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DESIGN AND ANALYSIS OF THE PERFORMANCE OF AN ARTIFICIAL REEF
TO PROTECT THE SOUTHERN RHODE ISLAND SHORELINE

BY

JENNIFER BRANDES

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
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OF
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ABSTRACT

This study aims at assessing the performance of an artificial reef to protect sandy shorelines from erosion. The approach includes a literature review in form of a summary of the relevant coastal and design parameters and a numerical case study using the numerical morpho-dynamic model XBeach along the southern Rhode Island shoreline to assess the impact of an artificial reef.

The literature review focus on artificial reefs or submerged breakwaters designs and summarizes the results of past laboratory and field works. It aims at assessing (1) the critical processes controlling the shoreline morphological changes associated to the reef and (2) summarizing the parameters used to optimize the reef design. It confirms that an optimal design is site specific, with shape and location depending on local wave climate and geo-morpho-dynamic processes. Based on theory, experiments, past case studies, as well as local test site characteristics, we have sited a test design offshore of Green Hill (Rhode Island). A sensitivity study to shape and location is performed using XBeach.

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LIST OF SYMBOLS

B	Structure crest width	[m]
cf	Drag coefficient	[-]
d	Water depth over the structure	[m]
d_s	Water depth at gap between segments	[m]
E	Mean wave energy density per horizontal area	[J/m ²]
f	Frequency	[s ⁻¹]
f_w	Wave friction coefficient	[-]
g	Gravity	[m/s ²]
H	Wave height	[m]
H_i	Incoming wave heights	[m]
H_r	Reflected wave heights	[m]
H_s	Significant wave height	[m]
h_b	Structure height	[m]
h_e	Water depth seaward of the structure	[m]
I_S	Beach Response Index	[-]
L_b	Length of structure	[m]
L_g	Length of gap between segments	[m]
n	Manning coefficient	[s/m ^{1/3}]
R	Reflection coefficient	[-]
S	Thickness of sand layer	[m]
T	Wave period	[m]
T_p	Peak period	[s]

X	Distance to shoreline	[m]
α	Wave angle	[°]
β	Structure alignment	[°]
ρ	Water density	[kg/m ³]
λ	Wave length	[m]

1. INTRODUCTION

Artificial Reefs (AR) are submerged structures implemented offshore, typically along coastlines. Recently there has been renewed interest from the research community because they are offering a variety of promising functions. They protect the shoreline by reducing the wave energy towards the shore, but also improve the consistency in beach nourishment material, increase the biodiversity, and potentially enhance wave surfability (Voorde et al., 2009). Ahrens and Cox (1990) showed that submerged reefs can dissipate and reflect up to 45% of the incoming energy, however there is considerable variability in published values of energy dissipation.

This study aims at assessing the performance of an AR to protect sandy shorelines from erosion. The study is conducted in two steps. First, a comprehensive literature review is attempted to summarize findings of past AR deployments and studies (field, experimental and numerical) performed to assess the relevant parameters to reef design optimization. In the second step, we assess the performance of promising designs (shape and location) which were chosen based on the literature review, using numerical experiments with the 2-D morpho-dynamic model XBeach (Roelvink et al., 2009). The test site is part of the Rhode Island (RI) southern shoreline, offshore of Charlestown and South Kingstown.

The objective of the numerical approach is to perform a sensitivity study of the design parameters, to allow for optimization of the reef shape, dimension, location, and distance to the shoreline. Resulting sediment transport of the ARs are analyzed, in the limits of the model capabilities. An optimal design minimizes the transmitted energy beyond the reef as well as the beach and dune erosion.

In this manuscript, the approach is limited to one environmental scenario defined by the sea state characteristics associated to the historical storm Irene (August 2011). The study assesses the impact of the reef during similar conditions as that historical storm. The impact of the reef is assessed by comparing changes in wave spectral energy beyond the reef and subaerial eroded volume along the beach, with and without a reef.

This work is a continuation of work previously performed in the Ocean Engineering modeling group at the University of Rhode Island in which nearshore wave data and pre- and post-storm beach profiles were used to calibrate and validate XBeach parameters (Schambach et al., 2018).

2. LITERATURE REVIEW

This literature review aims at finding a consensus in AR design optimization targeting beach erosion as the main factor to minimize. Based on theory and case studies, we provide a summary of the standard design parameters and their formulation.

Section 2.1 aims at presenting a compilation of the environmental factors, as well as induced hazards relevant in an AR design optimization, keeping in mind the mitigation of erosion along the sandy shoreline as primary criteria of optimization. Section 2.2 summarizes the findings of relevant past published studies. The focus is on the design parameters, as location, height, width, length and space between segments. Based on this chapter prototypes for the numerical case study in section 3 were selected.

