Water Research Laboratory

A Review of Artificial Reefs for Coastal Protection in NSW

WRL Technical Report 2012/08
June 2013

by
M J Blacka, T D Shand, J T Carley and A Mariani

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Executive Summary

ES.1 Overview

This report presents a comprehensive review of the use of submerged constructed reefs (SCRs) for coastal protection in NSW. SCRs are often proposed as a 'softer' protection option due to their simulation of natural processes and negligible visual intrusion. Conclusive information regarding SCRs is less well documented within the literature compared with more conventional coastal structures. While the review has primarily focused on the use of SCRs for providing coastal protection, and in particular the applicability of SCRs for the NSW coast, a broader range of aspects have also been considered. The study was based on an extensive review of international literature that considered in excess of one hundred and fifty (150) references.

ES.2 Design and Analysis Methods

The stability of rock armouring on submerged breakwaters has been studied in numerous detailed investigations and reasonable empirical design guidance is available. Most relevant design methods for submerged rock structures have been summarised in the report, and the state-of-the-art empirical design equations have been recommended. In contrast, the understanding of the behaviour of large sand filled geotextile containers under wave attack is not yet well developed. A small number of studies looking at the stability of geotextile containers and tubes have been undertaken with varying approaches and results. However, there is no single publication presenting stability design curves or equations for geotextile tube submerged reef structures. While it is generally stated that the geotextile mega containers used in reef construction are so large that they are inherently stable, experience from existing reefs has shown that the tubes are able to be dislodged, re-worked, and damaged by wave attack.

There has been considerable improvement in the understanding of the mechanisms driving shoreline response to submerged reef/breakwater structures over the past decade, nevertheless all completed studies have significant limitations. No single study has comprehensively tested the effects of primary structural and environmental variables on quantitative shoreline response and the shoreline response equations published are based on either approximate field measurements of a limited number of parameters, or un-calibrated and un-validated modelling (both physical and numerical). This suggests that the available empirical techniques for assessing shoreline response are suitable only for preliminary engineering calculation and not detailed design. Structures that are designed using these methods should be considered as trial or experimental only and shoreline response to these designs will inherently contain higher uncertainty than many other beach control structures.

Numerical models are well suited to assessing wave, hydrodynamic, and morphological aspects of reef structures, with the degree of certainty in model predictions proportional to the level of model calibration. Physical models should be used preferentially for assessing reef armour stability, wave and hydrodynamic processes, and can also be applied to gain valuable qualitative and semi-quantitative insight into morphological response, but scaling limitations mean that they do not provide a complete answer. It is recommended that detailed design of any SCR structure adopt a hybrid modelling approach, whereby the individual strengths of both numerical and physical models are utilised to arrive at the final reef design. Furthermore, it is recommended that modelling of any structure with environmental, social, or economic significance be underpinned by site specific data collection programs for wave transformation, water levels, and sediment transport.

ES.3 Existing Reef Projects

A review of existing SCR structures around the world was undertaken with key engineering, environmental and cost information for each structure summarised in the report. Based on this review, the key findings were:

- Of the thirty-two (32) SCR structures reviewed, twenty-nine (29) were intended to provide coastal protection as a primary or secondary objective;
- Approximately half of the "protection" structures had no significant accretionary impact on shoreline alignment compared to the predicted morphological response;
- 55% of submerged breakwaters were successful at providing increased coastal protection, though not all to the degree initially predicted;
- One of five multi-purpose reef (MPR) structures may be providing a reasonable level of coastal protection but this structure has only been monitored for two to three years. Three other MPRs provide only minor or negligible coastal protection compared to design, and the performance of the newest MPR (Borth) is yet to be determined;
- Eight artificial reefs were constructed with the objective of improving surfability and approximately half of these were considered at least partially successful;
- The resulting shoreline morphology behind reef structures often differed significantly from the design predictions, even when the best available design methods were applied;
- Most structures settled and/or suffered from localised scour which resulted in an actual crest level which differed from that specified by design and subsequently led to further maintenance and top up costs or under performance; and
- Approximate construction costs per linear metre of coastline protected were in the order of $1,500 to $5,500 for submerged breakwater structures and $7,000 to $10,000 for MPR structures, compared with $5,000 to $10,000 for a high quality engineered rock seawall on the open NSW coast. The relatively high wave climate of the NSW coast is likely to further increase the construction costs of the offshore structures relative to the precedent structures located in milder wave climates.

ES.4 Application of Submerged Constructed Reefs in NSW

In recent years SCRs and in particular MPRs have been proffered as a coastal protection option for some NSW communities, due to the perception of the benefits outweighing the limitations. However, within NSW the use of MPRs that combine surfing and protection objectives are likely to be limited in success by a number of factors including:

- NSW has a tidal range of approximately 1.5 m and a multi-directional wave climate with a wide wave height and period distribution. To accommodate surfing as a design objective the cross-shore dimension of a MPR has to be large enough to allow proper wave pre-conditioning under a range of wave and tidal conditions. This makes the structures relatively cost-inefficient at protecting any significant stretch of coast, unless used in series (which is expensive compared to other protection options);

- Most sections of the NSW coast are relatively rich in high quality natural surf breaks, resulting in high community expectations if surfing is a primary design objective; and

- Safety concerns for the various reef users results in reef designs that are not optimum for coastal protection or surfing.

As with all coastal protection structures being considered within the NSW coastal management framework, it is important that feasibility assessments of SCR structures give consideration to several key points:

- The existing hazards need to be well defined before a reef can be assessed for feasibility, if coastal protection is an objective;

- A range of alternative solutions should be considered at the feasibility stage to allow selection of the best option to achieve the management objectives;

- The reduction in hazard that can be achieved by the reef needs to be predicted through technical assessments and quantified in terms of present and future hazard/risk reduction; and

- The predicted reduction in hazard should be considered in terms of its environmental, financial, and social costs and benefits.

ES.5 Future Applications of Submerged Constructed Reefs

On a relatively simple, straight coastline, it is likely that an emergent offshore breakwater designed in accordance with published methods would form a locally widened beach, provided there is sufficient available sand. The uncertainty in beach response increases as the crest elevation is lowered and the structure becomes submerged. This appears to stem from the complexity of processes leeward of the reef hampering understanding of the morphological response to reef structures in a naturally variable environment. As a result there is inherently a larger uncertainty associated with these structures. This uncertainty needs to be considered in any feasibility analysis, as it presents a significantly higher risk in comparison with other forms of coastal protection.

Consideration of SCRs built to date shows a relatively large number of structures underperforming in coastal protection objectives, even for cases where significant effort was put into very technical designs. This cannot be ignored when considering the current ability to be able to successfully predict the processes surrounding a SCR with required accuracy. Furthermore, many failures have been as a result of structural problems due to complexities of building a structure in an active surf zone on loose unconsolidated materials. This highlights the considerable improvements that are still needed in the design and construction of submerged reef structures.

Regardless of these current limitations, the potential benefits of SCRs mean that they should continue to be considered as an option for hard coastal protection, so long as the design and expectations take into consideration the lower level of certainty in performance. Future construction and monitoring of SCRs will result in an improved understanding of the processes and refined methods for predicting shoreline response to these structures. Throughout this period of ongoing improvement, consideration should be given to trial and experimental structures to reduce uncertainty and to create structures which meet the desired objectives.

The difficulty in attempting to meet multiple objectives is that the success in meeting one objective may be diluted by the attempts to meet the others. While some community groups may continue to favour multi-purpose structures due to their perceived benefits, there is little doubt that focussing the objective of coastal protection structures on coastal protection rather than multiple objectives will achieve improved results with more reliability and increased efficacy.

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1. Introduction

1.1 Foreword

The NSW coastline is a dynamic environment, frequently exposed to high energy wave events, which have the potential to cause episodic erosion of the upper beach. Parts of the coastline are also heavily developed with public infrastructure and private property occasionally threatened by such erosion episodes. While periodic erosion caused by large storm events affects the entire open coastline of NSW, the coastline generally recovers over subsequent months and years as sand returns onshore. Some locations, however, suffer from ongoing or chronic recession, often due to negative sediment budgets. As sea levels continue to rise, it is expected that the number of locations suffering chronic recession will increase. These differences in coastal processes are an important consideration when determining appropriate management and adaptation techniques. Globally, approaches to combat short-term erosion or longer-term recession have included both 'soft' measures such as beach nourishment with additional or relocated sand or 'harder' protection measures such as the construction of seawalls to restrict the erosion of material from the backshore, groynes to trap sand and increase beach width, and offshore breakwaters to reduce wave climate and longshore current in their lee.

This report reviews the use of Submerged Constructed Reefs (SCRs) to provide coastal protection. SCRs are often proposed as a 'softer' protection option due to their simulation of natural processes and negligible visual intrusion. While design guidance and case studies for more conventional structures are well covered within the literature, conclusive information regarding SCRs is less well documented.

The possibility of using submerged structures for shoreline protection has been recognised since at least the early 1950s with Johnson *et al.* (1951) reporting on the wave damping action of submerged breakwaters. Griggs (1969) proposed that similar submerged structures could be used to enhance beach amenity by creating waves conducive to surfing. Studies by Walker (1971; 1974) defined surfing parameters enabling more precise design of structures for such use, however, it was not until the late 1990s when structures were built specifically for this purpose (Pattiaratchi, 1999; Skelly, 2002). Due to the high costs of construction, and diverse-stakeholder interest in SCRs, multi-purpose structures suitable for surfing, enhancing marine life and providing beach protection, have more recently been constructed (e.g. Narrowneck Reef at the Gold Coast, Jackson *et al.*, 2007).

1.2 Terminology and Glossary

The terminology applied to submerged reefs and breakwaters can be somewhat inconsistent, and this has led to misinterpretation of previous studies and their outcomes. A clear and uniform set of definitions has been applied throughout this report to avoid this issue. These definitions have also been applied to reviews of previous studies, where the original authors may have used differing terminology.

The terminology applied throughout this report is summarised in Table 1.1 along with the range of other terms widely used in literature to describe reef structures. The term Submerged Constructed Reef (SCR) has been used within this report to include submerged breakwaters, artificial surfing reefs (ASRs) and Multi-Purpose Reefs (MPRs). Submerged breakwaters are a variation of the classical emergent offshore breakwater whereby wave energy is reduced by wave breaking and diffraction around the breakwater ends. ASRs are submerged reef structures where the primary design objective has been to provide increased surfing amenity, while MPRs have been designed to achieve multiple objectives such as coastal protection, ecological habitat, and improved surfing, fishing, or diving amenity.

Table 1.1: Submerged Constructed Reef Terminology

Submerged Constructed Reef (SCR) Definition: Any submerged artificial structure. Includes ASRs, MPRs, and submerged breakwaters	**Artificial Reef** Definition: An artificial reef structure, typically submerged during most tides, with a single intended purpose. This may include enhancing surfing amenity, enhancing marine habitat and associated amenity or for coastal protection. Other Common Names: Artificial Surfing Reef (ASR), Artificial Fishing Reef, Offshore Artificial Reef (OAR)
	Multi-Purpose Reef (MPR) Definition: An artificial reef structure, typically submerged during most tides, where the structure is intended to achieve multiple objectives. These may include erosion protection, marine habitat, recreational amenity including surfing, diving, fishing, etc. Other Common Names: Multi-Purpose Artificial Reef, Multi-Function Artificial Reef
	Submerged Breakwater Definition: A submerged artificial structure, detached from the shoreline and intended primarily to reduce wave climate in the structure lee (typically for coastal protection) and accrete sand in its lee. A reef breakwater is a specific subcategory of submerged breakwater where the armouring is a homogeneous rubble grading which is designed to reshape under wave attack. Other Common Names: Low-crested breakwater, low-crested structure, submerged rubble mound, submerged detached breakwater

Other technical coastal engineering terms used regularly throughout the report are described below:

- *Erosion:* Short-term loss of sediment from the upper beach due to storm effects, and generally large waves combined with elevated water levels;
- *Recession:* Long-term landward shoreline movement associated with a negative sediment budget or sea level rise;

- *Accretion:* Accumulation of sediment at the upper beach due to either a positive sediment budget, a change in environmental conditions or trapping of sediment;
- *Salient:* Localised sediment accretion, typically in the lee of an offshore structure, whereby the sediment build-up does not connect sub-aerially to the structure;
- *Tombolo:* Localised sediment accretion, typically in the lee of an offshore structure, whereby the sediment build-up connects sub-aerially to the structure. (The transition between *Salient* and *Tombolo* is not precise – these features may transition from one state to the other);
- *Protection:* A reduction in the effects of erosion or recession on a beach as a result of the installation of a structure.

1.3 Study Objectives and Limitations

This review has primarily focused on the use of SCRs for providing coastal protection, and in particular, the applicability of SCRs for the NSW coast. The review was based on an extensive review of international literature and considered in excess of 150 references. The literature list reviewed for this study includes document categories such as:

- Conference papers;
- Journal papers;
- Site specific design reports;
- Data collection reports;
- Physical/numerical modelling study reports;
- Technical notes;
- Design manuals;
- Research theses (honours, masters, PhD);
- Scoping and proposal documents;
- Presentation slides.

In general, the review has put more emphasis on recent documents that are considered to be based on robust science/engineering research, as well as studies that present information for existing reef structures. Nevertheless, other pieces of literature were considered for their contribution to the review, even if only to highlight gaps in the current understanding or to identify other references worthy of consideration.

This review has been divided into four main areas:

1. Summary of design information (Section 2);
2. Assessment of the use of numerical and physical models for SCR design (Section 3);

3. Assessment of performance of existing SCRs (Section 4);

4. The application of SCRs within NSW coastal management (Section 5).

Based on this review, conclusions have been drawn regarding the design, construction and use of SCRs for protection of the NSW coast.

While considered comprehensive and robust, this study was based on currently available literature and has not analysed or reanalysed any data. Unpublished and/or confidential reports may exist which were not available to the present study.

2. Design of Submerged Constructed Reefs

2.1 Background on Development of Submerged Constructed Reefs

The historical applications for submerged constructed reefs have included shoreline protection, recreational amenity, ecological improvement or increase to fishing potential. Bleck (2006) cites the first reported utilisations of artificial reefs as being for this latter purpose with Japanese fishermen submerging wooden frames after observing increased fish in the vicinity of shipwrecks. This application has remained the major reason for construction of submerged reefs with significant literature and design guidance available (i.e. Thierry, 1988; Lukins and Selberg, 2004; NOAA, 2007; The Ecology Lab, 2008).

Offshore structures are used to provide protection of the shoreline by promoting deposition of sediment in their lee (CIRIA, 2008). This is achieved in emergent structures by reducing or preventing transmitted wave energy, thereby altering the wave crest patterns in their lee and locally reducing the longshore sediment transport rate. Submerged or low-crested structures are a sub-group of this type of coastal protection whereby the structure is submerged for all or a portion of the tidal cycle. These structures are intended to break larger, less frequent waves but to allow transmission of more typical waves. If correctly designed, this is intended to reduce hydraulic loading to a required level to maintain dynamic equilibrium at the shore (CIRIA, 2008).

Low crest level breakwaters are often preferred for aesthetic reasons, however, the effectiveness of narrow, low-crested structures reduces with less freeboard, particularly in tidal environments and where storm surge frequently occurs (Pilarczyk, 2003). Effectiveness may be improved with wider crest levels, i.e. designing artificial reef structures rather than submerged breakwaters. Such structures have become popular in Japan (Pilarczyk, 2003), however, required reef volumes and associated costs are significantly higher than for narrow crested structures.

While Johnson *et al.* (1951) presented findings on the wave damping action of submerged breakwaters, one of the first examples of construction of a submerged breakwater for coastal protection was reported by Abecasis (1964). Following damage to breakwaters protecting the port of Leixoes, Portugal, and with rapid repair required due to the approaching storm season, design was changed to that of a submerged structure (Bleck, 2006). Subsequent functionality and economy of the structure were reported as positive. While emergent breakwaters have remained a popular, and generally successful means of coastal protection (Ranasinghe and Turner, 2006), fewer examples of submerged structures have been completed. Lamberti and Mancinelli, (1996) report of over 50 submerged breakwaters constructed in Italy. Ranasinghe and Turner, (2006) report on 10 such structures constructed as of 2006. They report on their physical characteristics and their effectiveness in terms of beach response and conclude that while beach response in the lee of emergent structures is nearly always accretionary, submerged structures have resulted in both accretionary and erosive beach response.

While the idea of using such artificial structures to enhance recreational surfing amenity was first proposed by Griggs (1969) in Surfer Magazine, the concept was not implemented until the late 1990s with a string of artificial reefs constructed in Australia and the United States for the sole or combined intent of enhancing surfing amenity and providing shoreline protection (Pattiaratchi, 1999; Skelly, 2002; Jackson *et al*. 2007). Since then a number of additional reefs have been completed in New Zealand, India and the United Kingdom.

There are approximately 70 breakwaters/training walls on the open coast of NSW. Many of these have associated surf breaks. Most of these were designed and constructed in the late 1800s and early 1900s, so no surfing amenity was considered in their design. Nevertheless, recognised surf breaks are now associated with many of these structures.

2.2 General Design Considerations and Available Guidance

The European Union DELOS project investigated engineering, socio-economic, and environmental aspects specifically with regard to low-crested structures. This project had contributions from a wide range of European investigation teams and is the most comprehensive and holistic design guidance currently available. While numerous investigation reports were produced as a part of the study (not all are publically available), Burcharth *et al.* (2007) is a published book entitled "Environmental Design of Low-Crested Structures" and presents a compilation of the knowledge and methods determined from the project. While it is comprehensive and best-practice with regard to submerged breakwaters, Burcharth *et al.* (2007) does not present information specifically targeted at MPR structures.

Yoshioka *et al.* (1993) present a "Design Manual for Artificial Reefs" based on several studies undertaken in Japan. This manual focusses on the technical aspects of wave breaking, dissipation, and hydrodynamics associated with submerged breakwater structures.

Pilarczyk (2003) presents design guidance for submerged structures in the publication "Design of Low Crested (Submerged) Structures - an Overview". This reference has a particular focus on wave transmission, hydrodynamics, and morphological response.

CIRIA (2008) provides a detailed scoping investigation into the application of MPRs in the UK with many aspects relevant and applicable to NSW. The scoping study presents useful case studies from existing projects around the world. In particular, the report presents detailed descriptive information regarding the planning and requirements for the various purposes for which MPRs can be designed. Little technical design information is presented.

2.3 Design of Reef Stability

2.3.1 Overview

For emergent rock structures, the first published work on rock stability was published in Spanish in 1938 by the Spanish engineer Iribarren (1938), "A formula for the Calculation of Rock-Fill Dykes". This related the size of rock armour required to the incident wave height. It was translated into English by the US Beach Erosion Board in 1949. The first original English language publication on rock stability was the work of Hudson (1953), "Wave Forces on Breakwaters", which remains in use for initial approximations of rock size on emergent structures.

The stability of rock armouring on submerged breakwaters has been studied in numerous detailed investigations (predominantly based on physical model studies), and reasonable empirical design guidance is available. In contrast, the understanding of the behaviour of large sand-filled geotextile containers under wave attack is not yet well developed. A small number of studies looking at the stability of geotextile containers and tubes have been undertaken, with varying approaches and results, however, there is no single publication presenting stability design curves or equations for submerged reef structures constructed from either rock or geotextile containers.

Table 2.1 presents a summary of the investigations relevant to the stability of reef armouring.

Table 2.1: Summary of SCR Stability Design References

Reference	Relevance
Ahrens (1987)	Response of Reef Breakwater Rock Armouring
Ahrens and Cox (1990)	Response of Reef Breakwater Rock Armouring
Vidal *et al.* (1992)	Reef Breakwater Rock Armour Stability
Van der Meer (1990)	Submerged Breakwater Rock Armour Stability
DELOS Project	Submerged Breakwater Rock Armour Stability
Hudson and Cox (2001)	Geotextile Container Stability
Recio and Oumeraci (2009)	Geotextile Container Stability
Borrero et *al*. (2010)	Geotextile Container Stability

2.3.2 Rock Armour Stability: Ahrens (1987) Method

Early studies into the stability of rock rip-rap when used in submerged reef breakwaters were presented in Ahrens (1987), with results based on a series of two-dimensional physical model studies. This investigation looked at the stable crest elevation above the sea bed (h_c) for given rock mass and density, water depth, and wave conditions. Figure 2.1 summarises the terminology.

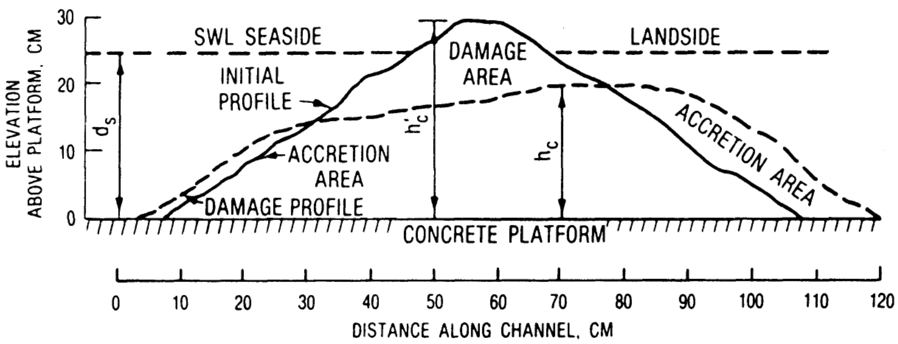


Figure 2.1: Diagram of Terminology for Reef Breakwater Rock Stability After Ahrens (1987)

This work characterised damage by the reef response slope which was observed to become flatter as waves become more severe, and followed Equation 1.

$$\frac{A_t}{h_c^2} = e^{(KN_S^*)} \qquad \textbf{Equation 1}$$

Where:

N_S* = Spectral Stability Number as shown in Equation 2.

$$N_S^* = \frac{\left(H_{mo}^2 L_p\right)^{1/3}}{\left(\frac{W_{50}}{W_r}\right)^{1/3}\left(\frac{W_r}{W_w}-1\right)} \qquad \textbf{Equation 2}$$

A_t = Cross sectional area of reef
h_c' = As built reef crest height above bed
h_c = Stable reef crest height above bed
K = Regression coefficient = 0.0945
W_{50} = Armour stone median weight
W_r = Specific gravity of armour stone
W_w = Specific gravity of water
H_{mo} = Spectral zero moment wave height
L_p = Airy wave length

Equation 1 is valid for values of N_S* greater than 6, as no reef reshaping occurs for lower stability numbers.

2.3.3 Rock Armour Stability: Ahrens and Cox (1990) Method

The two-dimensional investigations of Ahrens (1987) were later modified in Ahrens and Cox (1990) where the armour damage was determined as a more conventional damage parameter D', evaluated as shown in Equation 3.

$$D' = \frac{A_e}{d_{50}^2} \qquad \textbf{Equation 3}$$

A new equation estimating armour damage in proportion the the reef exposure was reported and is shown in Equation 4.

$$D' = C_0 + C_1 M + C_2 M^2 + C_2 M^3 \qquad \textbf{Equation 4}$$

Where:
M = Reef exposure parameter as shown in Equation 5

$$M = N_S^* \left(\frac{h_c'}{d_s}\right)^{3/2} \qquad \textbf{Equation 5}$$

C_0 = 19.4458
C_1 = -7.4546
C_2 = 0.760505
C_3 = -0.010478

2.3.4 Rock Armour Stability: Vidal et al. (1992) Method

Vidal *et al.* (1992) presented a two-dimensional stability analysis for low-crested reef breakwater structures, with stability charts presented for various parts of the structure:

- Front/seaward slope (FS)
- Crest (C)
- Back/leeward slope (BS)
- Total structure (TS)

The stability curves relate the Stability Number (N_s) to the structure freeboard for a range of different damage levels, with definition of damage levels as follows:

ID = Initiation of damage
IR = Iribarren's damage
SD = Start of destruction
D = Destruction

Damage was quantified using a dimensionless parameter (S) that was calculated by looking at the number of stones displaced from the structure per unit area, as well as by looking at the eroded area of armour. The equation for calculating S is shown in Equation 6.

$$S = \frac{ND_{n50}}{(1-n)X} \quad \textbf{Equation 6}$$

Where:
N = number of stones displaced per section of trunk length X
n = Porosity of rock armour

Thresholds of the various damage levels occurred at different 'S' values for the different sections of the structure, as shown in Table 2.2.

Table 2.2: Damage Parameter S for Different Damage Levels and Structure Sections

Damage Level	Front Slope	Crest	Back Slope	Total Structure
ID	1.5	1.0	0.5	1.0
IR	2.5	2.5	2.0	2.5
SD	6.5	5.0	3.5	4.0
D	12.0	10.0	-	9.0

The stability relationships are between the Stability Number (N_s) and the Adimensional Freeboard (F_d), where:

N_S = Stability Number as shown in Equation 7.

$$N_s = \frac{H_s}{(\Delta D_{n50})} \quad \textbf{Equation 7}$$

F_d = Non-dimensional freeboard as shown in Equation 8, with F = actual freeboard

$$F_D = \frac{F}{D_{n50}} \quad \textbf{Equation 8}$$

Stability charts for the various sections of the structure are reproduced in Figure 2.2.

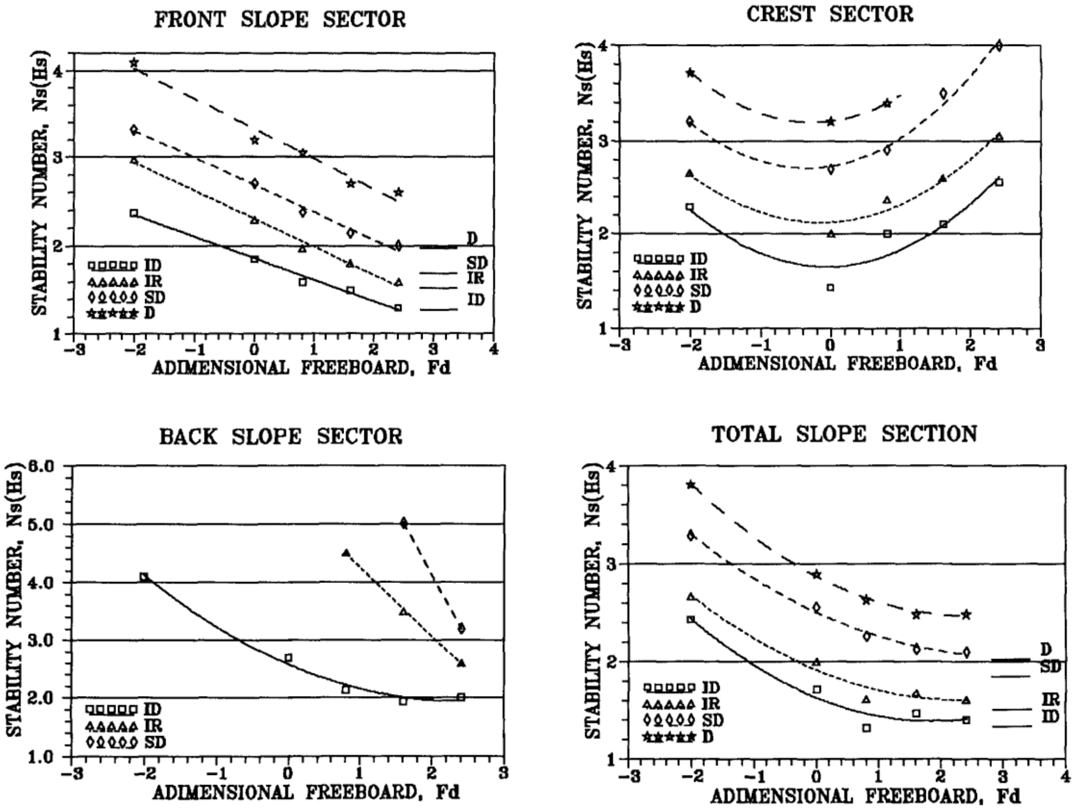


Figure 2.2: Stability Charts for Rock Armour After Vidal *et al.* (1992)

2.3.5 Rock Armour Stability: Van der Meer (1990) Method

Van der Meer (1990) presented two-dimensional stability formula and analysis for a range of low crested breakwater structures including reef breakwaters, submerged breakwaters, and emergent low-crested breakwaters. For reef type breakwaters, Van der Meer (1990) re-analysed the physical model data set from Ahrens (1987) along with another data set from Van der Meer (1988) to develop an updated armour stability equation as shown in Equation 9.

$$\frac{A_t}{h_c^2} = e^{(aN_S^*)}$$ **Equation 9**

Where:

The terms are as defined for Ahrens (1987) and shown in Section 2.3.2, with an updated regression coefficient, a, as shown in Equation 10 and $h_c = h_c'$ if h_c in Equation 10 is $> h_c'$.

$$a = -0.028 + 0.045C' + 0.034\frac{h'}{h} - 6.10^{-9}B_n^2$$ **Equation 10**

For statically stable rock armoured submerged breakwaters Van der Meer (1990) presents an analysis for structures with relatively steep side slopes of 1:1.5 to 1:2.5. It was found that structures with a submerged crest have more stable armour compared to emergent structures. The stability of submerged structures can be estimated by Equation 11.

$$\frac{h_c'}{h} = (2.1 + 0.1S)\exp(0.14N_s^*)$$ **Equation 11**

2.3.6 Rock Armour Stability from DELOS Project

Kramer and Burcharth (2003a) undertook a series of three-dimensional physical model tests to investigate rock armour stability for a wider range of structural and environmental parameters than previous research, and in depth limited breaking wave conditions. Based on data from previous investigations as well as the newer modelling, a more comprehensive equation for identifying the threshold for initiation of damage was proposed in Kramer and Burcharth (2003b) as shown in Equation 12. This equation was developed by fitting a curve under the envelope of all stability data from the various modelling studies, and therefore represents a conservative threshold for initiation of armour damage for various structural (slope, submergence, location, etc.), and environmental (water depth, wave height, wave period, wave direction) parameters.

$$N_s = 0.06F_d^2 - 0.23F_d + 1.36$$ **Equation 12**

Equation 12, considered to be applicable for non-dimensional freeboard (F_d) values between -3 and +2, is based on structures having a single armour grading over all parts and represents the initiation of damage for any part of the structure.

Under depth limited breaking wave conditions, on the assumption that the depth limited significant wave height was $H_s = 0.6d_s$, it was established from Equation 12 that the rule of thumb in Equation 13 applies for the minimum required armour size.

$$D_{n50} = 0.29h_c$$ **Equation 13**

Burcharth *et al.* (2006) presents the analysis for depth limited breaking wave conditions in slightly more detail and establishes that the minimum required armour size is given by Equation 14.

$$\frac{D_{n50}}{h_c} = \frac{\gamma/\Delta}{1.36-(\gamma/\Delta-0.23)^2/0.24}$$ **Equation 14**

Burcharth *et al.* (2006) went on to validate the rule of thumb in Equation 13 by comparison against constructed low-crested structures as a part of the DELOS project and also reported on validation against further physical model tests with depth limited waves. It was confirmed that Equation 13 was valid and that Equation 12 was also valid for depth limited breaking wave conditions.

While no specific testing was undertaken, Burcharth *et al.* (2006) made the observation that the toe armour stability equation proposed by Van der Meer (1995) was able to describe toe armour damage from the DELOS project physical modelling reasonably well. This relationship is shown in Equation 15.

$$N_s = \left(0.24\frac{h_b}{D_{n50}} + 1.6\right)N_{od}^{0.15}$$ **Equation 15**

2.3.7 Geocontainer Armour Stability

To date there have been few detailed and conclusive investigations into the stability of geotextile containers used in artificial reefs and indeed there are few empirical relationships for assessing the stability of submerged geotextile containers placed in SCRs. As discussed in Borrero *et al.* (2010), stability equations for rock or concrete armour units are not applicable to large sand filled geotextile containers and the considerations for material strength, structure settlement, and overall behaviour are also significantly different. Most sand filled geotextile containers used in SCRs are considerably larger (100s of tonnes) compared to rock armour (5 to 20 tonnes), and it is often reported that this makes them inherently stable under wave attack (Mead *et al.*, 2010; Borrero *et al.*, 2010). However, it is worthwhile considering that the large volume of the geotextile tubes also means that they each have a large surface area and therefore potentially large wave drag loading. Furthermore, the relative submerged density of sand filled containers is significantly lower than rock armour, which also has a significant impact on stability under wave attack.

Hudson and Cox (2001) examined the stability of submerged geotextile containers using 2D flume physical model testing. Based on the results of the model testing, Equation 16 was developed which relates the minimum stable size for a geocontainer to the primary environmental conditions.

$$N_s' = \left(H_c^{2/3} + L_p^{1/3}\right)/(\Delta B_h)$$ **Equation 16**

Where:
N_s' = Modified spectral stability number
H_c = Deepwater wave height
L_p = Airy deepwater wave length
Δ = Relative submerged density
B_h = Height of geocontainer

Recio and Oumeraci (2009) investigated the stability of sand-filled geocontainers by laboratory testing and derived stability formula using force-balance approach. By this approach a container is stable when the mobilising forces of wave-induced lift (F_L), drag (F_D) and inertial (F_M) are exceeded by the resisting forces of bag weight (W_{SFC}) and friction (μ). This is expressed generally as:

$$\mu(W_{SFC} - F_L) > F_D + F_M$$ **Equation 17**

While design equations were derived for geocontainers of fixed geometry, containers of alternative geometry require re-derivation (Borrero *et al.*, 2010).

Borrero *et al*. (2010) assessed the stability of existing geotextile tube SCR structures, including analysis of bag displacement, deterioration and failure. A numerical analysis of geotextile container stability was also presented using the Narrowneck reef as a case study. This numerical model solved the Navier-Stokes equation using a volume of fluid approach, to resolve the wave pressure field acting on individual containers and subsequently the dynamic load placed on the containers. The destabilising forces of the containers were compared to the resisting forces to estimate the stability of the containers. Borrero *et al*. (2010) suggested that for first-order assessment, the stability expressions of Recio and Oumeraci (2009) can be modified to a more generically applicable form using the cross-sectional area and length of the geocontainer rather than fixed height, length and width ratios.

2.3.8 Summary Box: Reef Armour Stability

The most up to date empirical equation for assessing the stability of rock armouring on SCRs is from Kramer and Burcharth (2003b) and Burcharth (2006), as previously shown in Equation 12, Equation 13, and Equation 14.

Equation 12 (repeated below) was developed using a combination of new data as well as data from most other previously published studies, and provides an empirical armour size design equation for a wide range of wave conditions and with structure non-dimensional freeboard (F_d) values between -3 and +2. This equation indicates the threshold of armour movement, and is therefore a useful first-pass conservative approximation for stable armour size on a reef:

$$N_s = 0.06F_d^2 - 0.23F_d + 1.36$$

For most NSW conditions, reef structures are likely to be located in breaking or broken waves during design wave events, and therefore Equation 14 as later presented in Burcharth (2006) is applicable for conservatively estimating required armour size:

$$\frac{D_{n50}}{h_c} = \frac{\gamma/\Delta}{1.36-(\gamma/\Delta-0.23)^2/0.24}$$

Few well developed methods exist for estimating the stability of large sand-filled geotextile tubes. In general, the empirical methods that do exist are based on containers that differ in shape and proportion to the tubes typically used in SCR structures. Given the large size of the containers, a first principle force-balance approach that considers the mobilising wave forces acting on the containers and the restoring forces of container mass and friction, as discussed within Recio and Oumeraci (2009) and Borrero *et al*. (2010), appears to be reasonable.

2.4 Design of Hydrodynamic Processes

2.4.1 Overview

Hydrodynamic considerations for SCR structures are typically focussed on wave transmission as well as localised currents generated as a result of the structure. The two processes are intrinsically linked and are the driving mechanism for sediment transport adjacent to and inshore of structures. The wave height observed in the lee of a structure is a function of both two-dimensional transmission over and through the structure, and wave propagation around the structure by refraction and diffraction processes.

