensure wave breaking occurs. Structures up to MSL would be more effective at stopping wave penetration to the beach, although the final design also incorporates redirection of the wave-driven currents (i.e. shorewards rather than alongshore) and are effective at retaining sand. The lower crest (i.e. 0.5 m above LAT) means less intrusion (visual mainly, but also access), less volume and hence less capital cost.

14 Table 3.1, Appendix 2

The average condition is 0.5 m wave
height with 8 second period. As noted
in Item 7 above, this may be more a
rationalizing of height bins, rather than
an actual average wave (note median
height offshore 0.66 m and the mean

was 0.82 m at 30 m).

Significant wave attenuation occurs as waves propagate into Orewa Beach between Tiri Island/Whangaparaoa and Kawau Island, as is evident in Figure 2.17 (a 1 m wave is reduced to ~0.4 m (prior to shoaling and breaking on the beach). Given that the mean wave height on the north eastern open coast is ~0.9 m, these heights within the sheltered Orewa embayment are valid.

Only up to 10 year storm tested. A 10 year storm is usually the upper limit for the design of coastal protection (e.g. the USACE use the 10-year return storm for coastal protection structures), 'normal', 1-year, 10-year and 100-year storms were simulated. These investigations were to look at the impacts of the reefs on hydrodynamics and sediment transport (some from morphological modelling, some inferred by current speed, direction and circulation). These results have been included as an appendix and summarised in the text. However, it is also important to reiterate that the project aim is to widen the beach, with a wide sandy beach being the best form of coastal protection. As is seen today at Orewa, after an erosive event, even if renourishment is not placed on the beach, some sand accretes due to natural processes during calmer weather, although it is limited since there is little sand in the upper beach system. With a wider beach similar offshore/onshore responses to storms would be expected, although to a lesser extent due to wave dissipation and current redirection, with the wider beach providing a buffer zone and source of sand to move in this natural process.

Unclear what wave direction was used or whether a range of directions were tested. Given the "annual" and extreme come from different directions what has been used to determine the annual equivalent wave climate.

A full range of wave directions, heights and periods were tested for 'normal' and storm events. Two kinds of boundary conditions were used for 2DBEACH modelling (for both hydrodynamic and sediment transport modelling). The first boundary condition was developed from a year of wave events, for example, the 'normal' condition boundary incorporated wave heights of 0.3-2.04 m, peak periods of 4.5-12.2 sec, and a directional spread of 28° (as found at the 7 m depth contour, e.g. Figure 4.21 of the feasibility study) and sinusoidal tidal changes – while each event is not comprised of a spectrum of wave heights, periods and directions (rather the peak statistics), a large variety of wave conditions are included. The second boundary type incorporates sinusoidal variations in the wave parameters centred on peak values, e.g. 1 year storm tide (mean 2.4 m, period 2500, amplitude 1.0 m, phase 180, i.e. at 3.4 m incorporates storm surge and is most 'high' tide to analyse the critical period), wave height 3.5 m, direction 0° (to the grid, which corresponds with a NE storm), and a period of 8 sec. A great number of additional model outputs from these model simulations have been put together and added as appendices to provide further understanding of the results.

Given the issue is extreme events, should a wider range of extreme events not be tested.

'Normal', 1 year, 10 year and 100 year return period events were tested for 1, 2, 3, and 4 reef configurations.

14 Pg 54, last para Appendix 2 and Appendix 5

The original design (single reef), it would appear that much of the consultation included in Appendix 5 was for this option and not for the final proposal (the V1 report in Appendix 2 is dated March 2008). It is uncertain how much consultation was done with the final proposal.

Consultation summary has been compiled by RMS

15 Wave model testing

Individual reef options were tested and information provided. Data provided is difficult to interpret and difference plots between no reef and reef would be useful. However, there is not final test of the final proposal.

Difference plots, plots scenarios without reefs, and with 1, 2, 3 and 4 reef final design configurations have been extracted from existing model simulations and new simulations recently undertaken.

16 Shoreline response modelling, pg 63

Initial concern is wave boundary conditions may not be representative of actual conditions, with all periods significantly greater than those recorded on site. If this represents Figure 2.26 this may also not represent representative annual climate.

Figure 2.26 represents wave heights and heights, not periods? Dot points 7 and 8 above refer to the close similarity between the 1 year of data and 10 years of data, as well as the representation of the periods – the periods that were modelled are in agreement with those measured on site.

It is unclear what datum was used for the tides, but the it would appear to be Chart Datum. No levels reach MHWS (2.9) or include higher events such as storm surge as per Table 3.1. Therefore not clear how extreme events were considered.

CD and MSL are the datums used, and both were developed from the measured sealevel data, which correspond well with the world and Hauraki Gulf tidal models (with respect to the local tidal coefficients). Storm scenarios used sealevels of up to 3.4 m, which includes MHWS and storm surge, while the models apply wave run-up and set-up.

It is not stated what sediment grain size was used and we note Section 2.3 (pg 7) has a range of grain sizes, reducing in size offshore. This could make signficant differences to the rate of change.

Sediment grain size for modelling was the mean grain size determined by T&T (1994) for the shallow neashore area (i.e. -0.5 and -1.0 m, with reef depths between -0.3 and -1.7 m), or 0.12 mm, which is considered similar to the intertidal sediments. These sediment data have been determined previously by Raudkivi, Smith (1986) and Tonkin and Taylor (1994). Beach fractions can be 0.13-0.17 mm, while grain size reduces to 0.1 mm at 3 m depth and to 0.09 mm at 4-6 m depth. T&T (1994) analysed sediment between -0.5 and -3.0 m CD and conclude that the “beach sediment is very uniform”. Note, heavier sand fractions would have higher accumulation rates on the beach, thus, this is a conservative approach.

Only one set of reefs considered. The final proposal should also be modelled and compared.

1, 2, 3 and 4 sets of reefs have been modelled, and as described in the original technical report (pg. 68), although hydrodynamic impacts are relatively confined to the vicinity of the reefs, 2 reef systems retain more sediment than 2x the volume retained by 1 reef system, 3 reef systems retain more sediment than 1.5 times 2 reef systems and 4 reef systems retain more sand than 1.33 time 3 reef systems. This is presented in model output graphics.

Fig 3.12 shows an extreme event classified as a 2 m wave event with a moderately high tide (unspecified). It is unclear if this is an extreme storm?

Figure 3.12 refers to a 'large wave event', which considering a mean wave height of 0.8 m for offshore wave data (i.e. significant wave height is lost before the beach), is accurate. This modelling exercise was undertaken to assess the stability and recovery of the salient in the lee of the reef during a 'large wave event', with the results suggesting that the "salient and shore protection is stable even after large wave events.

In general the bathymetry is up to 2 m CD, which is just above MSL (MSL 1.7 m) and shows very little change at the 1 m contour (between MSL and MLWS). What confidence do we have for the upper beach at MHWS (2.9 m CD) changing? Although it is not clear if the scale bar reflects the colour variation used on the plots. In addition, Figure 3.11 may have incorrectly labelled legends and an inverted scale (this occurs elsewhere).

CD and MSL datums were used throughout the project – all figure captions have been amended to include the datum used. The scale label refers to depth, rather than velocity, this has been amended.