The literature review shows that there is no universal optimal AR design. The functionality of designs needs to be tested through appropriate methods (numerical simulations controlled with field experiments and in-situ benchmarks) before implementation since each site has its own limitations and specific needs. Current methodologies are mostly based on either simplified theory (regular waves) or specific case studies.

2.1 ENVIRONMENTAL FACTORS AND HAZARDS

The wave climate, generally defined by significant wave height (H_s), peak period (T_p) and wave angle (α) largely controls the sediment transport in the surf zone, providing sedimentological and geomorphological characteristics of the shoreline (Birben et al., 2007). The significant wave height is directly correlated to the wave energy (E):

$$E = \frac{1}{16} \rho g H_s^2$$

When waves reach the breaking depth, breaking induces cross-shore and long-shore currents, resulting in sediment transport. The resulting wave height controls the wave energy and the induced velocity controls the pick-up rate of the sediment.

Introducing an AR with the objective of redirecting wave energy through reflection and diffraction as well as dissipation through friction and breaking, could potentially reduce the wave energy beyond the reef and ultimately reduce the shoreline erosion. However, since breaking creates cross- and long-shore currents, inducing breaking is a delicate choice.

The wave period is known to be a significant factor in sediment transport with long waves acting as a restorative factor (accretion) when associated with small wave heights or destructive factor (erosion), when associated with large wave heights. The wave angle controls the relative intensity of the cross- and longshore currents. Strong longshore currents induce loss of local sediments.

The storm surge acts as an additional factor in sediment transport raising the Mean Sea Level (MSL) and therefore increasing the range of wave action. A long term increase in water level such as Sea Level Rise (SLR) would change the equilibrium profile, resulting in long term or permanent shoreline erosion with a new equilibrium profile reached for a beach when it moves landward (Dean and Dalrymple, 2004). For a steady state wave climate, the equilibrium beach profile depends only on the sediment size, with coarser sediment resulting in an equilibrium profile with a steeper slope (Dean and Dalrymple, 2004).

These environmental parameters are strongly correlated and the resulting balance or equilibrium, once a ‘mitigating’ factor such as an AR, is introduced is very difficult to predict. Many designs have been implemented to protect the shorelines with various degrees of success. Most of these designs are full-length breakwaters (emerged breakwater) (Dally and Pope, 1986).

Rip currents are strong offshore directed flows with velocities up to 8 kilometers per hour (Society, 2011), they can develop around reefs and wave breakers, and within the gaps between reefs (Kennedy et al., 2008). Rip currents are created by the differential set up on the reef and gap due to the breaking. The waves break strongly on the reef and weakly in the deeper adjacent area resulting in a narrow offshore directed current and a wider shoreward return flow connected by feeder currents (Haller et al., 2002). On the next page the circulation patterns around an emerged breakwater are shown (Figure 1), as well as the occurrence of rip currents when it comes to overtopping (Figure 2).

These currents can create a serious hazard for swimmers – in fact over 80% of the 50,000 lifeguard rescues per year in the U.S. are caused by rip currents according to the United States Lifesaving Association lists (Society, 2011). Consequently these currents must be considered to optimize a reef design. While they are inherent to the concept and cannot be prevented, the relative position of the reef to the local topography can significantly affect their patterns (Kennedy et al., 2008).

Figure 1: Circulation pattern around emerged breakwaters (Woodroof, 2012).

Figure 2: Occurrence of a seaward return flow (rip currents) due to waves overtopping the breakwaters (Woodroof, 2012).

Besides these induced currents, breakwaters shelter the shoreward region against the waves, through loss of energy by breaking, friction and reflection. Diffraction occurs around the ends of the breakwaters, propagating the waves inward towards the center of the structure inducing sediment deposits. These accretion areas can eventually reinforce the longshore currents and induce additional downdrift erosion (Dean and Dalrymple, 2004).

2.2 REEF PARAMETERS

In the following section a compilation of approaches found in the literature used to design ARs and submerged breakwaters is presented. Results are summarized in Table 1 on the next page and further explained afterwards. The table includes examples of values from existing designs, which have been proven as successful for the purpose of minimizing wave energy, as well as references providing more details. For each design parameter respectively there are conceptual grounds listed which can be used to design ARs. Based on this review, the designs used in the case study (section 3.4) were chosen

Table 1: AR Parameters, example settings of successful designs and respective relations used to define the designs of this study.