There have been many studies investigating two-dimensional wave transmission over low-crested and submerged structures, ranging from the early studies of Dick (1968) through to the more recent studies such as Buccino and Calabrese (2007). Indeed there has been more research into wave transmission past (over, through and around) submerged structures than most other aspects. The general understanding of the processes has improved through time and modern empirical equations provide reasonable estimates of two-dimensional wave transmission if applied within their limits. Due to the large amount of research published in this field, only the more recent and widely accepted methods have been summarised in this review. These studies are considered to supersede the earlier research and provide the state-of-the art for analysis.

Current flow patterns generated as a result of SCR structures have been the focus of increasing research attention, as it has become apparent in trial field studies that these currents are responsible for localised scour at structure foundations (and resulting settlement), and also dictate the mode of beach response in the lee of the structure (erosion or accretion).

2.4.2 Wave Transmission after Ahrens (1987)

Ahrens (1987) as discussed in Section 2.3.2 also studied two-dimensional wave transmission for reef type submerged breakwaters. Based on a large physical model data set, an estimation of wave transmission was presented and is shown in Equation 18.

$$K_t = \frac{1.0}{1.0+0.0294(h_c/d_s)^{3.3338}(B_n)^{0.5857}}$$ **Equation 18**

Where the bulk number (B_n) is given by Equation 19 and the reef response parameter 'A_t' is defined in Equation 1.

$$B_n = \frac{A_t}{D_{50}^2}$$ **Equation 19**

2.4.3 Wave Transmission after Van der Meer and Daemen (1994)

Van der Meer and Daemen (1994) investigated the two-dimensional wave transmission coefficient for low crested and submerged rubble mound breakwaters, using the data set for reef breakwaters from Ahrens (1987) along with data sets from Van der Meer (1988), Daemen (1991), Daemrich and Kahle (1985) and Seelig (1980).

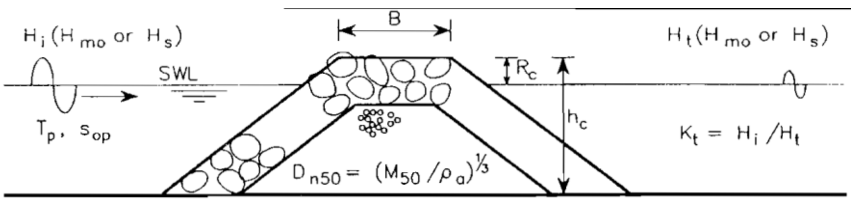


Figure 2.3: Variables for Investigation by Van der Meer and Daemen (1994)

The equation for wave transmission presented (Equation 20) is based on a linear relationship where wave transmission is proportional to crest freeboard and armour size. Two separate equations were presented for the parameter 'b', one for conventional submerged rock breakwaters and the second for reef submerged rock breakwaters (Equation 22 and Equation 23 respectively).

$$K_t = a\,\frac{R_C}{D_{n50}} + b$$ **Equation 20**

$$a = 0.031\frac{H_i}{D_{n50}} - 0.24$$ **Equation 21**

$$b = -5.42\,S_{op} + 0.0323\frac{H_i}{D_{n50}} - 0.0017\left(\frac{B}{D_{n50}}\right)^{1.84} + 0.51$$ **Equation 22**

$$b = -2.6\,S_{op} - 0.05\,\frac{H_i}{D_{n50}} + 0.85$$ **Equation 23**

Where the fictitious wave steepness, S_{op} is given by Equation 24

$$S_{op} = 2\pi H_S/gT_p^2$$ **Equation 24**

2.4.4 Wave Transmission after d'Angremond et al. (1996)

d'Angremond *et al.* (1996) further refined the understanding of wave transmission for submerged structures by building on the previous work and data sets from Van der Meer and d'Angremond (1991) and Van der Meer and Daemen (1994), as well as incorporating a series of new data into the analysis. In particular, d'Angremond *et al.* (1996) established revised equations for wave transmission by incorporating the effects of the armour/structure permeability into the analysis. Two separate equations were presented for the cases of a permeable submerged breakwater (Equation **25**) and an impermeable submerged breakwater (Equation 26).

$$K_t = -0.4\,\frac{R_c}{H_i} + \left(\frac{B}{H_i}\right)^{-0.31}\left(1 - e^{0.5\xi}\right)0.64$$ **Equation 25**

$$K_t = 0.4\,\frac{R_c}{H_i} + \left(\frac{B}{H_i}\right)^{0.31}\left(1 - e^{0.5\xi}\right)0.80$$ **Equation 26**

Where ξ is the Iribarren number given by Equation 27 with α being the structure slope

$$\xi = Tan\alpha/\sqrt{(H/L)}$$ **Equation 27**

2.4.5 Wave Transmission after Seabrook and Hall (1998)

Seabrook and Hall (1998) completed a series of 2D and 3D physical model tests of wide crested submerged breakwaters in order to refine the available empirical equations for estimating wave transmission. The physical model tests were compared to predictions made using one of Van der Meer's equations, and it was shown that for the wider crested breakwaters the equation failed to predict the physical model test results. A refined equation for wave transmission was presented, by Seabrook and Hall (1998) and is shown in Equation 28.

$$K_t = 1 - (e^{-0.65\left(\frac{R_c}{H_i}\right) - 1.09\left(\frac{H_i}{B}\right)} + 0.047\left(\frac{BR_c}{LD_{50}}\right) - 0.067\left(\frac{R_cH_i}{BD_{50}}\right))$$ **Equation 28**

2.4.6 Wave Transmission from DELOS Project

Burcharth *et al.* (2007) summarise the investigation undertaken by Briganti (2003) for wave transmission over submerged breakwater structures, which included a compilation of data from a wide range of physical model studies. It was concluded that the d'Angremond *et al.* (1996) equation for wave transmission of permeable structures (Equation 25) is applicable for structures with a narrower crest ($B/H_{si}<10$). For wider crested structures ($B/H_{si}>10$), an alternative version of the equation was suggested, as shown in Equation 29.

$$K_t = -0.35\,\frac{R_c}{H_i} + 0.51\left(\frac{B}{H_i}\right)^{-0.65}\left(1 - e^{-0.41\xi}\right) \qquad \textbf{Equation 29}$$

2.4.7 Wave Transmission after Bleck (2006)

Bleck (2006) approaches wave transmission from an energy conservation perspective with an analytical model developed for considering wave transformation with both breaking and non-breaking waves. The model considers a wider range of parameters than most established empirical methods, and therefore is able to be applied to a wider range of situations (as opposed to the empirical equations that are limited to the range of test conditions upon which they were based).

The model considers the following energy sink terms:

- Bottom friction;
- Internal friction;
- Vortex shredding;
- Wave reflection; and
- Flow resistance of reef and wave breaking.

The predictions of the analytical model when compared to physical model results indicate that it resolves the physical processes reasonably well.

2.4.8 Summary Box: Reef Wave Transmission

The empirical relationships for two-dimensional wave transmission past SCRs presented in Burcharth *et al.* (2007), as previously shown in Equation 25 and Equation 29, were developed from a wide range of physical model test data that encompassed most of the data used by other researchers around the world. These empirical relationships are the most evolved and therefore consider the widest range of reef and environmental parameters. Importantly, these empirical equations (repeated below) include the effect of cross-shore reef crest width, and are therefore able to predict wave transmission for both narrow crested submerged breakwater structures, as well as wide crested reef type structures. However, these equations still do not allow for wave refraction and diffraction around submerged structures, which is an important limitation when applied to reef type structures with a relatively small longshore crest length.

$$K_t = -0.4\,\frac{R_c}{H_i} + \left(\frac{B}{H_i}\right)^{-0.31}\left(1 - e^{0.5\xi}\right)0.64$$

Narrow Crested Structures with $B/H_{si} < 10$

$$K_t = -0.35\,\frac{R_c}{H_i} + 0.51\left(\frac{B}{H_i}\right)^{-0.65}\left(1 - e^{-0.41\xi}\right)$$

Wide Crested Structures with $B/H_{si} > 10$

2.4.9 Hydrodynamics and Forcing Mechanisms

Yoshioka *et al.* (1993), Ranasinghe and Turner (2006), and Ranasinghe *et al.* (2006) discuss the importance of offshore structure location (and to a lesser extent wave orientation and structure depth) on nearshore circulation patterns. In particular, the concept of circulation cells and the resultant modes of morphological response is presented in Ranasinghe *et al.* (2006), along with the possibility that submerged structures can result in erosion of the leeward beach.

Ranasinghe *et al.* (2006) examined the hydrodynamics and morphodynamics in the lee of a typical triangular MPR using a two-dimensional depth averaged numerical model (MIKE 21). A matching three-dimensional physical model was also developed and a reduced number of test conditions were run in order to be able to validate the hydrodynamic predictions from the numerical model. Qualitative assessment of current patterns in the lee of the reef were obtained using dye trace in the physical model for the validation exercise.

The numerical modelling investigation looked at a range of values for:

- Onshore/offshore position of structure from beach;
- Crest submergence; and
- Wave obliquity.

The study concluded that under shore-normal wave conditions either a two-cell or four-cell circulation pattern is established depending on the proximity of the structure to the shoreline. The influence of the reef distance from the shoreline and the resultant circulation patterns and modes of beach response (erosion or accretion) were found as follows:

- Structure located nearer to shore: two-cell circulation, divergent current, erosive; and
- Structure located further from shore: four-cell circulation, convergent current, accretionary.

Circulation patterns in the lee of a submerged reef structure that were presented in Ranasinghe and Turner (2006) are reproduced in Figure 2.4. Burcharth *et al.* (2007) discussed the difference in circulation patterns in the lee of submerged and emergent detached structures and also identified the two-cell/four-cell systems for submerged structures, governed by the structure proximity to the shoreline (Figure 2.5).

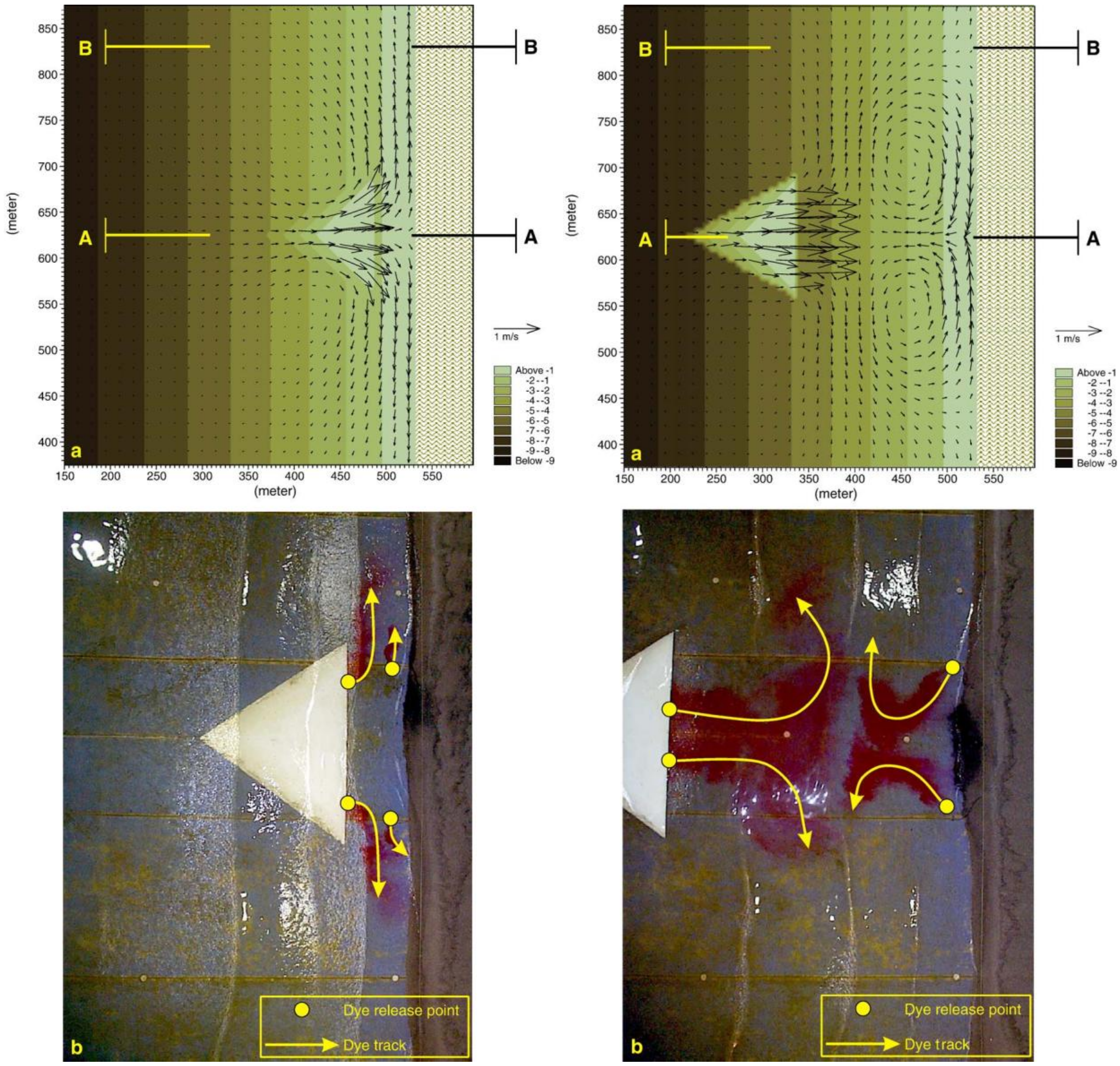


Figure 2.4: Circulation Patterns Around Submerged Structures (Ranasinghe and Turner, 2006)

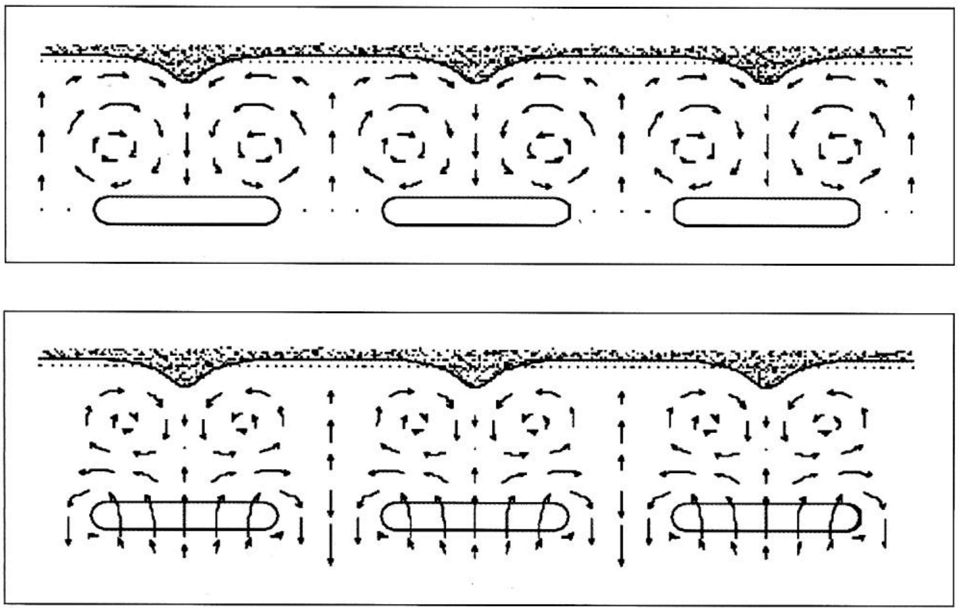


Figure 2.5: Circulation Patterns Around a Submerged Structure (Burcharth *et al.*,2007)

Ranasinghe *et al.* (2006) concluded that as a result of the driving circulation mechanisms, submerged structures can result in two contrasting modes of morphodynamic response, erosion or accretion, which contrasts with emergent structures where only accretion is expected to occur. These response modes are caused primarily by the cross-shore location of the structure, with structures nearer to the shore (typically inside the surf zone) resulting in two-cell circulation patterns where flows move over the reef to the shoreline before diverging alongshore and causing erosion. Structures further offshore (generally seaward of the surf zone) caused four-cell circulation patterns and promoted accretion. Salient growth was found to increase to some maximum value with increasing structure distance offshore, before decreasing if the structure is located even further offshore.

Yoshioka *et al.* (1993) also presented a summary of four different current pattern regimes that can operate in the lee of submerged reef structures, and more importantly, the range of reef parameters that result in each current pattern. Figure 2.6 shows the results of the Yoshioka *et al.* (1993) analysis where it can be seen that the current patterns identified are more complex than those presented in Ranasinghe *et al.* (2006) and extend to cover a wider range of cases (submerged long crested breakwaters and structures with very small crest length). It is important to note that like the conclusions of Ranasinghe *et al.* (2006), Yoshioka *et al.* (1993) also identified that there are some reef configurations that will result in erosion in the lee of the reef, while others will tend toward accretion.

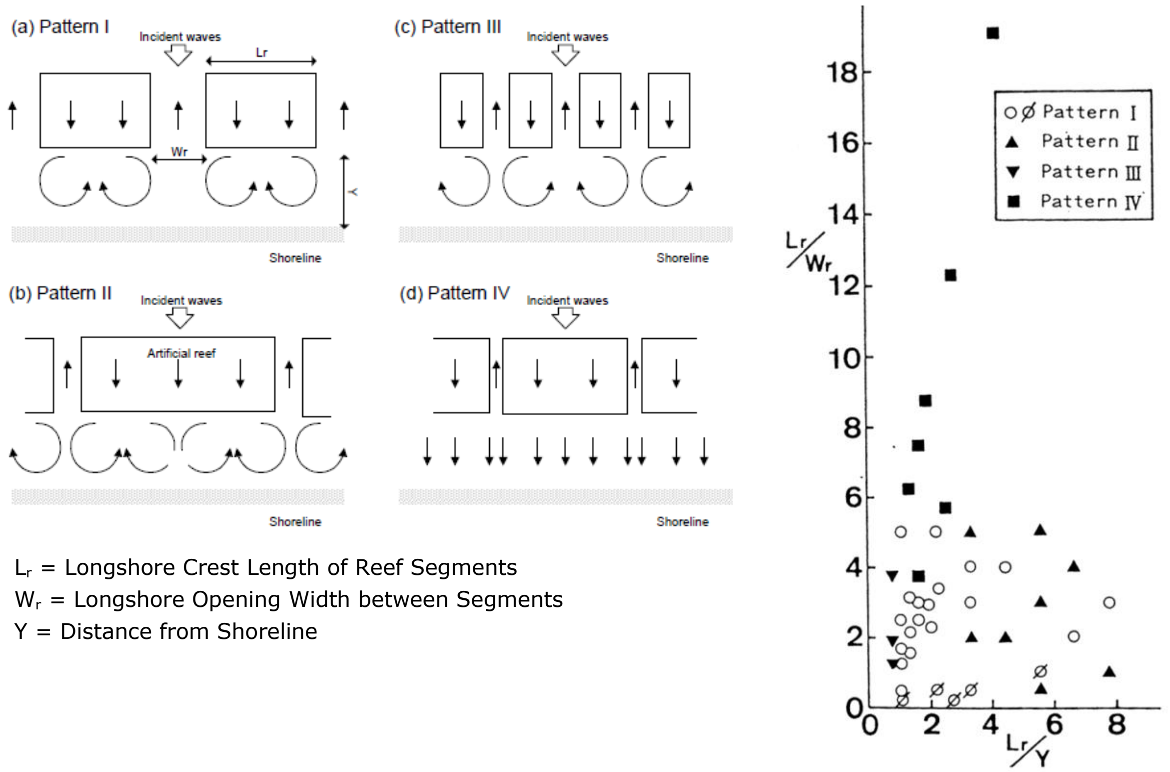


Figure 2.6: Current Patterns and Reef Configuration
(Yoshioka *et al.,* 1993)

2.5 Design of Morphological Response

2.5.1 Overview

Hanson and Kraus (1989; 1990; 1991) report that shoreline response to an offshore breakwater is controlled by at least 14 variables with 8 being primary controllers. While the principles behind submerged and emergent detached structures are similar, there are slight differences in the hydrodynamic mechanisms driving sediment transport and beach response, as was discussed in Ranasinghe and Turner (2006) and Burcharth *et al.* (2007). Nevertheless, the work by Hanson and Kraus highlights the complexity of morphological response to a detached (be it emergent or submerged) offshore structure.

2.5.2 Shoreline Response

Empirical relationships for shoreline response to emergent structures have been widely published in literature. Similar relationships for submerged structures have also been published but to a significantly lesser degree. Andrews (1997) investigated the beach response to naturally occurring islands and reefs, and used the data to develop relationships between structure location/geometry and equilibrium shoreline response. The analysis was undertaken using aerial photographs of salient and tombolo formations on the New Zealand and NSW coastline. A synthesis of the results are reported in Black and Andrews (2001), with definition of the terms reproduced in Figure 2.7. The formation of salients and tombolos in the lee of natural reefs were separated from the analysis of islands, and were predicted to occur as follows:

- No change to shoreline when: $\frac{B}{S} < \sim 0.1$
- Salient formation when $\frac{B}{S} < 2$
- Tombolo formation when $\frac{B}{S} > 0.6$

Where B is the structure longshore crest length and S is the offshore distance. The geometry of salients in the lee of natural reefs was proposed to follow a power relationship, according to Equation 30 and Equation 31:

$$\frac{X}{B} = 0.5\left(\frac{B}{S}\right)^{-1.27} \qquad \textbf{Equation 30}$$

$$\frac{Y_{off}}{D_{tot}} = 0.125\ (\pm 0.02) \qquad \textbf{Equation 31}$$

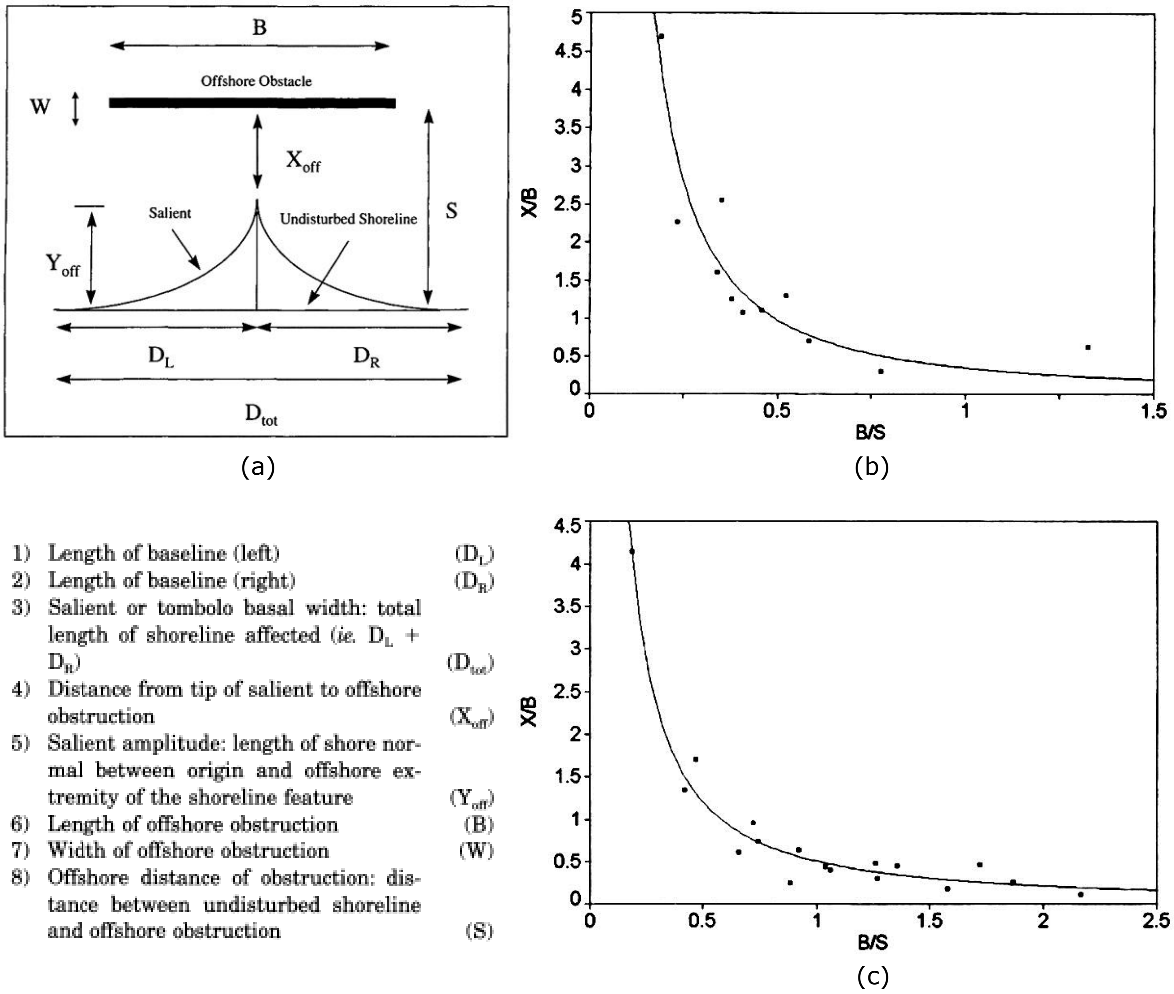


Figure 2.7: (a) Geometry Definition for Offshore Features; (b) Shoreline Response for Emergent Islands and(c) Shoreline Response for Submerged Reefs (Black and Andrews, 2001)

Evans and Ranasinghe (2001) used aerial photographs to estimate shoreline response comparing results with Black and Andrews (2001) and theoretical values by Silvester and Hsu (1997), (Figure 2.8). Evans and Ranasinghe (2001) did not differentiate between submerged and emergent features but did use visual examination to determine feature size. They suggested that it is likely that Black and Andrews (2001) underestimated reef size by not fully quantifying submerged reef portions, and therefore over-predicted relative shoreline response. No empirical relationship for shoreline response was presented in Evans and Ranasinghe (2001).

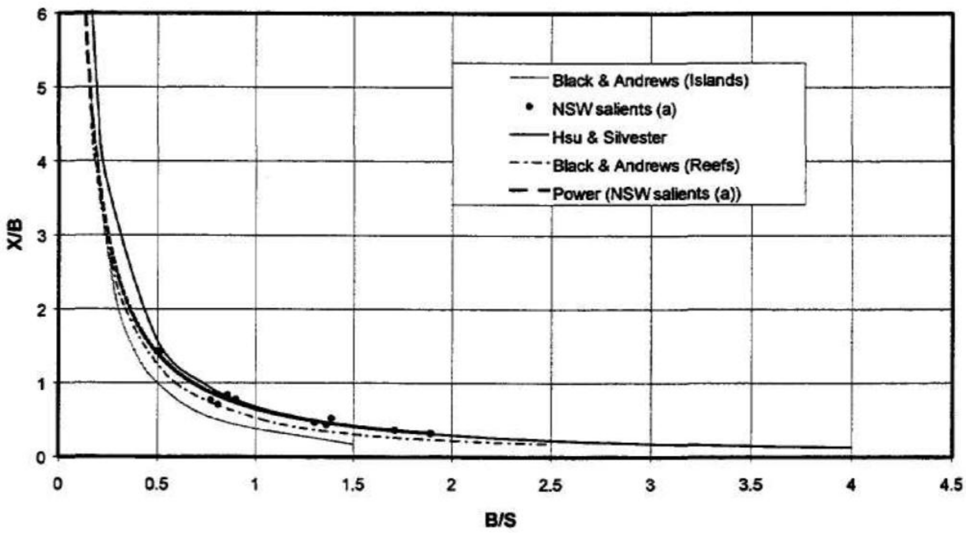


Figure 2.8: Analysis of Salient Features in NSW (Evans and Ranasinghe, 2001)

Pilarczyk (2003) summarises the work of Black and Andrews (2001), but also suggested that an empirical formula for shoreline response to a submerged structure could be developed by including a term for wave transmission/sheltering (1-K_t term) into the formula for emergent breakwaters. Possible equations proposed by Pilarczyk (2003) for submerged breakwaters were:

- Salient formation when: $\frac{B}{S} < \frac{1}{1-K_t}$
- Salient formation for multiple breakwaters when: $\frac{GS}{B^2} > 0.5(1-K_t)$

Where G is the gap between the breakwaters.

Though the morphodynamic response predicted in the modelling by Ranasinghe *et al.* (2006) was not calibrated or validated, relationships for morphological response were proposed for preliminary engineering calculations. In this case the ratio of the distance offshore (structure apex to undisturbed shoreline) to the natural surf zone width is plotted versus the ratio of salient width (Y) and the structure longshore crest length (B). This plot is shown in Figure 2.9 where it can be seen that if the structure is close to the shore (within the natural surf zone, Sa/SZW<1), then erosion is likely to occur. Likewise, if the structure is outside the surf zone (Sa/SZW>1.5), then accretion is likely to occur, though accretion is predicted to peak when the structure is located twice the distance offshore of the natural surf zone width (Sa/SZW=2.0).

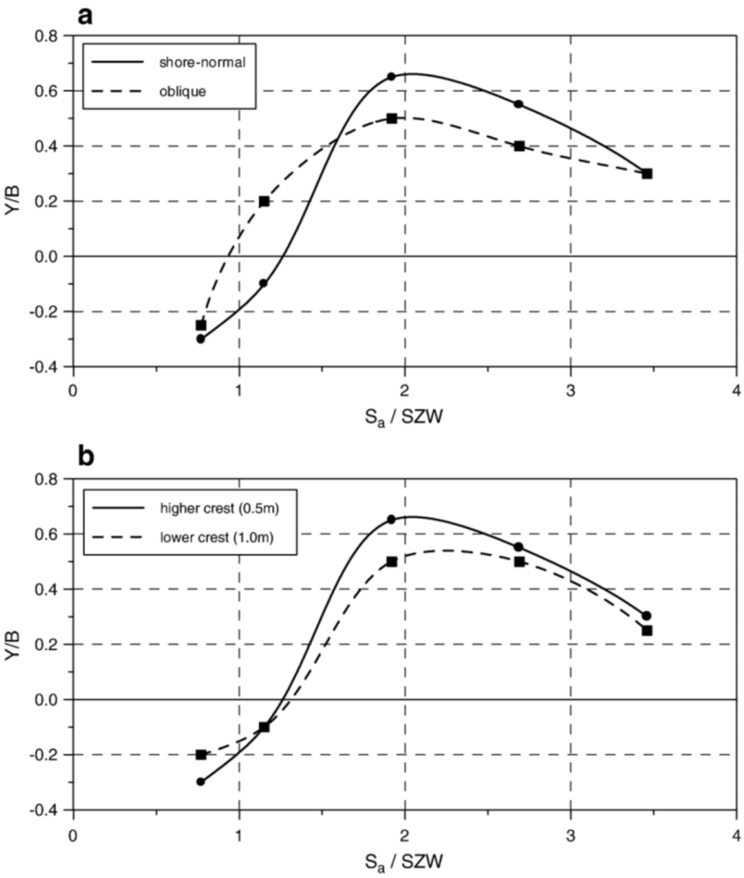


Figure 2.9: Shoreline Response to Submerged Structures (Ranasinghe *et al.*, 2006)

Savioli *et al.* (2007) examined a technique to predict the mode of shoreline response (erosion/accretion) to submerged shore parallel breakwater structures through the application of a two-dimensional depth averaged numerical model. This work was extended in Ranasinghe *et al.* (2010) who investigated the effect a wider range of structural and environmental parameters on shoreline response. The analysis was undertaken using both theoretical (dimensional analysis) and numerical modelling techniques to ascertain the relative importance of the governing variables on shoreline response mode. Based on the numerical modelling, the plot shown in Figure 2.10 was produced, which indicates a relationship between key structural/environmental parameters and the resulting occurrence of shoreline erosion or accretion. The derived expressions indicate that shallower structures, located further offshore tend to result in hydrodynamic response more conducive to accretion. Increased crest width was found to promote an accretional response in reefs at shallow submersion and tidal influence was not found to significantly affect response modes. Overall the method has potential as a first-order design tool but is presently un-validated, with further verification against field data and/or large-scale moveable-bed physical models recommended.

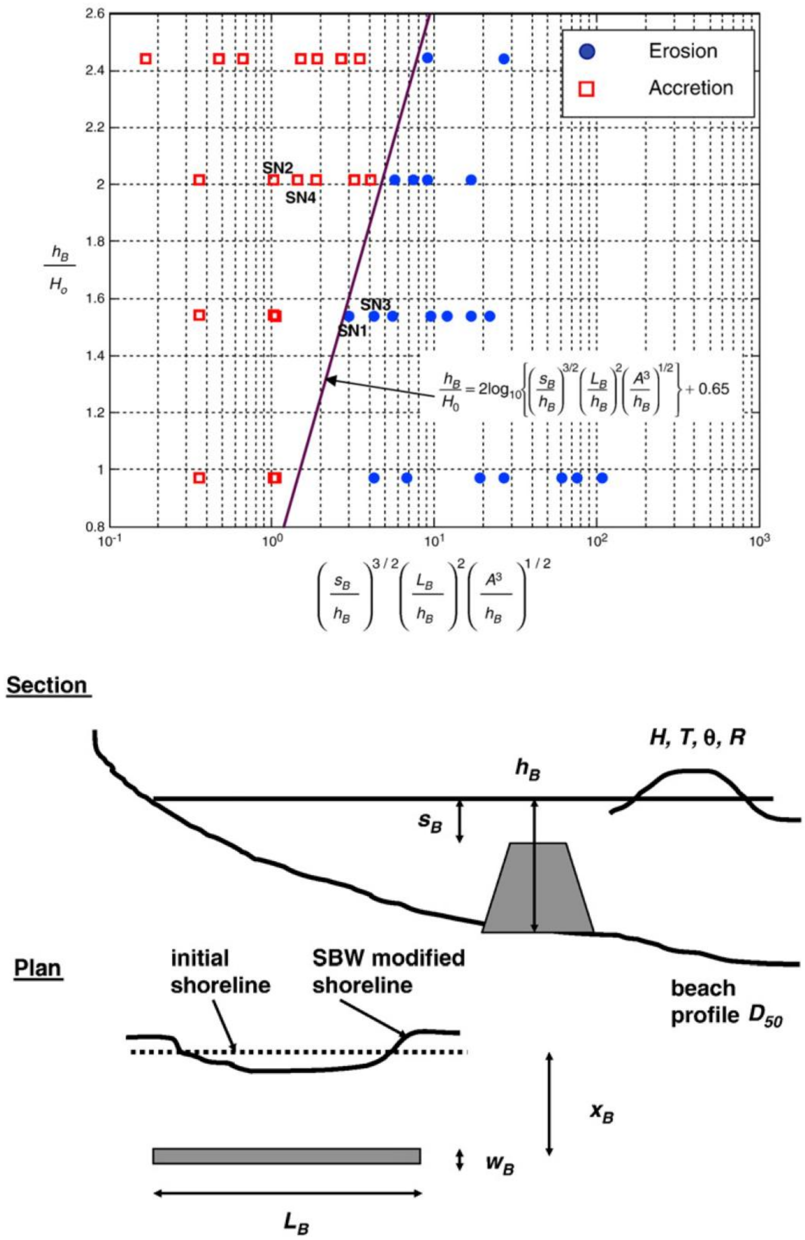


Figure 2.10: Mode of Shoreline Response to Submerged Breakwaters (Ranasinghe *et al.*, 2010)

As well as the empirical relationships presented above which make an attempt to predict the beach response to specific reef configurations, general guidelines and advice regarding the morphodynamic response of a beach in the lee of a submerged structure is available in several other literature sources. Lamberti and Mancinelli (1996) suggest that to retain and build sand in the lee of a submerged breakwater, the active toe of the inner beach should be landward of the submerged breakwater structure, i.e. active sand from the beach should not reach the landward toe of the breakwater. This can be achieved by ensuring the reduction in wave height caused by the breakwater is adequate to allow development of the full protected beach profile to the closure depth landward of the submerged breakwater. Yoshioka *et al.* (1993) suggested that to protect from beach recession due to an imbalance in littoral drift rate, then a target wave transmission coefficient can be determined on the basis of the required ratio in background littoral drift rate (Q_i) and the littoral drift that will occur in the lee of the SCR (Q), as shown in Equation 32. Presumably this rule can also be used to estimate the fraction of littoral drift sand that would be trapped by a SCR for a given wave transmission coefficient.

$$K_t = \left(\frac{Q_i}{Q}\right)^{2/5}$$ **Equation 32**

2.5.3 Summary Box: Morphodynamic Beach Response

While several different empirical equations have been proposed for prediction of shoreline response to submerged structures, all have fundamental limitations that restrict their use to preliminary design guidance. Nevertheless, the underlying principles of the methods used are not unreasonable and the equations do present valid preliminary methods for assessment. Understanding and predicting morphological response in the lee of SCR structures is more uncertain than other design parameters such as armour size and wave transmission.