17 Multiple reef units, pg 68, Appendix 2

This appears to say that the reefs groups that extend over 400 m (shoreline width) only influence 600 m of shoreline. Two sets were tested, but for a schematized coast (no bathy) and no variation in alongshore wave climate, whereas the bathy (Figure 2.29 shows differences and increased sheltering at northern end of beach). Test case for the two reef sets is 3.1 m wave height and water level of 2.7 m, i.e. not the extreme storm event or consistent with previous test cases(ref Table 3.1). Uncertain what the wave direction is.

The 3 reef system, that extends some 360 m alongshore, has an influence of some 600 m alongshore. This number, 600 m, is there length of beach with definite response, with beach width tapering away each end beyond this. Orewa Beach's flat gradient and tidal range of around 3 m, there is a very wide surf zone through a tidal cycle. Thus, the beach response to each single reef is somewhat 'blurred' by the varying sealevel. The three reef system helps to dissipate wave action over this wide surf zone, while also modifying currents by deflecting alongshore currents in the surf zone more shoreward. Together, these factors address a relatively rare beach situation. Full ranges of test cases have been undertaken and are included in the updated version of the report. Schematized bathymetries are generated from a beach profile at Orewa, which given the planar nature of the beach is valid, and in addition, whether schematized or measured bathymetry once the model is run with sediment transport it will change in response to the boundary inputs. Both schematized and 'natural' bathymetries are presented in the updated report.

Results difficult to interpret. Difference plots would be better. However, Figs

3.17 and 3.18 suggests there are differences in results for two reef systems. A conclusion is made that "similar results were obtained for 3 reef sets", but no information provided. Similarly no information is provided for 4 reef sets, or comments on results.

Full ranges of test cases have been undertaken and are included in the updated version of the report.

The model suggests relatively localised effects with a shore normal wave, but the problem ID is for non shore-normal wave attack during storms, so it does not really provide an indication of what could happen in an extreme event.

Modelling incorporates varied directions (26° spread) and storm direction (i.e. from the NE).

18 P73, Appendix 2

Figures show accumulation of sand. It is unclear what this sand accumulation comprises, i.e. does it show dry beach above MHWS? If so, it is unclear where this sand comes from as nod discussion on beach nourishment volume is provided. No justification is provided on the amount the beach will accrete as even areas outside the influence of the reefs are shown to be accreted.

Pg 73 is the introduction to the construction materials and methods section, so it is not clear what this is referring to. However, 100-120,000 m^3 of renourishment material is proposed for renourishment of the whole beach.

19 P77, Appendix 2

No details of beach management or planting are included in the application

Beach plantings in northeastern NZ comprise spinifex and pingao. Beach management strategies have been discussed with the RDC in relation to the esplanade enhancement plan. It is unclear whether this item should be part of the physical impact assessment?

20 P81, Figure 4, Appendix 2

Table 3.1 does not provide reef gradient specifications. No crest level indicated on figure.

A full set of drawings and specifications have been provided in the updated report

21 Section4.5, Appendix 2

No consents sought for the take or discharge of water.

This item has been referred to RMS

We note that the reefs are in shallow water (i.e. landward limit is around Chart Datum, 0.5 m below MLWS) and within the surf zone. Therefore, there may be constructability issues with barge mounted construction techniques, particularly in the shallow water. Has construction methodology being reviewed by a contractor?

ASR has a construction arm and the methods proposed have been successfully used previously. It is worth noting that while the shallowest extent of the reef system is around 0.5 m deep, MSL is 1.7 m, i.e. 2.2 m deep.

22 Section 4.6.4, Appendix 2

A geomat is proposed as scour protection and to prevent settlement. No geotechnical investigations are presented to confirm seabed conditions and subsurface geotechnical parameters. However, while it may be possible for a geomat to assist in reducing settlement, it is unlikely that a geomat will prevent scour as it is an open grid. The report identifies possibility of scour bags, but no detail or extent of these items are included in the application. Confirmation of geotechnical viability is required.

Combigrid geomat is the material recommended for the base of the structure. This material is a combination of a structural grid material and a non-woven geofabric – specs are attached to the updated report. Scour is around the structure is prevented by scour-bags, or T0.5 Softrock containers. These containers sit along the edges of the reef and following scour (which will mainly occur along the inshore edges of the structure), will settle into the scour holes and prevent undermining of the structure. Similar measures are used world-wide.

With respect to geotechnical viability, sand-filled containers represent sand on top of sand, with the bearing capacity of packed sand (as existing at Orewa Beach) being relatively high. Cores have not been taken over the entire subtidal area of Orewa Beach at reef locations, and based on the available information, they are not considered necessary. Anecdotal evidence during field work and the deployment of 3.0 m long 4” galvanised pipes of mooring attachment, using a 4” trash-pump to jet them into the seabed, indicates sand and shell fragments to depths of 3.0 m and 300 m offshore (from CD). International literature review (as attached in the feasibility study) shows that such structures have the capacity to settle between 0.3 and 0.5 m when placed on sand (see feasibility study appendices for case studies). Analysis of 13 bathymetry surveys of Mount Maunganui reef and 7 of the lower layer of Boscombe reef indicate that following initial settlement of 0.2-0.3 m, containers have remained at the same elevations over almost 4 years of monitoring.

23 Section 4.7, Appendix 2

This section identifies some 100,000 to ~~200,000~~ 120,000 m3 of sand will be required for beach nourishment. This is in addition to the 15,000 to 17,000 m3 per reef system (i.e. 60,000 to 68,000 m3 for 4 reef systems). No consent has been sought for this volume and no drawings are providing showing the resulting form of the nourishment.

As described in Section 4.7, it is envisaged that the RDC consent to extract sand from the Waitemata Groyne be utilized (as previously discussed with the RDC), up to 50,000 m^3/year has previously been extracted from this area. The project is based around monitoring the performance of the first reef system prior to constructing additional systems. As described in Section 4.7, it is expected that sources such as the offshore deposits in the Hauraki Gulf would be utilized, similar to the sand used to renourish Mission Bay.

24 Assessment of Effects Chapter 5, Appendix 2

In terms of effects assessment there is no testing of the final proposal in its entirety and with actual bathymetry to examine possible effects on the wider system, particularly on sediment transport and modifications any change to existing sediment patterns may have on the wider environment, including the wider bay area and in and around the estuary entrance and the southern part of the beach not protected by the reefs. It would also be useful to understand what the resulting wave climate is in the lee of the breakwaters (i.e. compare with existing) to determine if there will be a compositional change in sediment properties.

Full ranges of test cases have been undertaken and are included in the updated version of the report. As shown in this modelling, the impacts on currents are relatively localized. With respect to impacts on the estuary entrance/delta, it is noted that this feature is an important control point of the current beach alignment; the impacts of moving it in 1959 are largely responsible for the current status of the beach. However, there is some 600 m separation between the estuary entrance, and the formative mechanism is the tidal prism of the estuary; since the project will not change the tidal prism, the ebb tidal delta will not be impacted. In addition, the first reef system is to be built at the northern end of the beach and the monitoring programme will include full beach bathymetry surveys which will provide data, verification and understanding of the wider beach impacts for future systems.