Design Parameter	Successful Designs	Reference	Conceptual ground	Reference
[1] AR Orientation β	Parallel to shore	(Marrone et al., 2019)	Parallel to incoming wave crests (Orthogonal to wave direction)	(Dally and Pope, 1986)
[2] Distance to shore X	240 – 549 m	(Marrone et al., 2019)	In the closure depth	(Ranasinghe and Turner, 2006)
	50 – 600 m	(Ranasinghe and Turner, 2006)		
[3] AR length L_b	80 – 3000 m	(Ranasinghe and Turner, 2006)	$L_b = X * 0.8139$ (min) $L_b = X * 1.5156$ (max)	(Ahrens and Cox, 1990); (Pope and Dean, 1986)
	8 m	(Reguero et al., 2018)	$L_b < X * 1.667$	(McCormick, 1993)
			$L_b = \frac{1.78 - 0.809 * X}{S}$	(Nir, 1982)
[4] Gap between segments L_g	1 m	(Reguero et al., 2018)	$L_g = X * 0.8$	(Seiji et al., 1987)
			$L_g > X - 2 * L_b$	(Birben et al., 2007)
[5] AR height h_b	Sea surface to half of the water depth	(Ranasinghe and Turner, 2006)	$h_b = d * 0.95$	(González et al., 1999)
[6] AR width B	2- 20 m	(Ranasinghe and Turner, 2006)	$B > \lambda * 2.0$	(Narayan et al., 2016)
			$B > \lambda * 3.0$	(Gourlay, 1994)
			$B = \lambda * 0.06$	(González et al., 1999)
			$B = d * 0.75$	(Rambabu and Mani, 2005)

In natural reef environments, wave reduction is mostly influenced by the width of the reef (B), its relative submergence (ratio of water depth over the reef (d) to the water depth seaward of the reef (h_e)), and its relative width (ratio of the reef crest width (B) to the wave length (λ)) (e.g. Narayan et al., 2016). However, most of the design parameters are connected and influence the effectiveness of mitigating coastal erosion.

[1] AR orientation

Most erosion occurs when waves hit the shore orthogonally (Ranasinghe and Turner, 2006), at the same time this orientation minimizes the loss of sediment by longshore currents. The alignment of a submerged structure can influence the angle of the waves. Therefore, the relative wave direction is indeed relevant and the alignment of the AR needs to be chosen for each site individually.

[2] Distance to shore

Earlier studies based on traditional breakwaters related the volume of accretion directly to the distance to shore (e.g. Birben et al., 2007). In general, a smaller distance between the AR and the shoreline leads to more accumulation. Structures deployed seaward of the breaking point do not have a significant impact on the sediment transport along the coast. If they are placed too close to the shoreline, a tombolo, or spit of land between the shore and structure is created. In that case the divergent vortices can even lead to an increased amount of erosion (see Figure 3 and 4).

Figure 3: Nearshore submerged breakwater circulation patterns.

Figure 4: Offshore submerged breakwater circulation patterns.

[3] AR length

In early work, Pope and Dean (1986) related empirically the wave energy reaching segmented breakwaters to the beach response using eight US test sites. These test sites are located mostly in limited fetch wave climate with relatively low wave energy. They proposed a classification of the coastal geomorphological response based on non-dimensional parameters relating the relative breakwater “coverage” of the shoreline (ratio of the breakwater length (L_B) to gap length between segments (L_G)) to the hydrodynamic parameters (ratio of the distance to shore (X) to the average water depth at the breakwater/gap location (d_s)) (Figure 4). The water depth represents a proxy variable for wave energy since it controls the wave breaking height and consequently the amount of energy flowing through the gaps. The wave energy is assumed to be diffracted at the tip of the reef, with diffracted waves interfering at the center of the reef. This classification although largely used, has significant limitations (Thomalla and Vincent, 2004). It is applicable to emerged segmented breakwaters only and it is developed on the base of a very limited data set representing a narrow range of wave climates, reducing its applicability to similar wave climates.

Figure 5: Ratio 'distance to shore/average water depth' over ratio 'breakwater length/gap length' (Pope and Dean, 1986).