Black and Andrews (2001) and Evans and Ranasinghe (2001) both base their analysis of measuring salient/tombolo size in proportion to reef size for naturally occurring features. This technique has the benefits of being based on real world experience and eliminates problems associated with numerical or physical model analysis. However, the methods used to determine the size of the reef structures in both cases are simplistic and approximate with no detailed bathymetric survey of the structures taking place. The proposed equations do not relate the shoreline response to wave exposure or wave transmission, which logic and other research suggests both have a significant impact on shoreline response. These methods have also been criticised in literature as they inherently cannot predict a shoreline erosion mode as a result of a submerged structure (which has been shown in other studies to exist). The method of Black and Andrews (2001) has been applied during the design phase of several existing SCR structures and has been shown to over-predict salient size (examples include Narrowneck and Mount Maunganui reefs).

Ranasinghe *et al.* (2006) examined shoreline response using a numerical model that is partially validated against a physical model (only qualitative validation of hydrodynamics, no validation of morphodynamic response). While the driving mechanisms for beach erosion/accretion in the lee of submerged structures are examined in detail and are expected to be qualitatively correct, the prediction methods for magnitude of shoreline response and threshold of erosion/accretion modes should be considered with caution due to the relatively un-validated nature of the morphodynamic model used. Savioli *et al.* (2007) and Ranasinghe *et al.* (2010) both apply a similar numerical model, however, neither the hydrodynamics or morphodynamics of the model were calibrated or validated. The outcome of these studies indicates the dependence of shoreline response mode (erosion/accretion) on circulation cell patterns determined by key structural and environmental variables. However, it is clearly pointed out in the conclusion of the study that the dependency cannot be comprehensively tested due to limited availability of good quality prototype data.

While there has been considerable improvement on the understanding of the mechanisms driving shoreline response for submerged reef/breakwater structures over the past decade, all completed studies have significant limitations. No single study has comprehensively tested the effects of primary structural and environmental variables on quantitative shoreline response, and the shoreline response equations presented are based on approximate observations or un-calibrated and un-validated modelling. None of the discussed empirical methods are able to capture the underlying differences in processes between swash-aligned and littoral drift-aligned beaches, as are frequently experienced on the NSW coast. The shoreline response to a reef structure on these different beach types will also differ, with littoral drift beaches experiencing:

- Asymmetry in resulting shoreline alignment;
- Potential down-drift erosion as a result of the formation of a salient;
- Potential up-drift 'groyne' effect.

This suggests that the available empirical techniques for assessing shoreline response are suitable only for preliminary engineering calculation and not detailed design. Structures that are designed using these methods should be considered as trial or experimental only, as it has been shown from previous prototype experience that the available equations do not predict beach response with accuracy or reliability. This results in designs that inherently contain higher uncertainty than many other beach control structures and this should be considered during the options planning and feasibility phase.

Regardless of these limitations, the benefits of SCR structures mean that they should still be considered as an optional form of hard coastal protection, so long as the design and expectations take into consideration the lower level of reliability in prediction of the morphological response. Ongoing construction and monitoring of SCRs will result in a better understanding of the processes and refined methods for predicting shoreline response.

2.6 Surfing Amenity Considerations

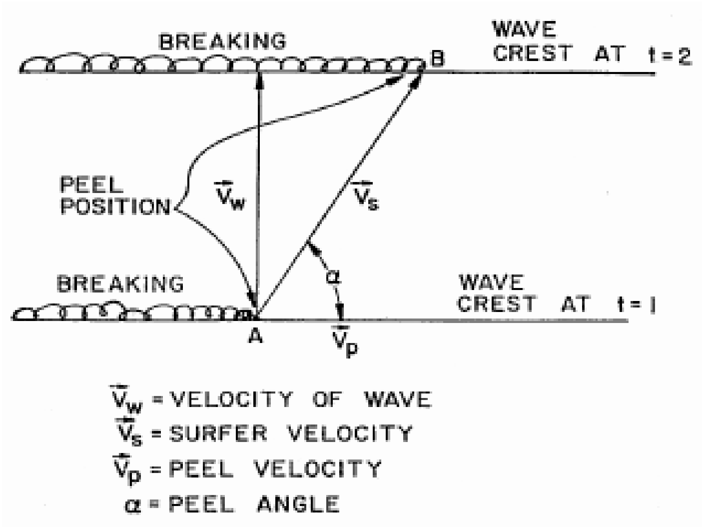
The concept of using artificial structures to enhance recreational surfing amenity was first proposed by Griggs (1969) in Surfer Magazine. Significant work has since been undertaken both in the laboratory and field to quantify wave breaking parameters relevant to surfing. Important parameters have been found to include wave breaking height, peel angle and wave breaking speed (Walker 1971; 1974; Mead and Black, 2001), wave breaker type (Galvin, 1968; Battjes, 1974; Dally, 1990; Smith and Kraus, 1991) and breaking intensity (Sayce, 1997; Black *et al.*, 1997; Mead and Black, 2001). These wave breaking characteristics were related to surfers' skill level (Walker, 1974; Hutt *et al*., 2001) and to specific world-class surfing locations (Mead and Black, 2001).

Mead and Black (2001) analysed 34 world-class surf breaks and classified the various morphological features into specific components based on their influence on wave processes. These components include:

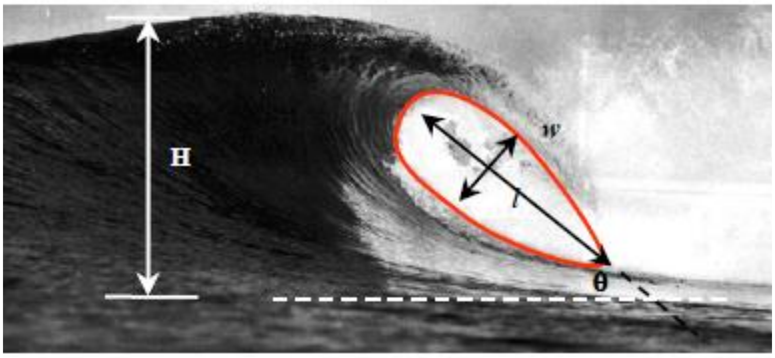
- Ramp;
- Platform;
- Focus;
- Wedge;
- Ledge;
- Ridge; and
- Pinnacle.

Each of these components influence either the pre-conditioning, break point or break type of surfing waves and should be considered if designing a reef with surfing amenity as an objective. Figure 2.11

defines the typical parameters used to characterise breaking waves for analysis of surfability. Figure 2.12 relates these parameters to a level of surfer skill.



(a)



(b)

Figure 2.11: Parameters used to Quantitatively Describe Surfing Aspects of Breaking Waves (a) Walker (1974) and (b) Black *et al.* (1997)

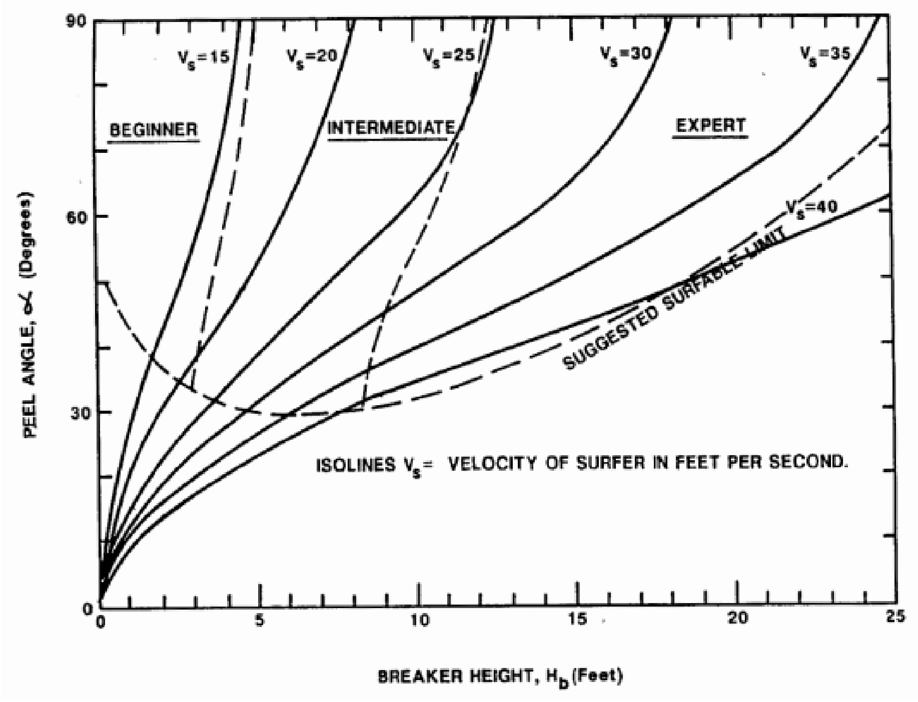


Figure 2.12: Breaking Wave Classification for Surfer Skill Level (Walker, 1974)

Using the relationships established between bathymetric components and resulting wave breaking characteristics, several artificial structures were constructed from the late 1990's for the sole or combined intent of enhancing surfing amenity and providing increased shoreline protection (Pattiaratchi, 1999; Skelly, 2002; Jackson *et al*., 2007). While many of these structures were not completed to specification due to construction difficulties or material failure, the surfing amenity produced by those that were completed has generally failed to live up to public expectation (Shand, 2011). However, Shand (2011) points out that the success of physical models and prototype structures during design (ideal) conditions indicates that the physical understanding of wave breaking parameters and bathymetric components required to obtain such breaking is likely sufficient to adequately design structures for surfing amenity. The major issue remains an expectation by the general public of consistent, quality waves during a wide range of environmental conditions.

In natural environments, consistency of surfing waves during a range of tidal conditions, wave heights, directions and periods is achieved by wave pre-conditioning and/or large scale breaking features (Mead and Black, 2000). The scale of ASRs constructed to date have generally been too small to allow adequate preconditioning and wave breaking during a range of conditions (Borrero and Nelson, 2003; Shand, 2011) but rather have been limited to specific environmental conditions. However, general public and media expectations have been for structures that produce surfable waves with high consistency. This disconnect between stakeholder expectation and realistic design outcomes has been detrimental to the public's confidence in such structures and to the future prospects of ASRs.

Predicting the consistency and range of conditions during which a reef will produce surfable waves is a crucial aspect of assessing the feasibility of MPRs, as the large required reef volume and limited longshore extent required to produce surfable waves also makes these structures inefficient at providing coastal protection to a long stretch of beach. While designers appear able to adequately predict, measure and classify the breaking waves that a reef will generate, using tools such as physical models, reefs in the past have still not performed to stakeholder expectations for several reasons:

- Translation of model predictions of surfability during a limited number of test conditions, to the wide range of naturally occurring conditions at the prototype structure;
- The “ruler edged” long crested model waves versus the “peakiness” of many real world swells;
- The single swell source of model waves versus the potential for multiple simultaneous swell sources in nature;
- Construction methods have prevented accurate construction of reefs to initial design specifications;
- Other factors such as public safety have required design modifications (i.e. lowering of crest level); and
- Unrealistic stakeholder expectations.

While it seems reasonable to consider providing improved high-level surfing amenity in the design of an ASR or MPR, Jackson (2002) highlighted that in reality, reef users encompass a much wider group, with wider views, requirements, and expectations as shown in Figure 2.13. Managing these wide community expectations while still designing a structure that provides coastal protection is a difficult combination (Jackson, 2002).

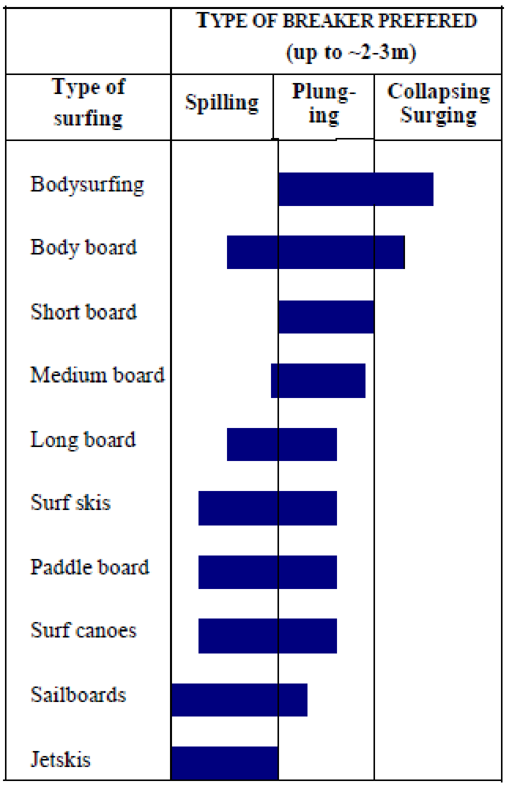


Figure 2.13: Type of Wave Preferred by Different Reef Users (Jackson, 2002)

2.7 Reef Construction Materials and Methods

2.7.1 Overview of Construction Materials

Construction materials for coastal structures are discussed in detail in CIRIA (2007) and summarised with respect to SCRs in CIRIA (2008). Lukens and Selberg (2004) also provided guidelines for reef materials with a focus on marine habitat. SCRs are typically constructed using the following materials:

- Rock/rubble;
- Geotextile containers and tubes;
- Pre-cast concrete units; and
- Other materials including sunken ships, car bodies, tyres, 'material of opportunity' (although these have generally been used for ecology focussed reefs).

Each construction material has a range of advantages and disadvantages, as well as suitable applications. Due to the site specific nature of reef installations, each material needs to be considered for its suitability when designing a new reef. Key aspects of material selection that should be considered include:

- Ability to achieve desired outcomes (ASR, MPR, submerged breakwater, ecological enhancement etc.);
- Stability under design conditions;
- Environmental impacts;
- Durability throughout design life;
- Constructability and placement methods for specific site;
- Safety and risks; and
- Ability to adjust and/or adapt design.

2.7.2 Rock Armour

Rock armour has long been used as a coastal protection material and is considered one of the most durable and adaptable materials available. In NSW rock has been successfully used to armour structures ranging from open coast emergent breakwaters through to revetments located within sheltered water bodies and estuaries. Rock also has the appeal of being a natural material and can be sourced to match the substrates that form natural reef systems within NSW. In Australia, rock was successfully used to armour the Cable Station ASR and it has also been used widely throughout Europe for construction of submerged breakwaters. The design guidance for assessing the stability of rock armour on submerged structures is also considerably more advanced than other armouring materials, making rock designs lower in uncertainty.

The longstanding use of rock armour in NSW means that the most suitable types of rock have been well defined, source quarries have been identified and costs for supply are well established. However, there are limitations to the types of locations where rock is considered a suitable armour material. Most SCRs to date have been constructed in water depths of 3 m to 5 m, with the exception of the Gold Coast Reef at Narrowneck which extends to a depth of 10 m, and Cable Station reef which extends to a depth of 6 m. Based on the empirical equation from Burcharth *et al.* (2006) for stable reef armour size in depth-limited breaking wave conditions (Equation 14), required rock armour sizes have been estimated for a reef with crest level 1 m below the water surface. The results are shown in Table 2.3 for a range of water depths. Given that the upper limit of armour stone mass that can be sourced in sufficient quantity from most NSW hard rock quarries is less than 10 tonnes, it is apparent that rock armoured reefs are only suitable on the open NSW coast when located in water depths of the order of 6 m or shallower. It can also be seen that rock may be a very efficient armour material for reefs in relatively sheltered areas where a reef would be located in shallower water depths of the order of 2 m.

Table 2.3: Required Rock Armour Size for Various Reef Depths (with Crest Submerged 1 m)

Water Depth at Structure (d_s) (m)	**Reef Crest Height, h_c (m)**	**(1) Assumed Significant Wave Height at Reef, H_s (m)**	**Required Armour Stone Size, D_{n50} (m)**	**Required Armour Stone Mass, M50 (kg)**
2	1	1.2	0.27	50
4	3	2.4	0.80	1,350
6	5	3.6	1.33	6,250
8	7	4.8	1.86	17,150
10	9	6.0	2.40	36,460

(1) Depth-limited wave conditions assumed with $H_s = 0.6d_s$

Construction of detached rock armoured structures typically occurs by means of barge mounted crane and/or excavator (Figure 2.14). Regular surveys of the reef during construction allow the structure to be trimmed and topped up as needed to closely match the design profile. This method is typically restricted to wave conditions of 0.5 to 1.0 m wave height (Abbot and Price, 1994) which on the NSW open coast would be exceeded 80 to 99 % of the time. Clearly this has implications for possible productive construction periods and flow-on effects for construction time and costs.

Figure 2.14: Placing Rock on the Cable Station Reef by Barge Mounted Excavator (Jackson and Corbett, 2007)

2.7.3 Geotextile Tubes/Containers

A number of existing reef structures have been built from very large geotextile tubes or "mega containers". The fabric used and methods of construction have evolved with time and this reef material has now been shown to produce satisfactory results (Jackson, 2010; Borrero *et al.,* 2010). Earlier reef constructions using geotextile tubes were plagued with difficulties and several such reefs failed to accurately achieve design as a result of difficulties during construction or poor performance of the geotextile material (Mead and Borrero, 2011). Smaller geotextile containers (approximately 8 m^3 individual volume) were used for construction of the Pratte's Reef and containers of the order of 0.75 to 2.5 m^3 individual volume are also readily used for other forms of coastal protection including revetments and groynes. While the maximum wave climate that the smaller containers can withstand is significantly smaller than the "mega containers", they remain suitable for construction of reef systems in sheltered areas such as within harbours or estuarine environments, so have also been considered in this report.

CIRIA (2008) reports a range of benefits of using sand filled geocontainers as opposed to precast concrete units or rock armour including:

- Their flexibility helps to distribute and dissipate loads within the containers as opposed to the brittle nature of precast concrete units;
- It is possible to modify, move, or remove geocontainers;
- The perception of a softer surface presents a safer option for surfing;
- They can be more cost effective than rock armour, particularly for reefs with a large volume; and
- They can be a more sustainable construction option.

Various methods have been used over time for the construction of reefs using geotextile tubes. Construction of the Narrowneck Reef at the Gold Coast was one of the early examples and a technique was adopted whereby the tubes were filled and placed by a split hull hopper dredge (Jackson and Corbett 2007). The geotextile tubes, each measuring approximately 20 m long, were hydraulically filled in the hopper of the dredge, before being dropped through the split hull and into place (Borrero *et al.*, 2010). Accurate placement of the geotextile tubes was a complication during construction and resulted in disparity between the design and the "as constructed" profile of the reef. This placement method was analysed by De Groot *et al.* (2004) who reported on physical modelling of the dynamics of geotextile tubes when released from a split hull vessel and confirmed a range of potential issues with this placement method including:

- Inaccurate placement due to currents, waves, and tube non-symmetry;
- Splitting/bursting of geotextile tubes due to fall velocity and touch down impact; and
- Failure of structure edge slopes where tubes initially land in an unstable position.

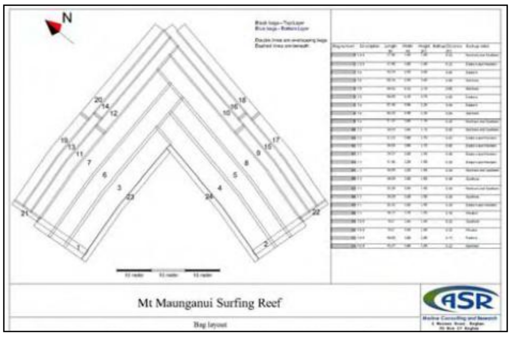
Evans and Ranasinghe (2001) also state that using a split-hull technique to lower geotextile bags is likely limited to a structure crest level at least 1 m to 2 m below the operating water level. Figure 2.15 shows a geotextile tube being placed from a split hull vessel during the construction of the Narrowneck Reef.

Figure 2.15: Placement of a Geotextile Tube by dropping from a Split-Hull Vessel (Photos: ICM)

Placement of large geotextile tubes has been optimised over time and recent reefs have been constructed using a technique whereby the empty tubes are initially fastened to the sea bed and then hydraulically pumped full in-situ (Jackson, 2010; Mead *et al.,* 2010). This technique (termed 'Rapid Accurate Deployment', RAD) was developed by ASR for the Mount Maunganui Reef in NZ and consists of the following process (ASR, 2008a):

1. A geotextile tube layout is developed to replicate the design configuration of the reef.
2. An underlying webbing is assembled on a geomat to form a framework for holding panels of reef bags together. This is undertaken on dry land.
3. The empty geotextile tubes are then secured to the webbing to form manageable sized panels of tubes.
4. Prior to deployment of the webbing/bag assembly, anchors are positioned on the seabed at precise locations.
5. The webbing/bag assembly is then folded and placed onto a barge and transported to the reef site during calm weather.
6. Leader lines from the webbing are then fed through the seabed anchors and the entire assembly of bags, webbing, and geomat is winched to the seabed.
7. The geotextile bags are then filled in place using a barge mounted submersible pump which extracts sand from a seabed source close to the reef site.

Figure 2.16 illustrates the RAD reef construction process at Mount Maunganui Reef. Similar methods have been used for newer reef locations including Boscombe (Figure 2.17) and Kovolam MPRs as well as the geotextile tube detached breakwater at the undisclosed location in the UAE (Figure 2.18). While this construction method has overcome many of the issues associated with the split-hull technique, it also has its own limitations and risks. Relatively calm conditions are required for transporting and fixing the unfilled containers in position, and the occurrence of moderate to larger wave conditions before the containers are filled has the potential to move or damage the panels of unfilled containers. As the containers are pump filled in-situ, there are limitations on the distance between the filling sand stock pile and the reef location. The filling process requires vessels and divers to work in the vicinity of the reef, which is also expected to be challenging in open coast NSW conditions.

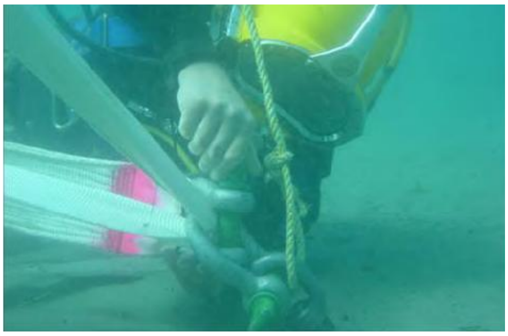


Bag Layout

Webbing and Bag Assembly

Loading Reef Panel onto Barge

Deployment of Reef Panel



Securing Reef Panel to Seabed Anchor

Filling Geotextile Tubes

Figure 2.16: RAD Reef Construction Method at Mount Maunganui (ASR, 2008a)

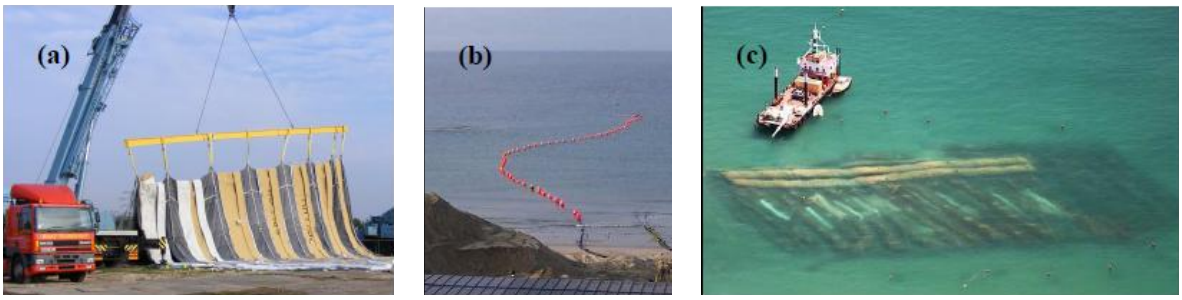


Figure 2.17: Construction of the Boscombe MPR (Mead *et al.,* 2010)

Figure 2.18: In-situ Filling of Emergent (Left) and Submerged (Right) Geotextile Tubes (Jackson, 2010)

Smaller geotextile containers are typically lifted into place by crane or excavator using either lifting slings (as shown in Figure 2.19 for Prattes Reef) or specially developed excavator buckets as shown in Figure 2.20 for a groyne at Clifton Springs Boat Harbour, Victoria).

The expected design life of geotextile materials has been difficult to quantify, especially since it is a relatively modern coastal protection material. However, modern staple fibre non-woven geotextiles have significantly improved in this respect and outer layer containers now have a design life of approximately 20 years (Geofabrics, 2013). Improvements to the geotextile bag materials such as vandal deterrent layers and improved ultra violet (UV) light resistance have contributed to achieving this life. Inner layer geotextiles protected from UV light, anchor and boat impacts, debris and vandalism have a longer life. The rapid development of marine growth cover layers in reef type situations (Corbett *et al.*, 2010) is also likely to extend the design life due to the reduced UV exposure, possibly as long as 40 years (ASR, 2008b), which is comparable to the 50 year design life for most rock structures (though rock in most coastal applications significantly outlasts a 50 year structure life if properly sized).

Figure 2.19: Placement of Smaller Geotextile Bags by Sling at Pratte's Reef (Borrero and Nelson, 2003; Jackson and Corbett, 2007)

Figure 2.20: Placement of Smaller Geotextile Bags by Special Excavator Bucket

2.7.4 Pre-Cast Concrete Reef Units

A large number of innovative precast concrete reef units have been used in recent years to construct artificial reefs. The main advantages of these novel systems are the simplicity in placement and constructability, as well as cost effectiveness (Pilarczyk, 2005). Another advantage is that, if not performing as expected, the structure can more easily be removed compared to traditional rubble mound structures. In general, the available units can be grouped into either submerged breakwater structures (focus on coastal protection) or reef mimicking structures (ecological focus), however, there have been selected applications in small wave environments where reef units with an ecological focus have been built into the form or a submerged breakwater and produced successful beach widening (Harris, 2009). The main limitations in application of the precast units for coastal protection in NSW is the lack of proper design criteria, in particular in terms of unit stability under wave attack. Furthermore, the brittle nature of the units mean that their use would be restricted to sheltered locations or enclosed embayments dominated by wind waves.

Available units are shown in Figure 2.21 to Figure 2.24.

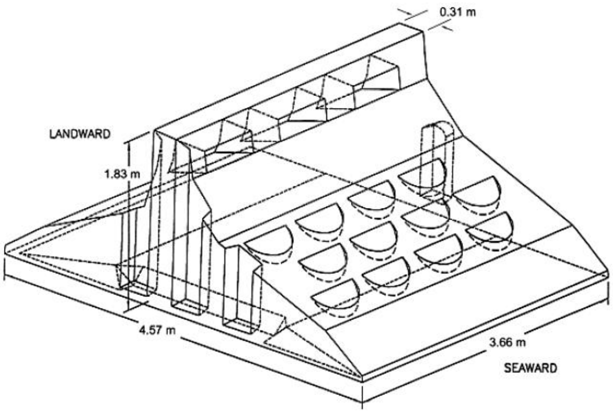


Figure 2.21: PEP Reef Units
(Stauble and Taber, 2003)

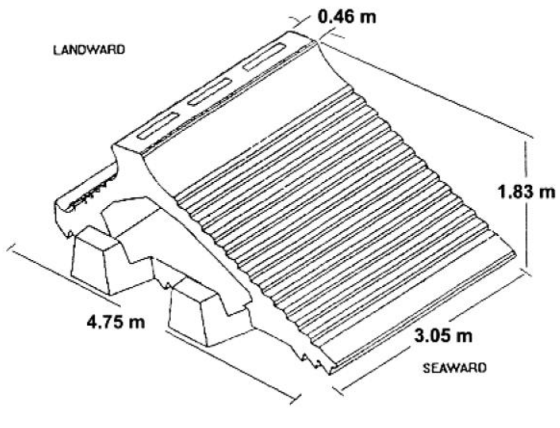


Figure 2.22: Beachsaver Reef Units
(Stauble and Tabar, 2003)

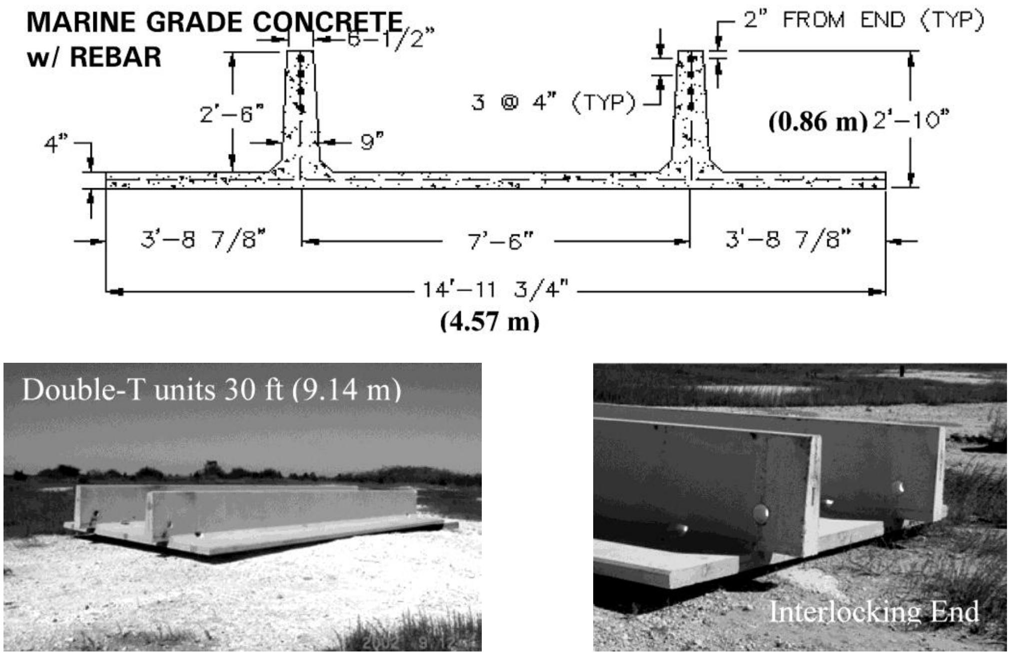


Figure 2.23: Double-T Sill Units
(Stauble and Giovannozzi, 2003)

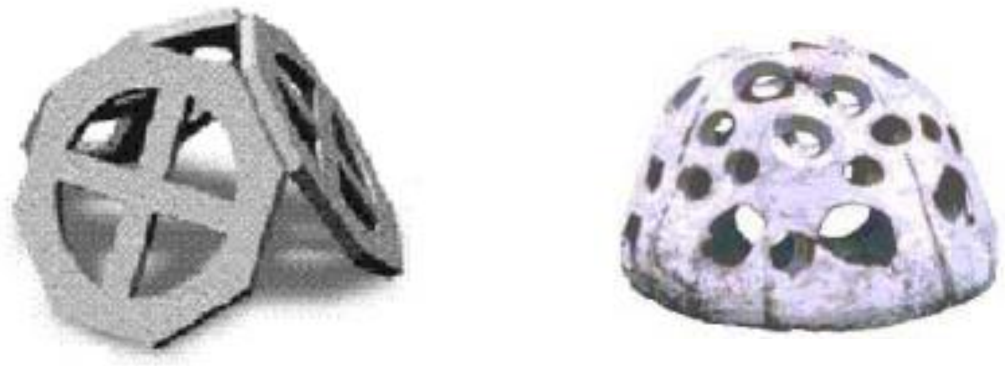


Figure 2.24: Precast Tecnoreef (Left) and Reef Ball (Right) Reef Units
(www.tecnoreef.com; Harris, 1995)

Beachsaver Reefs™ and Prefabricated Erosion Prevention (PEP) Reefs™ (Figure 2.21 and Figure 2.22) were built and tested in Florida and New Jersey, USA. Both systems use modular precast concrete units of triangular cross-section that are locked together in segments and in different configurations to form shore parallel narrow-crested submerged breakwaters. Stauble and Tabar (2003) report that these installations were subject to settlement and scour and were only appropriate in combination with groynes to form a closed cell perched beach. The large breaking wave conditions experienced during storms on the NSW coast would likely preclude the use of these units for open coast locations based on structural strength and unit stability. The relatively detailed and interlocking placement of these units would also be extremely challenging to construct at open coast locations. Nevertheless, the units could be applied in sheltered locations or protected embayments where shorter period and less energetic wind waves dominate.

Aquareef™ and Tecnoreef™ units were deployed in Japan and Italy respectively. The Iburi reef in Hokkaido Japan implemented concrete interlocking blocks as a primary armour layer above a rubble mound core (Hirose *et al.*, 2002). In Punta Marina Italy, a 350 m long pilot trial artificial reef was built in 2010 using Tecnoreef™ modules, with each module consisting of reinforced porous octagonal plates assembled to form pyramidal shapes. Extensive monitoring of the structure is ongoing. Localised settlement and scour was observed within a few months from construction (Sordini, 2011). Reef Ball™ units were used to construct several artificial reefs around the world. These concrete units are hollow and characterised by high porosity and roughness. They are mainly used for ecosystem restoration and in shallow water have been shown to help improve beach restoration and stabilisation (Harris, 2009). In NSW Reef Ball™ units have been installed for ecological enhancement at Botany Bay, Lake Conjola, Lake Macquarie, St Georges Basin and Merimbula Lake. Wave climates in these locations are typically restricted to locally generated wind seas and these units would likely be unsuitable for application on the open NSW coast.

2.7.5 Other Reef Materials

The use of other materials for forming artificial reefs (such as scuttled vessels, cars, tyres, etc.) has primarily been targeted at reefs where ecosystem enhancement and/or diving amenity are the primary considerations. Examples of this application already exist in NSW and include the scuttled Ex-HMAS Adelaide site on the Central Coast (WorleyParsons, 2009), scuttled ferries off Sydney and scuttled tug vessels on the Far South Coast near Green Cape. The use of such materials for coastal protection and in relatively shallow water depths (less than 10 m) would introduce additional environmental and safety risks, particularly for structures considered as MPRs. Furthermore, location of these reefs in the past has been targeted at avoiding occurrence of breaking wave conditions even during storms (due to structure integrity), which further reduces the ability to provide coastal protection.

2.7.6 *Summary Box: Suitability of Reef Materials for Application in NSW*

Table 2.4 summarises the recommended applicability of reef materials for NSW sites based on a review of other reefs constructed around the world as well as the specific characteristics of the NSW coastal environment (mean and storm wave climates). While some materials are not suited to the open coast, they would be highly suited to enclosed water bodies or sheltered sections of coast.