The beach sand out to some 3.0 m below CD has been described as very uniform (T&T's 1994), and so it is unlikely that any compositional change in sediment properties will occur. If slightly coarser grain sizes than presently occur on the beach are used for renourishment for future reef systems (i.e. following the first), then it is expected that some sorting will occur and the slightly coarser fractions will be found higher on the beach and would form cusps such as those observed on other sheltered east coast beaches (e.g. Takapuna Beach on the North Shore).

In terms of shoreline stability it is unclear, given that it is stated that there is no long term erosion trend and the beach is dynamically stable, why large scale nourishment, possibly with one reef system would not provide similar benefits without the intrusion of remaining reef systems.

Monitoring will help answer more clearly, however, the additional modelling outputs presented in the updated version of the technical report also provide more evidence to this effect. Due to the historical human impacts on the Orewa Beach system, there is no longer a dry beach. In order to create and stabilize a dry beach, structures to dissipate and modify wave and current action are required, along with additional beach sand in this sediment limited pocket beach. Following detailed monitoring of the first set of reefs, it will be determined whether no, 1, 2 or 3 reef systems are required, and indeed, due to the additional effectiveness provided by 2 sets compared to 1 (as shown in the modelling), it may be the case each time a new set goes in, i.e. monitoring, review and decide on the future direction. Numerical models may be the best tools we have, but they are only tools, which can be refined following the addition of data from monitoring schemes (i.e. for further calibration).

Without additional information on modelling and improved confidence of that modelling via calibration and modelling the required conditions, it is not possible to consider potential effects further.

Full ranges of test cases have been undertaken and are included in the updated version of the report.

25 5.4, Appendix 2

Navigation risk and mitigation should be assessed by qualified persons. Discussion of races changes (such as Thundercats, etc) also need to be carried out. Assessment of markings needs confirmation.

These issues have been assessed in consultation with the Harbour Master and RDC Environmental Services staff, and similar measures have been undertaken as those applied at Lyall Bay, where user groups include 2x SLSC's, an existing surfing break (The Corner), and a 200 m wide wetbike/jetski lane within the <1 km long bay. Lighted buoys and notification (via signage at local boat launching sites) and addition of the reefs to the navigation charts is proposed. The location of the existing ski lane is between reef systems 2 and 3, i.e. there is no change to this, and there is space available for more, noting that this predisposes the outcome of monitoring of the first reef system. It must be noted that while ASR has previous experience with respect to solutions for navigation issues/user conflicts, our expertise lies in physical and biological impacts. Previously, issues raised following consent notification have been successfully addressed through pre-hearing meetings with the effected parties, the Harbour Master, the applicants and the planners, with ASR providing any technical input required. Thus, while mitigation measures have been proposed in the technical report, it is expected that following notification any effected parties will have the opportunity to respond and be heard or have issues addressed prior to the hearing.

Appendix B: Scarfe assessment and commentary table

1 Pg 10, 3rd para
Main Application
"Significant storm waves"
What is the definition of a significant storm event?

[RMS document] Unlike statistical significance, a significant storm event at Orewa is one that causes loss of beach sand.

2 Pg 3, Figure 3
Main Application
Historical beach changes
The location of existing shore protection structures is not provide in a map, Figure 3 could be adapted to include existing tipped rock walls

[RMS document] Unsure of the relevance of this to the application?

3 Pg 4, para 2
Appendix B
It is commented that "various storm event have caused beachfront erosion" and it would be good to have a better

understanding of particular storms and
their character summarised in the report.

A very good understanding of the particular storms that remove any renourishment and move back to the Waitemata groyne was developed by Dr. Scarfe (supported by earlier work by T&T describing the zigzagging of sediment southwards offshore and down the beach), and can be found in Appendix 4 The Feasibility Study (while listed as a document provided to the reviewers, I am uncertain whether or not this was the case?)

Review of newspaper records, land owners person photos, video footage or some other evidence could be presented to show the scale of the problem. This could be included in an appendix.

The application is for the construction of offshore semi-emergent breakwaters. The historical data of research and consulting on Orewa Beach (from Raukivi's first report), through to findings that other than realigning the channel to the pre-1959 configuration, an offshore reef would provide the beach sand retention on Orewa Beach (UoA, T&T) were reviewed and are supplied in Appendix 4 and summarized in Appendix 2 – We consider this type of technical information of far greater relevance to the RC application than snapshots of anecdotal evidence provided by the media and third parties. This additional information has not been included in the updated technical report.

4 Pg 2, 2nd para
Main Application
"beach is currently stable…..prone to
erosion during storm events"
Provide summary evidence here, or reference to
reports/studies that back up evidence

Appendix 4 provides references to the variety of analyses undertaken on Orewa Beach profile data, the impacts of storms, the short residence time of renourishment without retention structures, etc, etc.

5 Pg 3, 4th para
Main Application
"problems associated with beach erosion
had not been evident prior to about 1960"
Provide summary evidence here, or reference to
reports/studies that back up evidence

Again, this is presented in Appendix 4, however, it is of little relevance, the technical reports, scientific investigations, beach profile data, etc, available from the early 1980's all indicate the current status of Orewa Beach, which is what the proposed solution in the RC application is addressing.

6 Pg 3, 5th para
Main Application
"….it was the change in the estuary mouth
alignment which resulted in the estuary
becoming a sediment 'sink'.
Provide summary evidence here, or reference to
reports/studies that back up evidence

The main application is supported by the technical reports – this information and relevant references are available in the Appendices.

7 Pg 4
Main Application
Review of previous reports
The reports are not referenced in a clear or typical manner,
and not all reports are included in the reference list

The main application is supported by the technical reports – this information and *all* relevant references (reports supplied to ASR by RDC) are available in the Appendices.

8 Pg 4, 7th para
Main Application
""… the most effective solution to retain
sand is offshore reefs"
Why did the University of Auckland study indicate that
offshore reefs are the most effective. Please provide further
details within the application.

The main application is supported by the technical reports – this information and relevant references (UoA and T&T) are available in the Appendices.

9 Pg 5, 1st para
Main Application
Discussion of historical storm events
Empirical information (observed dates, photos etc) of erosive
storm events would help put in context the extent of the
erosion problem.

The main application is supported by the technical reports – this information and relevant references are available in the Appendices.

10 Pg 5, 2nd para
Main Application
"The result of course was a net erosion and
retreat of the dry beach line"
Can this be shown from the 27 years of beach profile
monitoring?

We are unsure of the context of the quote. However, as above, the history of the beach has been well documented, the main application is supported by the technical reports – this information and relevant references are available in the Appendices.

11 Pg 5, 2nd para
Main Application
Discussion of rock walls causing wave
reflection during large storms
If this is a common and problematic process, photos/video or
other evidence of this could be presented

As described earlier, the proposed multi-purpose reefs are part of a broader initiative to maintain and enhance Orewa Beach. Design of any future rock walls would need to take full account of the potential for these structures to exacerbate erosion and will be presented to ARC in full as part of the proposed RDC applications for consent to upgrade some of the existing structures on the beach.