(Ahrens and Cox, 1990) developed a Beach Response Index (I_S) based on an adjustment of Pope and Dean's data (1986) using the ratio of the breakwater depth and distance to shoreline versus a discrete variable inversely proportional to the amount of accretion defining the morphology type (1 to 5; with 1 and 2 representing a permanent and intermittent tombolo, respectively, 3 and 4, a well-developed and a subdued salient, and 5, no sinuosity in the accretion pattern), expressed as:

$$I_S = e^{(1.72 - 0.41 \cdot \frac{L_S}{\bar{X}})}$$

This Beach Response Index results, for a segmented breakwater, in a ratio value of $L_s/X = 0.8$ to 1.5 for subdue to well-developed salient, which usually corresponds to the most desirable morphological change. Dally and Pope (1986) proposed a similar adjustment for single breakwaters providing a ratio $L_s/X = 1.5$ to 2 for similar morphological changes. Other authors expanded the data base providing slightly different coefficients for a and b resulting in slight variations in morphological thresholds.

(McCormick, 1993) refined the index noting that the value of the ratio $L_s/X = 1.7$ represents a more realistic threshold between tombolo and salient. (Nir, 1982) developed a linear adjustment to predict the thickness of the shoreward accretion, S (m), using the ratio X/L_s as independent variable, with,

$$S = 1.78 - 0.809 \frac{X}{L_s} .$$

Besides these adjustments based on field studies, other authors addressed the issue of reef design optimization with laboratory experiments developing similar relationships (Suh and Dalrymple, 1987) .

While the above indices and ratios, as structure length/distance to shore (L_s/X), were developed based on data associated to standard (emerged) breakwaters, (Ranasinghe and Turner, 2006) focused their research on submerged breakwaters. They compared the performance of ten submerged breakwaters. According to their review and analysis, only 30% of the projects built on standard breakwater design rules resulted in accretion, most of them (70%) led to erosion. They demonstrated the need for rigorous studies, as state-of-the-art hydro- and morpho-dynamic modeling to fully understand the complex local processes involved in the implementation of an AR (submerged reef/breakwater).

[4] Gap between segments

Multiple breakwaters are often deployed on long shorelines with optimal spacing following a ratio of the breakwater length (L_B) to the gap length (L_G) between 0.75 and 1.25. If the distance to shore is equal to the total length of the structures and the gap between them, the breakwater becomes unable to sufficiently reduce the waves and currents leading to sediment transportation (Birben et al., 2007).

$$\frac{2 \cdot L_s + L_g}{X} > 1$$

In the study of Seiji et al. (1987) cited in (Birben et al., 2007) the relationship between breakwater length (L_g) and distance from shore (X) to the gap erosion was specified. This relationship was evaluated with prototype data. The lower boundary for no erosion (< 0.8) was a good predictor of either accretion or very little erosion. Gap erosion occurred for ratios greater than 0.8.

$$\frac{L_g}{X} < 0.8 \text{ (Accretion or very little erosion)}$$

$$\frac{L_g}{X} > 0.8 \text{ (Gap erosion)}$$

[5] AR height

Gonzalez et al. (1999) theoretically and experimentally explored the concept of perched beaches, seeking to optimize the relative submergence of the reef to maximize the reflection coefficient. They concluded on the necessity to design a reef with a low submergence, which was numerically validated for solitary waves in Grilli et al. (Grilli et al., 1994).

This is illustrated in Figure 3 on the next page.

Figure 6: Reflection coefficient of different submergence of breakwater in respect to breakwater crest width divided by wave length (Gonzalez et al., 1999) and incoming wave height (Grilli et al., 1994).

Gonzales et al. (1999) used a theoretical approach based on the beach equilibrium profile and linear shallow water wave theory to assess the transmission of energy across a submerged reef, and in particular the importance of the wave reflection in energy reduction as a function of the reef crest depth. The problem is simplified to regular waves. The analytical solution for the reflection coefficient, R ($R = H_r/H_i$, with the incoming (H_i) and reflected wave height (H_r)), associated to wave propagation over an assumed impermeable reef (Losada et al., 1992) is shown in Figure 3 (left), with R , plotted as a function of the dimensionless breakwater crest width, B/λ , for different values of the dimensionless water depth, d/h_e , with λ , the wave length, B , the breakwater crest width, d , the water depth over the breakwater, and h_e , the water depth at the seaward side of the reef. Results show that the reflection significantly increases when

the ratio of reef submergence to water depth diminishes. Accepted reasonable values for optimum efficiency are less than 0.5 with optimal values less than 0.1. One can show however, that very small values of the reef crest might create resonant effects strongly impairing the reef efficiency (see Figure 3).

[6] AR width

The simplified analytic approach described above (González et al., 1999) shows that the reflection coefficient R is optimum with value between 0.7 and 0.8 for dimensionless water depth, d/h_e , of the order of 0.1 and a relative width, B/λ , within a range of 0.025 to 0.1 (see Figure 4).