Table 2.4: Suitability of Reef Materials for Application in NSW

Material	Potential Applications in NSW	Limitations
Rock	Potentially open coast reefs, sheltered coast reefs, protected embayments, estuaries	Typical available upper rock size limit in NSW limits applicability for large design wave environments
Geotextile Tubes and Mega-Containers	Open coast reefs	Difficult to place and/or fill in exposed wave environments
Geotextile Containers	Sheltered coast reefs, protected embayments, estuaries	Unstable in large wave conditions on open coast
Precast Concrete Units	Sheltered coast reefs, protected embayments, estuaries	Unstable and likely too brittle in large wave conditions on open coast. Site preparation and placement would also not be possible at open coast sites
Materials of Opportunity	More suited to dive/ecology reefs than coastal protection	Could be high environmental, health and safety risk if used in MPRs or ASRs

3. Modelling of Artificial Reefs

3.1 Physical Modelling

3.1.1 Overview

Physical models have long been considered a robust tool for assessing the stability and short-term wave and hydrodynamic response to coastal structures. For some applications such as small spatial- and temporal-scale or complex processes there are clear advantages over numerical modelling approaches, while for others such as for more simple, larger spatial- and temporal-scale processes this is not the case (Blacka *et al.,* 2006; Shand *et al.*, 2010). Many of the empirical formulae on which reef design is now based were developed on the basis of physical model studies. Furthermore, modern day physical models typically rely on the use of at least a numerical wave transformation model to define boundary conditions, so is nearly always considered "hybrid" modelling (Hughes, 1993). With application to SCR analysis and design, physical models are commonly used to investigate:

- Wave transmission (2D), transformation (3D), and surfability (3D);
- Hydrodynamics (3D);
- Structure stability (2D and 3D); and
- Immediate shoreline response (3D, qualitative or inferred).

As is the case with numerical models, application of physical models is an expert science, and requires careful consideration and understanding of modelling rules, assumptions and limitations. Modelling results should be interpreted within these limitations, and in general, application of physical models should:

- Ensure dominant processes and forces are reproduced with minimal scale effects;
- Consider the impact of scale effects on processes in the model where scale effects cannot be minimised, such as response of mobile beach sediments;
- Ensure predicted shoreline alignment is validated for pre-structure conditions if beach response is being analysed;
- Understand the limitations of the modelling, especially when simulating beach response; and
- Consider the impact of short, mid, and long term variability of environmental variables on structure performance.

3.1.2 Wave and Hydrodynamic Modelling

Physical modelling of waves and wave driven hydrodynamics is commonplace, with both two-dimensional models (for long crested submerged breakwater structures) and three-dimensional models (for shorter crested structures such as MPRs and ASRs) regularly used in the design of reef structures. For most reef structures where surfability is a key design parameter, physical modelling of the resulting wave field has been undertaken. Examples include:

- Cable Station ASR (Button, 1991; Lyon, 1992);
- Narrowneck MPR (Turner *et al.*, 2001);
- Mount Maungnui MPR (Black and Mead, 2009); and
- Boscombe MPR (Mead *et al.,* 2010).

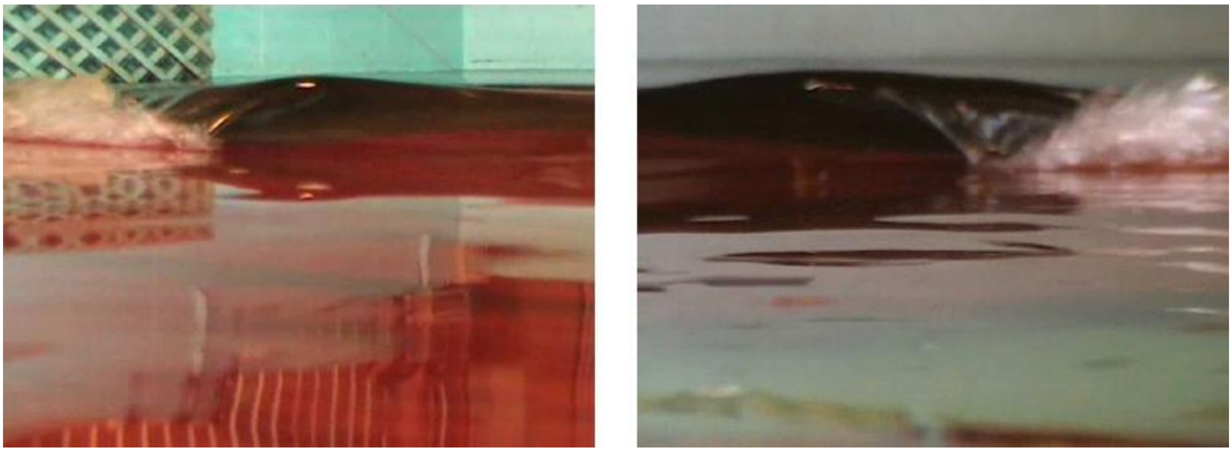


Figure 3.1: Physical Modelling of Wave Breaking on the Mount Maunganui MPR (Black and Mead, 2009)

Apart from modelling the wave field, physical models have also been used to predict the wave driven hydrodynamics around, and in the lee of, reef structures. The results have subsequently been used to validate numerical predictions and also to infer the likely sediment transport regimes (Blacka *et al.* 2009).

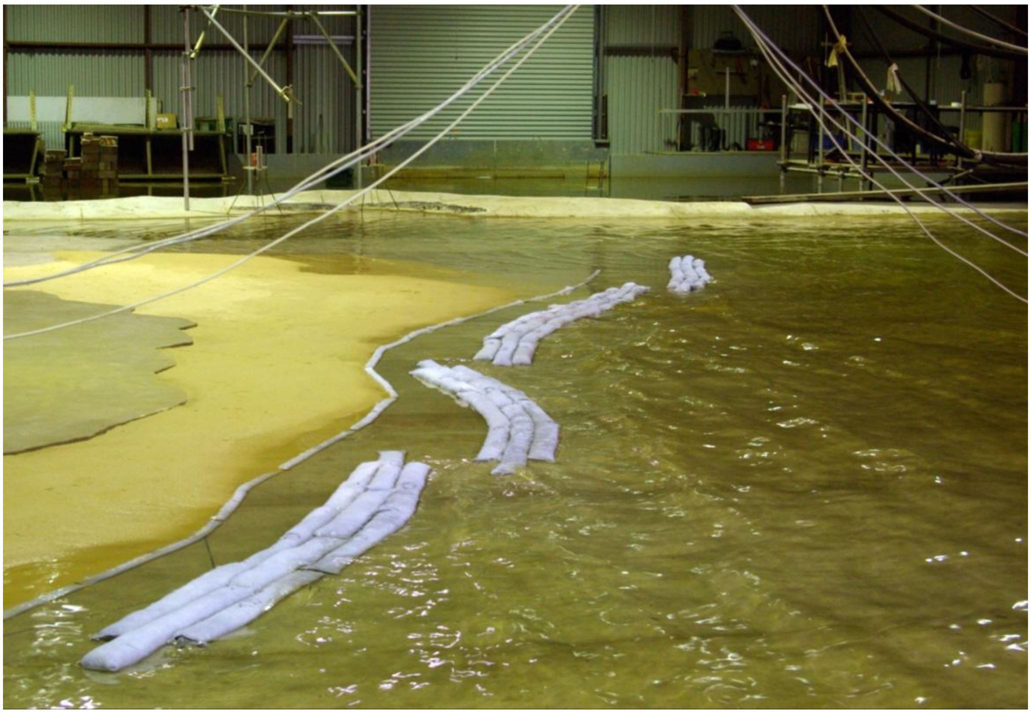


Figure 3.2: Physical Modelling of Wave Generated Currents around Submerged Breakwaters (Blacka *et al*., 2009)

Scale effects in short wave physical models are typically associated with surface tension and viscosity and most models are designed so that these parameters have minimal impact on results. This is almost always achieved by following a few basic rules, such as (Frostick *et al.*, 2011):

- Model wave length greater than 0.3 m (period greater than 0.4 s);
- Model water depth greater than 5 mm; and
- Model wave height greater than 20 mm, with design wave height greater than 50 mm.

It is more likely for inaccuracies to be introduced into physical modelling of waves and hydrodynamics through laboratory errors which are typically associated with model boundary conditions, wave generation, or misrepresentation of real world wave conditions (Hughes, 1993).

3.1.3 Structure Response Modelling

Stability formulae for rock or concrete unit reef armouring have all been developed on the basis of physical model testing. While these formulae are reasonable for initial/conceptual design, they have been developed on the basis of physical model tests with specific test conditions and limitations. It is therefore recommended that all significant SCR detailed design studies include at least 2D and preferably 3D physical modelling to confirm the stability of reef armouring, particularly if the reefs are relied upon to provide coastal protection and risk reduction. This modelling will provide confidence in the robustness of a reef design, and may reduce the cost of the reef through optimisation of armouring, which is not possible with empirical design equations. Structure response physical modelling also has the advantage of visual association with the design, and can often be undertaken in conjunction with modelling of other parameters.

Figure 3.3: Physical Modelling of Wave and Structure Interaction

As with modelling of waves and hydrodynamics, the impact of scale effects on model results for reef armour stability is typically negligible, so long as basic guidelines are followed. For armour stability modelling, scale effects should be negligible for both the wave and hydrodynamic processes

(Section 3.1.2) as well as for the wave/structure interactions. This is typically achieved by (Frostick *et al.*, 2011):

- Ensuring the dimensionless Reynolds Number for flow through the armour layer remains larger than 3 x 10^4; and
- Using rock armour with diameter greater than 25 mm.

Though response of structures built using smaller sand filled geotextile containers (less than 5 m^3) have been examined in physical model studies, the stability of the very large geotextile tubes (tens of metres long) is often not modelled. Scaling of such structures for physical modelling would be complex and would require careful consideration of container fabric and fill material in the model, as well as the behaviour of the model containers under wave attack.

3.1.4 Morphological Response Modelling

Hans Albert Einstein was one of the world's leading researchers in the field of sediment transport and was the son of the famous Albert Einstein. Albert was reported to have discouraged his son from studying sediment transport: "… the older Einstein is said to have warned his son strongly of the difficulties in dealing with sediment transport processes" (Vollmers, 1989 in Hughes, 1993). Hughes (1993) also gave his own opinion that: "Understanding sediment transport in coastal regimes is a perplexing challenge that in all likelihood will continue to frustrate coastal researchers and engineers for generations to come".

Physical modelling of morphological response to a structure is arguably one of the most complicated and least reliable types of coastal physical modelling. Due to the physics inherent with scaling coastal physical models according to Froude similitude and the required scales suitable for most facilities, it is well known that model sediment typically cannot be scaled for size according to the undistorted model length scale. The use of sediment that is not scaled in similitude subsequently results in sediment transport processes not being truly reproduced within physical models.

Previous experience has led to a range of empirical scaling adjustments that allow some sediment transport processes to be simulated with reasonable accuracy, when specific conditions dominate the real world sediment transport processes. A range of considerations for compensating for the inability to scale model sediment with true similitude have been presented over time:

- A moveable bed scale model is restricted to representing either suspended-load transport or bed-load transport, not both (Bijker, 1967);
- Scaling laws must be applied consistently across the model (Bijker, 1967);
- Wherever possible, sand should be used as the model material (Noda, 1972; Dean, 1985) with the size adjusted to the model law (Noda, 1972) or scaled to achieve similitude with the fall velocity (Dean, 1985);
- If lightweight (low density) material must be used for the model sediment in suspended-load models, its relative density should be greater than 1.3 (Noda, 1972);
- Model and prototype densimetric Froude numbers should be equal for suspended-load models (Kamphuis, 1975; Sager & Hales, 1979);
- Lightweight sediment cannot be used to simulate littoral drift transport processes (Kamphuis, 1975; 1991);
- Suspended-load dominated models must be undistorted (Dean, 1985);
- Suspended-load dominated models must maintain fall velocity scaling criteria (Dean, 1985);
- Models should be as large as possible (Noda, 1972; Dean, 1985); and

- Small-scale mobile-bed basin models should not be used for medium or long-term sediment transport studies (Kamphuis, 2000).

It has been shown that the equilibrium shoreline alignment can be reproduced in a model with reasonable accuracy when structures or significant coastal features dictate the nearshore wave alignment (Nielsen *et al.*, 2000). In these cases, the primary sediment transport mechanisms (nearshore currents and longshore distribution of sediment by waves) still occur correctly in a model, and so a mobile shoreline tends to evolve to an equilibrium alignment that resembles reality, even though the sediment transport processes are not scaled correctly. These conditions apply to shoreline change in the lee of submerged reef structures, and indeed this "mobile shoreline" physical modelling technique has been used in the past to investigate shoreline response to reef structures.

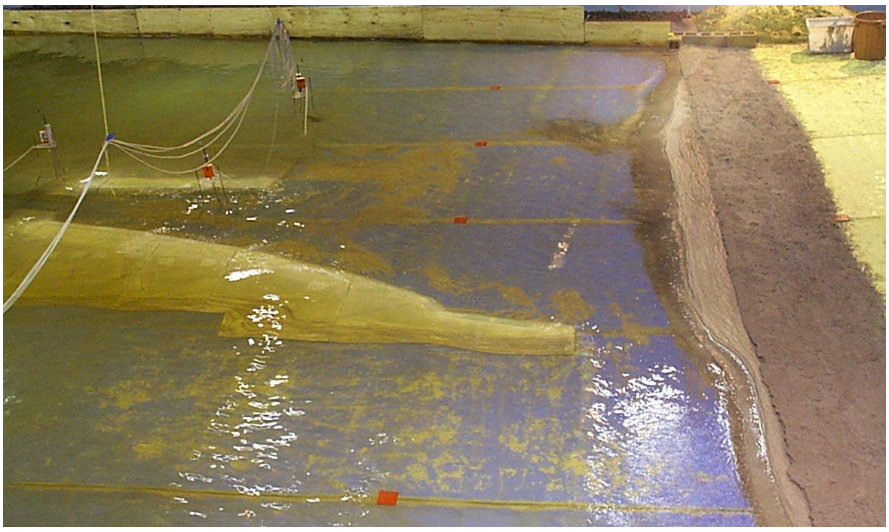


Figure 3.4: Mobile Shoreline Response Physical Modelling for Narrowneck Reef

Due to the empirical adjustments that are required in order to undertake mobile bed or mobile shoreline physical modelling, the results cannot be considered as truly quantitative. Instead the modelling provides only qualitative and semi-quantitative insights into sediment transport processes. This requires careful and limited interpretation of model results. Such models should only ever be undertaken where the shoreline response of an existing or previously observed case can be initially simulated (model validation), and the relative change in shoreline alignment induced by a structure can then be qualitatively assessed.

3.1.5 Physical Modelling Limitations

While considered one of the most robust techniques for assessing wave/structure interactions and insightful for investigating the effect of reef structures on beaches, physical modelling has definite limitations. These include:

- Limited spatial extent that can be considered in a model domain;
- Model boundaries can interfere with hydrodynamic and morphological response if caution is not taken;
- Restriction to simulation of short time durations, resulting in discrete event-based modelling, rather than evaluating long term processes;

- Sediment transport processes cannot be scaled with proper similitude, and empirical "corrections" or "adjustments" are required to approximate morphological response;
- Requires careful model design and interpretation of results, in order to properly transfer model results to the real world application; and
- Requires specialised large scale facilities.

3.2 Numerical Modelling

3.2.1 Overview

Roelvink and Reniers (2011) gave the following commentary on coastal morphological modelling:
"*Models are formalized representations of reality. Computer models of coastal morphology contain ideas about hydraulics, waves, sediment transport and sediment conservation that are captured in formulations. They produce interesting pictures and flashy animations of coastal behaviour but they cannot claim to represent reality in all its complexity. Rather, they quantify concepts we have in our heads and combine processes that are too difficult for us to combine just by reasoning... The problem arises when the people who put the concepts into the model are different from the people who use the model. Then model concepts may be used out of context and results produced that are clearly wrong*."

The use of numerical models in reef design and general coastal engineering applications has evolved since the 1970s. Today, such models are used to consider the dynamics of wave/structure/beach interaction with a higher level of complexity than is captured with the empirical solutions presented in Section 2. With respect to applications for SCR analysis and design, numerical models are commonly used to investigate:

- Wave response to a reef, including wave shoaling, refraction, diffraction, and breaking, as well as parameters used for classifying surfability such as peel angle, breaking intensity, etc.;
- Beach response during ambient and extreme conditions.

They may also be used to estimate structure loading under waves including prediction of the pressure and velocity field around structure components.

Due to the complexity of natural processes, all numerical models involve approximations and smoothing in both space and time. Models which are more theoretically correct and/or based on more advanced physics inherently contain additional coefficients or parameters. This subsequently introduces additional uncertainty, potential instability, may be much slower to run, and without commensurate field data collection will give less realistic answers than well-developed simpler empirical models.

3.2.2 Processes in Morphological Modelling

Generally, the primary objective of SCRs has been coastal protection, so the majority of modelling effort has focussed on morphological change. For an initial bathymetry and assumed water level (either averaged or varying with each time step), the logical steps/processes in modelling morphological change are:

- Waves;
- Currents;

- Sediment transport; and
- Bed change.

These steps/processes will also interact with each other (e.g. waves may be refracted by currents, bathymetry changes will alter waves and currents), and their interaction is an important part of the mathematical equations and numerical scheme.

3.2.3 Calibration and Validation of Models

While the complexity solved within numerical models has increased over recent decades, robust calibration of morphological models remains challenging. As with all numerical modelling, confidence in simulation of real world situations requires calibration of the model, preferably for the specific site of application and also for beach response to similar structure/s. This requires the availability of comprehensive morphological data from existing structures (pre- and post-construction) which is mostly limited (though does exist). It should also be recognised that the accuracy and reliability of even the best calibrated morphological models contain considerable uncertainty. Furthermore, a successful calibration does not guarantee successful prediction of the future where the processes and forcings may be different from the period of calibration (Figure 3.5).

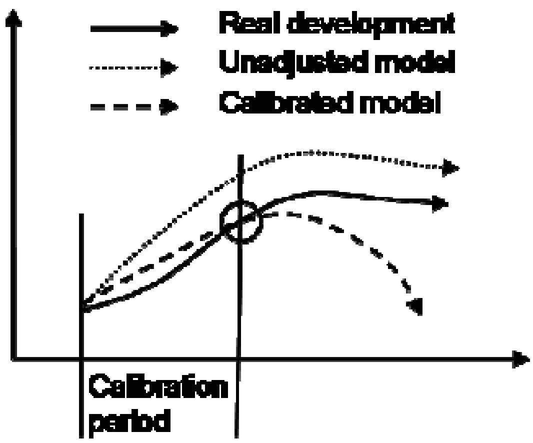


Figure 3.5: Illustration of Pessimistic Calibration Scenario (Roelvink and Reniers, 2011)

Any engineering design for a SCR that relies on numerical modelling for prediction and design of shoreline response should:

- Ensure that the morphological model is calibrated to existing observations at the site (i.e. the longshore transport rate), and preferably also be validated for pre- and post-construction conditions at an existing SCR site.

- Reproduce observed historical change at the site and/or at any nearby coastal structures which interrupt the littoral system, such as river training walls, groynes or breakwaters. There are approximately 70 training walls/groynes/breakwaters on the open coast in NSW and approximately 17 in south-east Queensland south of Fraser Island.

- Consider the impact of short, mid, and long term variability of environmental variables on performance.

3.2.4 Offshore Waves

For many locations beyond NSW, much effort is often required to define an offshore wave climate and time series. Measured data is considered superior to computer model output for defining the ambient and extreme wave climate, but is not available for many locations beyond NSW.

For NSW, a network of wave buoys commenced with the first installation in 1971 and now encompasses more than eight locations. Manly Hydraulics Laboratory (www.mhl.nsw.gov.au) operates seven long term installations, while Sydney Ports Corporation (previously Maritime Services Board) has operated a wave buoy off Botany Bay since 1971. The ports of Newcastle and Port Kembla also have nearshore wave measuring instruments, and there have been numerous short term deployments at other locations.

Time series of wave buoy data is available from the collecting institution, while the most recent analysis of NSW wave data is contained in Shand *et al.* (2011).

For NSW, the measured wave climate can be supplemented with numerical model output from:

- Wavewatch III which is available from 1997 to present (www.polar.ncep.noaa.gov);
- ERA-40 which is available from 1957 to 2002 (www.knmi.nl).

These models provide additional wave directional predictions (prior to directional wave buoys). ERA-40 provides a wave climate back to 1957, however, it is recommended that measured data be given precedence over model output whenever the measured data is available.

Hindcast waves for documented ocean storms affecting the NSW coast from 1880 to 1980 are provided in Blain Bremner Williams (1985), while Callaghan and Helman (2008) documented storms affecting the eastern Australia coast between 1770 to 2008.

For enclosed water bodies protected from ocean swell, wind wave hindcasting techniques are well documented in standard coastal engineering references such as SPM (1984) and CEM (2006).

3.2.5 Wave Transformation to Shore

In terms of wave processes, deep water is defined as half a deep water wavelength. For 10 and 12 s period waves, the deep water wavelengths are 156 and 224 m respectively, meaning that the limit of deep water is 78 and 112 m for these waves. The wave buoys operating off the NSW coast are typically located in water depths of approximately 60 to 100 m which is an intermediate water depth and close to deep water for wave periods of 10 s or less. Between deep water and the shore, waves may be transformed through the processes of:

- Refraction due to depth changes;
- Refraction due to currents;
- Diffraction;
- Shoaling;
- Bed friction;
- Breaking;

- Reflection;
- Growth through a following wind;
- Decay through an opposing wind; and
- Wave-wave interactions.

The earliest wave transformation models relied on Snell's law (from optics), whereby waves slow in shallow water – the shallow water wave equations. These early models applied wave transformation processes to monochromatic waves – those with a single period. All contemporary models still rely on this physical basis, but add varying levels of complexity and incorporate a range of the transformation processes listed above. Simple Snell's law refraction may be appropriate for simple bathymetry and is easy to visualise (Figure 3.6 and Figure 3.7).

Contemporary models are generally governed by the "mild slope equation" and may be spectral (e.g. SWAN, STWAVE) or use the "Boussinesq" approximation (e.g. FUNWAVE). Boussinesq models rely on vertically averaged velocities and still generally require mild slopes. Most models transform "spectral" waves, that is, waves having a range of periods as occurs in nature (frequency spectrum or spreading) and a range of directions (directional spectrum or spreading).

Mild slope equation models can model wave transformation over complex bathymetry, but are theoretically limited to "mild" (rather than steep) slopes.

The basic empirical observation that a wave will break in shallow water when its height is between 0.5 and 1.5 times the water depth is well accepted. However, the wave transformation process of shallow water wave breaking is the poorly understood and most difficult to model. All contemporary models use empirical approximations of wave breaking. The most common techniques used to model this include setting a depth limited wave height, the surf zone model of Battjes and Jansen (1978) or the breaker decay model of Dally, Dean and Dalrymple (1984).

The presence of an SCR will alter nearshore waves predominantly through alterations to wave breaking, refraction and diffraction. This perturbation will then create or alter wave driven currents. SCRs are of small scale, have complex interactions with waves and are possibly steeper than natural slopes. This means that different models may be required to simulate important wave processes in the vicinity of an SCR than the model used to transform waves from deep water to the vicinity of the structure.

3.2.6 Wave Transformation on the NSW Open Coast

The NSW open coast has the following characteristics which influence nearshore wave modelling:

- Nearshore waves are predominantly swell, with periods above 8 s (rather than sea);
- The continental shelf gradient is relatively steep – steep enough to make frictional losses negligible, but mild enough to be appropriate for "mild slope" models;
- The extreme tidal range is about 2 m and extreme water level range (from negative tidal anomalies to 100 year ARI storm surge level excluding wave setup) is less than 3 m;
- Accurate wave climate information is available from wave buoys at typically 60 to 100 m water depth.

These features mean that for the NSW open coast, the processes of bed friction, growth through a following wind, decay through an opposing wind and wave-wave interactions can generally be ignored for engineering studies, though caution and judgement are still required to make this assumption

The CEM (2002, p II-3-34) provides the following caution on wave transformation modelling:
"*The techniques provided …, if used carefully by an experienced engineer, can provide very useful information in a wide range of cases. However, there are some cases where they simply will not work. Anyone who applies these techniques should understand the limitations of the techniques, and be versed in understanding when they have been used inappropriately. The user should be aware that the models can provide realistic-looking answers that unfortunately are just wrong.*"

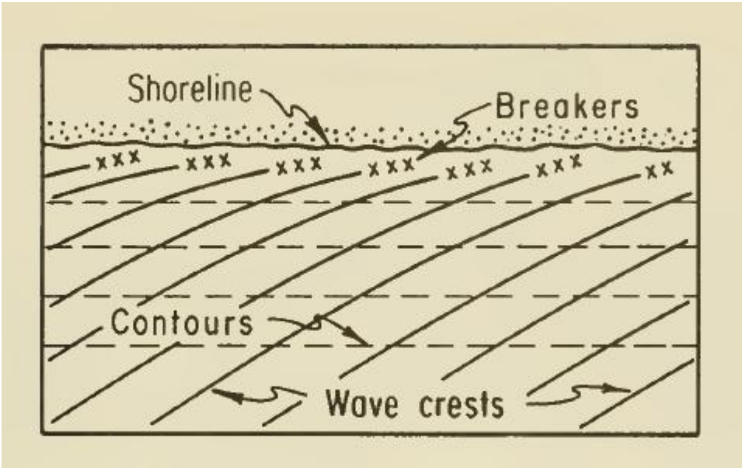


Figure 3.6: Refraction along a Straight Beach with Parallel Bottom Contours (SPM, 1984)

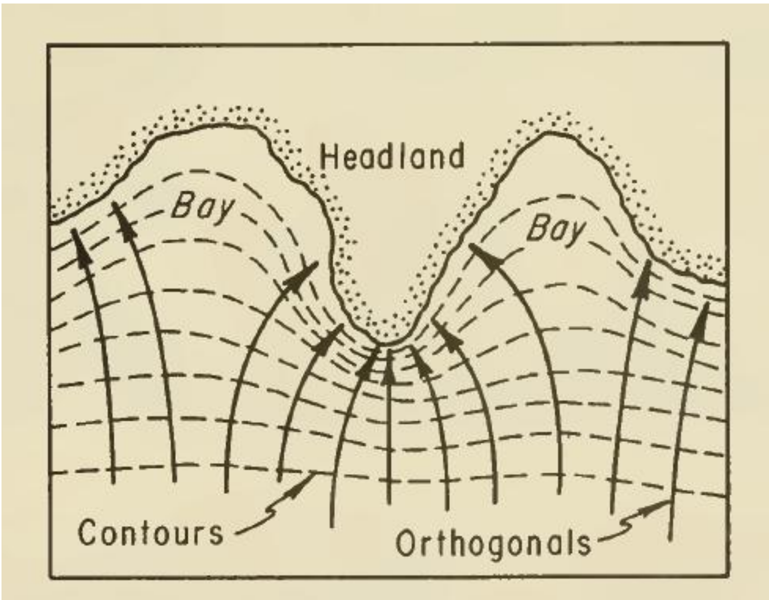


Figure 3.7: Refraction along an Irregular Shoreline (SPM, 1984)

3.2.7 Hydrodynamic/Surf Zone Circulation Models

Nearshore currents can result from wave transformation, tides, local winds and larger scale ocean currents. Tsunamis also have a rare but extreme impact on currents, but are not generally considered for design of SCRs or other coastal structures within NSW.

On the open coast of NSW, currents near to and inside the surf zone are predominantly driven by waves rather than tides or ocean currents. The predominant currents driven by waves are:

- Longshore currents due to waves breaking at an angle to the coast and applying stress to the surrounding water;
- Wave setup which results in higher water levels near the shore – there may also be alongshore gradients in wave setup;
- The mass flux over the crest of a SCR due to wave breaking and subsequent surf zone circulations and return currents.

These currents have a quasi-steady state component and episodic components or intensification, which may manifest as "flash" rips on natural coasts. The episodic component may be caused by groups of irregular waves, incoming long waves, and interactions of different swell sources. There is also usually a vertical velocity profile within the water column, particularly in the cross shore direction, where the surface current may be directed shoreward and a return "undertow" directed seaward (Figure 3.9).

Hydrodynamic numerical modelling of currents always involves approximations and numerical solutions of the Navier-Stokes equations. Simpler coastal profile and planform models may use empirical approximations of physical processes relating to wave angle and height to generate currents. Modelling of currents driven by waves may be undertaken three dimensionally (3D), quasi 3D using one or more layers, or by depth averaging 2D-H (Figure 3.8). For hydrodynamic modelling of a single SCR or field of SCRs, 3D or quasi 3D modelling is needed to properly model seaward return flows due to wave setup and mass flux over the reef crest (Figure 3.9). The use of a 2D-H model in these circumstances will overestimate seaward velocities in rips or the gaps between SCRs because the 2D-H model is unable to reproduce stratified flow. Conversely, for simple longshore currents, the complexity of a 3D model may be unwarranted.

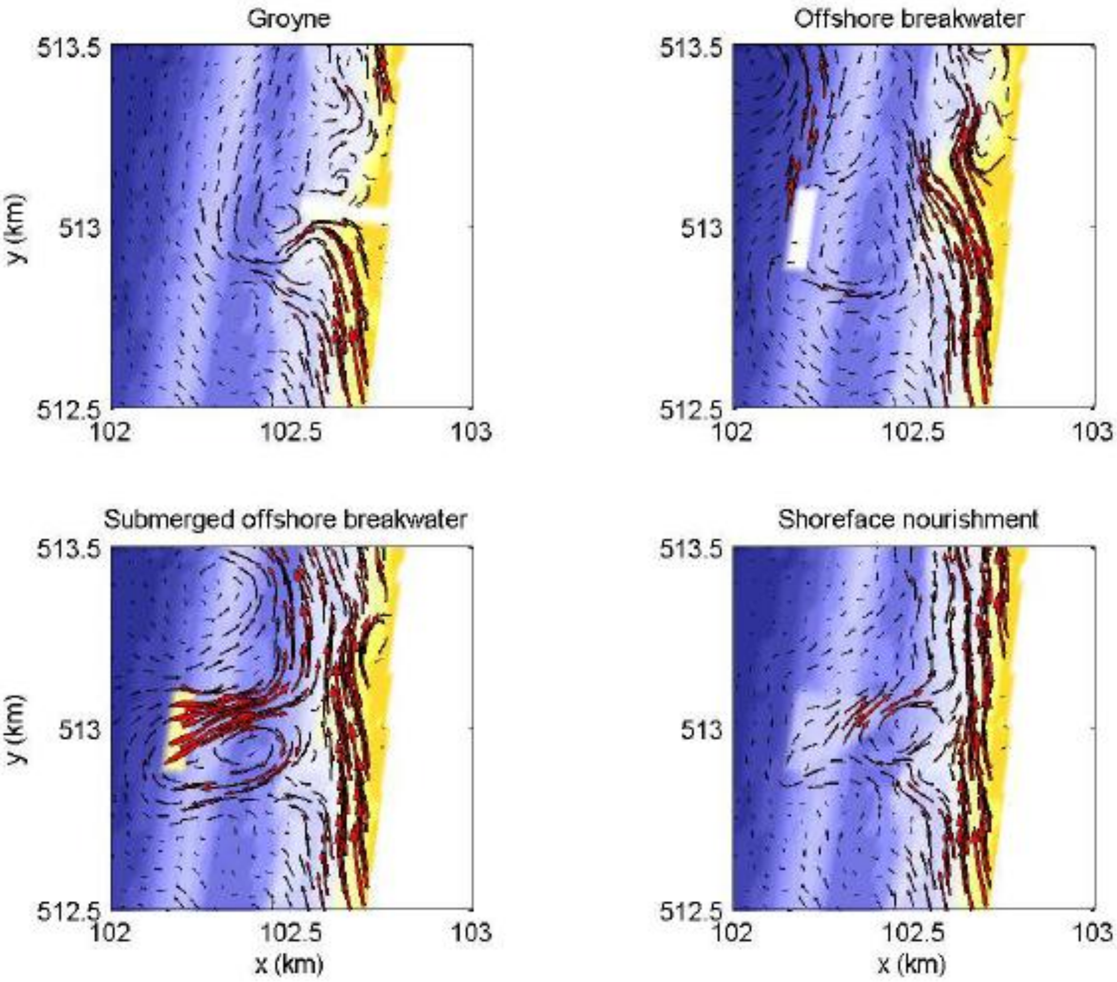


Figure 3.8: Example of Velocity Fields for Simplified Engineering Options (2D-H Model) (Roelvink and Reniers, 2011)

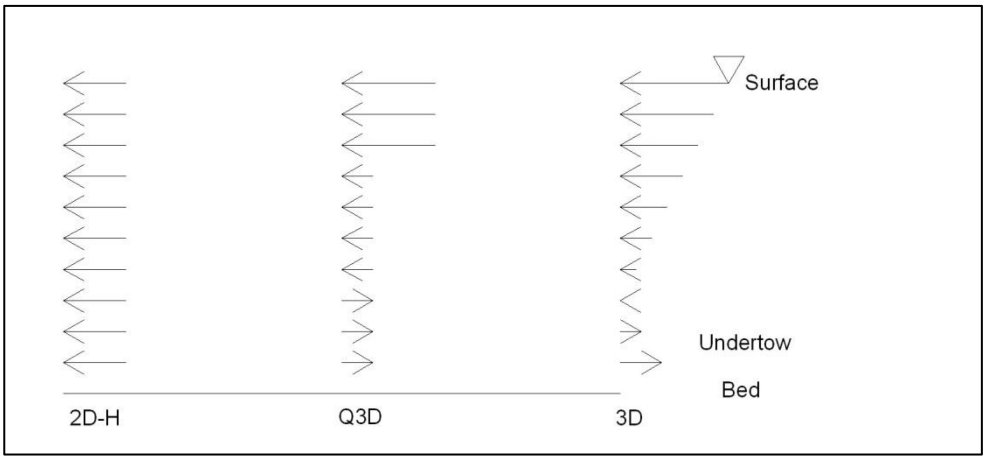


Figure 3.9: Illustration of Undertow, 2D-H, Quasi 3D and 3D Flow

3.2.8 *Sediment Transport, Morphology and Shoreline Response*

Sediment transport is a complex interplay between the sediment characteristics, water velocities and waves. Sediment may move as either bed load or suspended load.

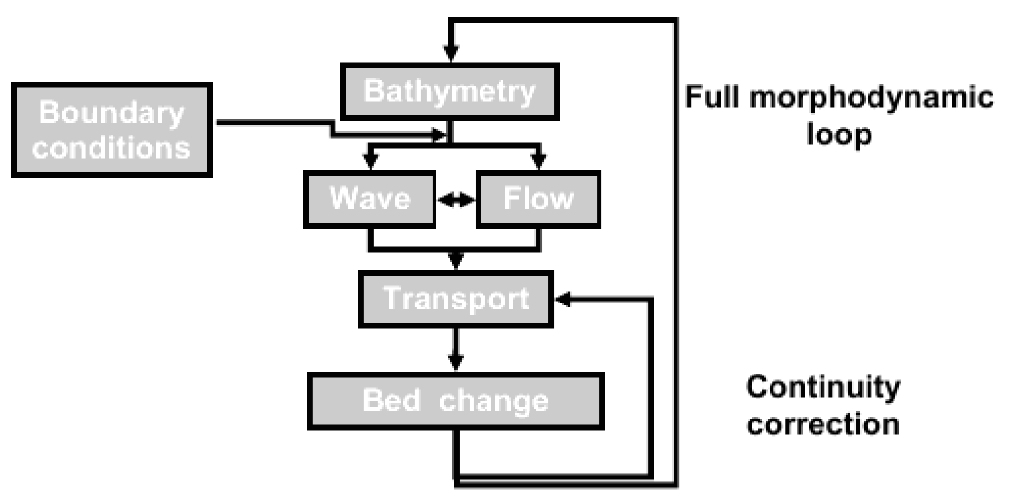
Sediment is characterised by composition, density, grain size, size distribution and shape. A large amount of research effort has been undertaken on sediment transport in rivers and engineered structures such as pipes and channels, however, on beaches the process is further complicated by waves which generate rapidly oscillating velocities, very high sediment concentrations at the breakpoint and complex flow patterns. These factors mean that the gross sediment transport due to waves may be one or more orders of magnitude higher than the net.