12 Pg 10, 3rd para
Main Application
Various statements are made in the
consent application about there being very
little dry beach, but without clear evidence
A analysis of beach profile data could be presented, or a map
showing beach width a high and low tides (including
spring/neap tide variation). This would also be useful for Pg
13 (2.3 Physical Beach Condition - beach profile).

The main application is supported by the technical reports – this information is common knowledge and relevant references are available in the Appendices.

13 Pg 17, 3rd para
Main Application
Where does the 0.5 m storm surge come from?
De Lange and Gibb (2000) analysed 40 years of storm surge at Tauranga and found that surge is generally 0.20-0.30 m, but could reach 0.60 m. Thus for the east coast of the North Island, 0.50 m is a very large surge. Was 0.50 m arbitrary or based on measurements? DE LANGE, W.P., and GIBB, J.G., 2000. Seasonal, Inter-annual, and Decadal Variability of Strom Surges at Tauranga, New Zealand. New Zealand Journal of Marine and Freshwater Research, 34, 419-434.

The 0.5 m storm surge value has been applied around the NZ coast line for several decades as a ‘maximum’ and was adopted in Coastal Hazards and Climate Change: a Guidance Manual for local government in New Zealand (NZCCO and MfE, 2004) and is used to establish the level of consequences of the hazard occurring. It is conservative, as should be the case for assessing environmental impacts.

14 Main Application
Generally speaking, the application does not reference well to existing knowledge and literature on morphological and hydrodynamic impacts relevant to the application.

Various physical effects from such a reef construction can be expected including a salient, tombolo or erosive shoreline response; strong wave induced currents over the reef crest; rhythmic shoreline alignment from infragravitiy engery or proto-salient development; scour hole development in the reef lee; some form of connection between wave induced currents over reef crest, currents through scour hole and any longshore trough currents; increase in rhythmic nature of surfzone expressed through more prominent bars and rips; modification of sea levels (radiation stress) caused by the reef; asymmetric morphological response where longshore transport is asymmetric; variation of reef construction platform depth from design to construction (due to wave action); natural surfzone and offshore bar morphological changes and how these interact with the reefs. With exception of salient response, none of these environmental effects have been discussed.

The main application is supported by the technical reports – this information and relevant references are available in the Appendices. However, some of these various physical responses were not included in the technical reports and have been in the updated report.

15 Pg 12-13
Main Application
Why is there no wave calibration data?
Considering not all processes are accounted for in the model, why is no calibration data presented to show the quality of the inshore wave climate? This is particularly important considering Pg 13, 2nd paragraph states that the wave climate is dominated by local seas from the east. Why is no local wind data analysed within the context of local wave climate?

The main application is supported by the technical reports – this information and relevant references are available in the Appendices. For example, Section 4 of Appendix 4 provides a detailed assessment of the offshore and local wave climate (undertaken by Dr. Scarfe in early 2004), while further wave/current data calibration/validation and wind assessment is provided in Appendix 2.

16 Pg 12-13
Main Application
Why was a refraction only model used to transform hindcast waves to the nearshore location?
Processes such as refraction/diffraction/local wind generated waves/swell waves/swell spectral width can be accounted for to transform offshore waves to inshore locations. There needs to be a discussion of important processes and detail on the caveats of the modelling.

Two refraction models (SWAN and WBEND) were used for wave transformation, with WBEND also incorporating a diffraction scheme, as is the world-standard – boussinesq wave modelling over such large distances is computer intensive and provides little if any extra benefits if there is data available to validate and calibrate against. Appendix 4 of Appendix 4 provides information on the development of the diffraction scheme developed for WBEND, which demonstrates the very close agreement with boussinesq modelling. The earlier (Appendix 4) WBEND modelling was also validated against SWAN (Appendix 2). As demonstrated in the updated technical report, the peak spetral periods are those that should be modelled (as is the world standard), and representative wave climates using probability weighted wave events include a range of wave heights, directions and periods. Model processes and limitations are provided in Appendix 6 of Appendix 4, as well as within the body of the text of the technical report. With the thorough analysis of wave data in 2004, which included hindcasts and data from two offshore buoys (Great Barrier Island and Tiri Tiri Matangi), 3 years of SWAN forecast modelling, and inshore wave measurements, we are very confident that inshore wave climate developed for Orewa Beach is a very close representation of reality.

17 Pg 17, 6th para
Main Application
"The combination of numerical modelling results provides very good evidence that a system of offshore multi-purpose reefs can ……."
It appears that the modelling alone is being relied on solely for the design, which is a subjective design method if not grounded by other forms of information. There is significant engineering literature on empirical breakwater and submerged breakwater constructions to infer various surfzone and shoreline impacts of the reef

While wave transmission such as that described in various literature on submerged structures is incorporated in the models (e.g. through equivalent height formula (Dally et al, 1995), other more basic empirical solutions (e.g. Black and Andrews, 2001a, b; Ranasinghe et al, 2006; Savolli et al, 2007), are somewhat simplistic to be applied to the Orewa Beach situation (i.e. very low beach gradient and small wave climate relative tidal range resulting in a wide surf zone during storm activity). Even so, these methodologies are discussed in the updated technical report. It is noted that numerical models are tools for investigation, they have been applied this way (with experience and common sense), and are the best tools available to the coastal scientist/engineer; empirical methods are used for general assessments for fast evaluations, with many under scrutiny these days as to their actual applicability (e.g. the Bruun Rule is now widely disputed, but applied in many areas around the world because of the ease of use).

18 Pg 18-19, Figure 9 - 10 Main Application
The modelling figures are not clear, or large enough Are the current patterns averaged over time, or instantaneous?
Geoferencing these modelling results over aerial photographs would improve clarity. If the aerial photographs were during a wave event, this wave event could be modelled to improve the

confidence in the model.

A large number of further modelling output graphics are provided in the updated technical report, and existing figures have been expanded for clarity. 10's of giga-bytes of out files and extracted graphics are also available.

19 Pg 35, Figure 18
Main Application
Why is there no sediment transport
modelling provided when all reefs are
built?
Only modelling of one set of reefs is included in the main
application

The main application is supported by the technical reports – full ranges of test cases have been undertaken and are included in the updated version of the report.

20 Pg 37, Figure 20
Main Application
This model output does not appear to
represent reality at Orewa where the
surfzone can be wide and dissipative.

The band of currents at 1m/s is presumably caused by initial wave breaking of a 3 m monochromatic wave presented in previous images. However, this imagery suggests that a lot of wave energy is lost all at once, but for a dissipative beach like Orewa the wave breaking and wave reforming will occur over a wide surfzone.

This model graphic is of a steady state run (i.e. no fluctuation in tide, wave height, period or direction), which gives the impression described above; the magnitude of the currents have also been limited to 1 m/s, and since there are higher velocities than (e.g. on the reef crests) it forces lower velocities to the 'blue' end of the scale. However, the breaker pattern from this run as shown in Figure 3.5b of Appendix 2 indicates the usual breaking and reforming. Full variable modelling in the updated technical report shows the often wide and reducing surfzone.

21 Pg 17, Section
2.5.3 Appendix B
The use of a monochromatic wave model
in the "Inshore wave climate section"
might not be the most accurate method
A discussion of methods, or comparison of different model results, needs to be included to validate the monochromatic model findings. How does wave spectrum, wave height and direction gradients along the model boundary and local winds impact on modelling accuracy and predicted wave climate.