More recent studies considering irregular wave train (e.g. (Narayan et al., 2016)) show that the relative width (B/λ) is most effective when the reef is more than twice as wide (B) as the wave length (λ) and placed in water not deeper than half of the wave length. Indeed, a wide reef induce breaking for a larger section of the wave spectrum and also increase the loss by friction.

There is however an agreement that the dimensionless water depth, d/h_e , is the most important factor under low submergence conditions. The crest width becomes significant under higher submergence conditions (e.g. (Seabrook and Hall, 1999)) as conditions of high surge.

3. CASE STUDY: SITING, IMPACT ASSESSMENT AND REEF DESIGN

Figure 7: Green Hill's shoreline facing East.

Based on the above literature, we defined expected reasonable AR designs to deploy offshore of the southern Rhode Island shoreline, offshore of Green Hill and South Kingstown Beach (see Figure 8). The characteristics of the resulting designs are defined in Table 3 following the correlations of the literature review summarized in the flowchart shown in Figure 10.

The prototypes were tested with 2-D numerical simulations using the state of art morpho-dynamic model XBeach (Roelvink et al., 2009). The main goal of the simulations isto optimize the distance between the AR and the shoreline as well as its dimensions for the specific site.

The impact of the reef to the shoreline was simulated using a unique design storm, the historical storm Irene (2011). Eleven different AR designs, with each being represented by one scenario in XBeach, were tested as summarized in Table 3 of section 3.4.

The impact of the reefs were assessed by comparing the eroded volume along the beach of the test site to the base case (scenario without AR).

3.1 TEST SITE GREEN HILL (RHODE ISLAND)

The site of interest is the sandy shoreline of South Kingstown and Charlestown in southern Rhode Island (USA) (Figure 5). It is part of a beach barrier system with a dune and coastal lagoon, the Ninigret and Greenhill ponds, open to the ocean through the Ninigret pond's inlet. The cartesian numerical grid extends on about 9 km in alongshore direction and 4 km in cross-shore direction.

Figure 8: Study site Green Hill, Rhode Island (left); and limits of the computational domain used in the numerical simulations (right).

3.2 IMPACT ASSESSMENT METHODOLOGY

Numerical simulations are performed using the morpho-dynamic model XBeach (Roelvink et al., 2009). XBeach is a state-of-art 2D-horizontal numerical model, which couples phase-averaged wave and depth-averaged nearshore circulation modules with two morpho-dynamic modules (morphology change and sediment transport), to simulate the natural coastal morphological response to time-varying storm conditions. The model transports and redistributes sand, once eroded and suspended, according to the flow forcing associated with the wave and mean current fields. The bathymetry is modified and updated in real time accordingly.

XBeach is used in surf-beat mode which is adequate to focus on averaged conditions in the swash zone and has been previously validated for the study area (e.g. Schambach et

al., 2018). The same values of the relevant calibration parameters are used for this study as in Schambach et al. (2018). The model and its set of parameter values used in this study were verified by reproducing Hurricane Irene and comparing the computations to beach profile observations of the same event and area as in Schambach et al. (2018). The subaerial eroded volume was measured along 4 field stations (cross-shore transects, Table 4) and predicted for the same locations through the model (Table 5). The values are in good agreement as found in Schambach et al. (2018).

Parallel to this study, the M.S. candidate in OCE, Michael Gardner, has studied the impact of an AR to mitigate coastal erosion using the phase resolving wave model FUNWAVE (Gardner, 2020). Comparison of XBeach and FUNWAVE results are discussed in Gardner (2020) to assess the epistemic uncertainty associated to the choice of the model. XBeach has the strong advantage over FUNWAVE to be computationally very efficient when used in its “surfbeat mode”, with the ability to easily run full 48 hours storms, while such long simulations are computationally prohibitive for FUNWAVE. Consequently, XBeach was used at its full potential for 48 hours simulations for the sensitivity study of the reef design (Simulations 1.1 – 1.10, Table 3); at the opposite, it was used for a limited period of 1 hour for the comparison with FUNWAVE (Simulations 2.1 – 2.3, Table 3).

3.3 MODEL SET UP

The simulation of Hurricane Irene using the current bathymetry (no reef) is considered as our base case (see Figure 7). Additional scenarios are performed to include the reef offshore the study area.

Figure 9: XBeach computational domain in UTM coordinates (East, North) (Zone 19N; grid origin (SE corner) is at East = 284,000 m and North = 4,580,000 m. Color scale is bathymetry (< 0 m) and topography (> 0 m) relative to NAVD88.