Suspended sediment concentration may be solved by:

- 3D Advection-diffusion;
- 2D-H Advection-diffusion.

Complex processes may be considered in wave driven sediment transport, however, almost all models simplify this, and many use simple "bulk" formulas which combine bed load and suspended load such as Soulsby-van Rijn (Soulsby, 1997).

Morphological (and shoreline) change result from gradients in sediment transport. While this results directly from the sediment transport calculations, some degree of numerical smoothing is needed to avoid instability. The updated morphology is then used as input for the next time step of model calculations as shown in Figure 3.10.



Coupled

Uncoupled

Figure 3.10: Flow Diagram of Tide Averaged Morphodynamic Model (Roelvink and Reniers, 2011)

Example output from the modelling of a range of coastal structure options is shown in Figure 3.11. This example provides a valuable comparison of the options, but is limited to a single (modal) wave condition, with no incorporation of tides and depth averaged (2D-H) velocities.

Many models update morphology less frequently than waves and flow, on the assumption that the model is relatively insensitive to small changes in the bed. For models which calculate morphology at each time step, an alternative scheme is to estimate change over a single tidal cycle, then multiply this by a "*morphological factor (morfac)*" to simulate a longer real world duration more efficiently, but this may not simulate the full variability in the natural forcing conditions.

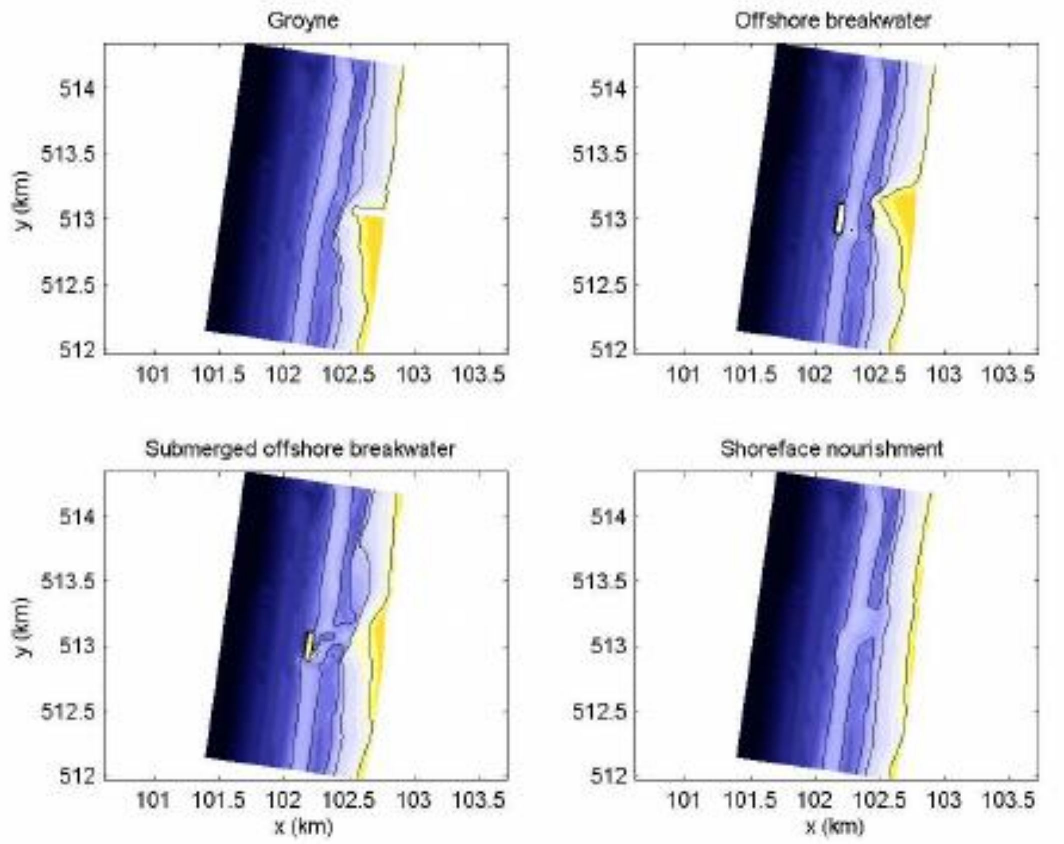


Figure 3.11: Example of Bathymetric Change for Coastal Engineering Options (30 Days of Simulation, Roelvink and Reniers, 2011)

3.2.9 Beach Profile and Planform Models

Simple and robust beach profile and planform models should be applied as an alternative or adjunct to more complex hydrodynamic models for the design of SCRs. These have a much longer track record and have greater verification after prediction. The reduction in complexity can be their greatest advantage. A principle behind these models is that cross shore and longshore processes can be separated because:

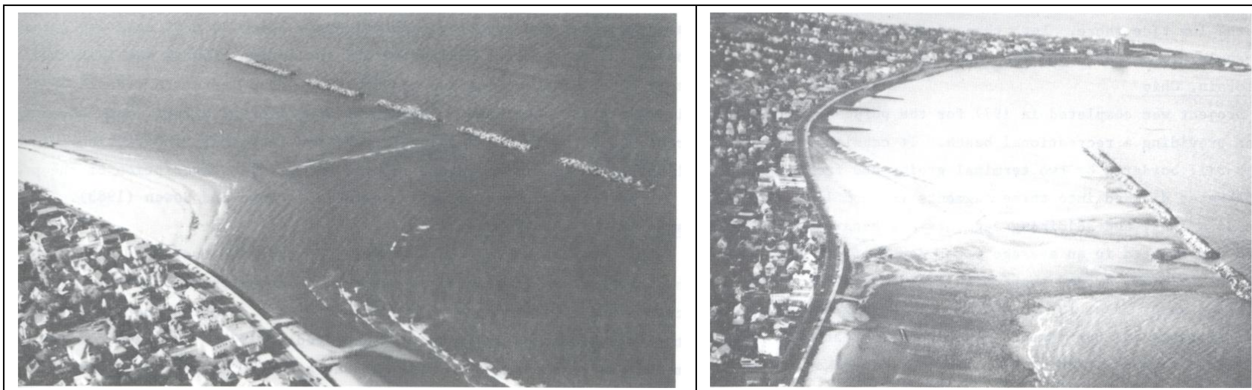
- The long term average cross shore profile reaches an equilibrium relatively quickly – of the order of 1 year if perturbed by a structure, nourishment or sea level rise, and is relatively uniform alongshore;
- Short term storm events drive most cross shore change (erosion and accretion), but the profile oscillates around a long term average shape, which may shift landward or seaward (recession and progradation);
- Long term (years to decades to centuries) coastal change (under steady sea level) is primarily driven by gradients in longshore littoral drift sediment transport.

Examples of profile and planform models in widespread engineering use include:

- The Bruun Rule – cross shore (Bruun, 1962 and 1988);
- SBEACH – cross shore (US Army Corps of Engineers);
- GENESIS – longshore (US Army Corps of Engineers);
- LITPAK – separate cross shore and longshore modules (Danish Hydraulics Institute); and
- UNIBEST – separate cross shore and longshore modules (Delft/Deltares, The Netherlands).

The relative simplicity of planform models means that many can be run using time series input data with time steps of 1 hour to 1 day, for simulation times of decades to centuries – both historically and as future projections. For locations where the long term coastal change is primarily driven by gradients in littoral drift, this simulation may be more realistic than complex hydrodynamic models. Planform models can be used to model the salient formation in the lee of a SCR, but cannot accurately model the cross shore shape of the salient (Figure 3.12), which may deviate from the equilibrium assumed.

An example of output from GENESIS modelling of an offshore breakwater is shown in Figure 3.13.



(High Tide) (Low Tide)

Figure 3.12: Salient Cross Shore Profile
(Dally and Pope, 1986)

Before GENESIS Modelling After Construction

Figure 3.13: GENESIS Modelling of Semaphore Park Offshore Breakwater

3.2.10 Surfability Models

As discussed in Section 2.6 the original concept of surfer speed as a function of wave height and peel angle was published by Walker (1974). Black *et al.* (1997) extended surfability assessment to include cross shore wave shape which is predominantly caused by offshore bathymetry and swell characteristics.

At the most basic level, surfability on an ASR can be assessed by the presence (or absence) of wave breaking on the crest of a structure – assuming the structure itself is to provide the surfing amenity and is not intended to modify incoming waves to provide surfing inshore of it. Higher level assessment involves extracting the breakpoint and wave height from a wave transformation model and calculation of peel angles and length of ride.

Mortensen (2010) produced a model and animation based on a Boussinesq wave model coupled with a volume of fluid wave model which appears to reproduce the wave breaking process from a surfing perspective. Modelling of surfability should be considered to be in the early stages of research.

There are no models available off the shelf which directly assess surfability. All require substantial post processing of model output, interpretation of results and interfacing of the model output with the prevailing tides, wind and wave climate.

3.2.11 Commonly Used Models

The underlying source code and published work for many of the principles underlying numerical models has existed since the 1970s. Much of the recent development has been in better data interchange between various components, user friendliness, utilising advances in computing and improved graphical output (including animations). Numerous consultants and institutions have "in-house" models suitable for coastal morphodynamic modelling. These may incorporate improvements on commercial or widely used (freeware) models, but such models may not be readily available for those outside the institution which developed them.

A list of commonly used morphological models and their underlying schemes is shown in Table 3.1.

Table 3.1: Summary of Commonly used Coastal Morphology Models

Model	Wave Driver	Flow Model	Sediment Transport	Morph Updating	Bed Comp	Grid	Availability
Simple Profile and Planform Models							
BEACHPLAN	Snell's law and depth lim	1D	CERC Longshore	Each time step	1D	Regular	Licensed
Bruun Rule	n/a	n/a	1D	1 step		n/a	Simple calculation
LITPAK	Snell's law, Battjes and Jansen	1D	Separate cross shore and longshore	Each time step	1D	Variable	Licensed
GENESIS	Snell's law and depth lim	1D	CERC Longshore	Each time step	1D	Regular	Licensed
SBEACH	Snell's law and Dally *et al.* (1984)	n/a	Cross shore	Each time step	1D	Variable	Licensed
UNIBEST	Snell's law, Battjes and Jansen	1D	Separate cross shore and longshore	Each time step	1D	Variable	Licensed
Hydrodynamic Models							
ADCIRC	Spectral	2D-H/3D	2D-H/3D	Online	2D-H	Unstruc, finite element/ volume	Licensed
Delft3D	Spectral wave av/ Short wave av	2D-H/3D	2D-H/3D sand and mud	Online with morfac	3D	Curve, finite diff	Open source
FINEL	Spectral	2D-H	2D-H	Online with morfac	2D	Unstruc, finite element	Not
Mike21	Spectral/ parabolic/ mild slope	2D-H/Q3D	2D-H/Q3D sand and mud	Offline or online with morfac	3D	Rect, curve, finite diff, unstruc finite vol	Licensed
RMA	External	2D-H/3D	2D-H/Q3D sand and mud	Offline or online with morfac	2D-H	Unstruc, finite element/ volume	Licensed
ROMS-SED	Spectral	3D	3D	Online with morfac	3D	Curve, finite diff	Open source
SWAN	Spectral	n/a	n/a	n/a	n/a	Rect	Open source
Telemac	Spectral	2D-H/3D	2D-H/3D	Offline or online with morfac	3D	Unstruc, finite element	Licensed
X-Beach	Spectral wave av/ Short wave av	2D-H	Q3D	Online with morfac	3D	Rect, finite diff	Open source

3.3 Example Applications of Physical and Numerical Models for Artificial Reefs

Many of the existing submerged breakwater, ASR and MPR structures around the world have been modelled during their design. In some cases this provides an opportunity to look back and consider the level of reliability with which the model predictions eventuated, while in other cases changes to structure design during or post construction as well as other influences on beach response (such as nourishment) make direct comparisons more difficult. Appendix A provides a review of the existing reef structures from around the world where information is available regarding the design, construction, and performance. Where suitable information is available to compare the modelled reef performance with the observed performance post construction, this comparison is made in detail in Appendix A.

3.4 Summary Box: Application of Physical and Numerical Models for SCR Structures

It is strongly recommended that detailed design of any SCR structure adopt a hybrid modelling approach, whereby the strengths of both numerical and physical models are utilised to arrive at the final reef design. Furthermore, it is recommended that modelling of any structure with environmental, social, or economic significance be underpinned by site specific data collection for wave transformation, water levels, and sediment transport. Numerical models are well suited to assessing simpler wave, hydrodynamic, and morphological processes over larger spatial and time scales, with the degree of certainty in model predictions proportional to the level of model calibration. Any engineering design for a SCR that relies on numerical modelling for prediction and design of shoreline response should as a minimum:

- Have validated hydrodynamic model predictions relating to wave induced currents around the structure (likely through comparison with physical model or previous study data);
- Ensure that the morphological model is calibrated to existing observations at the site, and preferably also be validated for simulating pre and post construction conditions at an existing SCR site; and
- Consider the impact of short, mid, and long term variability of environmental variables on structure performance.

Physical models should be used for assessing reef armour stability, wave, and hydrodynamic processes, and can also be applied to gain valuable qualitative and semi-quantitative insight into morphological response. Application of physical models should:

- Ensure dominant processes and forces are reproduced with minimal scale effects;
- Consider the impacts of scale effects on processes in the model where these effects cannot be minimised, such as response of mobile beach sediments;
- Ensure predicted shoreline alignment is validated for pre-structure conditions if beach response is being analysed;
- Understand the limitations of the modelling, especially when simulating beach response; and
- Consider the impact of short, mid, and long term variability of environmental variables on structure performance.

There are many example applications where numerical and physical models have been applied beyond their limitations, and while the modelling has been able to consider many complexities of the case and produce results, lack of calibration means that the results may not be any more reliable than could have been predicted with simple empirical solutions or the judgement of an experienced practitioner. Nevertheless, the analysis has been relied upon as what "will" happen and not what "may" happen, which (in the opinion of the authors) has contributed to structures not living up to expectations.

4. Performance and Review of Existing Reefs

4.1 Outline of Review of Existing Submerged Constructed Reefs

The objectives of this review were to identify, summarise and evaluate submerged constructed reefs both in Australia and internationally, with a focus on those reported in technical literature. In particular, the review has been undertaken to provide an objective summary of the coastal protection performance of SCR structures on the basis of information presented in technical publications. This information provides a useful overview of the current state of application of SCR structures and identifies trends in the types of structures that have performed more reliably. The review is presented in four parts:

- Detailed review of key existing reefs (Appendix A);
- Summary of existing reef characteristics (Section 4.2);
- Discussion of existing reef performance (Section 4.3); and
- Lessons learnt from existing reef projects (Section 4.4).

Where suitable information has been available, which is not the case for all reef structures, the detailed review in Appendix A and this evaluation have considered a range of aspects for the existing SCR structures, including:

- Summary of reef structural and environmental characteristics;
- Evaluation of coastal protection performance, including identification and discussion of primary variables influencing shoreline response;
- Environmental variables such as wave climate, tides, and sediment transport characteristics;
- Cost;
- Capability and success of construction methods; and
- Durability of construction materials.

Figure 4.1: Borth MPR June 2012, Wales
(Source: eCoast, 2012)

4.2 Summary of Existing Submerged Constructed Reefs

Lamberti *et al.* (2005), within the European research program DELOS, reported that submerged structures are more common in Europe with 20% to 30% of the breakwaters inventoried (total of 837) being fully submerged compared with less than 1% (over a total of 1548 inventoried) in Japan and a negligible number in the USA (total of 200 breakwaters inventoried). However, more recent statistics for Japan and USA may show an increase in use of submerged constructed reefs. While the use of submerged constructed reefs has not been widely adopted in Australia to date, there has been a trend for communities in Australia to consider submerged reef structures as a viable alternative coastal protection method due to the low visual impact, possible multi-purpose benefits and their active promotion by designers and builders. Australia is also home to the largest yet built MPR, the Narrowneck artificial reef at the Gold Coast.

Figure 4.2: Reefball^{TM} Submerged Breakwaters, Gran Dominicus, Dominican Republic (Source: www.reefball.org)

A list of major artificial reef projects is presented in Figure 4.3 with information for each project summarised in:

- Table 4.1: Structure type, intended purpose, success/failure;
- Table 4.2: Engineering features;
- Table 4.3: Environmental variables;
- Table 4.4: Estimated costs and cost rates.

This summary has been based on cases that are documented in published scientific literature. More information on the design and monitoring of these structures can be found either in the detailed review of key reef projects in Appendix A, or by consulting the available references for each project. Although not exhaustive, the list of field cases covers a wide range of hydrodynamic conditions with a variety of wave climates and tidal ranges considered, as well as different geomorphological settings such as longshore and cross-shore transport dominated coast, and exposed or sheltered beaches.

1. El Segundo, California, USA
2. Belmar, New Jersey, USA
3. Delaware Bay, Delaware, USA
4. Avalon, New Jersey, USA
5. Cape May, New Jersey, USA
6. Vero Beach, Florida, USA
7. Midtown & Dupont Palm Beach, Florida, USA
8. Yucatan, Mexico
9. Grand Cayman, Cayman Islands, Caribbean
10. Gran Dominicus, Dominican Republic, Caribbean
11. Borth, Wales, United Kingdom
12. Boscombe, England, United Kingdom
13. Pellestrina, Venice, Italy
14. Lido di Dante & Punta Marina, Ravenna, Italy
15. Marche, Italy
16. Lido di Ostia, Rome, Italy
17. Undisclosed Site, United Arab Emirates
18. Kovalam, India
19. Young-Jin Beach, South Korea
20. Iburi Reef, Hokkaido, Japan
21. Niigata, Japan
22. Keino Matsubara Beach, Japan
23. Cable Station, WA, Australia
24. Semaphore Park, SA, Australia
25. Narrowneck, QLD, Australia
26. Mount Maunganui, New Zealand
27. Opunake, New Zealand

Figure 4.3: Location of SCRs Around the World

Readers are directed to Table 1.1 for clarification on the terminology used and definitions for the different structure types in the summary (ASR, MPR, and Submerged Breakwater). It should also be noted that a structure has been classified as a MPR if design studies indicated that it would provide multiple outcomes. If a reef was constructed with a single objective, to provide enhanced surfing amenity for example, but design studies indicated that it would also provide a second outcome, coastal protection for example, then it has been classified as a MPR.

Table 4.1: Reef Summary: Structure Type, Intended Purpose and Successfulness

	Site	Structure Type	Primary Objective	Primary Successful [5]	Secondary Objective	Secondary Successful [5]
Australia	Cable Station, WA	ASR	Surfing Enhancement	Partially	na	na
	Narrowneck, QLD	MPR	Coastal Protection	Minor	Surfing Enhancement	Minor
	[1] Semaphore Park, SA	SB	Coastal Protection	Yes	na	na
New Zealand	Mount Maunganui, NZ	MPR	Surfing Enhancement	No[2]	Coastal Protection	Negligible[2]
	Opunake, NZ	ASR	Surfing Enhancement	No[2]	na	na
USA	Midtown Palm Beach, Florida	SB	Coastal Protection	No	na	na
	Dupont Palm Beach, Florida	SB	Coastal Protection	No	na	na
	Vero Beach, Florida	SB	Coastal Protection	No	na	na
	Avalon, New Jersey	SB	Coastal Protection	Partially	na	na
	Cape May, New Jersey	SB, Groynes	Coastal Protection	Yes	na	na
	Belmar, New Jersey	SB	Coastal Protection	Yes	na	na
	Cape May #2, New Jersey	SB, Groynes	Coastal Protection	Yes	na	na
		SB, Groynes	Coastal Protection	No	na	na
	Delaware Bay, Delaware	SB	Coastal Protection	No	na	na
	El Segundo, California	ASR	Surfing Enhancement	No	na	na
Mexico	Yucatan (5 sites)	SB	Coastal Protection	Yes	na	na
Caribbean	Gran Dominicus, Dominican Republic	SB, Natural Reef	Coastal Protection	Yes	Ecological Enhancement	Yes
	Grand Cayman, [4]Cayman Islands	SB	Coastal Protection	Yes	Ecological Enhancement	Yes

Table 4.1 Cont'd: Reef Summary: Structure Type, Intended Purpose and Successfulness

	Site	Structure Type	Primary Objective	Primary Successful [5]	Secondary Objective	Secondary Successful [5]
Italy	Lido di Ostia #1, Rome	SB, Groynes	Coastal Protection	No	na	na
	Lido di Ostia #2, Rome	SB, Groynes	Coastal Protection	Yes	na	na
	Lido di Dante, Ravenna	SB, Groynes	Coastal Protection	Yes	na	na
	Marche	SB, Groynes	Coastal Protection	No	na	na
	Pellestrina, Venice	SB, Groynes	Coastal Protection	Yes	na	na
	Punta Marina, Ravenna	SB, Groynes	Coastal Protection	Yes[3]	na	na
UK	Boscombe, England	MPR	Surfing Enhancement	Partially	Coastal Protection	Minor
	Borth, Wales	MPR, BW, Groynes	Coastal Protection	Unknown[3]	Surfing Enhancement	Unknown[3]
Japan	Iburi Reef, Hokkaido	SB	Coastal Protection	Yes	na	na
	Keino-Matsubara Beach	SB	Coastal Protection	No	na	na
	Niigata	SB	Coastal Protection	No	na	na
South Korea	Young-Jin Beach	SB	Coastal Protection	Partially	na	na
India	Kovalam, Kerala	MPR	Coastal Protection	Yes[4]	Surfing Enhancement	Yes[4]
UAE	Undisclosed	SB	Coastal Protection	Yes	na	na

Notes: (1) Trial structure originally built as emergent (+1 m AHD), crest was lowered to -0.9 m AHD following the rolling off of top geocontainers
(2) Construction incomplete
(3) Insufficient monitoring period
(4) Performance results of this reef are considered preliminary and subject to further monitoring and publishing of results
(5) A reef was considered successful if it achieved its objective or the outcome predicted in design studies, partially successful if it provided some improvement but not to the extent initially predicted, or not successful if it had no notable impact

Table 4.2: Reef Summary: Engineering Aspects

	Site	Material	Longshore Length [(1)] (m)	Cross Shore Width [(1)] (m)	Water Depth (m)	Offshore Distance [(2)] (m)	Submergence (m)	Nourishment (m)	Settlement or Scour
Australia	Cable Station, WA	V-shape, Rocks	140	70	3.0-6.0	275	1.0 to 2.5	N	Not Reported
	Narrowneck, QLD	V-shape, GCs	170	280	2.0-10.0	100-600	min. 1.0	Y	Y
	[(3)]Semaphore Park, SA	GCs then rock	197	13	2.0	200	1.0	Y	Y
New Zealand	Mount Maunganui, NZ	V-shape GCs	90	70	3.0-4.5	250	0.4	N	Y
	Opunake, NZ	GCs	30	100	1.8-3.0	~300	At LAT	N	Not Reported
USA	Midtown Palm Beach, Florida	PEP™ Units	1260	4.6	3.0	70	0.7	N	Both
	Dupont Palm Beach, Florida	PEP™ Units	168	4.6	2.2	53	0.6	N	Both
	Vero Beach, Florida	PEP™ Units	915	4.6	2.1-2.7	85	0.3	N	Both
	Avalon, New Jersey	Beachsaver™ Units	305	4.8	2.2-3.8	100-150	min. 0.4	Y	Both
	Cape May, New Jersey	Beachsaver™ Units	305	4.8	2.0-2.5	50-65	At MLW	N	N
	Belmar, New Jersey	Beachsaver™ Units	335	4.8	2.3-2.4	128	0.7 to 1.6	Y	Both
	Cape May #2, New Jersey	Beachsaver™ Units	220	4.8	2.7	120	At MLW	N	Both
		T-Sill Units	200	3.66	2.7	120	1.8	N	Both
	Delaware Bay, Delaware	Rubble Mound	300	Not Rep.	1.0	75	0.0	Y	Y
	El Segundo, California	V-shape GCs	64	29	3.0	50	0.9	N	Y
Mexico	Yucatan (5 sites)	GCs	4,300	1.8	1.0	MLWM	At MHW	N	Not Reported
Caribbean	Gran Dominicus, Dominican Republic	Reef Ball™ Units	~370	6	1.6-2.0	15-30	0.3-0.8	N	Not Reported
	Grand Cayman, Cayman Islands	Reef Ball™ Units	80	9-11	1.4-2.0	37	min. 0.1	Y	Not Reported

Table 4.2 Cont'd: Reef Summary: Engineering Aspects

	Site	Material	Longshore Length (1) (m)	Cross Shore Width (1) (m)	Water Depth (m)	Offshore Distance (2) (m)	Submergence (m)	Nourishment (m)	Settlement or Scour
Italy	Lido di Ostia #1, Rome	Rubble Mound	3000	15	4.0	100	1.5	Y	Y
	Lido di Ostia #2, Rome	Rubble Mound	700	15	3.0-4.0	50	0.5	N	Y
	Lido di Dante, Ravenna	Rubble Mound	770	12	3.0	150	0.5	Y	Y
	Marche	Rubble Mound	Not Rep.	10-12	3.0	100-200	0.5	N	Y
	Pellestrina, Venice	Rubble Mound	9000	14	4.5	300	1.5	Y	Y
	Punta Marina, Ravenna	Tecnoreef™ & Rocks	850	5	2.7	280	1.0	Y	Y
UK	Boscombe, England	GCs	80	90	3-5	225	0.5	Y	Y
	Borth, Wales	Rubble Mound	72 (Nth Reef) 85 (Sth Reef)	170 (Nth Reef) 45 (Sth Reef)	4.0	110 (Nth Reef) 140 (Sth Reef)	At MSL	Y	Not Reported
Japan	Iburi Reef, Hokkaido	Aquareef™ Units	150	19	3.5	?	At MHW	N	N
	Keino-Matsubara Beach	Not Reported	80	20	4.0	85	2.7	Y	Not Reported
	Niigata	Not Reported	540	20	8.5	400	1.5	N	Not Reported
South Korea	Young-Jin Beach	GCs	240	8.4	3.0	90-100	1.2	N	Scour
India	Kovalam, Kerala	GCs	65	85	1.3-3.7	90	0.2-3.4	N	Not Reported
UAE	Undisclosed	GCs	500	12	0.0-2.0	50-200	0.0	Y	N

Notes: (1) For V-shape reefs refer to max. alongshore and cross shore dimension
(2) Distance to the undisturbed shoreline prior construction
(3) Trial structure originally built as emergent (+1 m AHD), crest was lowered to -0.9 m AHD following the rolling off of top geocontainers, then reconstructed from rock with crest at +1 m AHD.

Table 4.3: Reef Summary: Environmental Conditions

	Site	Beach Slope [1] (1V:XH)	Typical Tide Range (m)	Typical Wave Period and Height (s,m)	Longshore Transport Rate (10^3 m^3/yr)
Australia	Cable Station, WA	Not Rep.	0.8	H=1-2 (summer), H=1.5-2.5 (winter)	Negligible?
	Narrowneck, QLD	50	2.0	H~1	500
	[3]Semaphore Park, SA	150	2.0	Not Rep.	80
New Zealand	Mount Maunganui, NZ	Not Rep.	2.0	T~11, H~1.5	60-80
	Opunake, NZ	Not Rep.	>3.0	Not Rep.	Not Rep.
USA	Midtown Palm Beach, Florida	25	Not Rep.	Not Rep.	100
	Dupont Palm Beach, Florida	Not Rep.	Not Rep.	Not Rep.	Yes
	Vero Beach, Florida	30	Not Rep.	Not Rep.	30
	Avalon, New Jersey	Not Rep.	Not Rep.	Not Rep.	Yes
	Cape May, New Jersey	Not Rep.	1.5	H=0.6 (summer), H=1.2 (winter)	153
	Belmar, New Jersey	Not Rep.	Not Rep.	Not Rep.	Yes
	Cape May #2, New Jersey	Not Rep.	1.5	H=0.6 (summer), H=1.2 (winter)	153
		Not Rep.	1.5	H=0.6 (summer), H=1.2 (winter)	153
	Delaware Bay, Delaware	Not Rep.	Not Rep.	Not Rep.	Negligible
	El Segundo, California	Not Rep.	1.6	T=6, H=0.9 (summer), H=1.5 (winter)	Not Rep.
Mexico	Yucatan (5 sites)	20	<1.0	T=4-8, H<1.0	50-100
Caribbean	Gran Dominicus, Dominican Republic	19	0.4	Not Rep.	170
	Grand Cayman, [4]Cayman Islands	Not Rep.	0.3-0.5	Not Rep.	Not Rep.

Table 4.3 Cont'd: Reef Summary: Environmental Conditions

	Site	Beach Slope (1V:XH)	Typical Tide Range (m)	Typical Wave Period and Height [(2)] (s,m)	Longshore Transport Rate (10^3 m^3/yr)
Italy	Lido di Ostia #1, Rome	20	0.4	H_s~2 (97^{th} %ile)	20-50
Italy	Lido di Ostia #2, Rome	10	0.4	H_s~2 (97^{th} %ile)	20-50
Italy	Lido di Dante, Ravenna	15	0.3 (neap) 0.8 (spring)	H_s~0.4 (average) T_p~4 (average)	Negligible
Italy	Marche	10	0.14 (neap) 0.47 (spring)	H_s~0.5 (average)	Negligible
Italy	Pellestrina, Venice	100	1.0	H~0.7 T_p~4	Not Reported
Italy	Punta Marina, Ravenna	Not Rep.	0.3 (neap) 0.8 (spring)	H_s~0.4 (average) T_p~4 (average)	Negligible
UK	Boscombe, England	Not Rep.	1.8	T~4, H~0.5	Not Reported
UK	Borth, Wales	Not Rep.	6.0	Not Reported	Not Reported
Japan	Iburi Reef, Hokkaido	Not Rep.	Not Rep.	Not Reported	Not Reported
Japan	Keino-Matsubara Beach	10 to 30	Not Rep.	H=0.4 (summer), H=1 (winter)	Not Reported
Japan	Niigata	50	Not Rep.	Not Reported	Not Reported
South Korea	Young-Jin Beach	Not Rep.	Not Rep.	H>4 m (storm, winter)	Not Reported
India	Kovalam, Kerala	Not Rep.	~1.0	T=11-15s, H=1-2.5	Not Reported
UAE	Undisclosed	12	1.0	T=5, H=1	Not Reported

Notes: (1) The beach slope is typically the natural gradient in the wave shoaling and breaking zone, though the definition is often not well reported in literature
(2) The wave height and period statistics that are reported in literature vary, with the numbers in this table only intended for relative comparison between the sites

Table 4.4: Approximate Reef Cost Rates

	Site	Structure Type	Cost ($M)[1]	Cost ($ per m^3 Reef Volume)[1]	Cost ($ per m Beach Protected)[1,2]
Australia	Cable Station, WA	ASR	2.51	457	NA
	Narrowneck, QLD	MPR	3.91	56	9,800 min.
	Semaphore Park, SA	SB	1.10	NA	5,500
New Zealand	Mount Maunganui, NZ	MPR	1.53	545	10,200 min.
	Opunake, NZ	ASR	0.93	274	NA
USA	Cape May #2, New Jersey	SB, Groynes	~0.88	NA	4000
		SB, Groynes	~0.28	NA	1400
	El Segundo, California	ASR	0.68	504	NA
UK	Boscombe, England	MPR	~4.91	384	9,800 min.
India	Kovalam, Kerala	MPR	1.41	351	7,000 min.
UAE	Undisclosed	SB	~6	NA	10,100[(3)]

Notes: (1) Cost rates have been corrected to approximate equivalent 2013 AUD.
(2) This is a crude approximation only, as the estimated length of coastline actually protected and the degree of protection provided by MPRs is difficult to quantify. It is only intended to provide a ball park estimate of protection cost rates for comparison purposes, and for MPRs is an absolute minimum as it is based on the absolute maximum salient length between the points where the coastal protection asymptotes to zero.
(3) This project included design and construction of the submerged breakwaters, but also included significant beach restoration and nourishment. The project was also undertaken to short timeframes, during high temperatures, and with long distance freighting of geotextile containers to a remote island.

4.3 Discussion of Existing Submerged Constructed Reef Performance

Of the thirty-two (32) SCR structures reviewed, twenty-nine (29) were intended to provide coastal protection as a primary or secondary objective. In considering well founded information available in published literature, approximately half of these structures did not have any significant accretionary impact on shoreline alignment, at least when compared to the predicted morphological response. A small number of structures resulted in increased erosion/recession, but these cases were rare and the design problems easily recognised. A slightly higher percentage of the submerged breakwaters reviewed were successful in providing coastal protection, though in general the majority of structures that have been associated with reasonable coastal protection were used in conjunction with other coastal protection methods such as groynes and/or nourishment. In these cases it is not possible to determine the extent of protection that can be directly attributed to the submerged breakwaters alone.

Eight artificial reefs were constructed with the objective of improving surfability and approximately half of these were considered at least partially successful, although the measure of success in this case needs to be cautiously balanced between realistic design objectives and stakeholder expectation. For the cases where ecological monitoring was performed, all the reefs were observed to promote colonisation by marine algae and aquatic species.

Figure 4.4: Successful Submerged Breakwater, Pellestrina Island in the Venice Lagoon, Italy (in conjunction with groynes) (Source www.venicethefuture.com)

Of the five (5) MPRs predicted to provide both improved surfing amenity and coastal protection, only one appears to have resulted in any significant positive effect on shoreline alignment to date (Kovalam). This structure has had less than three years of beach monitoring so far, which is insufficient to enable confident judgement of the long term impact on the shoreline position. An independent assessment of the morphological response of this reef is currently being undertaken by the Centre for Earth Science Studies, however, the results of the study were not complete at the time of this report. Recent unpublished observations indicate that damage to this reef is now hampering its effectiveness for providing coastal protection (E. Rendle, pers. comm., 2013).

While the Boscombe MPR was also initially reported to provide significant coastal protection (Mead *et al.*, 2010), further monitoring has shown that there has been no noticeable long-term impact of the reef on the shoreline (0 m contour) alignment (Rendle and Davidson, 2012). It should be noted, however, that the majority of accretion predicted to occur from this reef was predominantly around the -1 m bathymetric contour (K. Black, pers. comm., 2013) which was not reported by Rendle and Davidson (2012). The localised scour and bathymetric steepening that has occurred inshore of the reef was well predicted during design (K. Black, pers. comm., 2013). While the Boscombe reef appears to only provide minor coastal protection, aspects of the observed morphological response appear to have been predicted by modelling during design. The Borth project was only completed in 2012 so the performance of this MPR as a coastal protection structure is not yet known.

The majority of these MPR structures had significant detailed design effort, including numerical and physical modelling of sediment transport processes, however, most have not provided the degree of beach widening predicted during the design phase. This has typically been attributed to the constructed reef differing in profile from the design reef due to difficulties during construction or post construction damage. Regardless of the reason, this outcome highlights the complexity of designing and constructing successful MPR structures. The risk of these structures underperforming for coastal protection compared with design remains reasonably high despite having over a decade of experience, and this risk should be considered during the planning and feasibility assessment of any coastal protection project that incorporates the use of an artificial submerged reef.

Figure 4.5: Kovalam MPR, Kerala India
(Source: ASR, 2012)

Table 4.4 presents approximate cost information for the reviewed SCRs where suitable information was available. For ASRs and MPRs the cost rate typically expressed in previous literature has been a volumetric rate of dollars per cubic metre of reef volume (ASR, 2008a; Jackson and Corbett, 2007). Based on the analysis presented in this study, the ASR and MPR structures had volumetric cost rates typically in the range of $250 to $550 per cubic metre of reef volume. While this cost rate is useful for estimating the approximate cost of MPR and ASR structures based on their size and for comparing costs between structures, it is not so useful for assessing the financial feasibility for the extent of coastal protection offered. Costs of most common coastal protection structures (seawalls, revetments, groynes) are typically expressed as a cost per linear metre of protected coastline. For relative comparison purposes, the costs of the SCR structures have been normalised against the length of protected coastline generated, so that the financial feasibility for coastal protection purposes can be compared. This analysis is somewhat rudimentary and has a range of limitations that should be noted:

- The length of coastline considered to be protected by MPRs was the maximum length that the structure was reported to impact the shoreline. This cost rate is therefore the minimum cost per metre of protected shoreline. This cost also does not indicate the level of protection provided, which in terms of increased beach width was negligible for most MPRs analysed and at the extremities of the protected zone was minimal;

- The submerged breakwater structures analysed were all for environmental conditions that were less severe than the NSW coastline; and

- The design life of the structures varies but is expected to range from approximately 20 years (geotextile structures) to 50 years (concrete and rock structures).