There are several interwoven issues in this statement. The monochromatic waves are representatives of wave events for particular periods of time and occurrences – whether these are binned by probability and transformed from offshore to inshore or whether say 10 years of offshore wave data is transformed inshore, the results are the same, i.e. 40-60 wave conditions represent the wave climate – this is also the world standard with respect to development of an inshore wave climate; it should be noted that the validated inshore wave climate was used for boundary development was generated with SWAN, which is a third-generation wind-wave spectral model. The Hs, Tp and Dp are the wave statistics used in models to represent the particular wave conditions, and as can be seen from calibration and validation, these statistics match the measurements of waves and currents at the site. Secondly, both hydrodynamic and morphological modelling incorporate and range of wave heights, directions, periods, tidal phases and wind, as described above.

22 Pg 25, Section 2.5.4 Appendix B
The SWAN modelling is considered to be
a better method than the WBEND

modelling but also needs calibration data.
The comment that "anecdotal reports and observations suggest that the forecast system is accurate in predicting nearshore wave conditions" is not an established scientific method of calibration/validation. A time series of modelled versus measured wave heights is required.

Calibration/validation is provided in the updated report for three separate deployment periods and compared to the SWAN data (3 years of archived data), as well as above

23 Pg 27, para 2
Appendix B
Please clear show the margins of error of the model predictions.
The only wave calibration data used was the Waverider record and this effectively showed that the MDI was not accurately producing wave heights.
In addition, the presentation of the measured and modelled averages in this paragraph can distort the accuracy obtained. The established method for showing calibration is a time series of measured and modelled parameters. The lack of evidence puts in doubt the authors comments that "we are therefore confident in using the MarineWeather model output data.....". The model may be accurate, but it is not proven in the application.

This interpretation is incorrect. As above, with respect to 1 years versus 10 years and annual variability, these data sets are very similar, with the 'significant' energy increase from the 60° sector being relatively minor at <3% and at a greater probability than the 10 year data set providing a more conservative or worst case scenario, which is useful and quite often applied for modelling purposes.

Similarly with respect to the wave climate; two refraction models (SWAN and WBEND) were used for wave transformation, with WBEND also incorporating a diffraction scheme, as is the world-standard – boussinesq wave modelling over such large distances is computer intensive and provides little if any extra benefits if there is data available to validate and calibrate against. Appendix 4 of Appendix 4 provides information on the development of the diffraction scheme developed for WBEND, which demonstrates the very close agreement with boussinesq modelling. The earlier (Appendix 4) WBEND modelling was also validated against SWAN (Appendix 2). As demonstrated in the updated technical report, the peak spectral periods are those that should be modelled (as is the world standard), and representative wave climates using probability weighted wave events include a range of wave heights, directions and periods. Model processes and limitations are provided in Appendix 6 of Appendix 4, as well as within the body of the text of the technical report. With the thorough analysis of wave data in 2004, which included hindcasts and data from two offshore buoys (Great Barrier Island and Tiri Tiri Matangi), 3 years of SWAN forecast modelling, and inshore wave measurements, we are very confident that inshore wave climate developed for Orewa Beach is a very close representation of reality.

24 Pg 30-34
Appendix B

How does the tide rotate around the Whangaparoa Embayment and how does this influence the beach sediment transport and ebb-jet orientation and channel from the estuary?

The tidal patterns are likely to rotate around the embayment, travelling alongshore the Orewa beach. What effect, if any, will this have on the modelling quality, propagation of the ebb and flood jet currents and channel, or sediment transport?

The tidal currents along Orewa Beach were found to be very small (<5 cm/sec) and run parallel to the shoreline, which is in good agreement with the 3D tidal modelling of the Harauki Gulf (i.e. Black, Bell and Oldman, 2000. Features of 3-dimensional barotropic and baroclinic circulation in the Hauraki Gulf, New Zealand. New Zealand Journal of Marine and Freshwater Research.). These currents have an almost insignificant impact on sediment transport (the threshold of sediment movement for beach sand at Orewa is around 30 cm/s), however, are incorporated into the modelling (tidal currents will have a very minor impact on residual currents). Thus, the impacts of tides around the embayment are incorporated into the modelling. In the region of the ebb tidal delta, currents are relatively stronger and perpendicular to the shoreline. However, several factors mean that the reef systems will not impact on these currents and the ebb tidal delta. Firstly, the tidal prism inside the estuary drives these currents, and the project does not involve removal or deposition of sediment from within this tidal prism, i.e. no change. Secondly, the ebb tidal delta is a natural control point for the existing beach (the modification of this was a major factor leading to the current beach orientation), and so ensuring that there were no direct impacts on the tidal impacts that form it was part of the project criteria (in addition it can sometimes provide good surfing waves that we would not want to interfere with). And finally, the latter was achieved because the >600 m stretch of beach between the estuary channel and the SLSC has a wide sandy beach with native sand binding plants, i.e. it does not require 'managed advance'.

25 Pg 30-34
Appendix B

The tidal model calibration periods are too short.

This would be the case if tidal constituents needed to be derived, i.e. at least 14 days and preferably 28 days of data would be required. However, tidal constituents for the entire Hauraki Gulf and indeed most of NZ are available. The short term tidal current measurements allowed, calibration and validation of the bay and estuary.

Although the sea levels calibrate well, the modelled currents at on the delta and in front of the Surf Life Saving Club at times are half or double the speed of measurements. The currents are critical for sediment transport. The modelled ebb tidal delta measurement was 0.24 m/s while the measured was only 0.06 m/s. Thus the statement that a good model calibration was achieved is not supported by the presented model calibration. A much longer model calibration is required to determine if on average the performance is better over a longer time, or if the discrepancies are always present. The short calibration period makes the calibration statistically week.

The response to this statement is the same as provided for number 10 RRH above – the very low currents on the open beach and unstable eddy at the entrance account for the current velocity anomalies, however, the overall impacts of tides (i.e. very low shore-parallel currents) are incorporated in the modelling.

26 Pg 36 Table 2.9
and 2.10
Appendix B

Validity of measuring waves and wave induced currents for only 20 minutes is questionable

We do not agree with this comment. Standard burst data for waves 1024 or 2048 data points, which with an aqaudopp relates to almost 9 and almost 18 mins, respectively (at 2hz), while current measurement bursts are generally no more than 10 mins, and both bursts are usually take at 1.5, 3 or 6 hourly intervals. Given the environment (i.e. the surf zone) and the experiment (i.e. measuring a transect through the surf zone), 20 mins in each location is considered more than sufficient data.

As waves come in sets, will peaks and lulls, each 20 min measurement could miss the average conditions for the time period.

Similar to above, we do not agree with this comment. Sets of waves, or the surf-beat, are normally spaced at between 2-4 mins (i.e. the usual occurrence of infragravity wave activity), although generally IG waves of <120sec periods are known as near infragravity waves and infragravity waves with >120sec periods (up to 20 mins) are known as far infragravity waves, the latter are unlikely to directly associated with sets of waves. Traditionally, zero-down-crossing is used to analyse data sets of 9-18 min burst data to extract statistics such as Hs, Hmax, Tp, etc.