Computational grid

The numerical grid shown in Figure 9 is a cartesian grid with a resolution of 4 to 20 m in the alongshore direction and 2 to 20 m in the cross-shore direction, varying from the shoreline towards offshore respectively and resulting in a computational grid of about 1030 by 1264 grid cells. The bathymetry is interpolated from a high resolution (10 m) Digital Elevation Model (RIGIS, 2013).

Boundary conditions and model parameters

Model boundary conditions are specified with time series of storm surge and wave spectral parameters (Significant wave height H_s and Peak period T_p) for the selected event as specified in Schambach et al. (2018).

Time series of wave heights and peak periods used in boundary conditions are extracted from Torres's large scale simulations using SWAN coupled with ADCIRC (Torres et al., 2019). Surge and tide were extracted at the wave buoy (Lon., Lat.): NOAA-44097 ($-71.127, 40.999$) (NOAA, 2020). Simulations were performed for 48 hours of the event as shown in figure 10 and 11.

Figure 10: Significant wave height [m], Peak period [s] and Surge (including tide) [m] over time [h] of Hurricane Irene from August 21st 12:00 am to 30th 11:00 pm (216 hours). Red box shows the 48 hours of the storm when it hit the RI shoreline.

Figure 11: Significant wave height [m], Peak period and Surge (including tide) [m] of Hurricane Irene from 28th 12:00 am to 29th 11:00 pm (48 hours). Used as boundary conditions along the offshore boundaries of XBeach's computational grid.

Table 2: Parameters in JONSWAP wave spectrum file (XBeach manual) and average values of used data.

Parameter in XBeach	Description	1. Scenario (Irene)	2. Scenario (10-year synthetic storm)
Hm0	Hm0 of the wave spectrum, significant wave height [m]	See figure 7.	See figure 8.
Fp	Peak frequency of the wave spectrum [s-1]	$f = \frac{1}{T}$ See figure 7.	$f = \frac{1}{T}$ See figure 8.
gammajsp	Peak enhancement factor in the JONSWAP expression [-]	3.3	3.3
s	Directional spreading coefficient, law [-]	20.0	20.0
mainang	Main wave angle	270.0	270.0
fnyq	Highest frequency used to create JONSWAP spectrum [s-1]	1.0	1.0

In the 48 hour simulations one tidal signal, shown in figure 6, was imposed in all four corners of the domain and then spatially interpolated along the boundaries. The 1 hour simulations of section 4.3 does not include the tide but a constant storm surge 0.6 m relative to NAVD88.

The median grain size ($D50$) chosen for the study area is 0.00044 m, the $D90$ grain diameter is 0.00072 m based on field measurements conducted in 2018 (URI Beach Profile Survey unpublished data). The *facua* parameter was set to 0.3 (Schambach et al., 2018). The parameter *morfac* allows to decouple the hydrodynamical and morphological time and was set to 10 as in Schambach et al., (2018) .

The reef was assumed to be a hard non-porous structure. In the 2-dimensional XBeach domain, the area of the reef was set as a non-erodible layer. The friction is parametrized with a manning coefficient, reflecting the land use; most of the domain was set to the standard friction for sand, 0.02, except the reef area initialized to a value of 0.08 as suggested by (van Dongeren et al., 2013) for natural reefs. The relation between the manning coefficient (n) and the effective drag coefficient (cf) used in the flow module is,

$$cf = \frac{g * n^2}{d^{\frac{1}{3}}}$$

With d the water depth and g the gravity.

The wave friction coefficient (f_w) is by default 0.1, at the location of the reef it was set to a value of 0.6 as similarly recommended by (Quataert et al., 2015) for natural reef environments.

3.4 REEF DESIGNS

The primary aim in implementing an AR is to reduce the beach and dune erosion along the shoreline of the study area without negatively impacting adjacent shorelines. Most of the erosion occurs when the waves hit the shore in an orthogonal trajectory relative to the shoreline (Ranasinghe and Turner, 2006). As mentioned in Section 2.2 the alignment of a submerged structure can influence the angle of the waves. The relative wave direction of the waves is indeed relevant. However, in the Block Island sound most of the waves are coming from the S-SE sector (WIS station 79); when entering shallow water, they refract and reach the potential site for a reef relatively parallel to the shoreline. For this case study, an alignment parallel to the shoreline was chosen.

While the grid cell size limited the options in defining the shape of the reef, the AR implemented in the simulations have a simple trapezoid shape easily represented in our cartesian grid. The reef was represented in XBeach by altering the current bathymetry, setting the sea floor elevation to the reef crest elevation at the reef site, for the selected shape defined in Table 3. The reef slope was created by simple linear interpolation from the adjacent grid cells. Let's note that this shape is a crude approximation and a realistic slope should be implemented in further study.