The results indicated that MPR structures had a minimum cost rate of the order of $7,000 to $10,000 per linear metre of coastline protected, whereas submerged breakwaters had a lower cost rate of the order of $1,500 to $5,500 per linear metre of coastline protected. A high quality engineered rock seawall on the open NSW coast has a cost rate of the order of $5,000 to $10,000 per linear metre (including reinforced concrete crest footpath/wave return wall, excavated toe, landscaping, etc.), however, provides a more defined level of erosion protection and hazard reduction, albeit with a potential reduction in beach amenity.

4.4 Lessons Learnt from Existing Submerged Constructed Reef Projects

Vertical displacement of the structures of the order of 0.5 to 2 m as well as localised bed scouring was observed for most of the structures considered. This displacement was typically a combination of settlement into the seabed (particularly for hard/rigid structures), localised scour and large scale surf zone morphodynamics, as well as stretching/settlement of bag fill material for geotextile tube structures. Vertical displacement reduces the structure crest levels, which subsequently increases the wave transmission properties of the structure, reduces the effectiveness of coastal protection and also alters the expected performance in terms of surfing enhancement. Scour holes were regularly reported to form at the toe of the structures on their leeward side, at roundheads and in the gaps between reefs in the case of multi-segments reefs. This is an important consideration during design, as scouring can have serious implications for structural integrity.

Both the Boscombe and Mount Maunganui MPRs have underperformed from both a surfing amenity and coastal protection perspective in part, but not exclusively, due to differences between the design and 'as constructed' profiles. Physical problems with reefs that have resulted in loss of reef volume and profile include (Mead and Borrero, 2011; Rendle and Davidson, 2012):

- Ruptured containers during construction;
- Sediment inundation during filling;
- Unsealed filling ports;
- Hydraulic reshaping of fill material by waves;
- Damage by vessel impact; and
- Potential damage by wave impact.

While geotextile structures are promoted as having the ability to be used as a "trial" that can be easily removed, such removal comes at considerable expense if not specifically considered during the design phase. Removal can also provide an environmental risk if the geotextile is torn during the reef removal process. However, there is little doubt that removal of geotextile structures is easier than rock structures. Experience gained from the Pratte's Reef trial ASR is presented in Leidersdorf *et al.* (2011). Pratte's Reef was an experimental geotextile container reef structure built in 2000 and removed in 2008 after it failed to meet performance criteria. Notable aspects of the project include:

- Construction of the reef cost approximately $550,000 ($250,000 over the initial budget);
- Removal of the reef cost approximately $551,000 ($266,000 over the initial budget and the same cost as the reef construction); and
- Some forty (40) geotextile containers were also never recovered from the reef (through loss or tearing of the geotextile), posing potential environmental hazards.

4.5 Summary Box: Analysis of Existing SCR Structures

Based on the worldwide review of existing reef structures, the following key conclusions can be drawn:

- Of the thirty-two (32) SCR structures reviewed, twenty-nine (29) were intended to provide coastal protection as a primary or secondary objective;
- Approximately half of the "protection" structures had no significant accretionary impact on shoreline alignment compared to the predicted morphological response;
- 55% of submerged breakwaters were successful at providing increased coastal protection, though not all to the degree initially predicted;
- One of five MPR structures may be providing a reasonable level of coastal protection but this structure has only been monitored for two to three years. Three other MPRs provide only minor or negligible coastal protection compared to design, and the performance of the newest MPR (Borth) is yet to be determined.
- Eight artificial reefs were constructed with the objective of improving surfability and approximately half of these were considered at least partially successful;
- The resulting shoreline morphology often differed significantly from the design predictions, even when the best available design methods were applied;
- Most structures settled and/or suffered from localised scour which subsequently led to further maintenance and top up costs;
- Approximate construction costs per linear metre of coastline protected were in the order of \$1,500 to \$5,500 for submerged breakwater structures and \$7,000 to \$10,000 for MPR structures, compared with \$5,000 to \$10,000 for a high quality engineered rock seawall on the open NSW coast. The relatively high wave climate of the NSW coast is likely to further increase the construction costs of the offshore structures relative to the precedent structures located in milder wave climates.

5. SCRs and Coastal Protection in NSW

5.1 Overview of Submerged Constructed Reefs and their Application in NSW

The NSW coastline is managed through an established framework that consists of the Coastal Protection Act (1979), the NSW Coastal Policy (1997), and the Coastal Protection Regulation (2011). This framework requires development of Coastal Zone Management Plans (CZMPs) by Councils, which are established on the basis of Coastal Processes and Hazard Definition Studies, as well as Coastline Management Studies. The procedure for developing CZMPs and assessing coastline hazards in NSW is outlined in the document "Guidelines for Preparing Coastal Zone Management Plans" (DECCW, 2010) and accompanying technical guide notes (yet to be released at the time of writing). These documents when finalised will supersede the Coastline Management Manual (NSW Government, 1990). DECCW (2010) identifies a range of processes and hazards that could potentially be managed through protective works such as submerged constructed reefs, namely:

- Beach erosion;
- Shoreline recession; and
- Wave impacts (typically through reduction in runup and overtopping processes).

It is within these Coastal Hazard Definition and Coastal Management studies that vulnerable sections of the coast are identified and potential management options, such as submerged constructed reefs are canvassed.

Interest in SCRs and in particular MPRs has increased in recent years in terms of their desirability as a coastal protection option in some NSW coastal communities. This has also been seen in other states of Australia and around the world and is predominantly attributed to the perceived benefits of reefs outweighing the negative impacts when compared to other hard coastal protection alternatives, as well as active promotion of reef structures by designers and developers. In particular, the idea of low visual intrusion and the possibility of achieving multiple objectives with a single structure has made reefs popular with some community groups, though reefs are still frequently objected to on the grounds of "interfering with nature". Previous experience has shown that these structures have higher uncertainty than other coastal protection options in terms of their level of success. Principle 4 of the Coastal Policy (precautionary principle) requires a risk averse approach to coastal management decision making. In this context, the use of artificial reefs as a coastal protection option requires careful consideration.

5.2 Using Reefs for Coastal Protection Specifically in NSW

Yoshioka *et al.* (1993) presented a range of situations where reefs can be used to manage sediment transport processes, and when applied in the NSW context these include:

- Altering the planform of the central section of a beach that is controlled at each end by headlands. In particular if a beach has a deeply embayed or log-spiral planform that is pinned by end headlands, then it may be possible to locally advance a shoreline using a submerged detached structure;

- On sections of coast where littoral drift is prevalent and an imbalance is resulting in shoreline recession, a series of reefs can be used to locally reduce the littoral drift rate and subsequently build a wider beach. This may be particularly relevant for North-Coast NSW beaches that tend to have higher net littoral drift rates;

- In situations where a small group of properties are founded within a hazard zone (wave impact zone or zone of reduced foundation capacity), a reef or group of reefs may be applied to locally reduce the storm demand; and

- For sections of beach that are nourished, reefs may be able to provide a more stable nourished profile and reduce the volume of sand required to form a stable sub-aerial profile between the shoreline and the active profile closure.

The NSW coastline has a diverse range of environmental conditions, which range from open coast exposure to very large storm wave conditions, through to protected embayments where sediment transport processes are dominated by locally generated wind seas. As shown in Table 4.3, artificial reefs have been applied in various locations around the world that span the range of coastal environmental conditions present in NSW. While artificial reefs have had limited success as a coastal protection option to date and there are many technical obstacle to overcome, there is no reason why they could not be successfully applied in NSW. The difficulty arises when secondary objectives are required to be considered in the design of a structure. In NSW the use of MPRs that combine surfing and protection objectives will be limited in success (as it has been elsewhere) by a number of reasons including:

- Most sections of the NSW coast are relatively rich in natural surf breaks, so it will be difficult for MPRs to meet community expectations if a high quality surf break is the primary design objective;

- To accommodate surfing as a design objective the cross-shore dimension of a MPR has to be large enough to allow proper wave conditioning (Shand, 2011). This makes the structures relatively inefficient at protecting long stretches of coast, unless used in series (which may be expensive compared with other protection options); and

- Safety concerns for the various reef users results in reef designs that are not optimum for coastal protection (Jackson *et al.*, 2002).

These limitations are overcome by use of SCRs with only a coastal protection objective, such that the cross sectional profile of a reef can be optimised for coastal protection, allowing longer stretches of coast to be protected with higher efficiency. As with the approximately seventy (70) coastal breakwaters/training walls in NSW and several groynes in Queensland, the perturbation in bathymetry due to such structures may result in an enhanced surf break, although this may not have been a design objective.

5.3 Guidelines for Assessing the Feasibility of Reefs for Coastal Protection in NSW

The assessment of the feasibility of SCRs in NSW needs to be undertaken on a case-by-case basis within the NSW coastal management framework. In the majority of cases it is expected that the reefs will be targeted at reducing a hazard previously identified within a Hazard Definition Study and that the reef will be one of several options considered in a CZMP. Under the State Environment Planning Policy (Infrastructure), coastal protection structures such as SCRs are assessed by the NSW Coastal Panel in cases where a CZMP does not exist or does not cover the area where the protection works are proposed.

Installation of large scale coastal protection structures in NSW (such as SCRs) is typically a long duration process that has numerous stages including (but not limited to):

- Initial definition of hazards;
- Scoping of possible hazard management options including stakeholder considerations;
- Assessing the feasibility of selected management options and selecting the most appropriate option;
- Conceptual design of coastal protection option;
- Analysis of environmental effects;
- Detailed design of coastal protection option;
- Construction of coastal protection option; and
- Monitoring of beach/structure response and performance.

The outcomes of the feasibility assessment stage for coastal protection structures is typically a critical juncture point in the future management of a coastal site, as it identifies the optimum solution to a coastal management problem and sets the future direction that the coastal protection will take. It is therefore important that a feasibility assessment give consideration to several key points:

- The existing hazards need to be well defined and coastal processes well understood before the feasibility of structural options for coastal protection can be assessed;
- A full range of alternative solutions should be considered at the feasibility stage rather than a single option;
- The reduction in hazard that can be achieved by a reef needs to be predicted through technical assessments and quantified in terms of present and future hazard/risk reduction;
- The predicted reduction in hazard should be considered in terms of its environmental, financial, and social objectives; and
- A benefit/cost analysis should be undertaken that recognises the uncertainty and risk associated with the provision of protection.

The feasibility assessment of a reef should also take into consideration the impacts of ongoing sea level rise on the reef performance, and the costs required to top up the structure in order to maintain effective coastal protection into the future. Rising sea levels will reduce the effectiveness of reef structures through increased levels of wave transmission (and therefore reduced salient build up), as well as recession of the surrounding shoreline (effectively increasing the offshore distance of reefs).

5.4 Social Considerations for Reefs in NSW

Community involvement has long been an important aspect in the development of coastal protection structures in NSW, for example, intense opposition to the proposed Narrabeen/Collaroy seawall in 2002. Jackson *et al.* (2002) examined the social aspects associated with creating structures that combine coastal protection and water sports amenity and identified two key social aspects with respect to MPRs:

1. Involvement of the community as a stakeholder in establishing design criteria/conditions; and
2. Safety of public users of the structures.

Public consultation during the design phase of reef projects is required to clearly identify community requirements and expectations. Furthermore, contribution from the community as a key stakeholder is required for the following aspects of reef design (Jackson *et al.,* 2002):

- The types of activities that stakeholders feel should be supported and enhanced by the structure;
- The types of activities that would be beneficial to the greater community;
- Site selection and the implications of modifying existing conditions at a site;
- Types of structure that are acceptable;
- Level of coastal hazard reduction that is desired; and
- Funding arrangements.

While the surfing amenity of most artificial reefs constructed to date has been largely considered by community groups to have not met expectations (Shand, 2011), this has often been due to over-inflated community expectations. Community involvement at the project design phase is imperative for MPR structures to ensure that community groups have realistic expectations.

The safety aspects of MPR structures requires careful consideration during the design process and becomes a major concern for reef designers in situations where the reef is expected to be heavily utilised by a wide range of users (Jackson *et al.*, 2002). Despite a reef being designed to improve amenity for a particular water sport (typically surfing or diving), it will inevitably be used by a significantly broader group of people. The impact of a number of MPR design parameters on user safety was examined by Corbett *et al,* (2005), who also presented information to guide a risk assessment process for reef user safety. In particular, key risks for reef users were identified as:

- Impact with reef when coming off water craft;
- Impact with reef due to turbulent wave action in shallow water;
- Becoming trapped underwater due to gaps in structure;
- Drowning due to rips and unnatural currents;
- Dangerous marine organisms; and
- Conflicts between users.

Figure 5.1 presents an image of the Narrowneck MPR "sucking dry" during a wave trough to fully expose the geotextile container surface. To reduce this risk to a manageable level, the design of the Narrowneck MPR was altered by constructing the reef to a lower structure crest level. This was achieved by not topping up the crest to the original design level when it had settled. While a certain level of user risk is clearly still present, this design modification is one reason thought to have resulted in the structure underperforming for coastal protection compared to design.

Figure 5.1: Narrowneck Reef "Sucking Dry" During a Wave Trough
(Source: International Coastal Management)

5.5 Summary Box: Recommendations for Assessment and Use of Reefs in NSW

In recent years SCRs and in particular MPRs have been proffered as a coastal protection option for some NSW communities, due to the perception of the benefits outweighing the limitations. Within the NSW coastal management framework, SCRs can be used to manage a range of coastal hazards including:

- Beach erosion;
- Shoreline recession;
- Wave impacts (typically through reduction in runup and overtopping processes).

In NSW the use of MPRs that combine surfing and protection objectives will be limited in success by a number of factors including:

- NSW has a tidal range of approximately 1.5 m and a multi-directional wave climate with a wide wave height and period distribution. To accommodate surfing as a design objective the cross-shore dimension of a MPR has to be large enough to allow proper wave pre-conditioning under a range of wave and tidal conditions. This makes the structures relatively cost-inefficient at protecting long stretches of coast, unless used in series (which is expensive compared to other protection options);

- Most sections of the NSW coast are relatively rich in high quality natural surf breaks, resulting in high community expectations if surfing is a primary design objective;

- Safety concerns for the various reef users results in reef designs that are not optimum for coastal protection.

These limitations are overcome by use of SCRs that have a single objective to provide coastal protection. This allows the cross sectional profile of a reef to be optimised for protection, allowing longer stretches of coast to be sheltered with higher efficiency. Such perturbations may still result in the formation of surf breaks.

It is important that a feasibility assessment of SCR structures give consideration to several key points:

- The existing hazards need to be well defined and coastal processes well understood before a reef can be assessed for feasibility, if coastal protection is an objective;

- A range of alternative solutions should be considered at the feasibility stage to allow selection of the best option to achieve the management objectives;

- The reduction in hazard that can be achieved by the reef needs to be predicted through technical assessments and quantified in terms of present and future hazard/risk reduction;

- The predicted reduction in hazard should be considered in terms of its environmental, financial, and social objectives;

- A benefit/cost analysis should be undertaken that recognises the uncertainty and risk associated with the provision of protection.

6. Risk and Recovery from Failure

On a relatively simple, straight coastline, it is likely that a sufficiently elevated (emergent) offshore breakwater designed in accordance with published methods would form a locally widened beach (salient), provided there is sufficient available sand. The uncertainty in beach response increases as the crest elevation is lowered and the structure becomes submerged. Despite significant improvements in knowledge over the last decade, this still appears to stem from the present lack of understanding of the complex processes governing morphological response to reef structures in a variable natural environment. Furthermore, conditions in the lee of a submerged structure are subject to larger and more rapid temporal change due to changes in tides and waves compared to emergent structures, primarily as a result of the higher amount of transmitted wave energy. As a result there is inherently a larger uncertainty associated with reef structures and this uncertainty needs to be considered in the design and feasibility assessments, as reefs present a significantly higher risk alternative in comparison with other forms of coastal protection.

Regardless of this higher level of risk, the potential benefits of SCR structures mean that they should still be considered as a potential form of hard coastal protection, so long as the design and expectations take into consideration the lower level of reliability in prediction of the morphological response. Ongoing construction and monitoring of SCRs will result in a better understanding of the processes and refined methods for predicting shoreline response. In order to reduce the risk of poor performance, alternative strategies for reef implementation should be considered, including:

- Modelling reefs and shoreline response with methods and tools that have been demonstrated to work reliably for existing similar installations;
- Developing designs and construction methods that can be adapted to improve performance;
- Installation and monitoring of trial structures in order to calibrate/improve modelling and reduce uncertainty in beach response prediction;
- Keeping design objectives simple, focussed, and clearly understood by stakeholders; and
- Ensuring budget is available to modify or remove trial structures if required.

While design methods do not appear to have evolved to a degree that the morphological response to a reef can be accurately predicted, it is generally accepted that by following basic empirical guidance, a reef is unlikely to result in adverse shoreline impacts (such as increased erosion). Design problems have been clearly identified in the small number of previously installed submerged breakwater structures that resulted in erosion and this experience has led to improved empirical prediction methods. By using methods such as Ranasinghe *et al.* (2006) and Ranasinghe *et al.* (2010) during preliminary design, a reef should theoretically be able to be positioned so that it either creates accretion or has no impact on the shoreline alignment (i.e. does not create erosion). On this basis, it is unlikely that a reef would ever need to be completely removed, but more likely that it needed to be modified to alter performance. Nevertheless, careful consideration should be given during the project design stage, to the cost of modifying or removing reefs that have already been built. As discussed in Section 4.4, Leidersdorf *et al*. (2011) present a range of useful experiences regarding the removal of the Prattes Reef (geotextile bag) in California.

7. Conclusions

7.1 Overview

This report has presented a comprehensive review of the use of submerged constructed reefs for coastal protection in NSW. While the review has primarily focused on the use of SCRs for providing coastal protection, and in particular the applicability of SCRs for the NSW coast, a broader range of aspects have also been considered. The study was based on an extensive review of international literature that considered in excess of one hundred and fifty (150) references.

7.2 Design and Analysis Methods

The stability of rock armouring on submerged breakwaters has been studied in numerous detailed investigations and reasonable empirical design guidance is available. In contrast, the understanding of the behaviour of large sand filled geotextile containers under wave attack is not yet well developed. A small number of studies looking at the stability of geotextile containers and tubes have been undertaken with varying approaches and results. However, there is no single publication presenting stability design curves or equations for geotextile tube submerged reef structures. While it is generally stated that the geotextile mega containers used in reef construction are so large that they are inherently stable, experience from existing reefs has shown that the tubes are able to be dislodged, re-worked and damaged by wave attack. The stability of these containers requires further consideration in the future.

While there has been considerable improvement in the understanding of the mechanisms driving shoreline response to submerged reef/breakwater structures over the past decade, all completed studies have significant limitations. No single study has comprehensively tested the effects of primary structural and environmental variables on quantitative shoreline response and the shoreline response equations presented are based on either approximate field measurements of a limited number of parameters, or un-calibrated and un-validated modelling. This suggests that the available empirical techniques for assessing shoreline response are suitable only for preliminary engineering calculation and not detailed design. Structures that are designed using these methods should be considered as trial or experimental only, as it has been shown from previous real world experience that the available equations do not predict beach response with accuracy or reliability. This results in designs that inherently contain higher uncertainty than many other beach control structures.

Numerical models are well suited to assessing wave, hydrodynamic, and morphological aspects of reef structures, with the degree of certainty in model predictions proportional to the level of model calibration. Physical models should also be used for assessing reef armour stability, wave and hydrodynamic processes, and can also be applied to gain valuable qualitative and semi-quantitative insight into morphological response, but scaling limitations mean that they do not provide a complete answer. It is recommended that detailed design of any SCR structure adopt a hybrid modelling approach, whereby the individual strengths of both numerical and physical models are utilised to arrive at the final reef design. Furthermore, it is recommended that modelling of any structure with environmental, social, or economic significance be underpinned by site specific data collection programs for wave transformation, water levels, and sediment transport.

7.3 Existing Reef Projects:

Based on the worldwide review of existing reef structures, the following key conclusions can be drawn:

- Of the thirty-two (32) SCR structures reviewed, twenty-nine (29) were intended to provide coastal protection as a primary or secondary objective;
- Approximately half of the "protection" structures had no significant accretionary impact on shoreline alignment compared to the predicted morphological response;
- 55% of submerged breakwaters were successful at providing increased coastal protection, though not all to the degree initially predicted;
- One of five MPR structures may be providing a reasonable level of coastal protection but this structure has only been monitored for two to three years. Three other MPRs provide only minor or negligible coastal protection compared to design, and the performance of the newest MPR (Borth) is yet to be determined;
- Eight artificial reefs were constructed with the objective of improving surfability and approximately half of these were considered at least partially successful;
- The resulting shoreline morphology behind reef structures often differed significantly from the design predictions, even when the best available design methods were applied; and
- Most structures settled and/or suffered from localised scour which resulted in an actual crest level which differed from that specified by design and subsequently led to further maintenance and top up costs.

7.4 Future Applications of Submerged Constructed Reefs

On a relatively simple, straight coastline, it is likely that an emergent offshore breakwater designed in accordance with published methods would form a locally widened beach, provided there is sufficient available sand. The uncertainty in beach response increases and the effectiveness for coastal protection decreases for submerged structures. The present lack of understanding of the morphological response to reef structures in a naturally variable environment results in higher uncertainty in terms of coastal protection, and this uncertainty needs to be considered in any feasibility analysis, as it presents a significantly higher risk in comparison with other forms of coastal protection.

Consideration of SCRs built to date shows a relatively large number of failures, even for cases where significant effort was put into very technical designs. This cannot be ignored when considering the current ability to be able to successfully predict the processes surrounding a SCR with required accuracy. Furthermore, many failures have been as a result of structural problems due to complexities of building a structure in an active surf zone on loose unconsolidated materials. These conclusions confirm that considerable improvements are still needed in the design and construction of submerged reef structures.

Regardless of these current limitations, the benefits of SCRs mean that they should continue to be considered as an option for hard coastal protection, so long as the design and expectations take into consideration the lower level of certainty in performance. Future construction and monitoring of SCRs will result in an improved understanding of the processes and refined methods for predicting shoreline response to these structures. Throughout this period of ongoing improvement, consideration should be given to trial and experimental structures to reduce uncertainty and to create structures which meet the desired objectives.

Even when the morphological response to submerged structures can be predicted with improved accuracy, the coastal protection provided will not be as effective as a well-designed emergent structure. The difficulty in attempting to meet multiple objectives is that the success in meeting one objective may be diluted by the attempts to meet the others. While some community groups may continue to favour multi-purpose structures due to their perceived benefits, there is little doubt that focussing the objective of coastal protection structures on coastal protection rather than multiple objectives will achieve improved results with more reliability and increased efficacy.

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Appendix A: Constructed SCR Projects

A.1 Cable Station ASR, Perth, Western Australia

A.1.1 Project Description

Cable Station ASR is discussed in detail in Bancroft (1999), Pattiaratchi (1999), and Pattiaratchi (2003), and the information presented in these references has been used to compile this summary. The project is also summarised in Jackson and Corbett (2007) and Ranasinghe *et al.* (2001). Though not designed at all for coastal protection, this SCR has been included in this review as it provides useful information regarding the use of design tools for assessing wave conditions generated by SCRs.

The site, south of Cottesloe on the coast of Perth (Figure A.1), was selected for the development of an ASR for numerous reasons including localised wave climate, bathymetry, foreshore geomorphology and use, location, and environmental impacts. It was identified during the investigation of possible locations that the shoreline position at the site was relatively stable due to the rocky nature of the beach.

The final reef design was V shaped in planform, with a longshore length of approximately 140 m, cross shore width of 70 m, and crest submergence of 1 - 2.5 m below MSL (Figure A.2). The structure is located approximately 275 m offshore.

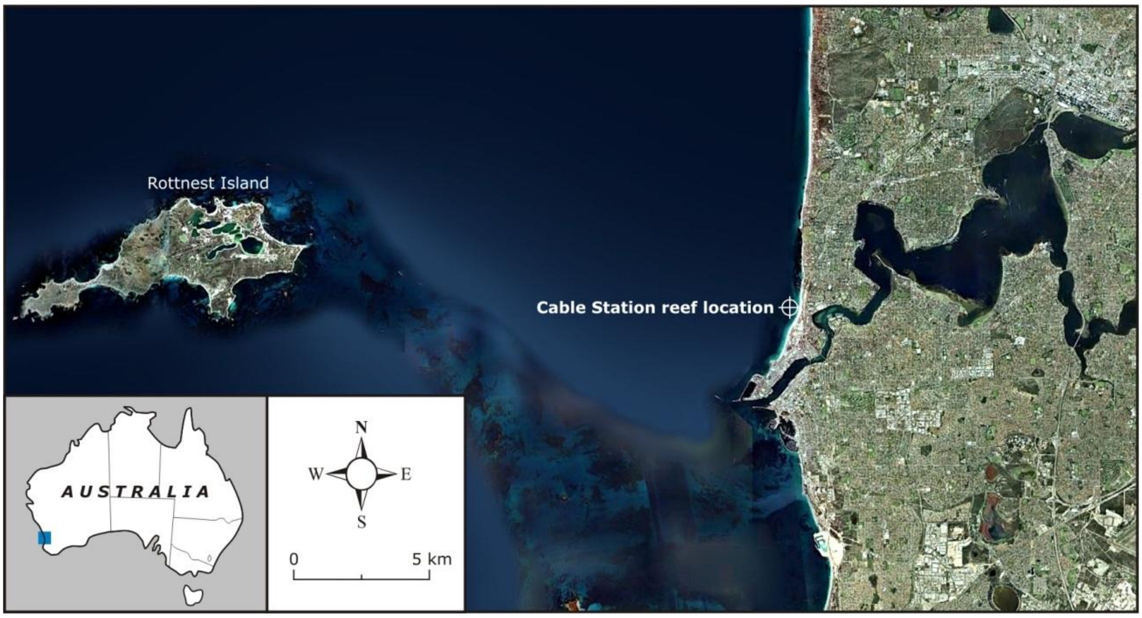

Figure A.1: Location of Cable Station ASR

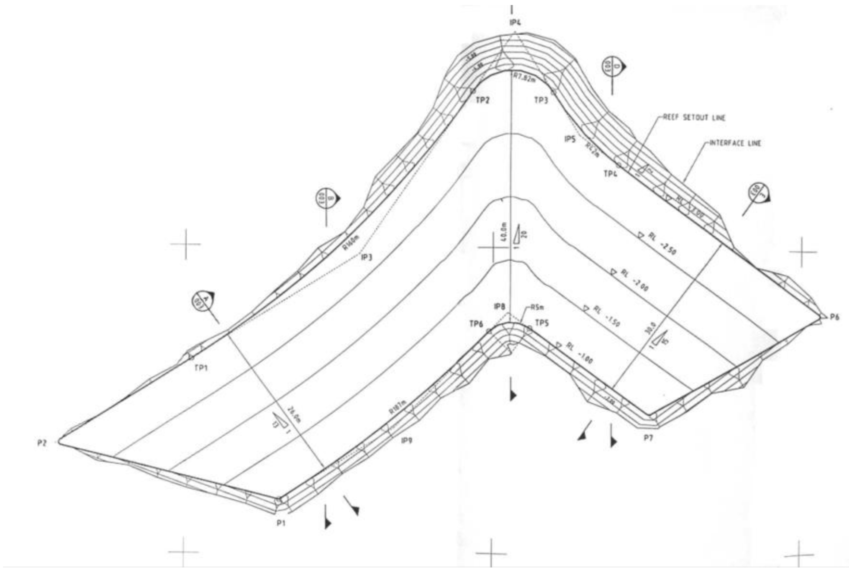

Figure A.2: Cable Station ASR Design (After Lyons, 1992)

A.1.2 Design Investigations

Detailed engineering design was coordinated by the Department of Marine and Harbours (WA). Numerous design investigations were conducted for the reef which included 2D and 3D physical modelling of the wave form aspects, as well as several numerical modelling studies of the wave form and of the likely changes in surfing amenity (as summarised by Ranasinghe *et al.*, 2001). Environmental investigations were also undertaken which considered the impact of the reef on existing ecology.

A.1.3 Construction Materials and Methods

The reef was constructed using 5,500 m^3 of 1.5 t and 3.0 t grade granite boulders, placed by excavator from a barge. The reef was constructed using the smaller 1.5 t stones as the body, keyed in place by a perimeter of the larger 3.0 t stones. No information is available regarding the long term stability and durability of the reef.

A.1.4 Performance

Post construction monitoring of the reef performance focused on the surfing amenity aspects (this was the primary objective of the reef and the primary consideration of the design studies). Monitoring of the reef ecology has also been undertaken. Pattiaratchi (2003) presented the results of a video monitoring study of the post construction surfing conditions. In this investigation the reef was classed as having "surfable" conditions if surfers were visibly present in the recorded images (Ranasinghe *et al.* 2001). The general result of the investigation concluded that the reef is performing as well, if not better than, the initial design investigations predicted (with respect to the number of days of breaking and "surfable" waves). The success of the reef and the generated surfing conditions have been disputed somewhat by some within the surfing community. Subjective anecdotal reports from surfers who have regularly surfed the reef is that the wave quality is good during big swells, but that the wave modelling overestimated the local wave height and that the number of good surfable days per year is small.

A.1.5 Cost

Documented total costs for the reef vary from $1.51 million (Ranasinghe *et al.*, 2001) to $1.8 million (Jackson and Corbett, 2007). The normalised cost (AUD, 2013) is estimated to be $2.5 million, with an approximate cost breakdown of:

- Feasibility Studies: $0.5 million;
- Construction: $2.0 million.

A.2 Narrowneck MPR, Gold Coast, Queensland

A.2.1 Project Description

Narrowneck is the largest SCR project undertaken to date in the world, and is located at the Gold Coast in Southern Queensland (Figure A.3). The stretch of coast at the reef location is relatively long and straight, and prior to reef construction and nourishment, has suffered ongoing recession combined with intermittent storm erosion. The beach at the site has a net littoral drift of approximately 500,000 m^3 per year to the north. The mean significant wave height is 1.0 m, and the coast is subject to semi-diurnal tides with a range of approximately 2 m (Jackson *et al.*, 2007).

The reef was developed as a part of the Gold Coast Beach Protection Strategy, and was broadly designed to provide the following multi-purpose criteria:

- Provide a control point for a nourished wider beach and dune;
- Allow over 80% (420,000m^3/year) of the net littoral drift (500,000 m^3/year) to bypass;
- Improve surfing amenity.

The final shape was a relatively complex split V formation, with an original design crest level of -1.0 m AHD (approximately 1.0 m below MSL). With this reef shape and crest level, numerical modelling undertaken by the University of Waikato predicted a salient extending seaward approximately 80 m from the mean shoreline position. This was somewhat confirmed by a physical modelling study undertaken by WRL, however, this was not the objective of the physical modelling (as discussed in Section A.2.4). After review of the modelling, and with consideration of safety aspects, it was decided by Gold Coast City Council (GCCC) and project managers International Coastal Management (ICM) to adopt a lower reef crest level of -1.5 m AHD, with the possibility of crest top up depending on how the structure performed. Construction of the reef commenced in August 1999 with the major phase of reef construction completed in December 2000. Additional work to raise the crest of the reef and trim the shape was undertaken in 2001, 2002, 2004, and 2006.

Due to the extensive design investigations as well as quantitative pre and post-construction monitoring, Narrowneck MPR provides what is probably the best indicator of the ability of empirical, numerical, and physical modelling to accurately predict the outcomes of a given reef design. As such, this case study has been presented in detail.

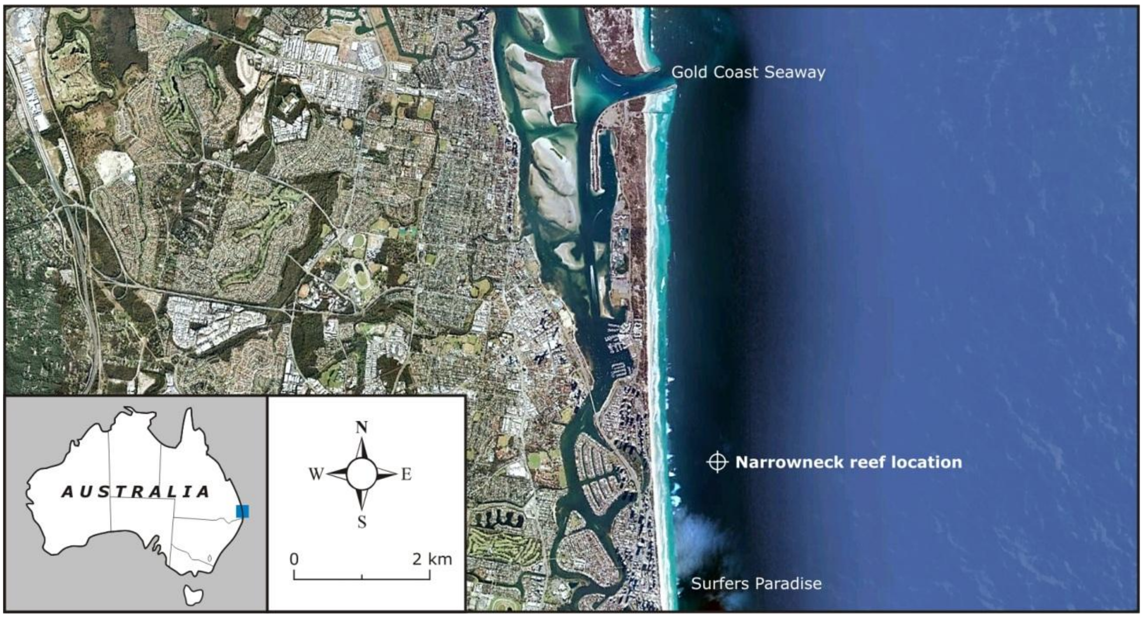

Figure A.3: Location of Narrowneck MPR

A.2.2 Design Investigations Summary

Jackson *et al.* (2007) noted that nine separate study briefs were undertaken by specialist consultants for the reef design, with coordination of studies undertaken by ICM. Studies relevant to engineering aspects of the reef included:

- Physical modelling: wave breaking characteristics and forces at the reef (WRL);
- Numerical modelling: wave breaking characteristics (University of Waikato);
- Numerical modelling: sediment movement and budget (WRL/Griffith University);
- Physical modelling: sediment movement (WRL);
- Numerical modelling: nourishment profiles/quantities and storm erosion (WRL).

It is important to note that these studies were undertaken using a reef design that had the originally proposed crest level of -1.0 m AHD, as opposed to the lower crest level (-1.5 m AHD) that was actually constructed.

A.2.3 Summary of Numerical Modelling Investigations

GENESIS modelling was undertaken by WRL to gain an understanding of the existing sediment processes, including the effect of the nearby Gold Coast Seaway (downdrift of Narrowneck). This modelling was also used to help identify the location of the reef, assess the impacts of the coincident beach nourishment, and to gain an understanding of the effect of wave transmission past the reef on the resulting shoreline planform alignment. This modelling indicated that a wave height reduction of 30% across the reef would be required to move the shoreline 50 m seaward.