Thus the variation presented in the tables could be from the natural variation in wave patters from sets, or from changes in the surfzone currents are the depth decreased. The difference in timing between the two tables e.g. 11:30, 11:50... waves and 11:40, 12:00... currents; is not explained. However, the empirical measurements do give some indication of wave and current character for small wave events. However, the reef has been designed for storm events which to date to not appear to have empirical information as validation.

We do not agree with this assessment of the validity of the wave statistics and this should be discussed in more detail at the next ARC workshop. With respect to empirical information as validation, we assume that this refers to the response for 17 above (i.e. Black and Andrews, 2001a, b; Ranasinghe et al, 2006; Savolli et al, 2007), as empirical information to validate the inshore wave climate is described great detail in Section 4 of Appendix 4 and expanded on in Section 2 of Appendix 2, which leads to my conclusion that we are very confident that inshore wave climate developed for Orewa Beach is a very close representation of reality.

27 General wave climate analysis Appendix B

Considering during storm events local wind seas will be superimposed on longer period swell, it is important to present empirical analysis of wave spectra in order to justify model boundary conditions

Monochromatic wave modelling is more suitable for narrow wave spectras. What is the wave spectra like during storms at Orewa and how does this impact on the model results?

It is unclear what is being referred to here with respect to empirical analysis of wave spectra? Two methods can be applied to achieve this, *spectral methods* and *wave-by-wave (wave train) analysis*, although this will analysis will not change the Hs, Tp or Dp for the wave event. However, as described above in several responses, and demonstrated via modelling in the updated report, the Hs, Tp and Dp are the statistics that are used in wave physics, transcribed into model code and via variation of the calibration coefficients replicate the coastal environment, as was achieved with 2DBEACH. As described in the model description appendix, this model does model individual waves (as does a boussineqs model), rather it uses a unique Lagrangian scheme on the peak wave statistics to effectively bring together all the hydrodynamics occurring in response to a reef (wave heights, wave angles, current speed and direction, wave set-up, etc.) and provides predictions of beach response. 2DBEACH has capacity to predict features such as rip currents, sand bar movement, beach transformations, storm erosion and the build-up of beaches after storms. Ranges of wave heights (Hs), wave periods (Tp) and direction (Dp), derived from validated offshore wave data, which was transformed inshore using world-standard procedures and was validated against field measurements used to calibrate numerical models were used in the modelling procedure to reflect the range of conditions at Orewa Beach.

28 Pg 7, 3rd para Main Application

Need to identify and discuss the processes for salient formation, and how each process contributes to the predicted salient response.

Wave rotation and dissipation are discussed, however Black and Mead (2007; Shore and beach) state that the mechanisms are "(1) wave sheltering generating a shadow zone; (2) wave crest rotation as the waves cross the reef reducing the longshore currents; by more closely aligning the wave crests at the breakpoint with the inshore isobaths; (3) wave breaking on the reef reducing the set-up of water level at the beach in its lee; and (4) counter-rotating vortices in the lee of the reef helping to direct sediment into the salient. Diffusion of sediment into the lee of the reef also plays a strong role." Not all of these mechanisms are discussed in the main consent application document.

The main application is supported by the technical reports. However, more description of mechanisms and effects are provided in the updated technical report.

29 Pg 17, 5th para
Main Application

"The detailed design effort continued on from the basic reef parameters determined in the preliminary design and examined in greater detail the ability of the preliminary design to sufficiently ……."

What are the basic reef parameters? The purpose of this paragraph is not clear as the multi-reef design appears to share only little in common with the preliminary design.

The main application is supported by the technical reports – this information is well summarized in the Executive Summary of Appendix 2, with a great deal of additional information in the main text of both Appendix 2 and 4.

30 Pg 18, 2nd para
Main Application

The paragraph states that the reefs will not have a significant impact beyond the immediate vicinity of the reef in terms of modification to waves and currents. However this statement contradicts findings from the study of the Mount Maunganui reef.

The PhD study of the Mount Maunganui reef showed that the morphology of beach is significantly effected by the reef over a large area. The reef is 2500 m^2 in size, but morphological impacts extended to at least 1,000,000 m^2 (including the intertidal beach). Thus, the process of morphological coupling (iterative feedback between hydrodynamics and morphology) has caused the surfzone hydrodynamics to be modified over an area significantly larger (~400 times) than the surface area of the reef structure itself.

The main application is supported by the technical reports – This statement refers to local current patterns, however, feedback between systems reinforcing sediment retention, local scour in the lee of the reefs, etc, etc, will occur, i.e. it is envisaged that the reef systems will have an impact on some 1.5-2M m^3. of subtidal, intertidal and dry beach, noting that as with the Mount reef (which is in a very different environment and so caution should be take with any comparisons) the majority of effects are very small). These effects are defined and described in the updated technical report.

31 Pg 18, 2nd para
Main Application

There is a comment that even under extreme conditions the reefs did not have a strong interaction with each other as evident in Figures 9 & 10.

How is this evident? There looks like lover lap between currents modified by each reef suggesting that they do interact. Combined with the morphological evidence from the Mount Maunganui reef, it is possible that the hydrodynamics (currents, waves, sea levels) and morphology induced by the reefs will interact. The comments on p18 are also confusing combined with to p21 paragraph 1 which states that there is feedback between the reef systems (in the sediment transport context)

Full ranges of test cases have been undertaken and are included in the updated version of the report which explains this seemingly contradictory section (additional explanation has been given above).

32 Pg 20, 2nd para
Main Application

A better method of calculating the relationship between chart datum and Mean Low Water Springs needs to be provided?

Chart datum is a reasonably arbitrary datum, used to ensure that vessel can navigate safely. Recent conversations with former NZ Navy head hydrographic surveyor (Commander David Mundy) revealed that the separation between chart datum and a tidal can vary significantly within a chart, and between beaches. The use of method on p28 (section 2.5.4 of Appendix B) to calculate the separation between chart datum and other tidal datum's is not correct, putting the model findings in doubt. For navigational safety I would recommend calculating exactly the relationship between chart datum and the local orthometric heights (e.g. Auckland Vertical Datum 1946). For construction purposes it is more accurate to transfer the datum from an orthometric height than a chart datum (which varies along coastlines). Chart datum for the Navigational Chart nz5321, which includes Orewa, is 4.00 m below LINZ mark EB2K on Tiritiri Matangi Island. The EB2K chart datum mark is also ~17km from Orewa, and therefore some validation of the chart datum separation presented need to be provided.

As discussed and agreed at the first workshop, the datums were derived from actual measurements of sealevel at known locations. Bathymetry survey will be undertaken prior to construction and corrected with an on site pressure gauge to provide depths in terms of the measured water levels, which is the important factor –the height of the reef crests will be relative to the water level. In addition, as discussed, the difference of several mm in each direction is well beyond the reality of construction tolerances for coastal protection structures.

33 Pg 28, Table 2.6

Appendix B

The tide levels from the chart are wrong, or have been updated. The latest LINZ chart uses the following values for Tiritiri Matangi Island MHSW 3.0 m MHWN 2.5 m MLWN 0.8 m and MLWS 0.3 m. This puts the datum used for the modelling in doubt and the proposed crest heights for the reefs.