The reef designs were defined for a range of distances to the shoreline, with the reef's lengths defined following (Ahrens and Cox, 1990)'s formulation, and the gap lengths between reef segments prescribed following (Seiji et al., 1987)'s formulation. The distance to shoreline is a simple function of the water depth and the equilibrium beach profile slope, easily estimated from the sediment size. In this study, a range of distances are arbitrary selected from the offshore limit of the surf zone to the closure depth .

Reef heights and crest widths were chosen based on their respective water depth following (González et al., 1999) and (Rambabu and Mani, 2005). Figure 12 summarizes these relationships and the resulting reef design prototypes are shown in Table 3.

Figure 12: Reef parameter design flow.

Table 3: Characteristics of AR Designs included in the study.

AR design/ Simulation ID	Distance to shoreline X (m)	Average water depth along AR d (m)	AR length L_b (m)	Gap between AR L_g (m)	AR crest width B (m)	AR height h_b (m)	Number of segments in AR
DEEP WATER REEFS							
1.1	Base case - Simulation without AR (48 hours)						0
1.2	500	-10	750	350	7.5	9.5	9
1.3	700	-11	1050	490	8.3	10.5	6
1.4	900	-12	1350	630	8.9	11.3	5
1.5	1100	-13	1650	770	9.4	12.0	4
1.6	1300	-13	1950	910	10.0	12.6	4
1.7	1500	-14	2250	1050	10.5	13.3	3
1.8	1700	-14	2550	1190	10.8	13.7	3
1.9	1900	-15	2850	1330	11.3	14.3	3
1.10	2100	-16	3150	1470	11.7	14.8	2
SURF ZONE REEFS							
2.1	Simulation without AR (1 hour)						0
2.2	220	- 6	200	-	1	5	1
2.3	220	- 6	200	-	7	5	1

4. RESULTS

For the validation of the model the same transects as in (Schambach et al., 2018) were used. The locations of the transects are shown in Figure 13 and Table 4.

Figure 13: Location of transects T1 to T4 in XBeach computational domain in UTM coordinates (X;Y) (Zone 19N), where beach profile measurements were made 3 days before and 3 days after Hurricane Irene's peak arrived on August 28, 2011, and FEMA's transects F1 and F2 (red triangles) (i.e., FEMA's transects 19 and 20 for Washington County, RI (FEMA, 2012)). Color scale is bathymetry (<0) and topography (>0) (m relative to NAVD88) (Schambach et al., 2018)).

Table 4: Coordinates in UTM of the transects showed in Figure 12

Transect	Starting point (UTM)	Ending point (UTM)
T1	275,328.2; 4,580,506.6	275,274.3; 4,580,598.4
T2	277,629.9; 4,581,273.4	277,622.2; 4,581,371.7
T3	280,325.8; 4,582,167.1	280,273.4; 4,582,257.5
T4	281,946.4; 4,582,469.6	281,949.4; 4,582,566.5

To validate the model used in this study the sub-aerial eroded volume along 4 different transects was computed and compared to the observed values and the values computed previously by (Schambach et al., 2018) using XBeach. The results are shown in Table 5. Figure 14 shows the bed level along transect 1 to 4 of the first time step (blue) and the last time step (red) of the 48 hour Base Case simulation.

Table 5: Comparison of computed sub-aerial eroded volume V (m^3/m above NAVD88 datum) along transects T1-T4 by XBeach to observed values from field data.

Transect	Observation	Schambach (2017)	Base Case (1.1) of this study
T1	19.7	28.3	35.9
T2	27.1	29.0	30.3
T3	34.3	33.6	32.0
T4	19.5	15.6	30.2

Figure 14: Transect 1,2,3 and 4, first time step (blue) and (96th) last time step (red) after 48 hours of XBeach simulation.

4.1 SENSITIVITY OF EROSION AND ACCRETION TO THE REEF DESIGN

None of the AR designs deployed in deep water (Simulation 1.2 – 1.10) were able to result in a significant reduction of erosion during the selected storm event; actually most of the cases increased the erosion. These results could either mean that the chosen designs are not suitable for the study area or that the model doesn't represent a realistic picture of the wave energy reduction. This could be caused by the small reef widths being inefficient to induce breaking of the long waves simulated in the surfbeat mode of XBeach. Since the phase of short waves is not simulated, the designs might induce more friction and breaking than the results of the model represent.

However, the results are in agreement with the theory that submerged reefs induce sheer vortices, enhancing accretion when deployed close to the surfzone, or enhancing erosion when deployed further offshore, in deeper water (Woodroof, 2012). A detailed analysis of the associated hydrodynamic processes would be desirable in future work.

Table 6: Erosion along transects T1 to T4 in m³/m.

AR Design/ Simulation ID	T1 [m ³ /m]	T2 [m ³ /m]	T3 [m ³ /m]	T4 [m ³ /m]	Average value of transects [m ³ /m]
1.1 Base Case (without AR)	35.9	30.3	32.0	30.2	32.1
1.2	37.1	33.8	35.5	33.8	35.1
1.3	37.7	39.5	41.6	39.5	39.6
1.4	35.7	32.9	34.6	32.9	34.0
1.5	35.5	31.4	33.1	31.4	32.9
1.6	42.0	36.1	37.9	36.1	38.0
1.7	40.7	29.5	31.1	29.5	32.7
1.8	34.4	27.3	28.9	27.3	29.5
1.9	36.4	36.8	38.7	36.8	37.2
1.10	35.4	39.2	41.3	39.2	38.8

Table 7: Calculated Erosion (negative bed level change) and Accretion (positive bed level change) in m³/m with XBeach for Simulation 2.1,2.2 and 2.3 (designs specified in Table 3).

Simulation ID	Bed level change above 0 (m³/m)	Bed level change above – 2 m (m³/m)
2.1	0.127	-0.02
2.2	-0.04	-0.05
2.3	0.03	0.07

On the other hand, an AR deployed closer to the shoreline, behind the surfzone shows a reduction in erosion (Table 7). These simulations (ID=2; Table 3) are however restricted to 1 h of simulation at the peak of the storm and are not comparable with the previous 48 hours simulations with designs placed in deep water. They were restricted to 1 hour to be compared with FUNWAVE simulations performed in parallel by another M.S candidate (Gardner, 2020).

The AR Designs of simulation 2.2 and 2.3 only differ in the reef width. Location and other parameter are identical. Thus, the difference in bed level change between these two cases must be caused by a difference in friction, illustrating the ability of the reef to limit the wave energy through friction.

5. CONCLUSION

Although there are many different studies concluding on best reef design practices, reef design is very site specific and previous designs can not become more than inspirations or references rather than hard rules to follow. The literature review proved that AR or submerged breakwater design is not well understood at this point (Ranasinghe and Turner, 2006) and illustrates the need of appropriate methods that can test designs site specifically, like XBeach. Section 4.2 has also shown that XBeach is applicable for short time simulation and confirmed its ability as a validation tool for other models.

Most of the designs tested and assessed didn't have a positive impact on the shoreline response of the extreme event Irene (2011) compared to the base case without an AR. From the 9 different AR designs of the 48 hour simulations (Cases 1.2 - 1.10), only case 1.8 decreased the average erosion along the transects compared to the base case without a reef. The short simulations (cases 2.1, 2.2, 2.3) of section 4.2 proved that a wider AR can increase accretion towards the shoreline. It indicates that the reef width could have a considerable influence on the shoreline response.

One can assume that negative coastal protection results could be due to the short reef widths, proving too short for the long waves simulated in XBeach. In a real life scenario or using a model that represents short waves, these reef designs would likely induce more friction and breaking on the short waves, better protecting the coastline. For future work, it is of interest to further investigate. This could potentially be performed through XBeach's non-hydrostatic mode, which covers all processes including short waves.

APPENDICES

Table 8: Used parameter values of the main input file in XBeach (params.txt)

Grid parameters	
depfile	bed.dep
dtheta	90
posdwn	-1
nx	2108
ny	2880
alfa	0
vardx	1
xfile	x.grd
yfile	y.grd
xori	0
yori	0
thetamin	225
thetamax	315
thetanaut	1
tidelen	97
Initial conditions	
zs0	0.6
zs0file	tide.txt
Model time	
tstop	174600
Wave boundary condition parameters	
instat	jons
Wave-spectrum boundary condition parameters	
bcfile	filelist.txt (file containing the wave boundary conditions)
Flow parameters	
bedfriction	manning
bedfricfile	bedfricfile.txt
Morphology	
facua	0.3
morfac	10
Sediment Options	
D50	0.00044
D90	0.00072
Output variables	
tstart	0
tintg	3600
outputformat	netcdf
nglobalvar	4

H	<i>Hrms wave height based on instantaneous wave energy</i>
Qb	<i>Fraction breaking waves</i>
zb	<i>Bed level</i>
zs	<i>Water level</i>

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