The empirical methods of Black and Andrews (2001) were used to estimate the beach response in the lee of the reef, and to estimate the equilibrium beach position. It was determined using this method that the equilibrium beach planform would have a salient extending 80 m seaward of the mean shoreline position. More detailed numerical modelling was also undertaken by University of Waikato (GENIUS, 3DD and POL3DD) to analyse wave conditions and develop the shape of the reef. This modelling is described in Black (1999) and summarised in Ranasinghe *et al.* (2001). The modelling was also used to assess the potential impacts of the reef on shoreline evolution. The models were calibrated using recorded field data and run using idealised monochromatic wave conditions (Jackson *et al.*, 2007). A potential oversight in the original numerical modelling was that the structure (in particular its alongshore length) was parameterised by its base dimension rather than effective dimension.

A.2.4 Summary of Physical Modelling Investigations

Turner *et al.* (2001) presented information regarding the physical modelling study completed by WRL. Two aspects of the reef design were investigated in the physical modelling study:

- Quantitative wave breaking processes and surfing amenity aspects;
- Semi-quantitative sediment transport.

For the wave and tide conditions tested, wave height transmission to the region immediately in the lee of the reef crest was approximately 0.5 (50%), though there was wide scatter in the results. Strong diffraction, particularly around the southern side of the reef, contributed to higher wave height transmission at the shoreline of approximately 0.9 (90%). Based on the GENESIS modelling, it was predicted that this level of wave transmission to the shoreline would generate significantly less than a 50 m salient.

Due to scaling problems inherently associated with physical modelling of sediment transport processes, this aspect of the physical modelling could be considered as semi-quantitative at best. The modelling was undertaken using a hybrid fixed bed and tracer method, where light weight plastic particles were used to track sediment movement in the vicinity of the shoreline. The model was validated in a two stage process that considered existing conditions as well as an idealised detached breakwater case, and was considered to be well validated when compared to empirical predictions.

The purpose of the physical modelling was to confirm the maximum likely salient extent, and that a tombolo would not be created by the reef (which could have resulted in significant down drift erosion). Conditions likely to result in maximum salient growth were tested in the modelling, with the equilibrium beach profile indicating a 30 – 40 m increase in beach width in the reef shadow zone during these idealised conditions (Turner *et al.*, 2001).

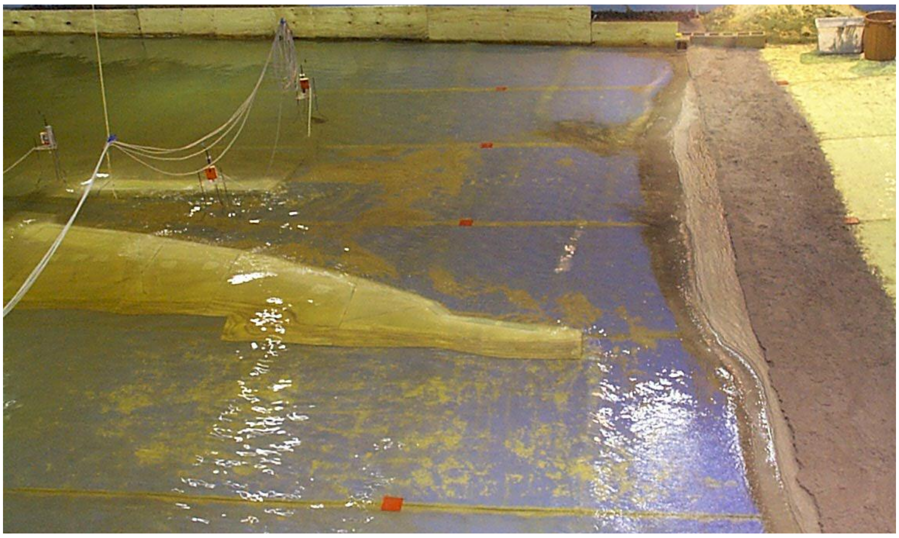

Figure A.4: Equilibrium Shoreline Physical Modelling for Narrowneck MPR at WRL

A.2.5 Construction Materials, Methods, and Maintenance

Construction details of the reef are summarised in the publications of Jackson *et al.* (2002), Jackson *et al.* (2007) and Black (2001). The initial reef was constructed using just over 400 Terrafix geotextile "mega-containers", each measuring 20 m in length, 3 – 4.5 m in diameter (approximately the dimensions of a bus), and weighing 160 – 300 tonnes. The containers were pre-manufactured then filled and placed using a split-hull hopper dredge. Positioning of the individual bags was monitored using GPS. Ongoing "maintenance" of the reef has seen an additional 50 bags placed on the reef between 2001 and 2007, as shown in Figure A.5. It can also be seen in Figure A.6 that during the peak of the construction, of the order of 90 – 100 containers were placed per month.

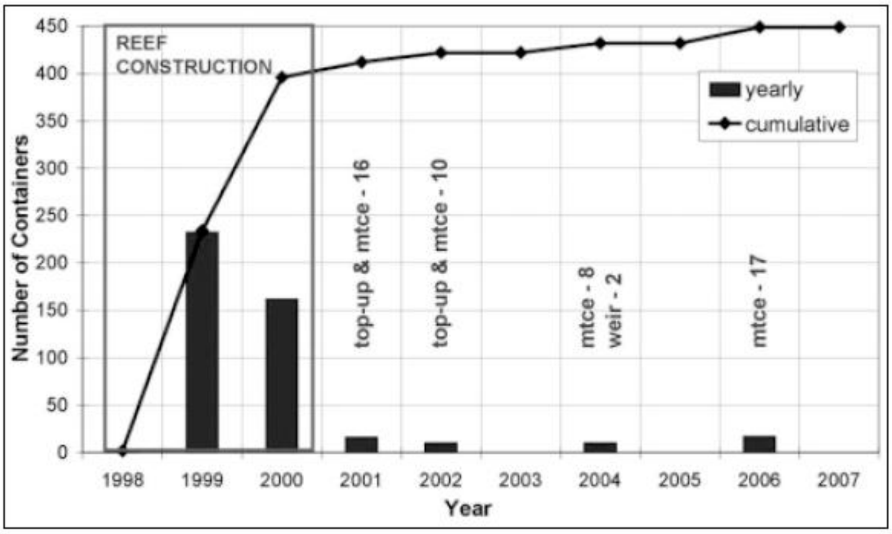

Figure A.5: Container Placement Schedule (Jackson *et al.*, 2007)

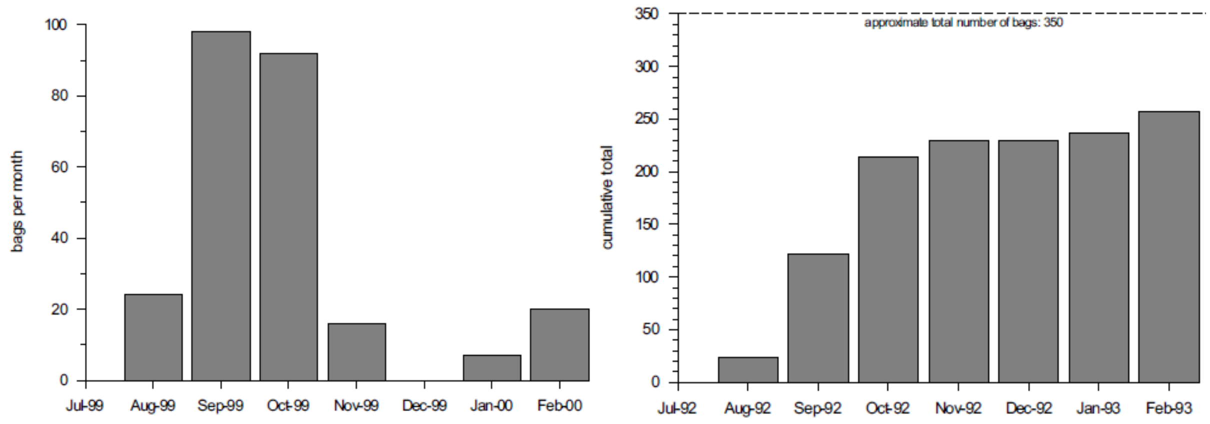

Figure A.6: Peak Container Placement Schedule (Turner *et al.*, 2000)

The containers were reported to be stable in waves up to 8 - 10 m (Black, 2001), however, this would be the maximum (rather than significant) wave height. Jackson *et al.* (2002) reported that at that time, the reef had been subject to nine storm events with wave heights exceeding 5 m, as well other impacts such as boat anchors, spears, knives, surfboards, etc. This experience led to the incorporation of composite geotextiles in the later bags that were placed on the reef to help reduce the impacts of these issues.

A.2.6 Performance and Comparison with Design

The reef has primarily been monitored for coastal protection and surfing amenity aspects, though documentation of other aspects such as ecology and safety is also available in published literature. Monitoring of the reef and beach extending several kilometres north and south was undertaken by WRL for the period August 1999 to July 2008 (nine years). Monitoring was undertaken using an automated ARGUS coastal imaging system which captured hourly snap, time exposure, and variance images of the coast during daylight hours. The images were analysed routinely to extract information regarding changes to the shoreline position and the extent of wave breaking on the reef, with data presented in a series of six monthly reports.

The outcomes of the coastal protection monitoring are discussed in detail due to the relevance to the current study. Monitoring was complicated by the fact that approximately 1.5 million cubic metres of nourishment sand was placed south of the reef prior to construction of the reef. The final beach monitoring report completed by WRL (Blacka *et al.*, 2008), as well as previous six monthly monitoring reports, have been largely used by industry to provide comment on the coastal protection achieved by the reef. It appears that much of the published literature has inadvertently misinterpreted beach width data published by WRL, reporting that the reef has created a salient some 20 to 50 m in cross-shore dimension. In fact this is likely a misinterpretation of the monitoring report data. The figures for beach width shown in the report are relative to an arbitrary dune reference line, and it is clearly pointed out in the reports that this line dips away from the coastline alignment in the vicinity of the reef, making the beach appear wider in this area, compared to beach regions further north and south. This is simply an artefact of the somewhat arbitrary location of the reference line.

Turner (2005) documented the shoreline evolution over a 1000 m study area centred at the reef location, for the periods 2001-2002, then 2001-2003. The ARGUS shoreline data was analysed using an "odd-even function analysis" approach to quantitatively define underlying signals in the planform shoreline alignment. The results of the analysis indicated that there was general beach accretion of the order of 20 m in the vicinity of the reef, likely due to the combined salient and groyne effects.

Blacka *et al.* (2008) documented that on average over the eight year monitoring period since construction, the shoreline was receding at approximately 2.8 m per year south and north of the reef, while in the lee of the reef the recession was more like 4.3 m per year. The shoreline position at the Gold Coast relative to August 2000 is shown in Figure A.7, where it can be seen that by mid-2008 at Narrowneck the shoreline had receded some 15 m further than sections of beach further north or south of the reef (relative to the August 2000 shoreline position). This implies that the general widening observed by Turner (2005), was likely no longer present by 2008. The beach position data also suggests that the extensive nourishment in the vicinity of the reef likely resulted in a "groyne" effect which maintained a wider beach south of the reef for several years longer. However, this was compensated by an increased rate of erosion in more recent years, with the beach position now receded by some 20 m by August 2008, relative to the 2000 beach position.

Figure A.8 shows the monthly shoreline position for years 1999 to 2007, where only a minor planform perturbation in the lee of the reef can be detected by the naked eye (likely of the order of 10 m). While no detailed analysis of the effect of the reef on planform beach alignment has been undertaken since 2003 (Turner, 2005), WRL intend to publish this information in the near future so as to reduce the ambiguity surrounding the coastal protection offered by the reef.

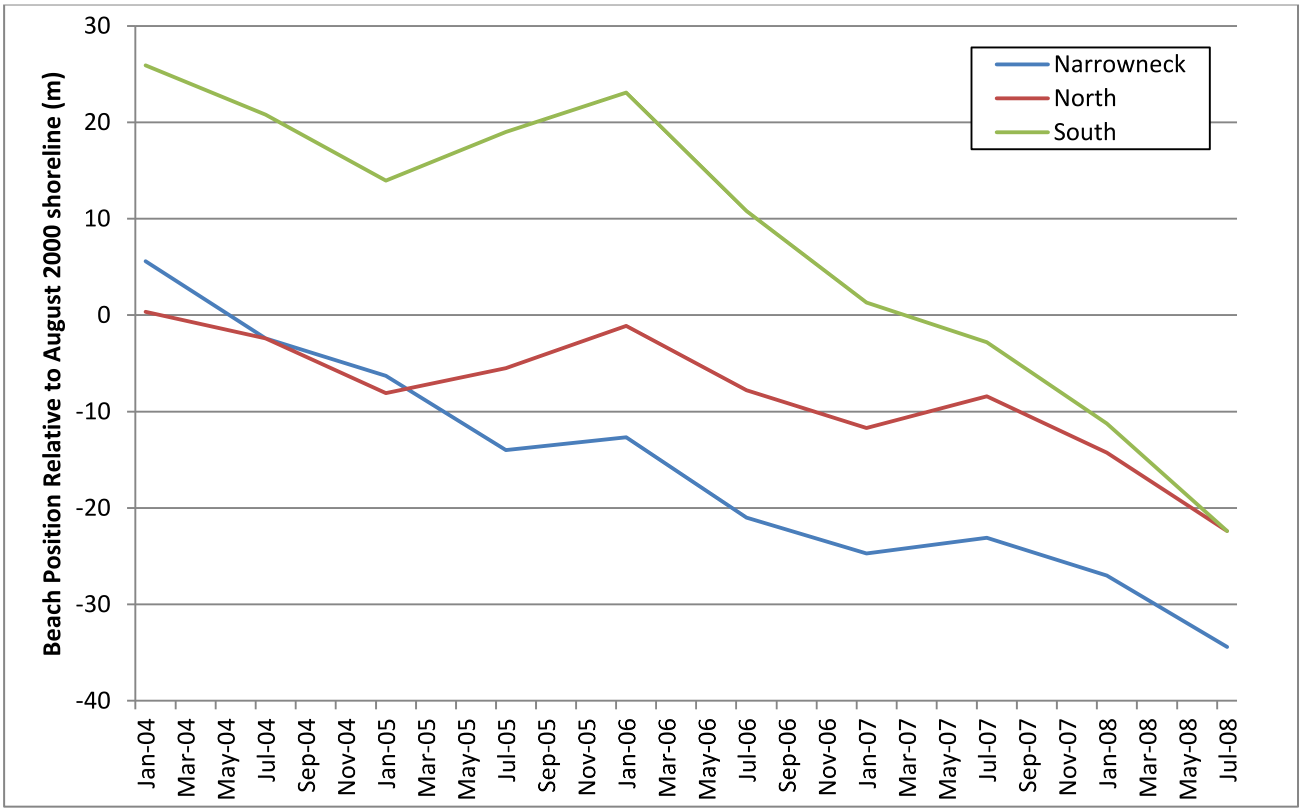

Figure A.7: Beach Position Relative to August 2000

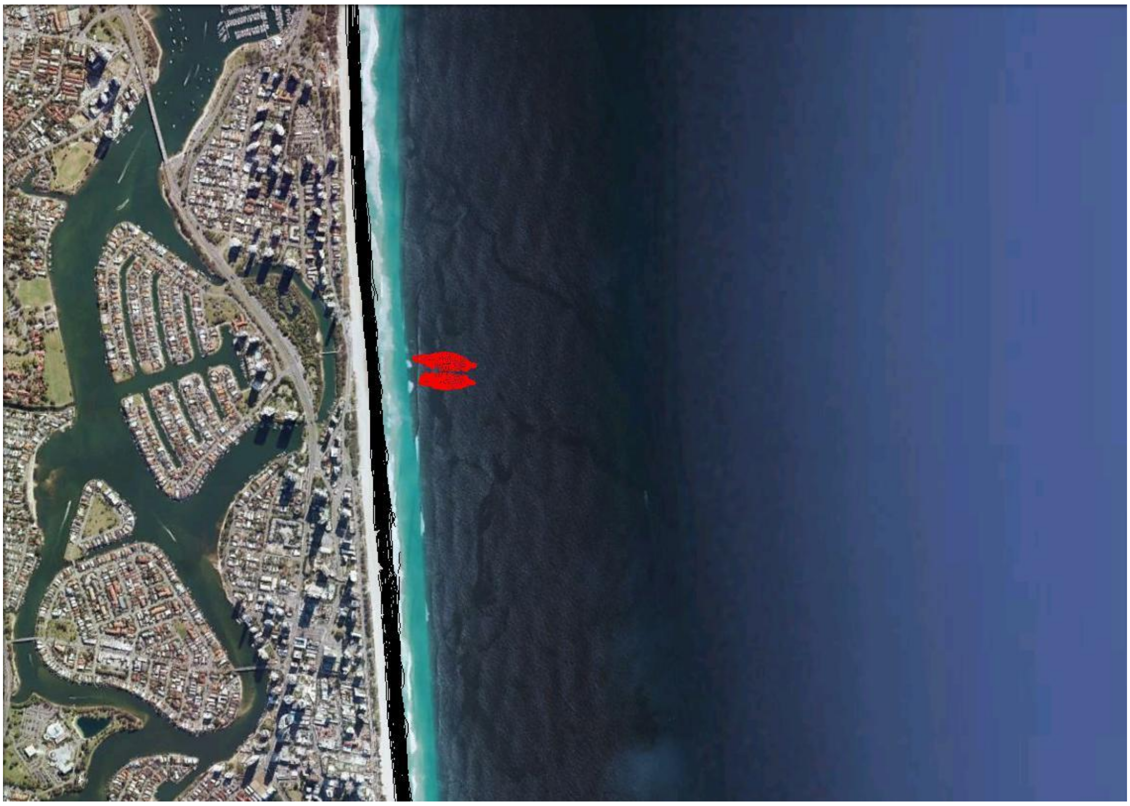

Figure A.8: Monthly Shorelines at Narrowneck, 2000 – 2007 (Black Lines)

Jackson *et al.* (2007) provided a summary as to how the reef has compared with the initial modelling that was undertaken, with the following conclusions drawn:

- The reef needs long period clean swell to reproduce wave conditions shown in the physical and numerical modelling (though modelling was undertaken with monochromatic waves of this nature);
- In reality, actual waves experienced at the Gold Coast are typically shorter period and therefore somewhat unlike the conditions that were modelled;
- Modelling is a useful tool, but only provides information for a limited number of scenarios, and these scenarios need to be carefully selected;
- Modelling results need to be interpreted carefully.

A.2.7 Cost

McGrath *et al.* (1999) summarised economic figures regarding the value of surfing and beach health to the Gold Coast. In particular it was noted that a 25 year ARI erosion event would cost the Gold Coast approximately $305 million (1996 AUD), and that a minor erosion event such as a 5 year ARI could cost approximately $47 million (1996 AUD). The surfing industry on the Gold Coast was valued at approximately $160 million per annum.

Jackson and Corbett (2007) indicated that the total cost of the reef was $2.8 million (including top up and maintenance) which equates to approximately $3.9 million in 2013. Ranasinghe (2001) indicated that the cost was approximately split as two thirds for construction and one third for studies and planning. Clearly the construction costs of the reef are an order of magnitude less than the potential loss in tourism and surfing revenue caused by a series of erosion events.

A.3 Mount MPR, Mount Maunganui, NZ

A.3.1 Project Description

Numerous investigations have been published that detail the design, construction, and performance of the Mount MPR located at Mount Maunganui, New Zealand. Information presented in Mead and Black (1999), Mead *et al.* (2007), Weppe *et al.* (2007), Black and Mead (2007), Black and Mead (2009) and Mead and Borrero (2011) has been used to compile this summary. The Mount MPR was developed as a research project to investigate surfing amenity and beach protection aspects of MPRs. Construction of the reef began in 2005 and was completed in June 2008.

The study site is at Mount Maunganui in the Bay of Plenty on the north-eastern coast of New Zealand (Figure A.9). This section of coast is reported to have a net littoral drift of the order of 60,000 to 80,000 m^3/year (Healy, 1980 in Weppe *et al.,* 2007). However, it is expected that this rate is relatively small in comparison to the gross littoral drift at the site which has variable direction.

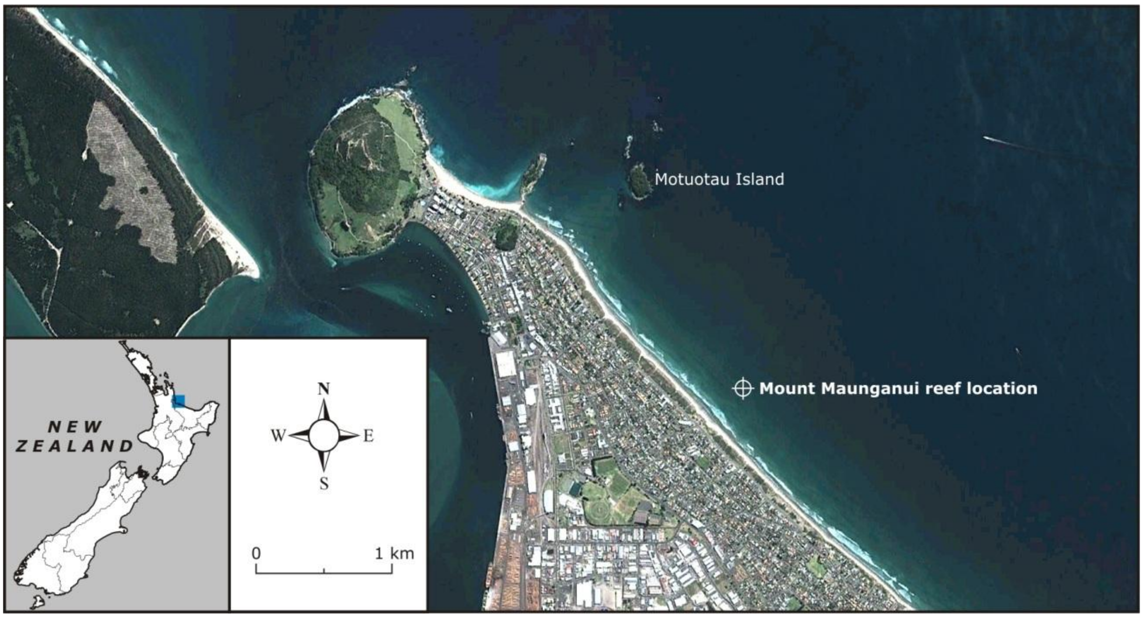

Figure A.9: Location of Mount MPR

In terms of coastal protection assessment, the following data was collected for the project:

- Pre-construction echosound bathymetry;
- RTK-GPS foreshore survey data as shown in Table A.1.

Table A.1: Beach Survey Data for Mount Reef Analysis

Survey Date	Reef Stage (% Complete)
19/08/2004 *	0
28/10/2004	0
26/04/2005	0
08/09/2005	0
17/11/2005*	10
20/04/2006	25
15/08/2006 *	70
23/01/2007 *	70
15/05/2007 *	70
14/03/2009 *	100

* Indicates complete shoreline contour along site

A.3.2 Modelling and Design

Design of the reef was undertaken in various stages, initially for consent approvals and subsequently for detailed design. The final design and impact assessment was developed using numerical (WBEND and 3DD) modelling and was subsequently physical modelled in order to achieve final design adjustments. The physical model testing was used to assess the constructability of the reef design, and also to consider the quality of the waves generated under a range of tide and wave conditions. The final reef design was a symmetrical 'delta' shape with a wave focussing zone (shallower gradient) and a wave breaking zone (steeper gradient), intended to produce a single peaking wave break (Mead *et al.*, 2007).

Black and Mead (2007) used the empirical methods of Black and Andrews (2001) and Ranasinghe *et al.* (2006) to predict the presence and size of a salient for the Mount MPR. The model 2DBeach was subsequently used to simulate the sediment transport processes at the site. This analysis was undertaken following construction and a period of monitoring of the reef, with the intention of analysing the effectiveness of a calibrated numerical model to predict and understand the key morphodynamic processes in the vicinity of reef structures.

A.3.3 Construction

The Mount MPR was constructed in two separate halves using the RAD (Rapid Accurate Deployment) technique. The individual geotextile mega-containers ranged in filled dimensions from 30 m long x 1.5 m wide x 1.0 m high to 50 m long x 7.2 m wide x 3.2 m high, with each half of the reef comprising 12 containers.

A.3.4 Performance

The coastal protection aspects of the Mount Reef have been examined by Weppe *et al.* (2007) using an odd-even function analysis to interpret the available survey data, as was used by Turner (2005) for the Narrowneck reef. The analysis of shoreline change was completed for three separate data sets, firstly the preconstruction survey data (change from August 2004 to November 2005), secondly the pre to post construction data (change from November 2005 to March 2009), and thirdly an alternative pre to post construction data set (change from January 2007 to March 2009).

The pre-construction even function analysis indicated a general trend of accretion south and north of the reef with minor erosion in the proposed reef area during the period considered. The odd function analysis showed an alongshore gradient in beach change, with accretion in the north-west and erosion in the south east. This is complementary with the observed wider beach widths to the north-west, possibly due to sheltering from the offshore islands or localised dredge spoil disposal. The analysis indicated complex alongshore variations in shoreline alignment, with variations in beach width of 20 m – 30 m evident.

Analysis of the shoreline change during the pre to post construction interval showed a total function that indicated fluctuating shoreline change alongshore, varying from 30 m accretion at about 100 m to 200 m north-west of the reef through to regions of slight erosion. There was little net accretion in the region directly in the lee of the reef, indicating that there was very little salient type formation. The odd function analysis indicated a potential secondary groyne effect being generated by the reef, with a significantly steeper beach width gradient spanning the region from 200 m north-west to 200 m south-east of the reef. It is also possible that the structure resulted in other discrete changes to the beach alignment, evidenced by an increasing number of beach width fluctuations alongshore as a result of rhythmic bar/rip formations.

In summary, with regard to coastal protection:

- Any effects of the reef on shoreline position are subtle and within the natural variation of shoreline position;
- No noticeable salient developed;
- There was probably a slight localised groyne effect from the structure.

The numerical modelling of Black and Mead (2007) (calibrated to the post construction bathymetry data), was shown to be able to broadly capture the processes and development of key sand bar features that have been observed in the vicinity of the reef. While the modelling was able to estimate quantitative details for the morphological response (sand bar crest levels and on/offshore position etc.), the level of accuracy was shown to be very sensitive to the modelled environmental conditions (wave conditions and initial bathymetry). The information presented in Black and Mead (2007) provided some confidence that a calibrated morphological model is useful for understanding the processes occurring around a reef; however, also confirms that the complexity of the processes result in quantitative model predictions that remain only reasonable approximations, and that the degree of uncertainty requires quantification if the modelling is to be used for assessment of the coastal protection provided by a reef.

A.4 Dupont Submerged Breakwater, Palm Beach, Florida, USA

A.4.1 Project Description

Stauble and Tabar (2003) summarised the findings of a two year submerged reef/breakwater study between 1988 and 1990 at the town of Dupont, in Palm Beach County, Florida (Figure A.10). The section of coast studied had suffered storm erosion. Existing coastal protection structures including groynes, a seawall, and a revetment were already present and created an irregular shoreline at the site. Details of the submerged reef structure are as follows:

- Constructed from PEP precast modules;
- Placed approximately 50 m offshore;
- Water depth of 2.4 m;
- Crest submergence of approximately 0.6 m below MSL; and
- Alongshore crest length of 160 m.

Three monthly profile surveys were undertaken at the site over a two year period from preconstruction in March 1988 through to March 1990. Seventeen (17) cross shore profiles were monitored at the site, with a longshore spacing of 30 m in the vicinity of the reef and 61 m north and south of the reef.

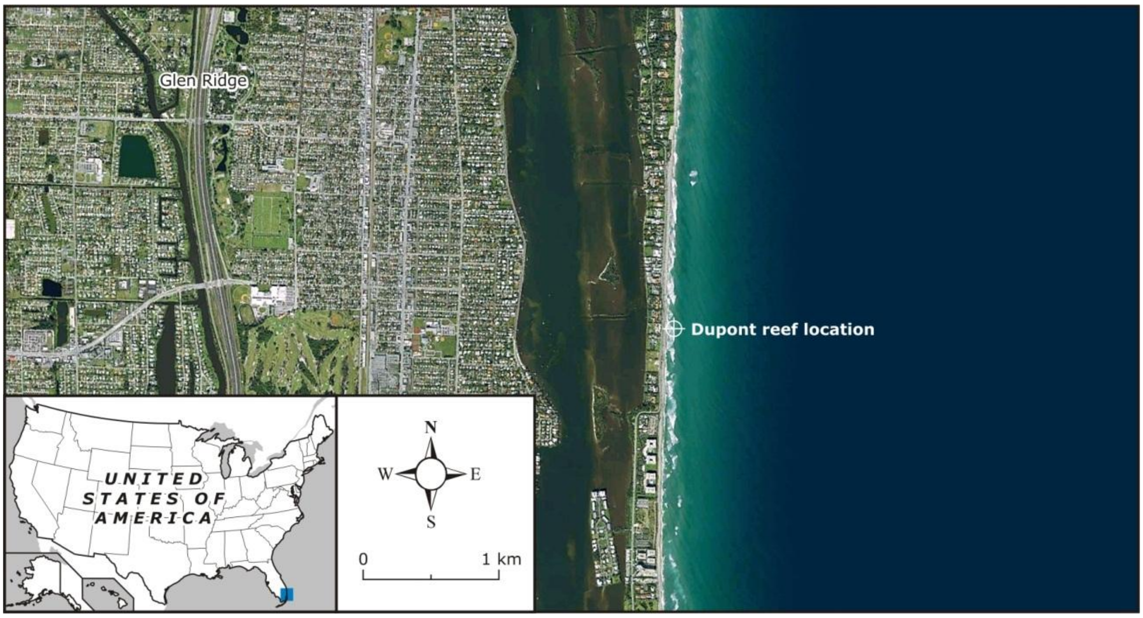

Figure A.10: Location of the Dupont Submerged Breakwater

A.4.2 Performance

In general, the shoreline position at the completion of monitoring was landward of the original shoreline position. In the lee of the reef structure there was little change in shoreline position between the start and end of the monitoring. The volumetric change in the sub-aerial beach was variable and it was difficult to separate background changes in sand movement from the effect of the reef. Furthermore, the existing coastal protection structures generated additional variability in the beach response. It was concluded that there was little benefit for shoreline position generated by the reef.

The reef units were observed to be displaced during a storm midway through the project, and had to be realigned. No further movement of the units was observed during the project.

A.5 Midtown Submerged Breakwater, Palm Beach, Florida, USA

A.5.1 Project and Reef Description

Dean (1996), Martin and Smith (1997), and Browder *et al.* (2000) presented the findings of a four year submerged reef/breakwater trial at the Town of Palm Beach, Florida (Figure A.11). The section of coast investigated was suffering ongoing erosion due to the installation of a trained harbour entrance updrift of the reef location (100,000 m^3/year net littoral drift). The reef was constructed from "Prefabricated Erosion Protection" (PEP) precast modules, and placed approximately 70 m offshore in 3 m water depth. The crest submergence was approximately 1.2 m below MSL, and the reef stretched continuously alongshore for approximately 1260 m. In terms of artificial reef structures, the PEP units have a narrow 0.3 m crest width and width at the base of only 4.6 m. The section of coast was contained by a back beach seawall, and had a narrow 8 m sandy strip seaward of the seawall before the breakwater was constructed (1992). The objectives of the reef were to reduce wave height and to increase beach width.

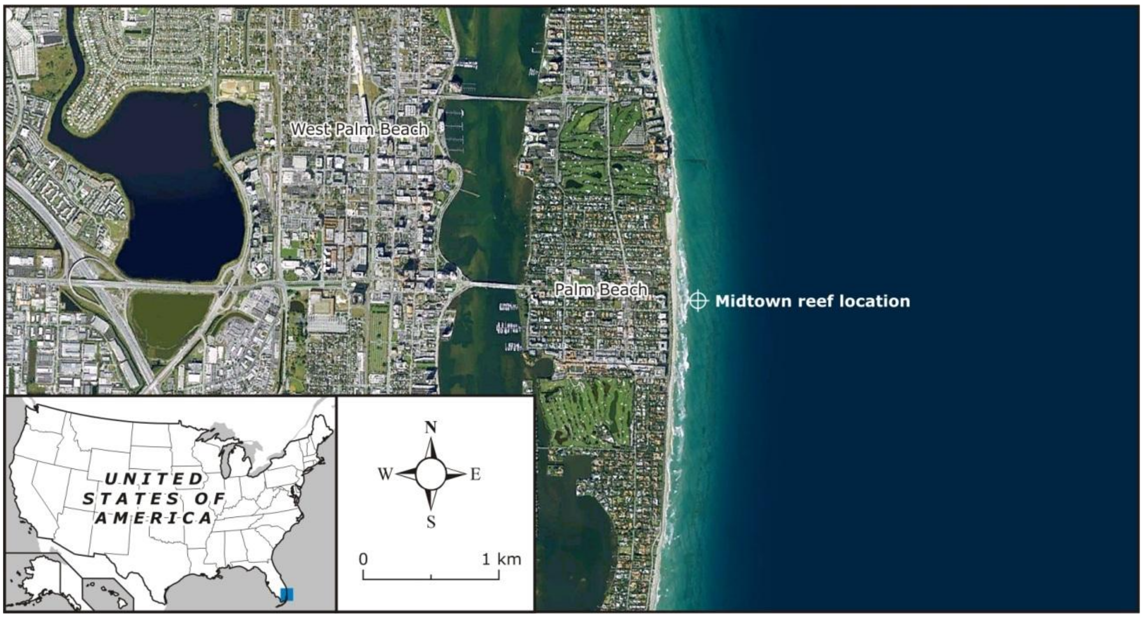

Figure A.11: Location of the Midtown Submerged Breakwater

A.5.2 Performance

An extensive monitoring program, including both wave and morphological measurements, was undertaken during and after construction of the reef. Wave transmission past the reef was measured at 0.85 to 0.95. Following installation of the reef, the shoreline was observed to suffer ongoing erosion, until the beach completely eroded back 8 m to the seawall two years after construction was completed. It was identified that a strong current pattern generated by flow over the reef caused longshore flow to then travel along the shore and return past the ends of the reef. These flow patterns were a key in driving the sediment transport processes that resulted in ongoing erosion of the beach. The transport of water over the reef was seen to cause "ponding" in the lee of the structure which could not return seaward due to the presence of the reef, and a portion of the current velocity was projected alongshore. These alongshore current velocities were identified as being proportional to:

- Relative crest height;
- Crest length;
- Offshore Distance;
- Wave Height.

Over the three year post construction monitoring period, the reef caused an estimated erosion rate of 2.3 times the background erosion. The PEP units were also found to suffer moderate settlement of 0.6 – 0.9 m (varied with construction technique). As a result, the reef units were removed late in 1995, and some reconfigured to produce groynes. The beach was then nourished.

A.6 Vero Beach Submerged Breakwater, Indian River County, Florida, USA

A.6.1 Project and Reef Description

Stauble and Tabar (2003) summarised the findings of a four year submerged reef/breakwater study between 1996 and 2000 at the town of Vero Beach, Florida (Figure A.12). The section of coast at this site is reported as having suffered ongoing erosion, with 95% of the beach at the study site backed by hard coastal protection structures. Ranasinghe and Turner (2006) reported that the littoral drift at the site is approximately 30,000 m^3 per year. The nearshore surfzone has underlying natural reef that runs shore parallel and has a cross-shore width ranging from 300 m to 600 m.

The submerged breakwater at the site was constructed in 1996 in an effort to reduce the ongoing erosion at the site. The structure had the following design details:

- Constructed from PEP precast modules;
- Placed in 11 short segments with crest length varying from 51 m to 96 m, gaps between adjacent segments, and total crest length of 914 m; and
- Breakwater segments alternated with 61 m/76 m offshore distance, 2.1 m/2.7 m water depth, and 0.3 m/0.9 m crest submergence.

The staggered and segmented configuration of the breakwater was established from physical model testing, as well as from the experience gained at the Midtown Palm Beach project, where a similar structure was built with a continuous long crest and resulted in scour inshore of the structure (Section A.5).