As previous and discussed at the workshop, our datum was derived from the actual measurements in relation to the known tidal constituents, which incidentally were in close agreement with the tidal chart information.

34 Pg 18, 1st para
Main Application

The discussion on rebuilding of the dry beach does not include the effect of upwelling and downwelling from onshore/offshore winds

This type of sediment transport can be significant, especially if working in conjunction with an erosive wave event. More detail on the magnitude of sediment transport during different wave and wind event combinations could be provided.

Upwelling and downwelling effects at Orewa Beach are likely to be insignificant due to the very low gradient of the beach, i.e. the great upwelling to occur in NZ occurs off the Farewell Spit which is on the edge of the continental shelf – upwelling and downwelling refer to the transport of nutrient rich or nutrient deplete waters into or away from the coast, and would be relevant to the whole of the Hauraki Gulf area, rather than individual beaches. Wave driven-currents are the dominant process through the wide surf zone present during storm activity. I believe that this statement is more in reference to the Dean equations (Dean, R., 1988. Prediction of Eroded Versus Accreted Beaches. Coastal Engineering Technical Note, US Army Corp of Engineers, CETN-II-2 6/88), which determine whether a beach will erode or accrete due to beach slope, sediment grain size and deepwater wave period – basically local 'steep' seas will cause erosion, while longer period swell (groundswell) will accrete the beach. These phenomena are regularly seen on the north east coast of NZ, where very big seas from distant tropical cyclones often result in a great deal of accretion rather than erosion due to their long period and local calm or offshore wind conditions. The effects of differing periods are incorporated into the models, and other than types of storm events, considering the type of beach response for each combination of possible wind and wave conditions would be of little value, and in addition, the 'normal' conditions that have been modelled to assess the development of the salient incorporate a range of conditions.

35
Main Application

No map of sediment character over the beach is presented.

Demonstrating an understanding of existing distribution of sediment is important if an artificial nourishment consent is being sort

The main application is supported by the technical reports. Manighetti & Carter's (1999) map of Hauraki Gulf sediments is provided in Appendix 4, with the beach sediments being described as very uniform and of order 0.12 mm (as described above from T&T's 1994 investigation).

36 Pg iii, para 2
Appendix B

The public is generally opposed to groins

What about the 'submerged groin' effect observed during the Mount reef PhD. Will the reefs cause a groin effect in the surfzone and on the shoreline?

The public's perception of groynes is of large structures on the beach that prevent access along the beach, are aesthetically unpleasant and cause downcoast erosion. It is a large leap of rationalization to consider the slight depth changes on either side of the Mount reef as a 'groyne' effect, with these changes also being linked to the longer-term wave climate, e.g. it is well understood that alongshore sediment transport along Mount Beach reverses due to changes in the La Nina/El Nino phases of the Southern Oscillation; this would result in reverse patterns of subtidal seabed changes.

37 Pg v, figure 5
Appendix B

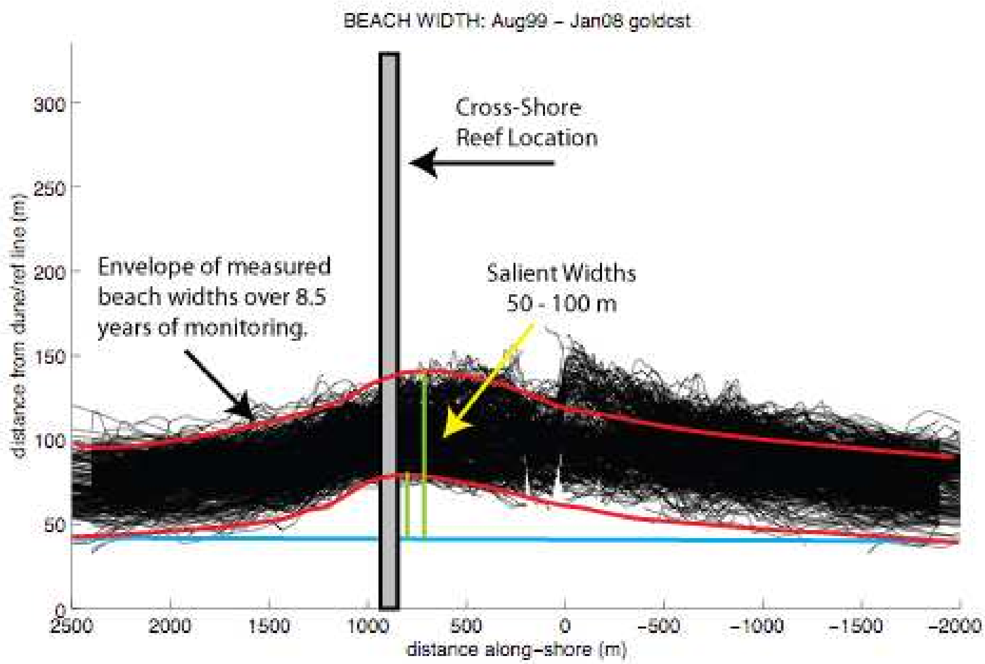
Why is sediment transport modelling of all reef systems not presented?

Full ranges of test cases have been undertaken and are included in the updated version of the report

38 Pg 49, para 1
Appendix B

Figure 3.2 is misleading because the peer-reviewed publication on the shoreline response to the Narrowneck reef (Turner, 2006) shows that the reef has only caused, on average, a 20 m wide salient.

This photograph shows no indication that the salient is 20 m, 40 m or 70 m wide? Indeed, the Gold Coast high tide mark fluctuates an average of 18 m each day, up to 60 m/day during events, and there are peer-reviewed papers stating that the Gold Coast salient is on average, 20 m wider, 40 m wider and 70 m wider… In order to determine a better understanding on these different claims, Dr. Jose Borrero digitized rectified time series images of salient at Narrowneck, as shown below. It is noted that the metocean conditions at Narrowneck are far different from those at Orewa, with the asymmetric salient extending several kilometers to the south of the reef; indeed the figure below indicates the impressive beach response achieved by the GC reef, some 3.5 km.



In terms of multi-purpose reefs, only one has ever been built with coastal protection in mind, the Narrowneck Reef on the Gold Coast in Australia. This structure was designed by ASR personnel in 1998. It has now been a decade since this award-winning project was completed, and numerous peer-reviewed papers have been written which describe the coastal protection achieved by the structure, including the one referred to:

Turner, I.L., Discriminating Modes of Shoreline Response to Offshore-Detached Structures, *Journal of Waterway, Port, Coastal and Ocean Engineering,* Vol. 132, No. 3, May 1, 2006

in which the shoreline response caused by the reef is quantified. Following the summary of the Narrowneck Reef project are some extracts from one of the earlier peer-reviewed papers on the monitoring of the beach response, and then a link to the great number of reports that describe the results of almost 10 years of monitoring of the ARGUS remote sensing system.

http://www.wrl.unsw.edu.au/coastalimaging/public/goldcst/index.php?page=goldcstMonitoringReports.html

While the monitoring reports can be quite dry and present the results without much discussion of the processes involved, GCCC Engineer John McGrath provided the following comments in discussion with John Hearin from ASR America in December 2008 (Mr. Hearin’s contact details can be provided, on request). These comments can be taken as an interpretation of Table 7.1 in the Blacka *et al*., 2008 monitoring report online at the above link:

Summary of the beach changes at Narrowneck based on monitoring data (J. McGrath pers. comm.):

1. Prior to reef construction, the Gold Coast beaches had been nourished on average every ten years since the 1970's. The last nourishment was done in 1987 (prior to 1999 project).