A.6.2 Performance

A range of monitoring was undertaken for the project during the four year study period including:

- 40 cross-shore profiles spaced at approximately 60 m and covering the breakwater length, as well as control profiles with spacing of 150 m extending 1500 m each side of the breakwater;
- Profiles were monitored approximately quarterly starting just prior to reef installation;
- Aerial photography;
- Settlement/crest level measurements;
- Scour measurements; and
- Wave measurements.

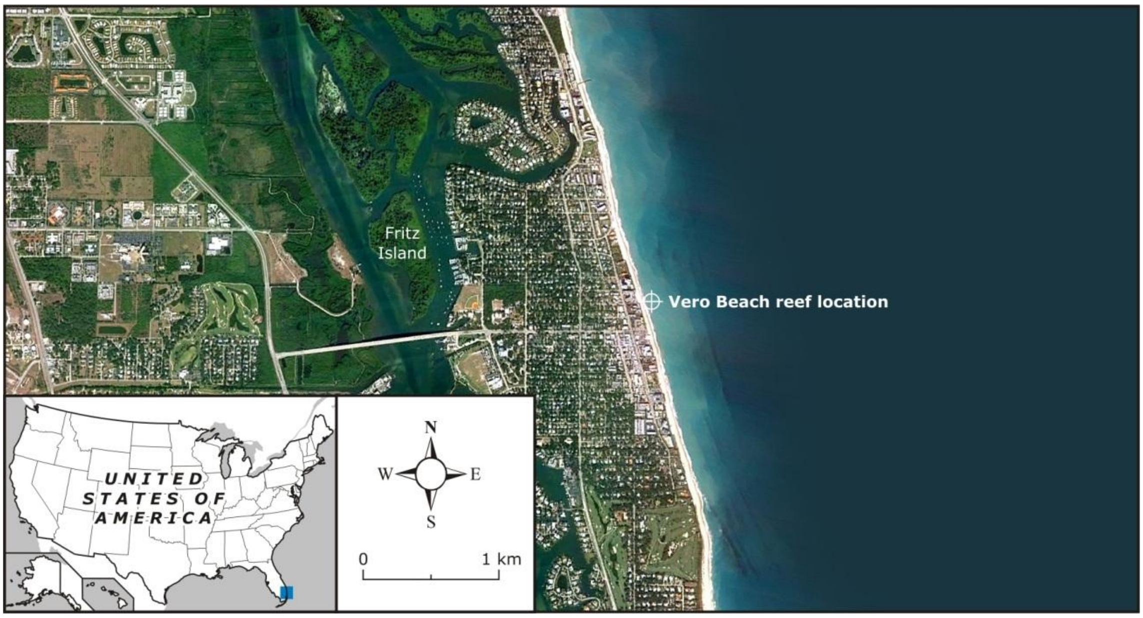

Figure A.12: Location of the Vero Beach Submerged Breakwater

In terms of coastal protection, for the majority of the monitoring period the control area to the north of the reef accreted with a seaward shift in shoreline position of approximately 10 m, while there was varying landward and seaward shoreline migration over the southern control area. The section of beach in the lee of the reef suffered erosion, with the shoreline migrating 3 m to 5 m landward in the two years following installation of the breakwater. Throughout the last year of the monitoring period, the beach at both the northern and southern control zones, as well as the breakwater zone all accreted. At the completion of the four year monitoring period the cumulative shoreline accretion was 9.7 m over the north control zone, 7.6 m over the southern control zone, and 5.3 m in the lee of the breakwater. In terms of overall volumetric morphological response the following results were obtained:

- 11.7 m^3/m of accretion in the lee of the reef;
- 15.3 m^3/m of accretion in the southern control zone; and
- 31.9 m^3/m of accretion in the northern control zone.

Based on the analysis of historical aerial photographs as well as the existing beach processes, it was difficult to separate the effects of the PEP breakwater from the underlying trends. It is thought that the breakwater possibly exacerbated the underlying trend of erosion at that specific area of the beach, however, the effects were minimal and localised.

Wave attenuation measurements were taken by five separate gauges over a two year period. The initial wave reduction across the breakwater was measured at 12%, however, after settlement of the units this dropped to approximately 8%. Measurements of wave attenuation over the southern control area indicated that the natural reef at the site was reducing wave height by up to 8%, and hence it was concluded that the breakwater was doing very little to reduce wave heights, particularly following the settlement of the units. This was attributed to the very narrow crest width of the PEP units.

In general, scour was variable around the units. A scour trench developed on the leeward side of the reef and persisted throughout the study. The scour trench varied from 3 m to 9 m in width and ranged in depth from 1.2 m to 1.8 m. This scour was likely due to wave structure interaction and localised wave driven currents around the units. Scour on the seaward side of the breakwater units was observed post storm, however, this naturally filled in with time.

Initial settlement of approximately 1 m was observed for the breakwater units, except where the units rested on hard bottom. Following this initial settlement the units remained essentially stable for the remainder of the monitoring period.

A.7 Avalon Submerged Breakwater, New Jersey, USA

A.7.1 Project Description

Stauble and Tabar (2003) summarised the findings of a two year submerged reef/breakwater study between 1993 and 1995 at the town of Avalon, New Jersey (Figure A.13). The project was a part of the New Jersey Pilot Reef Project. The section of coast at this site is reported as being naturally dynamic as it is a part of a barrier island system adjacent to Townsends Inlet, and therefore the coastal processes such as entrance currents, sand shoals, and littoral drift play a significant role in the local geomorphology. The area has been further complicate by human intervention including historical sand mining, nourishment, entrance clearing and training, and coastal protection.

The submerged breakwater at the site was constructed in 1993 in an effort to retain significant quantities of sand that had been placed on the beach during several nourishment campaigns throughout the previous five years. The structure had the following design details:

- Constructed from Beachsaver precast concrete modules;
- Placed approximately 100 m - 150 m offshore;
- Water depth of 2.2 m – 3.8 m;
- Crest submergence of minimum 0.4 m below MSL; and
- Alongshore crest length of 305 m.

The reef was located such that at one end it was attached to an existing rubble mound/groyne structure, effectively forming a sand trap.

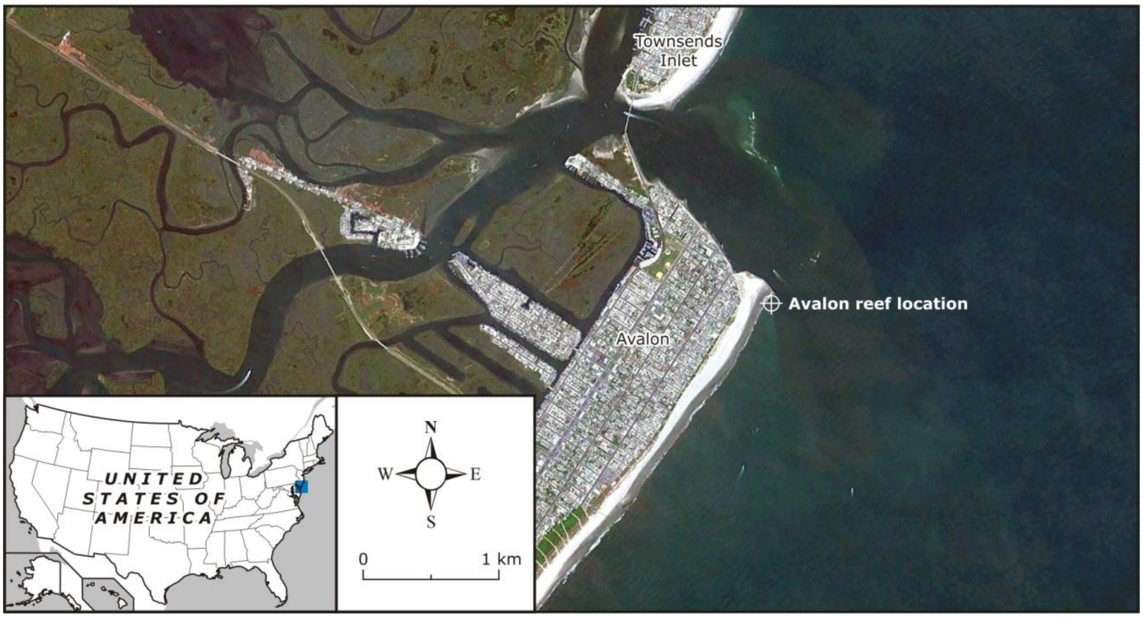

Figure A.13: Location of the Avalon Submerged Breakwater

A.7.2 Construction

The Beachsaver units were placed from barges onto geotextile, and were fixed into place by steel H-piles driven through holes in the units and into the subsurface soil. Midway through the monitoring period additional scour protection in the form a stone filled geo-mattress was placed along the landward side of half of the structure.

A.7.3 Performance

A range of monitoring was undertaken for the project during the two year monitoring period including:

- 27 cross-shore profiles spaced at approximately 30 m and covering an 820 m length of coast;
- Profiles were monitored approximately every four months starting just prior to reef installation;
- Settlement/crest level measurements;
- Scour measurements;
- Wave measurements;
- Hydrodynamics with a dye trace study.

In terms of coastal protection, the combined reef installation with beach nourishment makes it somewhat difficult to draw definitive conclusions regarding the morphological response directly attributable to the reef. In general, the results presented in Stauble and Tabar (2003) suggest that the beach south of the reef eroded back to the pre-nourishment position within one year, while the section of beach leeward of the reef eroded more slowly. Volumetric analysis of the profiles indicated that in the first year following nourishment and reef installation:

- Erosion to the south of the reef was approximately 180 m^3/m;

- Erosion at the southern (open) end of the reef was 156 m^3/m; and
- Erosion at the northern (closed) end of the reef was 100 m^3/m.

Wave attenuation measurements were taken as a part of two separate data collection exercises. The maximum reduction in wave transmission over the reef was recorded as 20%, while it was noted that once the submergence over the reef crest exceeded 80% of the water depth, there was no wave height reduction generated by the reef.

In general, scour was measured in the nearshore area landward of the reef and even more so immediately on the landward side of the reef (over 1 m scour depth). Accretion was observed against the seaward face of the reef along the entire reef length. Settlement of the reef units was measured over a 21 month period and was reported as highly variable. Settlement at the northern end of the reef was of the order of 0.5 m, while at the southern end it was more significant at over 1.5 m.

A.8 Cape May Point Submerged Breakwater Study Number 1, New Jersey, USA

A.8.1 Project Description

Stauble and Tabar (2003) summarised the findings of a two year submerged reef/breakwater study between 1994 and 1996 at Cape May, New Jersey (Figure A.14). The project was a part of the New Jersey Pilot Reef Project. The section of coast at the site was reported as being highly dynamic, as it is adjacent to the entrance of Delaware Bay and subject to strong tidal currents from the Cape May channel. The coast had suffered ongoing erosion due to littoral drift processes, and as a result, a groyne field with nine separate timber and rock groynes had been constructed. The groyne field reduced the littoral drift erosion, however, cross shore erosion was still resulting in a receding coast. The littoral drift at the site is approximately 153,000 m^3/year, with mean wave heights of 0.6 m in summer and 1.2 m in winter, and a mean tidal range of 1.2 m (Curtis *et al.,* 2000).

The project investigated the use of Beachsaver reef units in conjunction with the existing groyne field, in an effort to reduce the cross shore erosion processes. Two of the beach compartments were "capped" with beachsaver reefs joining across the seaward end of the perimeter groynes, effectively creating a 305 m long reef that spanned two of the compartments. The adjacent beach compartment was considered as a "control" and monitored for comparison purposes. The structure had the following design details:

- Constructed from Beachsaver precast concrete modules;
- Placed approximately 50 - 65 m offshore;
- Water depth of 2.1 m – 2.4 m MLW;
- Crest submergence of -0.15 to 0.61 m below MLW; and
- Alongshore crest length of 275 m.

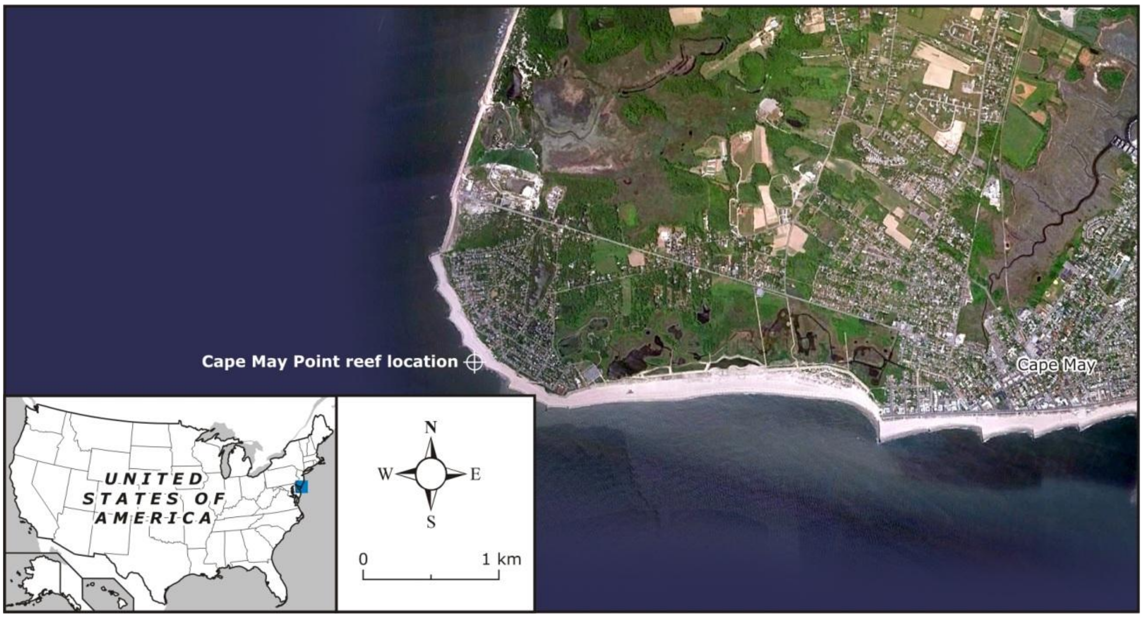

Figure A.14: Cape May Point Research Site

A.8.2 Construction

The individual breakwater units were placed by jack up barge. The breakwater units were connected to the existing groyne structures using a capstone, and steel H-piles were driven through the units into the seabed to help hold the units in place. A double thickness (compared to the Avalon project) geotextile filter was placed under the reef units to reduce scour and settlement. A stone filled geo-mattress was placed along the seaward and leeward edges of the reef units to further reduce scour effects.

A.8.3 Performance

A range of monitoring was undertaken for the project during the two year monitoring period including:

- 12 cross-shore profiles (five in each compartment behind the reef and two in the control compartment);
- Settlement/crest level measurements;
- Scour measurements;
- Wave measurements; and
- Hydrodynamics with a dye trace study and current meters.

In terms of coastal protection, the reef was considered to have stabilised the beach in its lee. During the monitoring period the beach in the lee of the reef was observed to widen by 13 m in one cell and 5 m in the other cell, while the beach width in the control cell was observed to narrow by 5 m. Volumetric analysis of the profiles indicated that during the monitoring period:

- The two cells protected by the reef gained a volume of 2,111 m^3 and 380 m^3; and
- The control cell lost a volume of 660 m^3.

Wave attenuation measurements were limited for this project, and data was only collected during a single day with wave heights less than 0.5 m. Under these conditions the breakwater was observed to reduce wave heights by approximately 10%.

The placement of the Beachsaver units on the filter cloth and geotextile mattress limited settlement of the units to approximately 150 mm. A consistent scour trench formed along the length of the breakwater units on the landward side. The scour trench measured up to 0.6 m in depth during the course of the study, however, eight years after placement the scour trench measured up to 2 m in depth.

A.9 Cape May Point Submerged Breakwater Study Number 2, New Jersey, USA

A.9.1 Project Description

Stauble and Giovannozzi (2004) and Stauble *et al.* (2005) summarised the establishment and findings of a second submerged reef/breakwater study between 2002 and 2003 at Cape May Point, New Jersey. The investigation was a part of the US Army Corps of Engineers Section 227 Project. In this project a further two beach cells at Cape May Point were protected by submerged breakwater structures, with the first being constructed from Beachsaver units and the second from Double-T sill units. Both breakwaters were constructed to completely enclose the beach cells by connecting each end of the reefs to cross-shore groyne structures, with the monitoring program including both of the protected beach cells as well as an adjacent unprotected beach cell.

The Beachsaver reef structure had the following design details:

- Constructed from Beachsaver precast concrete modules;
- Placed approximately 50 - 65 m offshore;
- Water depth of 2.7 m;
- Crest submergence of 0.9 m below MSL, or at MLW; and
- Alongshore crest length of 220 m.

The Double-T sill reef structure had the following design details:

- Constructed from Double-T sill precast modules;
- Placed approximately 50 - 65 m offshore;
- Water depth of 2.7 m;
- Crest submergence of 1.9 m below MSL; and
- Alongshore crest length of 201 m.

A.9.2 Construction and Costs

Both the Beachsaver and Double-T sill breakwater units were connected to the existing groyne structures were built using similar methods:

- Placed by barge mounted crane;
- Beachsaver units placed on geotextile mat while Double-T sill units placed directly on seabed; and
- Rock was used to infill between the ends of the breakwaters and the adjacent groynes.

Placement of the 72 units in the Beachsaver reef was undertaken over 25 working days at a cost of approximately $4,000 per linear metre (equivalent 2013 AUD), while placement of the 22 Double-T sill units took only four working days at a cost of approximately $1,400 per linear metre (equivalent 2013 AUD).

A.9.3 Performance

A range of monitoring was undertaken for the project during the one year reported monitoring period including:

- Four cross-shore profiles in each cell;
- Settlement/crest level measurements;
- Scour measurements;
- Wave measurements; and
- Hydrodynamics with current meters.

In terms of coastal protection, the Beachsaver reef was observed to result in a general widening of the beach and increase in beach volume compared to the adjacent control cell and the Double-T sill reef. Wave height attenuation by the Beachsaver reef was measured to be approximately 12%, while the Double-T sill reef surprisingly resulted in a slightly higher attenuation of approximately 18% though this was postulated to be due to other environmental variables at the site. Despite the use of a geotextile underlayer under the Beachsaver reef units and not the Double-T sill units, settlement in the range of 0.6 m to 1.2 m was observed for both reef structures during the initial 6 months after construction.

A.10 Belmar Submerged Breakwater, New Jersey, USA

A.10.1 Project Description

Stauble and Tabar (2003) summarised the findings of a 1.5 year submerged reef/breakwater study between 1994 and 1995 at Belmar/Spring Lake, New Jersey (Figure A.15). The installation was a part of the New Jersey Pilot Reef Project. The section of coast had suffered long term erosion problems, and as a result was almost completely protected by seawalls as well as several groynes trapping sand from the littoral drift system. The site is absent of tidal currents that were present at the Cape May and Avalon sites, though has a larger wave climate and steeper offshore slope.

The submerged breakwater was constructed in conjunction with beach nourishment, and had the following design details:

- Constructed from Beachsaver precast concrete modules;
- Placed approximately 128 m offshore;
- Water depth of 2.3 m – 2.4 m MLW;
- Crest submergence of 0.7 to 1.16 m below MLW; and
- Alongshore crest length of 335 m.

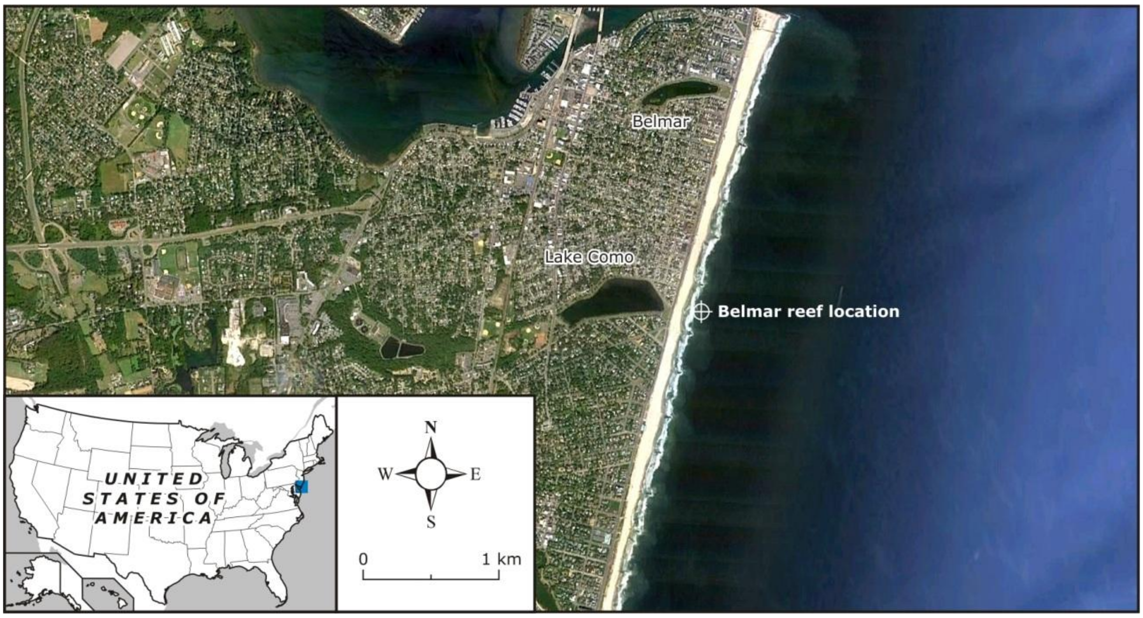

Figure A.15: Location of the Belmar Submerged Breakwater

A.10.2 Construction

The breakwater units were connected to the existing groyne structures using a capstone, and steel H-piles were driven through the units into the seabed to help hold the units in place. A double thickness (compared to the Avalon project) geotextile filter was placed under the reef units to reduce scour and settlement. A stone filled geo-mattress was placed along the seaward and leeward edges of the reef units to further reduce scour effects.

A.10.3 Performance

A range of monitoring was undertaken for the project during the study period including:

- 15 cross-shore profiles covering a longshore distance of 945 m (six profiles in the lee of the reef, four profiles to the north, and five profiles to the south);
- Settlement/crest level measurements; and
- Scour measurements.

In terms of coastal protection, the nourished shoreline in the lee of the breakwater retreated a distance of approximately 15 m during the study, while in the control area the shoreline moved seaward approximately 6 m. The majority of the shoreline recession in the lee of the reef was attributed to sand migrating from the beach face into a detached bar as a result of storm attack. Volumetric analysis indicated that a loss of 3.3 m^3/m occurred in the lee of the reef during the study period.

Analysis of reef crest levels showed that the reef units settled by 0.1 m to 2.0 m during the study. Settlement ceased once the base of the units migrated through the upper 1 m sand layer and reached a subsurface clay layer.

A.11 Prattes ASR, El Segundo, California, USA

A.11.1 Project Description

Prattes ASR was constructed at Santa Monica Bay, El Segundo California and was the first man-made surfing reef in the United States (Figure A.16). The objective of the project was to mitigate the effects on local surfing that had occurred as a result of the construction of the groyne at the El Segundo Refinery and the subsequent backfilling of the adjacent Dockweiler beach (coastal protection was not an objective). The project was mandated by the California Coastal Commission, with the primary goal of enhancing local surfing amenity. Permits for construction of the reef listed the project as experimental and temporary (10 years) with the possibility of extension subject to the success of the project to achieve the goal of improved surfing amenity.

Numerous documents discuss various aspects of the reef including Leidersdorf *et al.* (2011), Borrero and Nelson (2003), and Jackson and Corbett (2007). Information presented in these papers has been used to develop this summary of the project.

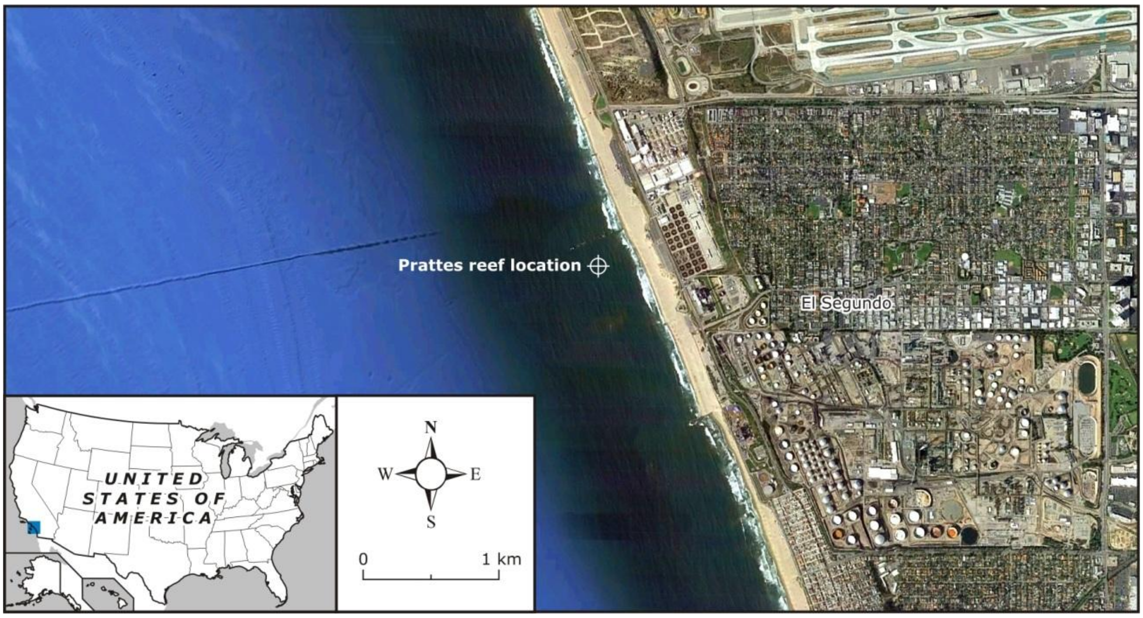

Figure A.16: Location of Prattes MPR, El Segundo, California

A.11.2 Design and Modelling

The reef design was significantly restricted in size by the available budget, with the initial design consisting of:

- 110 geotextile bags;
- Bag dimensions of 3.0 m x 2.1 m x 1.2 m;
- Approximate bag fill of 80% - 90%;
- Bag weight of approximately 12.7 tonnes;
- V-shaped plan layout, with apex of "V" pointing offshore;
- Water depth of 3 m MLLW; and
- Crest submergence of 1.8 m MLLW.

The initial reef design is shown in Figure A.17 and was intended to modify the wave breaking characteristics at the site, such that the waves would be more suitable for surfing. This was to occur through the reef forcing swell to break further offshore and have an increased peel angle.

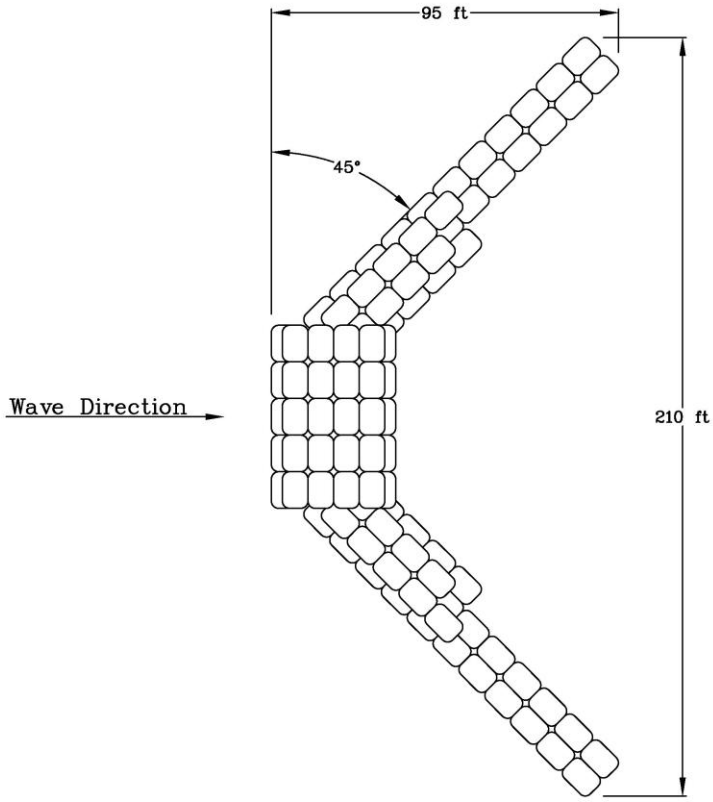

Figure A.17: Design of the Prattes ASR (after Leidersdorf *et al.,* 2011)

Shortly following the completion of the initial reef construction, dive inspections revealed that the seafloor had scoured to a depth of approximately 4 m (at least 1 m of scour depth from initial bed level).

Due to the inadequate size of the reef which was realised almost immediately, a second phase of construction saw an additional 90 bags placed on the reef, increasing the reef volume by some 80%. This increased the height of the structure such that the crest submergence of the apex was only 0.9 m below MLLW, and also increased the footprint of the reef.

A.11.3 Construction

The reef construction was undertaken as follows:

- Woven polypropylene bags with nylon lifting handles used for initial construction;
- Bags filled at Port of Los Angeles and transported to site by barge;
- Bags lifted into place by barge mounted crane; and
- Woven polyester bags used for second stage of construction.

A.11.4 Cost

The budget for the project agreed by Chevron (the owner of the El Segundo Refinery), the California Coastal Commission, and Surfrider Foundation was $300,000 (USD in 2000). This budget was used for the design and construction of the initial reef in 2000. The reef was extensively increased (80% additional geocontainers added) in 2000 with an additional budget of $250,000 (USD), giving a total construction budget of $550,000 (USD).

A.11.5 Performance

An extensive monitoring program was undertaken for the project to identify the impact of the reef on local surfing conditions, shoreline features, and nearshore bathymetry. Only limited monitoring was undertaken prior to construction of the reef, with the majority of the monitoring program occurring post-construction. The monitoring consisted of:

- Monthly beach profile surveys updrift, adjacent, and downdrift of the reef;
- Quarterly ADCP bathymetric surveys of the reef and surrounding seabed;
- Quarterly dive inspections of the reef; and
- Near daily observations of the surf conditions at the reef.

After several years of beach and bathymetry monitoring the reef was reported to have no discernible impact on either the surrounding bathymetry or the beach. With regard to surfing amenity, immediately following the stage 2 construction the reef was reported to have produced near rideable waves that broke only briefly. With the subsequent settlement of the reef over the following months, the reef lost all ability to improve surfing amenity at the site.

With regard to structural performance of the reef:

- In the year following the stage 2 construction, the reef crest dropped by 1.5 m, likely due to a combination of displaced bags, compaction of bag fill, and settlement from scour of the seabed; and
- Progressive deterioration of the bag material.

In general, the reef was considered unsuccessful at achieving any of the targets for the project, and as such, in following through with the initial project permits the reef was removed commencing in 2008.

A.11.6 Removal

Planning for removal of the reef was initially facilitated through two wading and diving inspections of the reef in the first half of 2008. It became apparent during these surveys that the reef and geocontainers were in relatively bad condition, with Leidersdorf *et al.* (2011) reporting:

- Many geocontainers were buried or partially buried;
- Geocontainers were in clusters, with areas of bare seabed in between;
- Torn geotextile fabric and nylon lifting straps indicated damaged geocontainers; and
- Degraded state of the polypropylene bags, such that the material could be torn by hand.

There was no prior experience with removing geocontainers from a submerged reef, however, the experience obtained from removing armouring (in the form of geotextile bags) from the subaerial beach face on man-made islands in the Alaskan Beaufort Sea was utilised. This experience indicated that so long as the geocontainers are located within suitable reach, and that the containers were exposed above the sea floor, removal could be undertaken rapidly using a conventional excavator. The efficiency of this land based extraction technique was noted to decrease dramatically if the geocontainers were beyond the reach of a conventional excavator. Based on this information, land based dredging or extraction was ruled out as an option at the Prattes Reef site due to:

- Likely tearing and spreading of geotextile;
- Disturbance of the sea bottom; and
- Generation of turbidity.

Similarly, the method of cutting the geocontainers and jetting the fill material out was eliminated due to:

- Generation of turbidity; and
- Difficulty of operation in the surf zone.

As a result two alternative methods were proposed for removing the Prattes Reef geocontainers:

- Vessel based removal using deck mounted winch; and
- Shore based removal where a bulldozer and winch were used to drag the containers ashore.

Both methods required a commercial diver to attach the removal cable and slings/nets to the geocontainers for extraction, and to also puncture the geocontainers so that the fill spread in a controlled manner. These methods are illustrated in Figure A.18.

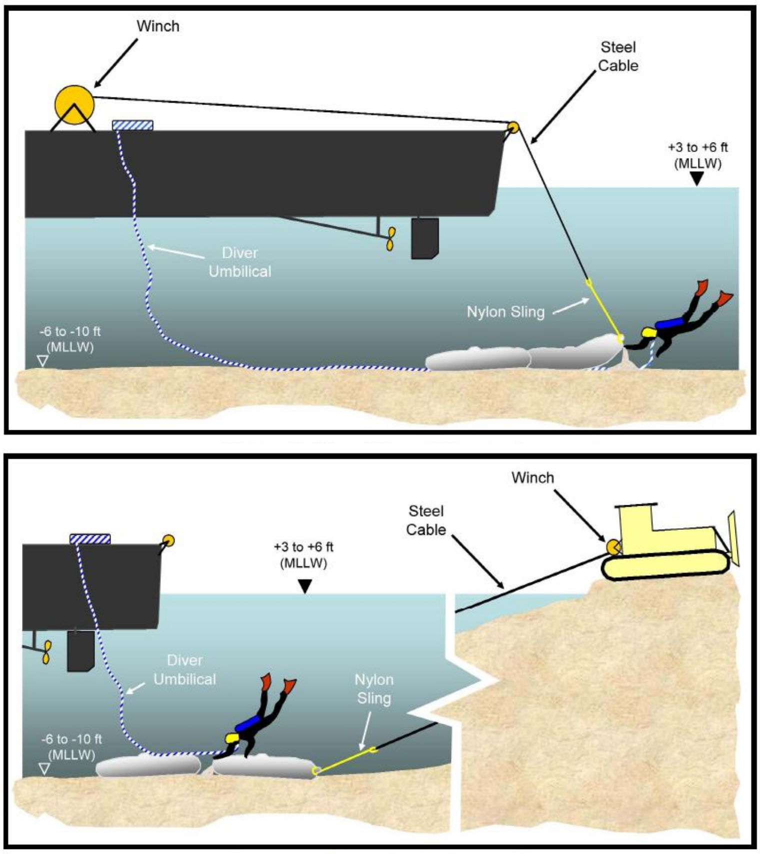

Figure A.18: Removal Methods used for Prattes Reef
(after Liedersdorf *et al.,* 2011)

The budget available for the reef removal was set at $285,000 (USD), and was intended to cover mobilisation, 12 days of on-site operation, as well as 4 days of downtime. Despite being well planned, equipped, and having experienced staff, the extraction process proceeded at a rate of only just over 6.3 bags per day. The vessel based extraction had a higher rate of 7.8 geocontainers per day, in comparison to the land based extraction which averaged only 3.5 geocontainers per day. Following the expenditure of the complete initial extraction budget, a significant quantity of geocontainers remained at the reef site. Two years later additional funding was secured and a second stage of reef extraction was completed. Only the vessel based extraction method was used. However, due to numerous reasons the extraction rate was limited to an average of less than four geocontainers per day. The cost of the second extraction stage was $266,000 (USD).

In summary, at the completion of both stages of the extraction process:

- 142.5 geocontainers were removed (including full and equivalent partial containers);
- 1 geocontainer remained visible in place;
- 42.5 geocontainers originally placed were deemed to have been torn up and washed away or buried; and
- Total extraction cost of $551,000 ($1,000 more than initial construction).