2. The average rate of beach recession was ~5 meters per year prior to the reef project. (Current rates are -1.5 to -3.6 according to WRL 2008/06). The current reduced beach recession is caused by a combination of over nourishment and reef stabilization.

3. The beach in the lee of the reef was purposely over-nourished in 1999-2000 (more so than the other sections of the project) with the understanding that the erosion rates would be higher at that location as the beach system moved towards dynamic equilibrium. In other words were higher erosion rates behind expected the reef and they expect that to continue until a dynamic equilibrium had been achieved.

4. The south reach experienced accretion for the first 6 years as the salient moved towards equilibrium (WRL 2008/06 table 7.1.).

5. Now that the south reach has achieved relative equilibrium the excess sediment is moving northward resulting in the slightly erosional trend of the last two years (-1.5m/yr).

6. The erosion trend is expected to equalize along the project shoreline once the entire system has reached equilibrium.

7. The reef has significantly stabilized the nourishment project and reduced the total rate of beach recession rate along the entire project.

8. It is not anticipated that any nourishment projects will be required in the area until 2030. In other words, the reef has extended the renourishment cycle well beyond the 10 year cycle that was experienced before the reef.

While not designed for coastal protection and too shoreward to have an optimum effect, recent investigations by Simon Weppe at the University of Waikato applying the same odd/even discriminating modes as Turner (2006) to the Mount reef indicate a salient response of 150-200 m alongshore and 20 m across shore at site.

39 Pg 8, Figure 4
Main Application

Quality of aerial photographs

Why are such low quality aerial photographs used? Are there no existing high-resolution photographs that exist where a surfzone is presents? This would provide an overview of where the reefs are located relative to the existing surfzone. Is it out of the question to collect photographs for this project as part of the design process? This would provide empirical information to support the modelling.

41 Pg 26, Figures 16-17
Main Application

Images to small and of low quality

Need better quality images that specify construction method, including potential issues and delays, and how they will be resolved.

42 Pg 27, 6th para
Main Application

Removal of reefs

If the reefs were to be removed, what would the impact of the artificial sediment be on the beach?

The sediment to be placed inside the containers is real sand, not artificial sediment. The impacts of releasing some 15,000 of m^3 sand of a similar grain size (or slightly bigger) into the Orewa Beach environment will be minor to insignificant and short term. The smothering effect on the local biota will be of lesser impact than a local storm that would suspend and move sand over the entire nearshore and intertidal region. The sand will migrate onshore and southwards to accumulate at the Waitemata groyne, and in the longer term (less than a year) would have no noticeable effect. It is noted that between 25,000 and 50,000 m^3 of sand was extracted from the Waitemata groyne and placed on the beach annually from 1994 to 2001 (a total of 235,000 m^3 in less than an 8 year periods) with no long term change to Orewa Beach – these data are present in the technical report that this review pertains to.

43 Pg 93, para 3
Appendix B

Settlement comments are not entirely correct

The study by Jackson et al. (2002) took a few point measurements with a staff and total station. The method is not considered robust enough and I completely reject the methodology used. The multibeam surveys of the Mount Reef clearly show that the bags settle, and that the settlement varies around the reef structure. Therefore a complete coverage method needs to be employed (such as multibeam) to really understand settlement. In the Mount reef case, when the sediment in the bags settled the crest of the bags lowered (~0.10-0.40 m) and the sides of the bags bulged out. In places settlement is > 1m. 3D visualisation of multiple surveys confirmed this behaviour. This settlement will not necessarily be detrimental to the reefs performance, but does need to be discussed in a realistic manner in the application.

Settlement effects are discussed above and in the technical report, which is in agreement with the findings of the multi-beam survey, i.e. 0.3-0.5 m of settlement. With respect to the Mount Reef, 13 surveys of the reef have recently been analysed and the results are in agreement with the previous analysis in 2007, i.e. apart from initial settlement, the containers have not settled – similar findings have resulted from analysis of 7 surveys of the Boscombe reef in the UK. Allowance for settlement has been built into the design.

44 Pg 93, para 3
Appendix B

Detail to back up scour comments are not provided

At the Mount reef scour has occur in the lee of the reef in the form of a large hole. However, offshore of the reef, the bags have been subject to inundation with sediment. Thus the comments that there will be scour needs to be balanced with any expected covering of sediment.

Scour patterns and prevention of undermining of the structure by scour are discussed in more detail in the updated technical report.

45 Pg 29, 2nd para
Main Application

Use of words "comprehensive monitoring programme"

This is a subjective phrase, and the monitoring presented does not appear to be detailed or comprehensive

The monitoring programme presented incorporates beach and seabed response, biology and recreational aspects, and include the incorporation of 5 new beach profiles in the lee of the reef to be surveyed quarterly, along with 6 monthly bathymetric surveys. It is expected that monitoring will be discussed and agreed upon further as part of the application and hearing process, as per the normal course of events. However, the monitoring programme is considered appropriate for determining beach response to the structures, which is of utmost importance with respect to initiating construction of future reef systems.

46 Pg 29, 3rd para

Main Application

"The bathymetry of the area and the beach
profile of the seashore and fore dunes have
been, and are continuing to be monitored"

The council has been doing monitoring of the beach profiles, but who, how frequent, over what area, how accurate and how dense has the bathymetric monitoring been? Why is a summary of this monitoring not presented in the application.

Monitoring is described in Section 5.6. of Appendix 2, which accompanies the main application, i.e. is part of it.

47 Pg 29, 4th para
Main Application

Can further details on the method for monitoring recreation and surfing be provided?

Based on the discussion in the application, it sounds possible that at times there might be no one to do the monitoring. The monitoring plan needs to be rigorous and well resourced before application of a consent.

48 Pg 29, 4th para
Main Application

What is the definition of a full bathymetry survey? And why is it dependent on local sea conditions

What are the accuracy requirements for the monitoring? What are the density of measurements and technology requirements?

49 Pg 29, 4th para
Main Application

Why is no beach safety monitoring been proposed?

Coastal structures increase the cellular rip circulation, making beach more dangerous for swimming. Problems with swimmer safety have occurred at the Mount reef.

Rip-cell development and swimmer safety has been incorporated into the effects section of the updated report.

50 Pg 29, 4th para
Main Application

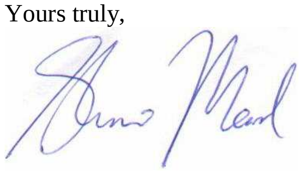
Why is no monitoring of the reef structure itself proposed in the main application?

Refer to – Section 5.6, Appendix 2.

This is proposed in Appendix B but not in the main consent application.

51 Pg 22, 4nd para
Main Application

Yours truly,



Dr. Shaw Mead
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APPENDIX 6

Geotextile Specifications: