DELFT UNIVERSITY OF TECHNOLOGY

MULTIDISCIPLINARY PROJECT

CIE4061-09

Modular Hybrid Coastal Protection Structure

Pilot site: Montego Bay, Jamaica

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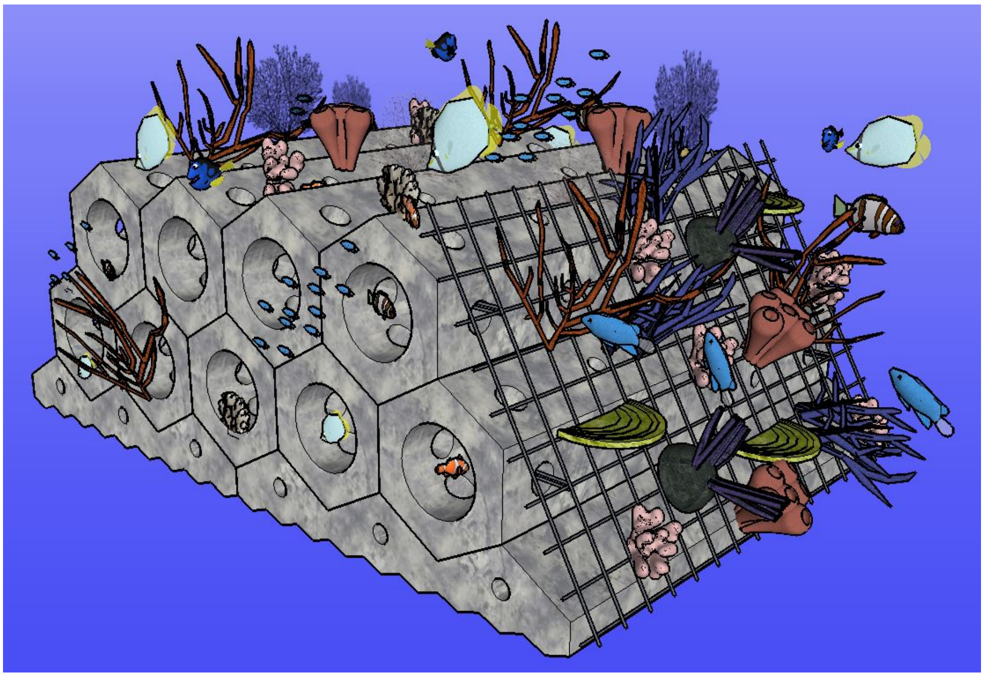


June 5, 2018

Preface

Starting from the 15th of February, a group of four students from the Delft University of Technology began the project, together with the company in Kingston named Smith Warner International Ltd. (SWIL). This report is the result of eight weeks of research and practical work.

The co-operation between Smith Warner International Ltd. and the students found its origination from the course *Multidisciplinary Project (CIE4061-09)* . This course is an elective in the MSc. curriculum at the faculty of Civil Engineering. The goal of this course is to tackle an actual multidisciplinary engineering problem with a team.

The project arose from the fact that Smith Warner International Ltd. and other stakeholders are looking for a sophisticated solution against beach erosion, while at the same time enhancing marine life. The pilot location of the project is called the Hip-Strip and is located in Montego Bay, Jamaica.

For the stay in Kingston, Jamaica, our team co-operated with several companies. The team wants to show their gratitude towards the following partners:

- **Smith Warner International Ltd.**
- **Delft University of Technology**
- **IV-Groep b.v.**
- **Deltares**
- **Koninklijke Boskalis Westminster NV**

We would like to thank David, Philip and Jamel for the opportunity and guidance through the project. Furthermore, our gratitude goes to the team of Smith Warner International Ltd., consisting of Chris, Graham, Elisabeth, Danielle, Renée, Cabral, Ashley, Roberto, Condrae and Dyonne for accepting us as part of their team and their support and expertise during different phases of the project. Special thanks to Cabral for the daily pickups to work.

Apart of the company members, also the following persons helped us during our pleasant stay in Jamaica. Special gratitude for our hosts Maureen and Keri-Ann, and personal taxidriver Mr. Brown.

Our last acknowledgements goes to Henk-Jan Verhagen for raising the opportunity for this project and to our supervisors Jeremy Bricker and René Braam for their time and insights.

Kingston, 6th of April 2018.

Summary

Tourism is one of the main sources of income of Jamaica. As most of them are beach tourist, protecting them is of great importance. However, at the moment the beaches are retreating. An example is the erosion of Hellshire Beach, showing a retreat of ten meters in only seven years. To preserve the beaches effectively, a new concept is requested. The main requirements of the system are wave attenuation and the marine life enhancement.

The literature study showed, a variety of coastal management techniques exist. However, none of those solutions are capable of attenuating waves and enhancing marine life in an effective way. Ranking criteria are given and the following concept groups are generated: boulders, gabions, marine blocks, big (open) blocks and 'lego' (interlocking) blocks. The Multi-Criteria Analysis shows that the big (open) blocks are the most viable and two concepts are designed within this concept group: a triangular and a hexagonal block structure.

In the hydrodynamic and wave models (Delft3D), a study is performed to find the relation between breakwater dimensions and wave attenuation. Also, three different conditions are modelled: daily conditions, hurricane conditions, and one-year storm conditions at the Hip-Strip in Montego Bay, Jamaica. Using the results from the hurricane model, the flow- and wave forces are calculated using the Morison equations for lift and drag. Three initial lay-outs for submerged breakwaters are tested in the model. This leads to A final lay-out, which is a combination of the three initial lay-outs.

Following the Delft3D models, a structural analysis is done with the flow- and wave forces from the Morison equation. The structural analysis focuses on the sliding and uplift of the submerged breakwater. The hexagonal structure shows a better stability than the triangular blocks in hurricane conditions and therefore chosen as the final concept. A sensitivity analysis is performed with regard to the friction coefficient, the force-time profile and the placement errors. The placement errors turn out to be crucial and a connection between the top block and the base is needed to retain stability.

The final dimensions (l x w x h) of the hexagonal blocks are 3 x 0.75 x 0.93 meter. The blocks can be made from a low strength class concrete and reinforcement is needed to provide strength during lifting. To enhance the marine life enhancement properties, fish condos of 4" and 6" are provided, the surface is made more permeable and the pH of the concrete is altered by curing.

The final design of the blocks on the pilot site in Montego bay, shows a total of 2 028 hexagonal blocks and 299 base blocks to be used. The construction time for the project is estimated at 140 days with an estimation of total costs of US $2 517 760.

The structure shows great future potential and can be built soon. It is a state of the art structure, the stability is high, it enhances the marine life, the final dimensions will precisely agree with the drawings and there is no need for a nearby quarry. However, to all this benefits, there is also a drawback; the cost. The cost is a multiple of the conventional armour stones. Recommendations are given to bring expenses down. Placement in shallow water is preferred and replacing steel reinforcement by fibre reinforcement is worth investigating. Those recommendations will decrease the cost and will increase the viability of the Honeycomb block concept.

Contents

Chapter 1. Introduction

Over the past few decades the coastline of Jamaica has been suffering from coastal erosion. At some locations, such as at Hellshire beach, the beaches have retreated with several meters (see Fig. 1.2). The top photograph represents the condition in 2009 and the bottom in 2016, showing a retreat of several meters in only seven years.

Since tourism is an important source of income for the Jamaican economy it is of great importance to preserve the beaches. In order to defend the coastline and preserve the beaches, numerous coastal protection structures have been built. Most of these projects are carried out at the touristic locations. The hotels and resorts are willing to invest in the beach, as it is their main selling point. At the public beaches such as Hellshire however, less attention is given to maintenance and preservation of the beach.

Figure 1.2: Hellshire beach in 2009 (top) and 2016 (bottom) [Gabriell Taylor, 2016].

1.1 Location

The project site is located in Montego Bay at the north-west coast of Jamaica (see Fig. 1.3). It consists of 3 public beaches which were reclaimed by the construction of several breakwaters and submerged sills in the 1970's. The combined length of the beaches is about 900 meters.

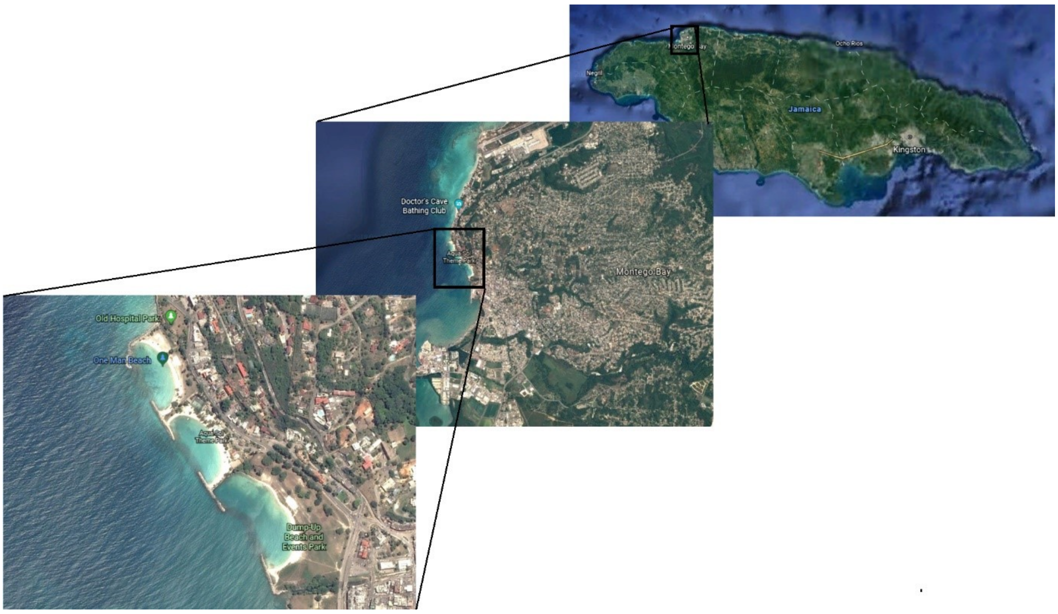


Figure 1.3: Project location at Montego Bay [Google Earth, 2018].

In order to protect the beaches and the area behind the coastline, a series of groynes and breakwaters were build, for a detailed lay-out and description of names see Fig. 1.4.

The most Northern groyne currently ranges between +0.9 m to +1.2 m and is severely eroded. At some locations of this groyne, the elevation is less than mean sea level. The two breakwaters, Gun Point and Dump-Up, are damaged as well. At Dump-Up the damage is more severe and at some locations the top elevation is less than mean sea level. At the northern part of the Dump-Up breakwater, the elevation ranges between +1 m to +1.6 m. At Gun Point the damage mostly concentrates at the ends, which is most vulnerable to heavy wave impact.

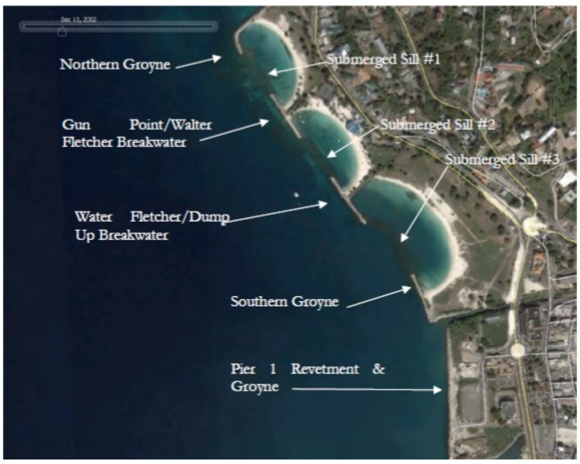


Figure 1.4: Lay-out of the project site, including naming of the structures [Smith Warner International Ltd., 2018].

Between the North/South groynes and the breakwaters, submerged sills are located. These should break the incident waves to reduce the forces on the beaches and prevent them from eroding.

During a field trip to the project site, the condition of the breakwaters is investigated. Also the state of the corals and marine life at the submerged sills is documented. For the results of the survey, see Appendix A.

1.2 Problem description

Since the construction in the 1970's the groynes and breakwaters have not been maintained. This has led to some severe degradation of the integrity and functionality of the structure. The main damage occurs when large swell from tropical storms affects the area or when the breakwaters are under severe wave attack by hurricanes. In order to investigate the sites condition and make recommendations for improvements, a survey was done by Smith Warner International Ltd. in 2013 [1]. The result of this survey was the heights of the protection structures given in Section 1.1. They proposed a solution in which a new armour stone layer would be placed to withstand a 50-year design hurricane.

Since degradation on coastal defence systems is not limited to Montego Bay, the idea arose to create a system that can be used at different locations under different wave conditions and water levels. However not only the coastal protection is of importance. The construction of coastal defence structures often influences the local ecosystem. A solution that sustains or can even enhance the conditions to support marine life would not only be beneficial for the environment, but could also create more opportunities for recreational activities.

1.3 Aim of the project

The aim of the project is to design a modular system that can serve a dual role as coastal protection, e.g. a breakwater, and marine life enhancement. With a modular system it is possible to apply the concept on different locations and to adjust the dimensions of the structure for different conditions. This can either be realized by a 'hard engineering solution' by using man-made structures with stones or concrete, or by designing a 'soft engineering solution' in which ecological principles are used to reduce erosion. In the following, those principles and a combination of them will be analysed.

Chapter 2. Problem analysis

The first part of this chapter will show the demands and wishes for the design of the sill at the project site of Dump-Up beach. The second part is a general stakeholder analysis. The stakeholders which are generally involved in the planning and executing stages of such projects are elaborated on.

2.1 Program of Requirements

The final design which tackles the problem described in Section 1.2 and is in line with the aim of the project described in Section 1.3, must satisfy a number of requirements. The requirements are ordered from most important (1) to less importance. A small elaboration on each requirement will be given.

1. Wave reduction
2. Enhancement of the ecosystem
3. Modularity
4. Stability during hurricanes and tropical storms
5. Constructability
6. 50 year lifespan

Wave reduction
The use of the submerged sills is of great importance for a coastline. On the one hand they create an open connection to the sea such that there is a continuous supply of seawater into the bays. On the other hand, it reduces the wave impact on the beaches by breaking them further offshore. This prevents the beaches from eroding. Maintaining the beaches is one of the most important requirements, since it has a high recreational value for the area. The survey done by Smith Warner in 2013 showed a direct link between beach erosion at Aqua Sol Beach and the status of submerged sill #2; the sill which is highly damaged (see Fig. 1.4).

Enhancement of the ecosystem
Normally, coastal defence structures disturb the local environment. In order to enhance instead of disturb the marine environment, an innovative design should be formulated. During this project multiple concept to achieve this will be considered. An electrified frame and permeable concrete could enhance the ecosystem, as well as fish condos. Those means will be further explained in the next chapters. Creating a well-functioning ecosystem will be beneficial for the local biodiversity, and thereby increase the recreational possibilities. It would attract people to do snorkelling and diving. Also, the growth of coral and sea grasses will help to attenuate waves.

Modularity
The protection structure must be as general and modular as possible to be able to use the concept all around the world. A modular structure could be dimensioned according to project specific conditions.

Stability during hurricanes and tropical storms
The design should account for extreme conditions that occur during hurricanes or tropical storms. Although these conditions will not occur very often, they will do most damage to the structure and the coastline. In this report an investigation will be done to see if the level of protection during hurricanes and to see if the structure remains stable during these extreme conditions.

Constructibility
During the entire design phase the constructibility will be kept in mind. The fabrication will be considered, as well as the materials and equipment needed.

50 year lifespan
The construction will be designed to perform the proposed requirements for 50 years. After this time, the structure may fail to perform the requirements and a survey must be performed for the remaining lifetime.

2.2 Stakeholder analysis

The coast erosion and beach retreat problems described in the Introduction affect multiple stakeholders. Every stakeholder has its own interest and wishes. The interest of different stakeholders may be different and thereby conflicting. To identify these potential conflicts or forthcoming complaints in advance, a stakeholder analysis can be used as a powerful tool. This section gives a general overview of stakeholders which may be encountered during this type of projects.

A short overview is given in Tab. 2.1, which is then further elaborated on.

Table 2.1: General stakeholder overview.

Stakeholder	Type	Remarks
National/local authorities	Client	Most common go/no go decision maker.
Hotels & resorts	Client/Third party	Satisfy tourists and tries to maximize profit.
Local inhabitants	Third party	Can impact decision maker by complaints.
Nature activists	Third party	Can influence decision maker by lobbying.
Tourists	Third party	Important economic factor for Caribbean countries.
Private clients	Client	Not very common.

National/Local authorities
Most projects can be tendered for organizations which are directly or indirectly part of the authorities. The two main goals for authorities are the minimization of cost and at the same time the maximization of the effectiveness of the solution (social welfare). This stakeholder is also the decision maker for these types of projects. However, the decision can be greatly influenced by third-party stakeholders mentioned later.

Hotels & resorts
While hotels and resorts may or may not directly be involved in the decision, they are an important stakeholder to take into account. In the Caribbean they are an important factor in the economic welfare of certain countries. Their aim for these projects is to maintain their beach, increase the number of tourists and increase their profit, all with an sophisticated solution.

Local inhabitants
Protecting coastlines can be important to protect the inhabitants from flooding. By increasing tourism the infrastructure may change. Firstly, new economic opportunities may arise (e.g. bar establishments, shops etc.). Next to that, negative consequences may also be experienced. The area might become overcrowded due to the tourists, or increased littering. The local inhabitants do have a powerful voice in the decision making process.

Nature activists
One of the influential stakeholders are the nature activists. By lobbying, they can greatly influence the decisions taken. Therefore, it is always important to hear their point of view before the project is in an advanced stage. Their main aim is to preserve the ecosystem and make sure it is taken care of in a sustainable way.

Tourists
Tourists, as mentioned before, are for most countries in the Caribbean the most important contribution to economic welfare. Increasing the satisfaction will generate an increase in tourists. The way of increasing satisfaction may be to make beaches wider, to make the water clearer and to enhance possibilities for recreation (snorkeling, diving etc.).

Private clients
Not a common stakeholder. A private client might have a wish, for which a company is hired to find a solution. For these clients money is mostly not an issue and aesthetics is important for these projects. In general, these projects do not have many other stakeholders to account for.

Chapter 3. Breakwaters & submerged sills

In the first part of this chapter, different sorts of breakwaters and their stability are analysed. In the next section, formulas to predict the wave attenuation will be given. Also, the boundaries of the given equation will be shown. In the last graph, the wave attenuation of the formula is compared with values attained by modelling software SWAN.

3.1 Concrete armour breakwaters

Most often breakwaters are made from natural stone blocks, but when the availability of these blocks is limited or a specific shape is desired, the use of concrete becomes an interesting alternative. In general the stability of concrete armour systems are approached in three ways [2]:

- Stability by *weight*, usually with simple, bulky shapes (see Fig. 3.5a).
- Stability by *interlocking* elements, usually with a specific shape to improve interlocking (see Fig. 3.5b).
- Stability by *friction* between elements, usually by blocks placed in a specific pattern, mainly used in block revetments or sea dikes (see Fig. 3.5c).

(a) Weight. **(b)** Interlocking. **(c)** Friction.

Figure 3.5: Concrete Blocks [Deltares, blockwallsTM, TU Delft].

3.1.1 Stability

In the current situation in Montego Bay, the breakwaters consist out of rubble mound stones. The stability of a rubble mound breakwater is usually approached with van der Meers equation over Hudson's equation and is given by Eq. 3.1, in which $H/\Delta D$ represents the stability number. The structure is considered stable for $H/\Delta D$ smaller than 2.

$$\frac{H_s}{\Delta d_n 50} = 6.2P^{0.18}\frac{s}{\sqrt{N}}^{0.2}\xi^{-0.5} \qquad \text{for plunging waves} \tag{3.1}$$

with

H	=	Significant wave height [m]
Δ	=	Relative mass density $(\rho_s - \rho_w)/\rho_w [\mathrm{kg\,m^{-3}}]$
D_{n50}	=	Nominal median rock diamter [m]
P	=	Permeability factor (0.6 for homogenous structure)
S	=	Damage number (for small damage S = 2-3)
N	=	Number of waves (for equilibrium damage N = 7500, for storm conditions N = 3000)
ξ	=	Iribarren number $\xi = \frac{\tan\alpha}{\sqrt{\frac{H}{L_0}}}$ 1.5 for plunging

When designing the breakwater, the significant wave height (H_s) should be replaced with the wave height occurring at the chosen return period (H_{ss}). Under these storm conditions most damage will be done so choosing the significant wave height from regular wave data will lead to an under-designed structure.

3.2 Submerged breakwaters

The main function of the submerged breakwater is to break waves to reduce the impact on the shoreline and prevent erosion (see Fig. 3.6). Therefore, it is important that the height of the submerged breakwater is sufficient to break the waves. Also the crest width should be wide enough to absorb the wave energy over the breakwater.

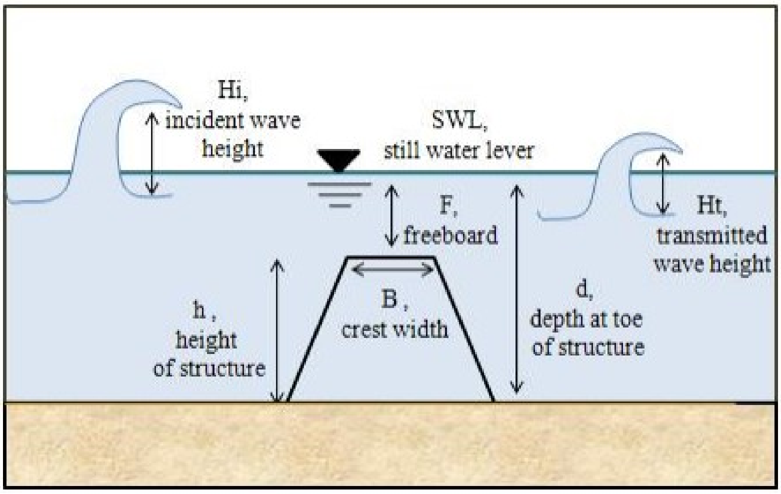


Figure 3.6: Effect of submerged sill on wave propagation [Arnouil, 2006].

A simple formula is used to indicate at what depth a wave will break and is given by the ratio below. The depth d should be small enough with respect to the incoming wave height H. In the case of a submerged breakwater the depth d should be replaced with the freeboard F.

$$\frac{H}{d} \geq 0.78$$

To measure the effectiveness of a submerged sill the transmission coefficient is used. H_t is the incident wave height and H_i the transmitted wave height. The larger the coefficient, the less the wave is attenuated.

$$K_t = \frac{H_t}{H_i}$$

Over the years many empirical relations have been derived for predicting the transmission coefficient, such as Ahrens (1987), Friebel and Harris (2004), Armono and Hall (2003). Ahrens derived his formula for rubble mound breakwaters, meaning it is less applicable for interlocked concrete armour units. Armono and Hall developed a model for wave transmission based on two dimensional tests, which included the use of ReefBalls. Finally Friebel and Harris derived a new empirical wave transmission formula from data collected from five physical model studies (Seelig (1980), Daemrich and Kahle (1985), Van der Meer (1988), Daemen (1991), and Seabrook and Hall (1998)). Since this model is not specified to a specific concept such as the Reefballs and it is based on several projects, it will be used in this research [3]. The formula from Friebel and Harris is given by:

$$K_t = -0.4969 \exp^{\left(\frac{F}{H}\right)} - 0.0292\frac{B}{d} - 0.4257\frac{h}{d} - 0.0696 \ln\left(\frac{B}{L}\right) + 0.1359\frac{F}{B} + 1.0905 \tag{3.2}$$

with

$$\begin{aligned}
F &= \text{Freeboard [m]}\\
H &= \text{Wave height [m]}\\
B &= \text{Crest width [m]}\\
d &= \text{Water depth [m]}\\
L &= \text{Wavelength [m]}\\
h &= \text{Height of structure [m]}
\end{aligned}$$

This formula holds for the following input intervals:

$$0 \geq \frac{F}{H} \geq 8.7; \qquad 0.286 \geq \frac{B}{d} \geq 8.75; \qquad 0.44 \geq \frac{h}{d} \geq 1; \qquad 0.0024 \geq \frac{B}{L} \geq 1.89; \qquad 0 \geq \frac{F}{B} \geq 1.05$$

3.3 SWAN validation

Before Eq. 3.2 will be used in further design of the submerged breakwaters, it will be validated by using a simple one-dimensional SWAN model. The situation sketched in Fig. 3.6 will be modelled in SWAN.

The boundary conditions for the SWAN model are given below. Tab. 3.2 gives an overview of the different submerged breakwaters, differentiating in height and crest width, and the resulting transmission coefficients per type of submerged breakwater. The supporting figures can be found in Appendix C.

$$\begin{aligned}
d &= 3.0\ \text{m}\\
H_s &= 1.21\ \text{m}\\
T_{m01} &= 6.6\ \text{s}
\end{aligned}$$

Table 3.2: Results of SWAN model and formula of Friebel and Harris.

	Height [m]	Crest width [m]	$\mathbf{H_{tr,SWAN}}$ [m]	$\frac{\mathbf{H_{tr,SWAN}}}{\mathbf{H_s}}$ [-]	$\mathbf{K_t}$ [-]
No breakwater	-	-	1.21	1.0	-
Type 1	1.5	3	1.07	0.88	0.80
Type 2	2.0	3	0.95	0.79	0.68
Type 3	2.5	3	0.44	0.36	0.52
Type 4	2.0	2	1.00	0.82	0.70
Type 5	2.0	4	0.90	0.74	0.66
Type 6	2.0	6	0.80	0.66	0.63

From Tab. 3.2 can be concluded that there is some difference between the SWAN model and the presented equation, Eq. 3.2. The presented equation mainly consists of experimental data [3]. The difference between the two resulting transmission values can be explained due to the fact that one is a theoretical model and the other is highly empirical equation. It may also be the case that SWAN isn't the best model to validate this equation. SWAN doesn't take reflection of the wave into account. A better model which could be used is SWASH. However, when dimensioning the submerged breakwaters later in this report, Eq. 3.2 will be used.

Chapter 4. Coastal management techniques

For the development of our concept an overview of alternative solutions is given. The benefits of the solutions will be explained in the first section. Management techniques such as Biorock, Reefballs and Wave Attenuation Devices will be shown. Also a short explanation of the Building with Nature concept is given. A more comprehensive explanation about the concepts can be found in the Appendix B. The chapter ends with a conclusion with regard to the relevant and useful properties of the alternatives, which could be combined in the final design.

4.1 Coastal management techniques

4.1.1 Biorock®

Biorock (see Fig. 4.7) is a product to create artificial reefs. This is done by placing a steel wire mesh in the sea and electrifying it. The electric current causes a deposition of limestone on the steel frame. As the limestone is the ideal base of corals to settle, they will flourish on the frame. The low current in the frame enhances the marine life by the positive influence of the electric field generated.

Biorock is a proven technology and studies have shown the effectiveness. In the Maldives, a coral reef was recovered with Biorock, which led to a stable 15 m growth of the beach width in 2-3 years. Also, increased growths rates (8x), recovery rates (20x) and resistance against acidification (50x) of coral were shown when electrified frames were used [4].

Benefits
An electrified frame increases development of corals and enhances marine life in the vicinity.

Figure 4.7: Biorock frame with coral in sea.

4.1.2 Reef Balls

Reef Balls (see Fig. 4.8) are the most commonly used structures in reef restoration projects. More than 3500 project are conducted with a total over 500 thousand deployed Reef Balls [5]. Reef Balls consist of single hollow concrete elements with holes and a permeable surface. The holes create a specific flow of water through the balls and a safe habitat for marine life. The current provides extra stability and distribution of valuable nutrients for the marine life. The concrete is adapted to match the pH of sea water and the surface is adapted to promote settlement of coral larvae. In addition, these blocks have a limited wave attenuation effect. Furthermore the Reef balls are reinforced with non-corroding fibres.

Benefits
Fish condos create a sheltered and nutrient-rich environment for fishes. The rough concrete surface promotes the settlement of coral larvae.

Figure 4.8: Reef Balls with coral in sea.

4.1.3 WAD

Wave Attenuation Devices (see Fig.4.9), shortly WADs, are designed to reduce the wave energy and wave height at the shore. The pyramid-shaped blocks are placed in a project specific orientation and lay-out. The WADs are commonly just emerge from the sea. Due to the geometry and weight of the blocks, they remain stable in category 5 hurricanes. Also, the holes in the WAD increases the fish population, comparable to the holes in the Reef Balls.

Benefits
The wave attenuation property reduces the wave energy and the fish condos enhance the marine life.

Figure 4.9: Wave Attenuation Devices (WADs) in sea. They emerge partly above water level.

4.1.4 Building with Nature

Building with Nature (see Fig. 4.10), shortly BWN, is a new philosophy in hydraulic engineering that utilizes the forces of nature, thereby strengthening nature, economy and society [6]. A project with the Building with Nature philosophy uses a combination of *grey* and *green* measures to combat the hydraulic challenge. Applying the BWN approach leads to long term solutions, as the *green* nature will help the performance of the *grey* structure. As an illustration; a *grey* structure which helps the coral flourish will have increased wave attenuation performance in future, due to the wave attenuation of the coral itself.

Benefits
Long term solution, in which the nature helps the future performance of the *grey* structure. Approach which fits in sustainable thinking and environmental engineering.

Figure 4.10: Building with Nature in Java, Indonesia. Natural materials are used to create permeable dams to prevent coastal erosion (See Appendix B).

4.2 Conclusion

The current coastal management techniques all have some benefits, but there is no concept on the market which has good wave attenuation as well as good marine life enhancement properties. During the concept development the goal is to develop a structure with a combination of the benefits from the mentioned alternatives. The electrified frame of the Biorock, the geometry and surface properties of the Reef Balls and the wave attenuation of the WAD will be combined in one viable concept. In the rest of the report, different concepts are ranked and analysed in depth to see the viability of a combination of those techniques.

Chapter 5. Concept selection

In this chapter a promising concept is looked for, which can comply with all design criteria. In the first section, the procedure of selection and ranking is explained. The design criteria are given and elaborated on. A Multi-Criteria Analysis will be performed to rank the concept groups. Next, two concepts within the most promising design group are worked out and elaborated on. Special attention is given to the main challenges of the structures. In the coming chapters, the challenges are closer looked at and substantiated by hydraulic and finite element models.

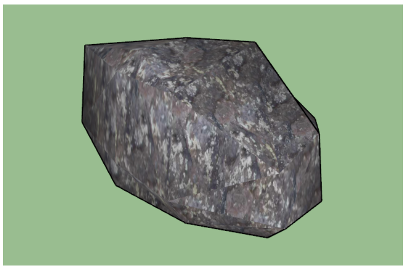
5.1 Concept design (group) selection

5.1.1 Procedure of selection

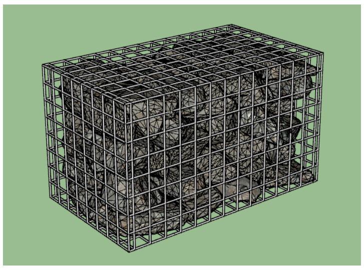
To make a justifiably selection, concepts are placed in multiple concept categories. The ranking categories are formulated, and weight is given to each of them. As modularity is a demand, only block-like concepts are analysed. Next, the concept categories are closer looked at and grades between 1-10 are assigned to each criterion. When grades are assigned to all concept categories, the most promising group scores highest in the Multi-Criteria Analysis. Next, within this concept category, different designs are made. The two most promising concepts will be worked out in depth to see the feasibility of these solutions.

5.1.2 Design groups and ranking criteria

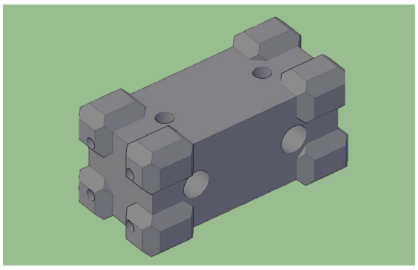
The used division of concept groups is based on the geometry and material mainly. The groups are visualized in Fig. 5.11. Boulders (see Fig. 5.11a) and Gabions (see Fig. 5.11b) are already existing blocks. Marine blocks (see Fig. 5.11c) are blocks designed by Smith Warner. The Big (open) blocks (see 5.11d) and Lego blocks (see Fig. 5.11e) are newly designed blocks.



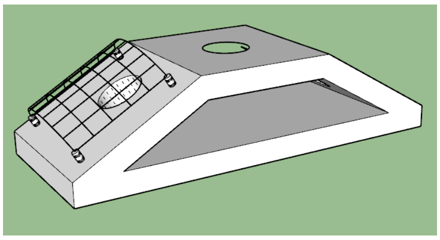
(a) Boulder.



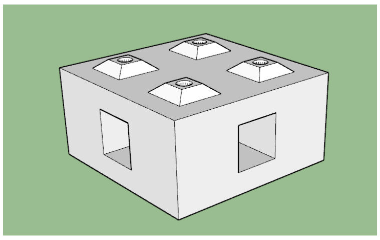
(b) Gabion basket.



(c) Marine block.



(d) Big (open) block.



(e) Lego block.

Figure 5.11: Concept groups analysed in Multi-Criteria Analysis.

The five concept groups can be described as follows:

1. **Boulders**
   Boulders are stones with a close to circular shape. The stones are of natural material and extracted from a quarry. Boulders are the conventional building blocks for coastal defence, used for decades and proven to be capable of attenuating waves. However, during hurricane conditions they can encounter stability issues.

2. **Gabion baskets filled with stones**
   The blocks of this group consist of an iron wire mesh filled with moderately sized stones. The iron cage determines the shape and makes sure the stones stay in place. The stones give weight and give the blocks a permeable structure. The blocks can easily be stacked to form a bigger structure.

3. **Marine blocks**
   This block is designed by Smith Warner and suggested as the coastal defence structure of a beach in Negril, Jamaica [7]. With dimensions of 1.6 x 1.6 x 1 m, these blocks are relatively easy to handle and to produce. The holes of the blocks are produced by using PVC pipes. The geometry of the block is optimized to generate turbulence and to diminish the waves, as well as to enhance marine life.

4. **Big (open) blocks**
   This group consists of blocks with relatively large dimensions, with a hollow structure. Dimensions are in the order of multiple meters. Dimensions are limited by the weight of the block (preferably maximum 5-10 ton) and handleability. Big blocks tend to reduce the time needed for placement of the coastal defence structure on site. The blocks can be optimized to enhance marine life in multiple ways. Fish condos in the blocks will create currents and shelter for fishes, special treated permeable surfaces will improve the adhesion possibilities of coral and the application of electrified frame will increase fishes and coral presence further.

5. **Lego blocks**
   The design of these blocks is inspired by the well-known Lego. These blocks are smaller than the big (open) blocks and the blocks have a less open structure. As the blocks are smaller in dimensions, they have a better handleability. However, as stacking should be done with sufficient accuracy, attention should be paid to placement of the blocks.

Table 5.3: Criteria used in Multi-Criteria Analysis and their weights.

Wave attenuation	20%
Marine life enhancement	20%
Cost	15%
Placement on site	15%
Modularity	10%
Stability	10%
Constructibility in factory	5%
Durability	5%

5.1.3 Ranking of concept groups

Table 5.4: Multi-Criteria Analysis of five different concept groups. Ratings are from 1-10, with 1 being the lowest and 10 being the highest. The least favourable concept score is shown in red, the most favourable in green.

	Wave att.	**M.L.E.**	**Cost**	**Place.**	**Mod.**	**Stab.**	**Constr.**	**Dur.**	**Total**
Boulders	7.5	5.3	7.5	7.8	6.8	5.8	8.3	6.5	**6.8**
Gabion basket	6.0	5.0	6.3	6.8	5.3	4.5	6.8	3.8	**5.6**
Marine blocks	5.3	6.8	6.3	8.0	5.0	6.8	6.3	7.3	**6.2**
Big open blocks	7.3	8.3	5.0	5.0	7.8	8.0	5.5	8.0	**7.0**
Lego blocks	7.5	7.5	4.8	4.8	8.8	7.3	4.5	7.0	**6.8**

Boulders are proven to be efficient wave attenuators. Marine enhancement is low, as the blocks are not enhanced with a more porous structure or electrified frame. Placement on site is easy and relatively fast. Structures of different dimensions can be made, which gives the structure a relatively good modularity. During storm conditions, the stones can fall which leads to a damaged structure. Stones are taken from a quarry, which is done with ease. But getting large boulders can be hard. The costs are relatively low, but can increase due to low durability.

The wave attenuation properties of **Gabion baskets filled with stones** are less than boulders, as they can not withstand high energy waves [8]. Gabion baskets are not equipped with properties to enhance marine life. Placement on site can be done with ease as the baskets have a low to medium weight and are easily stackable. The modularity of the total construction is relatively low and the connection between the blocks has to be looked at. As the blocks are not sloped, so stability of stacked structures is low. The investment cost of the structure is low. However, as the durability is very low [9], the expenses during life time of the construction are high. The constructibility in the factory is good.

Marine blocks are capable of attenuating waves, but only for relatively shallow waters. Attenuation of higher waves is not possible as the blocks can not be easily stacked. Marine life enhancement is medium. The holes in the blocks do enhance fish, but no coral enhancement is present [7]. Placement does not have to be very accurate so it scores good on placement. As the blocks do not have a suitable shape for making a stable stacked structure, the modularity is low. The stability of individual block is high, but stability of stacked blocks low. The cost of the blocks is okay and the placement can be done fairly quick. The openings in the block are created by PVC pipes, leading to a constructable mould. Meaning the constructibility is average. The durability of the blocks is expected to be high.

Big (open) blocks do have good wave attenuation properties and excellent marine life properties because of the enhanced currents, the possibility to apply an electrified frame and the possibility to apply permeable/rough concrete surface. The construction on site will need medium to high accuracy, as the blocks should interlock. However, the blocks can be shaped, to be self-guiding towards the right position. The total construction is very modular, as they can easily be stacked to form the complete coastal defence structure. The stability of a single block will be high, as the structure will have a high mass and wide bottom surface. Attention must be paid however to the interlocking of the blocks, as the blocks themselves might not be not stable. The costs for small scale will be high, however, as the projects become bigger, the open blocks will become more attractive. Besides that, the blocks are expected to be durable. The blocks are not easy to construct. However, as the blocks are similar, the same mould can be used many times.

The wave attenuation properties and marine life properties of **Lego blocks** are very good. The blocks are comparable to big open blocks, but are much smaller. This gives more difficulties staking them. The margins of placement is smaller and more time is needed to build the construction. As the blocks are smaller, the stability of the individual blocks is lower.

5.1.4 Choice of concept

As can be seen in Tab. 5.4, Big (open) blocks are most promising. To come up with the most viable concept, different options within this concept group are analysed in a structural manner.

Big open block can be designed in all kind of different shapes, types, modularity and placement. Below, those different parameters are analysed and their strong and weak points are indicated.

Shape

- Square
  + Stable bottom layer
  - No interlocking between different layers
- Triangular
  + Blocks slide into position → Increases placement accuracy
  - Unstable bottom layer → Sideways movement of blocks
  - No interlocking between different layers
- Hexagonal
  + Interlocking of different layers
  - Base block needed for stable placement

Type

- Blocks
  + Modularity in width
- Beams
  + Modularity in height
- Complete structure
  + Fast placement
  - Weight and dimension limitations of transport and placement

Constructability

- Perpendicular to shoreline (width-direction of construction)
  - In-line
    + Efficient use of material
    + More predictable wave attenuation
  - Random
    + Placement accuracy low → Fast placement
- Parallel to shoreline (length direction of construction)
  - Brick-like built-up
    + Interlocking in length direction
  - Exactly on top
    + Freedom of shape of breakwater → Curved breakwater possible

In the next section, two concepts are worked out with a combination of the above mentioned parameters.

5.2 Conceptual designs

Two concepts within the Big (open) blocks concepts are worked out. Both designs will be closer looked at from different perspectives. Extra attention is paid to the placement on site and stability.

5.2.1 Triangular block concept

The first concept is a concept with a triangular shaped cross sections. The blocks are beam like and the placement will be in-line. The construction can be built-up both brick-like and exactly on top. Depending on the site, one of both is preferred with respect to the shape/curvature of the structure. The blocks have the following dimensions:

Table 5.5: Dimensions of triangular block.

Dimensions (l x w x h)	3 x 0.89 x 0.77	m

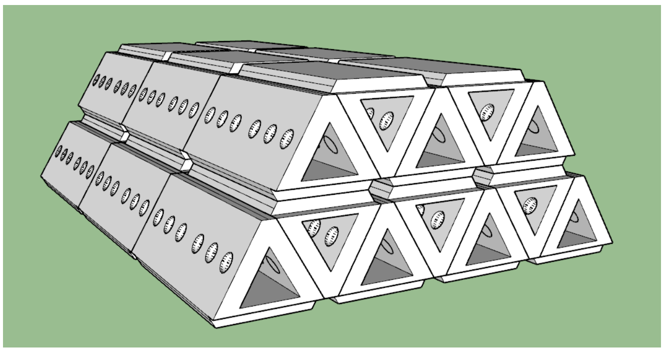


Figure 5.12: Basic lay-out of a 3-row protection structure, built up from triangular elements.

For this concept, a stability check must be performed. The uplifting of the blocks at the top layer must be looked at. Also it must me checked if the outer blocks will slide. The friction coefficient of concrete-concrete will be comparable to the seabed-concrete friction coefficient, so on both interfaces, there is potential to loose stability. Ideally, the friction force of the blocks with the seabed is high enough to prevent sliding. Also, stability against sliding of the blocks at the side of the second layer needs to be looked at. These blocks have a lower downward force (no blocks stacked on top), which gives a lower maximum friction force. The maximum friction force for both situations can be determined with the following basic physics formula:

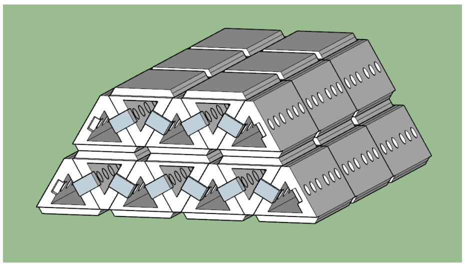
$$F_{\text{friction}} = \mu N$$

with

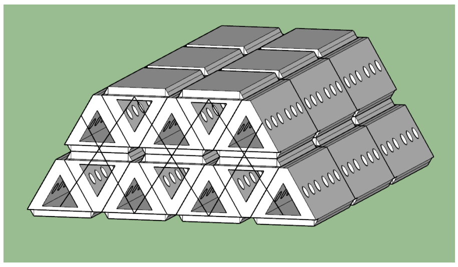
$$\begin{aligned} F_{\text{friction}} &= \text{Maximum friction force} \\ \mu &= \text{Friction coefficient} \\ N &= \text{Normal force} \end{aligned}$$

The value of the friction coefficient μ can be found by tests. These must be performed at the same conditions as the one of application. The triangular shape of the block has a beneficial effect, as the wave force gives a downward directed force, which adds to the normal force by the weight. Stability calculations are done by ANSYS (see Chapter 7).

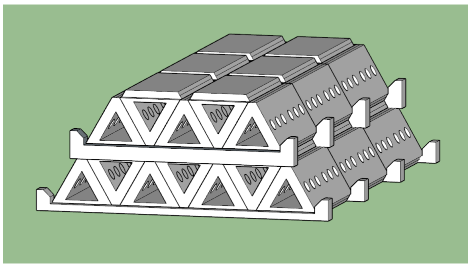
When the friction force is not sufficient, measures should be taken. Possible measures are connecting blocks with clamps or cables, placing blocks on a beam with upright edges or increasing the weight of (only the sliding/uplifting) blocks (see Fig. 5.13). Also, the seabed could be prepared or the blocks can be anchored.



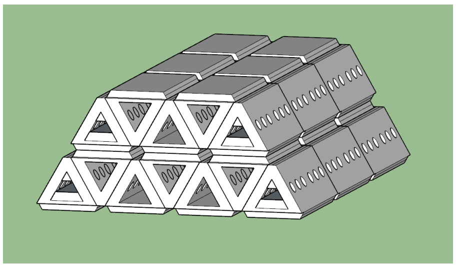
(a) Connection with clamps.



(b) Connection with cables.



(c) Connection with beam with upright edges.



(d) Connection with increased weight of side blocks.

Figure 5.13: Visualization of different connection mechanisms to prevent sliding and uplifting of blocks.

The triangular shape of the blocks allows for relative margins in accuracy of placement in the direction perpendicular to the shore. However, in the along-shore direction, placement must still be done accurately. The blocks can also be used to make an emerged breakwater by adding extra blocks op top. However, special blocks may be needed to form a vertical wall.

5.2.2 Hexagonal block concept

The second concept is a concept with a hexagonal shaped cross sections. The blocks are beam like and the placement will be in-line. The construction can be built-up both brick-like and exactly on top. Depending on the site, one of both is preferred with respect to the shape/curvature of the structure. The blocks have the following dimensions:

Table 5.6: Dimensions of hexagonal block and bed.

Block			**Bed**		
Dimensions (l x w x h)	3 x 0.75 x 0.93	m	**Dimensions (l x w x h)**	3 x 2.26 x 0.42	m

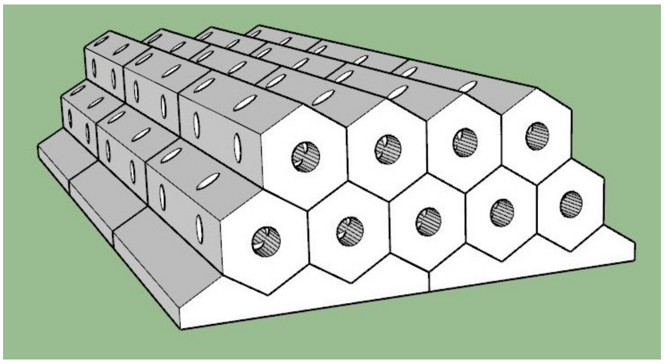


Figure 5.14: Basic lay-out of a 3-row protection structure, built up from hexagonal elements.

For this concept, the stability is more favourable compared to the triangular system. The blocks at the bottom are more stable, because of the underlying bed. Also, the stability of the second layer is clearly enhanced. The blocks interlock, which increases the resistance against sliding.

Because of the placeability, the width of the bed is limited to two and a half blocks. In this way, also the modularity of the system is increased. It is possible to change the structure base width, height and crest width separately, while the stability of the structure is maintained.

The variables for this structure are the outer dimensions of the bed and the blocks, the diameter of the longitudinal hole, and the amount and shape of the fish condos. As said, the concept will be enhanced by placing an electrified frame at the (beach-)side of the structure. The blocks will be made with the modified concrete (see Chapter 8), which enhances the coral attachment.

Just like the triangular concept, an emerged sea wall could be made from this concept. However, different blocks need to be designed for this application.

5.2.3 Dimension of blocks

The concepts mentioned before are based on the shape of the blocks, instead of the size of the blocks. As can be seen in Fig. 5.15, similar total construction dimensions can be accomplished with different sized blocks. As,

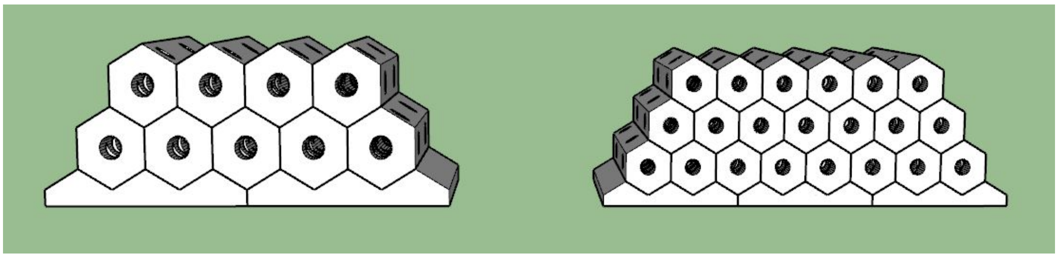


Figure 5.15: Two constructions with similar dimensions, built-up from different sized blocks.

the chosen concept group suggests, Big (open) blocks, big blocks are preferred. The size of the blocks is mainly limited by the weight and in a lesser degree by the handleability of the blocks. For placement, a maximum of 5 ton is preferable. In the next chapters, the final dimensions of the structure are determined, based on the maximum weight (upper boundary) and the stability and strength (lower boundary).

Chapter 6. Delft3D: Data & models

This chapter discusses the build-up of a Delft3D model of the project site. In the first section the different data, which is gathered and to be used in the model, are concisely discussed. A more in-depth view is given in the Appendix D.

The second part of this chapter serves to get insight in the built-up of this model and also treats the results. The Delft3D model will consist of different parts with each serving its own purpose.

- **Daily conditions model:** An on-line run of Delft3D Flow together with Wave to analyze daily conditions.
- **Hurricane Wave model:** Delft3D Wave stand-alone runs to model waves resulting from one in 50 years hurricanes.
- **Detailed hurricane model:** Again on-line runs of Delft3D Flow & Wave using the leading wave heights from the second part. The results from this part can be used for the structural analysis.
- **Breakwater design model:** Again on-line runs of Delft3D Flow & Wave including new submerged breakwaters to model their effects on wave heights and morphological bed changes.

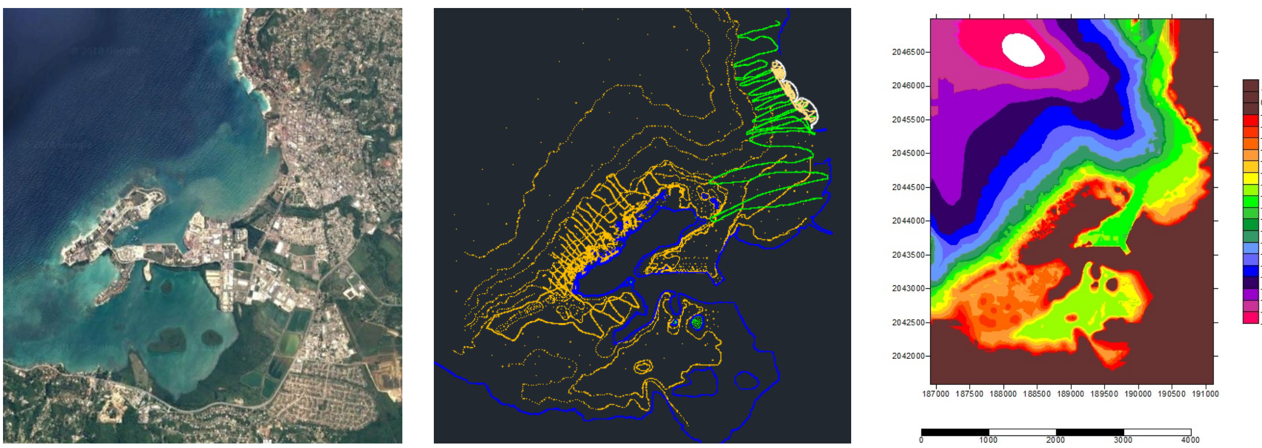
6.1 Data

As stated before, the first section of this chapter will discuss the data used in the Delft3D model. The different types of data, which are collected can be seen in the list below:

- Bathymetry (6.1.1)
- Wind data (6.1.2)
- Wave data (6.1.3)
- Tidal data (6.1.4)

6.1.1 Bathymetry

The first step in setting up the model is creating a detailed bathymetry map. The required data which is needed to create the depth profile is retrieved with use of an Odom Echotrac sounding system and the accompanying location is recorded with a Tremble GPS. At the time when the field investigation was done, the equipment was not available. Therefore data from a bathymetric survey done in 2013 by Smith Warner is used. In Fig. 6.16b the path made in the bathymetric survey can be seen. Each data point contains information about its location (x,y) and the depth (z) at that location. In order to extract the raw data and create a map with depth contours the software program Surfer 8 was used. The resulting bathymetry map is shown in Fig. 6.16c.



(a) Montego Bay. **(b)** Bathymetric survey. **(c)** Bathymetry map.

Figure 6.16: Bathymetric survey of the Hip-Strip in Montego Bay.

Since the data was very detailed, also an accurate bathymetry map at the site location could be made (see Fig. 6.17). In this figure an aerial view is added to get a better feel of the location and conditions. The submerged breakwaters can clearly be identified and are most clear at submerged sill 3, the most southern one. At the seaside of the breakwaters the depth increases rapidly.

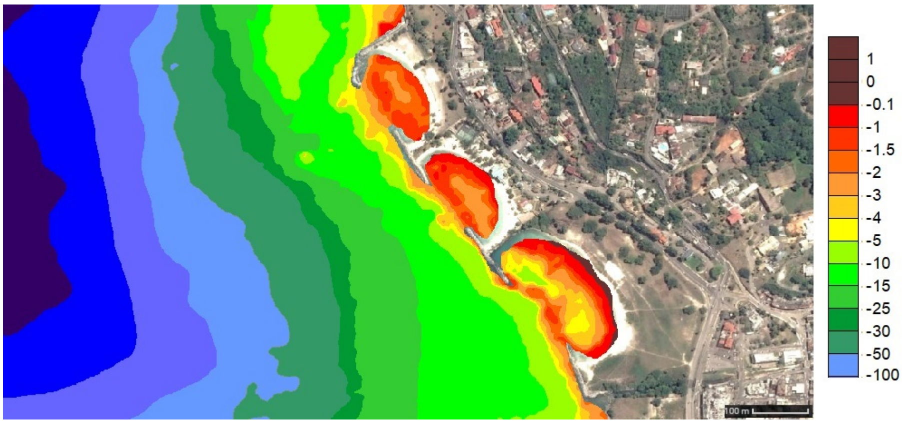
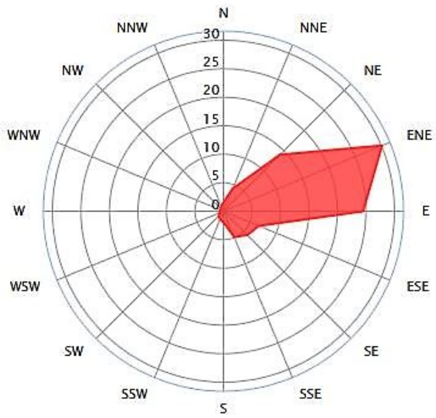


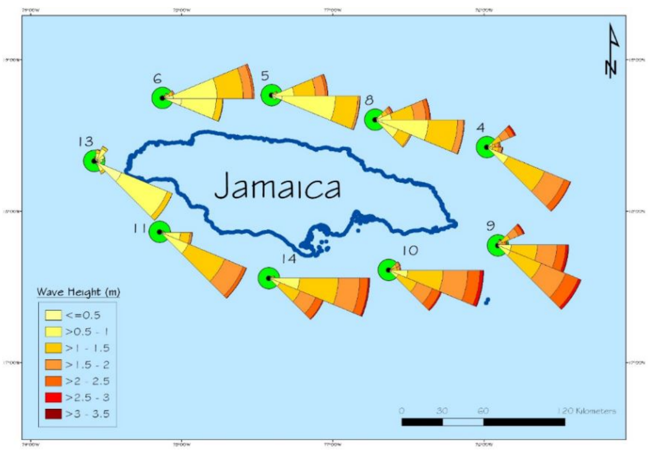
Figure 6.17: Bathymetry map of site location.

6.1.2 Wind data

Secondly wind and wave data were collected. Fig. 6.18a shows the direction and speed of the occurring wind in Montego bay. The vertical axis shows how many hours per year the wind is blowing from a certain direction. As can be seen the wind direction is generally east-north-east with a most occurring wind speed ranging between 12-19 km/h.



(a) Windrose [km/h].



(b) Waveheights [m].

Figure 6.18: Wind & wave data Montego Bay [Meteoblue, NOAA].

In the storm models other winds are used, which are generated by the model HurWave (more information in Appendix D.2). The wind of storms are used which are able to attack the project site directly, which means from the west and south-west. The storms have a return period of 50 years. The corresponding values can be found in Tab. 6.7.

Table 6.7: HurWave wind results.

Storm direction	**Return period** [years]	**Wind direction** [°]	**Wind speed** [$\mathrm{m\,s^{-1}}$]
West	50	270	26.8
South West	50	225	28.1
North	50	315	24.5
West	yearly	270	9.0

6.1.3 Wave data

In the models a wave climate is used which is generated from a model; Wave Watch 3 made by NOAA. The set retrieved, shows regular day-to-day wave data from a point north of Jamaica. This point in the sea shows the significant wave height, period and direction per time step in deep water conditions. In Fig. 6.18b the point, from which the data is retrieved, is node 5. The resulting wave climate statistics are calculated from this data. Together with Delft3D Wave the propagation of the waves at the site location can be computed. In node 5 the significant wave height, mean wave period, and mean wave direction read:

$$H_s = 1.51m \qquad T_m = 6.54s \qquad H_{dir} = 99°$$

Table 6.8: HurWave wave results.

Storm direction	**Ret. period** [years]	**Wave direction** [°]	**Sign. wave height** [m]	**Mean wave period** [s]
West	50	270	7.0	11.2
South West	50	225	6.8	11.0
North West	50	315	6.7	11.0
East	50	90	11.6	15.4
West	1	280	2.5	8.1

For hurricanes other conditions hold, and therefore a simulation is done with the model HurWave. From this model, the storm surge height and wave height can be calculated and used in further two-dimensional models. Also a yearly occurring typical tropical storm is extrapolated from the wave data. The results can be found in Tab. 6.8 and more detailed data in the Appendix.

6.1.4 Tidal data

The tidal variation is retrieved from a location just north of the project site. The measurements started in 1990 and are still updated till this moment. The total tidal range is approximately 0.4 meter. To limit simulation time, only the day with the maximum tidal amplitude together with one day before and after are simulated. The tidal data is used for simulation of the daily conditions.

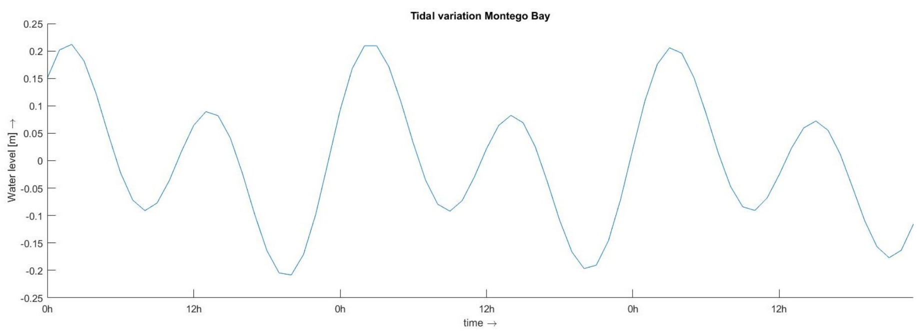


Figure 6.19: Tidal variation at project location.

6.2 Delft3D Flow & Wave models

The program used to model the wave conditions is Delft3D. With use of Delft3D, several interventions can be modelled to end up with a good solution. In Delft3D there are several modules in which the model can be built up. For this research the most important modules are Grid, Flow and Wave, which will all be discussed briefly in the next section and in more detail in Appendix D.

In this section all the models listed at the start of this chapter will be discussed. The sections will include the built-up for every model and accompanying results.

6.2.1 Daily conditions model

The first model to be constructed serves the purpose of modelling the daily conditions in the near area of the Hip-strip.

Grid & bathymetry

The first step is to define the grid in the section RGFGRID. A very small grid size will lead to the best results, however this will also increase the computation time. At the location of interest the grid should be small enough to get good results, further away the grid size can be larger to reduce the computation time. The ratio is found through trial and error. Since node 5 was at a large distance from the project site in Montego Bay, also two larger grids were made and were used as nests (Tab. 6.9). The use of nesting is to get realistic propagation of off-shore waves into the domain of interest. On all grids the yearly wind data is used from Section 6.1.2.

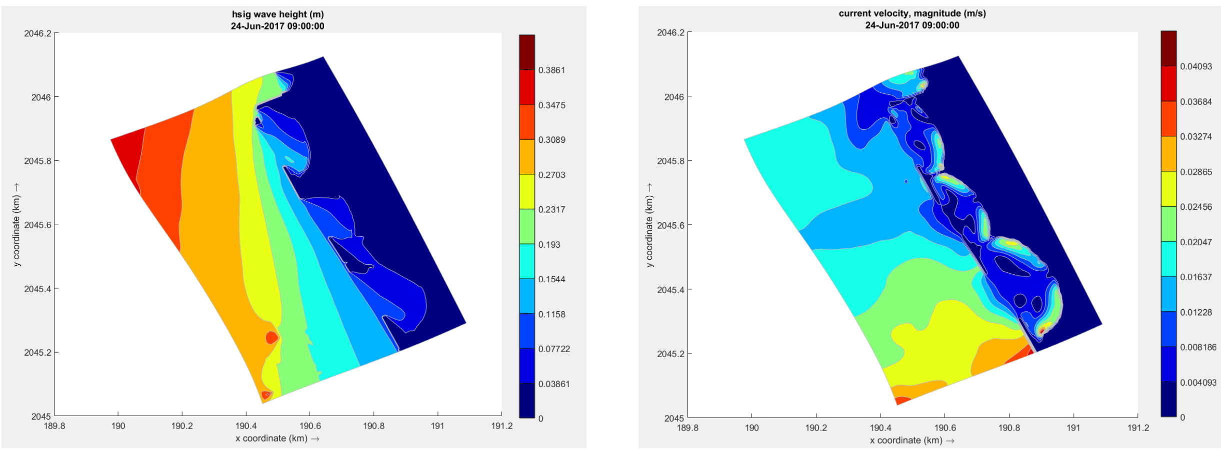
Table 6.9: Used grids for daily conditions model.

Grid	Nested in	Delft3D modules used
Large nest	-	Wave
Small nest	Large nest	Wave
Fine grid	Small nest	Flow & Wave

Now that the grid is defined, a depth file or bathymetry can be created. This can be done in the section QUICKIN. First a land boundary is made to indicate the water-land border. Then the data from the bathymetric survey, which contains all the depth information, can be imported. Together with the grid a bathymetry can be created using triangular interpolation between all data points. In QUICKIN also all breakwaters can be implemented. This is done by adding thin dams, which block all flow through them. In the Wave module of Delft3D the breakwaters are also given their realistic wave-breaking capabilities.

Boundary conditions

Inside the Flow module the data is combined with the bathymetry created in the previous section. At the boundaries of the grid the tidal data can be imported. The north, west and south boundaries are connected to the sea, so in order to get the tidal variation, shown in Fig. 6.19, inside the model the tidal information is assigned to the west boundary. To the north and south Neumann boundaries are applied which imposes a water level gradient as a function of time. The Neumann boundaries are both set to zero. This value is justifiable since the scales on the fine grid are much smaller than the tidal wavelength.



(a) Significant wave height under daily conditions.

(b) Flow velocities under daily conditions.

Figure 6.20: Results daily conditions model.

Results

The results of the daily conditions model was in line with the observation (Appendix A) done at the project site. The conditions are very mild and not interesting enough to design a breakwater for. The resulting magnitudes of the significant wave heights and flow velocities are shown in Fig. 6.20.

The fine grid of the modelled area (see Fig. 6.20) is used in all of the further models with the exception of the model discussed in Section 6.2.2.

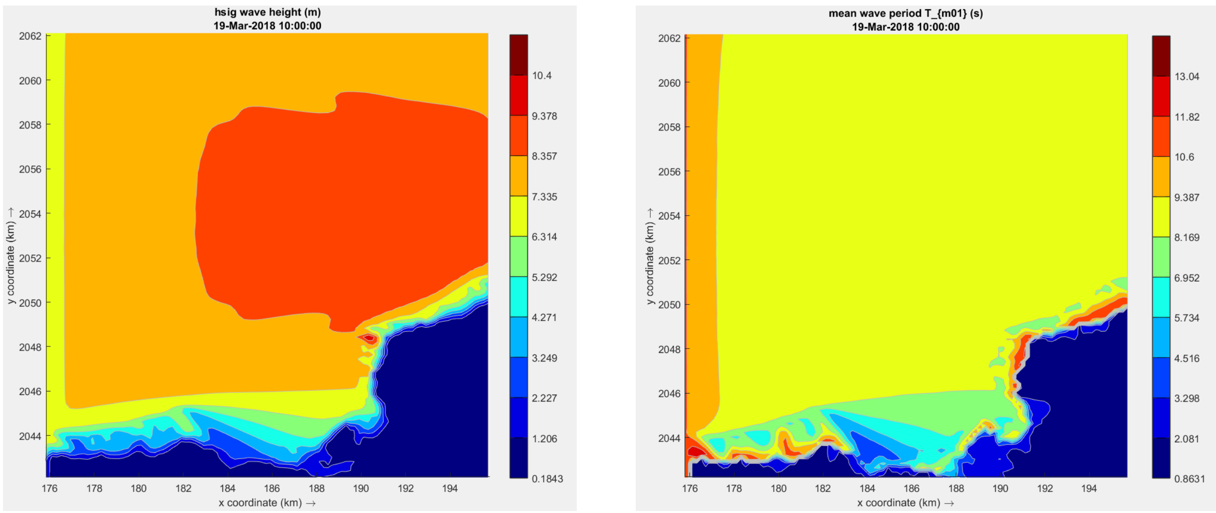
6.2.2 Hurricane Wave model

In order to determine the design conditions for the structure, the model is simulated with data from HurWave. The modelling is done for both waves and wind, for a hurricane with a return period of 50 years and different directions. To keep the simulation time down, first a coarse grid is used to get a look at wave heights from different directions. Resulting pictures can be found in Appendix D.4. In Tab. 6.10 the results are shown. These results are taken from a point at 1 km offshore. The direction with highest resulting significant wave height will be used in a detailed model run, to look at the wave heights at the structures at the site location. It would be superfluous and time consuming to run all the directions in a detailed model run.

Table 6.10: Wave results for different hurricane directions.

Hurricane direction	**Sign. wave height** [m]	**Wave period** [s]
West	6.8	7.6
North West	6.2	7.6
North	6.6	7.2
North East	5.8	6.1
East	2.8	3.6

As can be seen from Tab. 6.10, the highest waves occur with a hurricane from the west. Now this data is used in Delft3D with a fine grid to model wave conditions near the structure. The results can be seen in Fig. 6.22. The flow velocities and waves at locations where land is supposed to be, can be explained by the water level set-up due to storm surge, drop in atmospheric pressure and wind set-up. Combined these are enough to flood the beach area. The modelled significant wave height H_s just before the breakwaters is around 4.2 m. The highest flow velocities occur at the tips of the breakwaters and over the submerged breakwaters and are in the range of 1.4 $\mathrm{m\,s^{-1}}$.



(a) Significant wave height western hurricane 1/50 years.

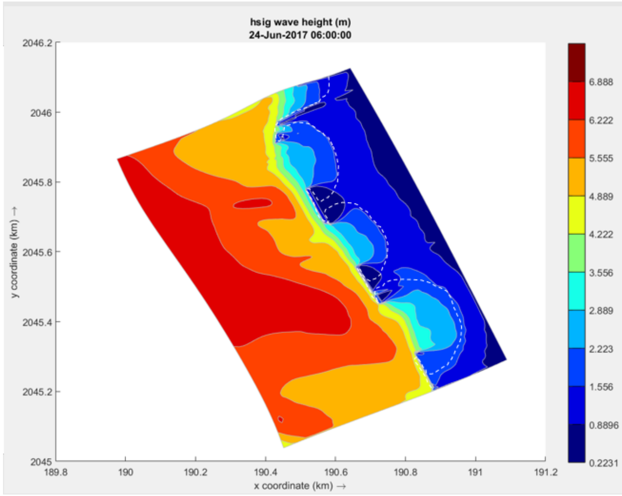
(b) Mean wave period western hurricane 1/50 years.

Figure 6.21: Wave data from western hurricane.

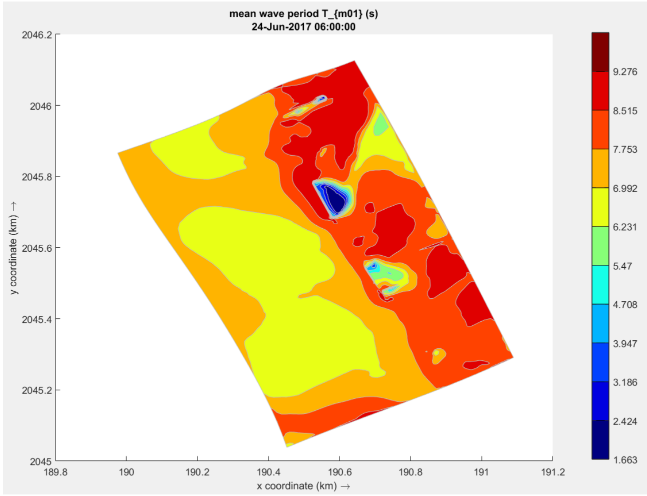
6.2.3 Detailed hurricane model

After the model from section 6.2.2, a detailed model is ran on the fine computational grid. In this model the small nest grid (with the wave boundary conditions) and the fine grid are used. The wave climate resulting from the last model ran are used as input in this model.

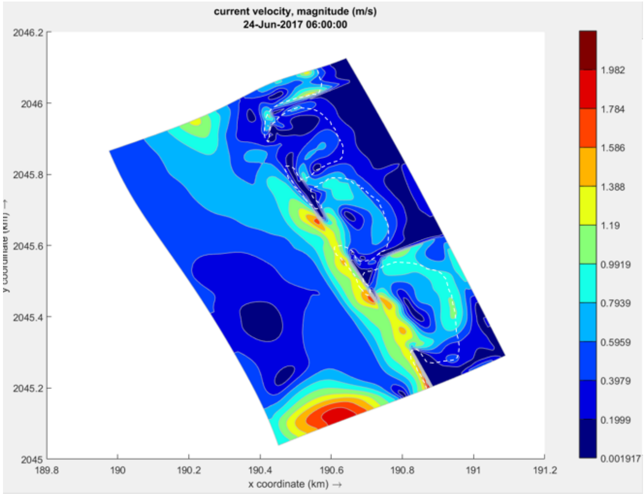
The resulting significant wave height and flow velocity can be read from Fig. 6.22. The governing values can be found in the accentuated area's in figures. The numbers of these governing conditions can be found in Tab. 6.11. These results are used in the calculation of the forces on the modular submerged breakwater structure (see Appendix E).



(a) Overview significant wave height.



(b) Overview mean wave period.



(c) Overview flow velocities.

Figure 6.22: Resulting hydrodynamic and wave conditions following an one-in-fifty years hurricane from the west.

Table 6.11: Resulting conditions used for the structural analysis.

Significant wave height	3.0	m
Mean wave period	8.0	s
Flow velocity	1.4	$m\,s^{-1}$

6.2.4 Breakwater design/sediment model

To look at the effects of the submerged breakwaters, three different lay-outs will be modelled (see Fig. 6.24). Since the daily conditions modelled in Section 6.2.1 are not interesting to design a breakwater for, a new reference case is taken of an once in a year storm to function as new design conditions. The new conditions for the breakwater model, which are located off-shore from the project's location, can be seen in Tab. 6.12. The same storm surge is assumed as it would occur in an one in fifty year hurricane. the Fig. 6.23 visualizes the results of first control run.

Table 6.12: Used conditions for one year storm/breakwater model.

Significant wave height	2.5	m
Mean wave period	8.0	s

Then three lay-outs of breakwater placements are tested to check their effectiveness. First three long submerged breakwaters are implemented in Delft3D. In the second and third run, multiple smaller submerged breakwaters in different orientations will be modelled. For more information see Appendix D.5.

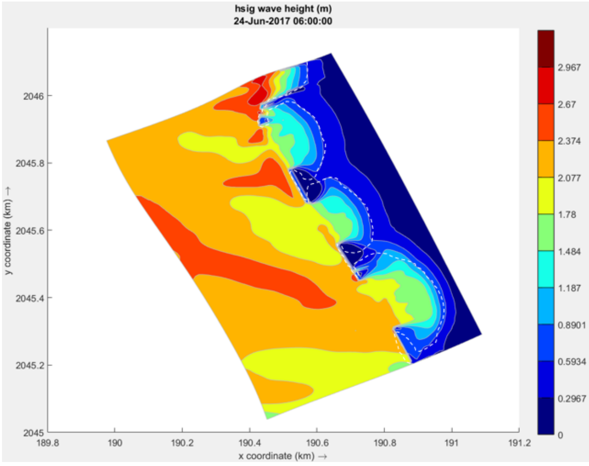


Figure 6.23: Significant wave height for an one year storm.

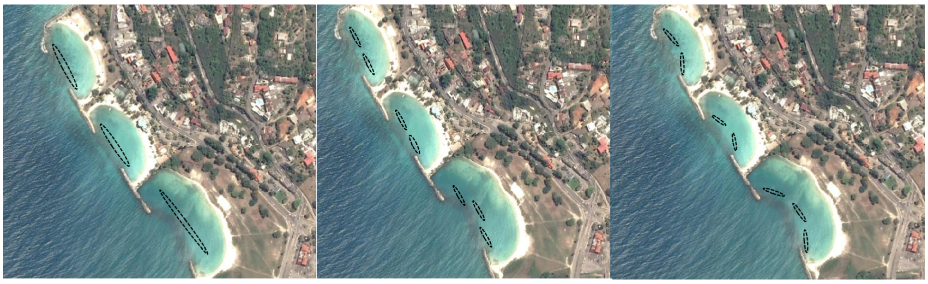
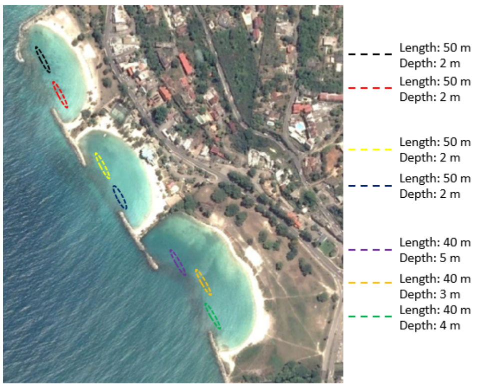
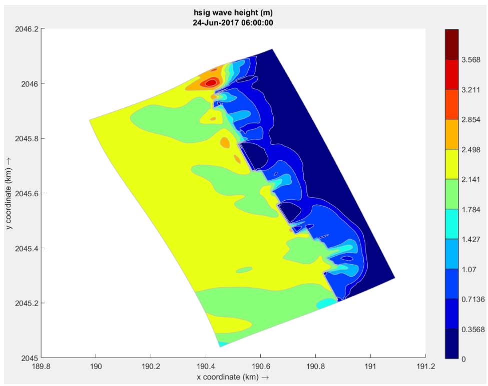


Figure 6.24: Three different submerged breakwater lay-outs [Google 2018]. Left: lay-out 1. Center: lay-out 2. Right: lay-out 3.

Beach erosion due to wave attack will determine the allowable transmitted wave over the submerged breakwater. The wave attenuation factor K_t is calculated according to Eq. 3.2. A balance has to be found between overall volume and wave attenuation, while still meeting the criteria for beach stability. In practice a maximum wave attenuation of 50 percent can be reached. The model run with the breakwaters (all lay-outs) included, will attenuate the waves with the given percentage. The results for one lay-out, number two, is given in Fig. 6.25. The results of the other lay-outs are shown in Appendix D.5.



(a) Lay-out 2 [Google 2018].



(b) Wave heights breakwater lay-out 2.

Figure 6.25: Results breakwater lay-out 2.

With elements from the three lay-outs, one final design will be made and presented chapter 9. The resulting dimensions of all breakwaters will also be presented, following from Tab. D.11. In the same chapter attention will be given to the difference in erosion at the site caused by the new implemented breakwaters. To do this, two new runs are performed with sediment transport enabled, one of the control run (no breakwaters) and the final breakwater design. The nominal diameter (d_{n50}) used in these model runs is 300 µm.

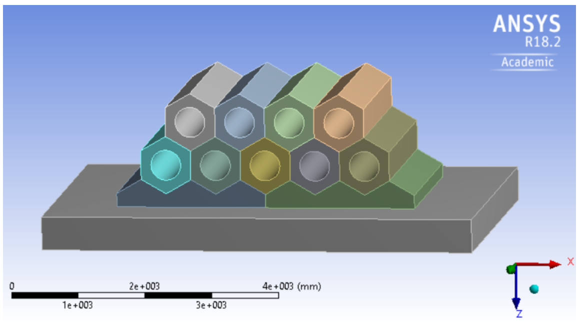
Chapter 7. Structural analysis

A structural analysis is performed to evaluate the concepts in terms of strength and stability. First, the properties, dimensions, forces and boundary conditions of the analysed constructions are given. Next, the difference in behaviour between the hexagonal and triangular construction is analysed. Conclusions with respect to stability and strength are drawn and one the most viable concept is analysed further in detail. The chapter continues with a sensitivity analysis with regard to the friction coefficient μ, the force-time profile, the placement accuracy and dimension deviations. The fourth section of this chapter covers the structural design of the blocks and base itself. The final dimensions and the reinforcement layout will be calculated based on hand calculations and finite element models.

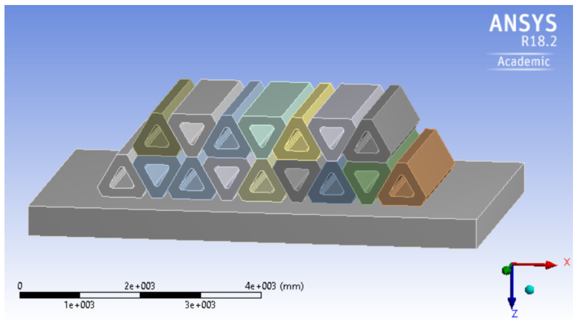
7.1 Properties, dimensions and weight of model

7.1.1 Dimensions

The dimensions of the individual blocks used for the analysis in ANSYS are the same as in the last Chapter Concepts (Tab. 5.5 and Tab. 5.6). As the dimensions of the blocks and the complete structure are nearly equal, the two concepts can be compared. The hexagonal structure has a height of 1.72 m and a crest width of 3 m. The triangular structure has a height of 1.43 m and a crest width is 3.69 m. To make a fair comparison, the forces are represented by pressure values, which take into account the exact geometry. In the ANSYS geometry, no fish holes (holes along the long edge) are present. The influence of those holes is expected to be low from both weight, stability and strength perspective. Excluding those holes from the geometry makes the models much less computational expensive. The modelled geometries can be seen in Fig. 7.26.



(a) Hexagonal construction model.



(b) Triangular construction model.

Figure 7.26: Constructions as modelled in ANSYS.

7.1.2 Materials

Concrete

The concrete type chosen for the analyses is a low strength class which is widely available. This class is chosen, to be able to produce the block with widely available concrete. The concrete properties used are given in Tab. 7.13.

Table 7.13: Properties of concrete.

Density	2400	$kg\,m^{-3}$
Young's modulus	30	GPa
Poisson's ratio	0.15	-
Tensile yield strength	2.9	MPa
Compressive yield strength	28	MPa
Tensile ultimate strength	2.9	MPa
Compressive ultimate strength	28	MPa

Seabed

From the site survey and local expertise, the soil conditions are determined. The seabed commonly exist of a thin layer of fine sand on a limestone foundation. However, as the sand layer is usually thin or the seabed is prepared before construction, the seabed is assumed to behave like limestone. The properties of limestone are taken from the ANSYS database and shown in Tab. 7.14. For the non-linear behaviour the Morh-Coulomb model is used.

Table 7.14: Properties of limestone seabed.

Density	2300	$kg\,m^{-3}$
Young's modulus	37.845	GPa
Poisson's ratio	0.3077	-
Initial inner friction angle	0.6	°
Initial Cohesion	88	MPa
Dilatancy angle	0.6	°
Residual Inner friction Angle	2.9	°
Residual cohesion	40	MPa

7.1.3 Boundary conditions

The seabed is fully fixed both at the bottom and at the sides. All the contact surfaces are modelled as friction planes. The friction coefficient of the blocks with the limestone base is set to $\mu = 0.3$ and the friction coefficient of block to block is set to $\mu = 0.3$ too. The concrete-seabed friction coefficient is conservative, as in literature values above $\mu = 0.4$ were found [10]. The coefficient of the concrete-concrete is set to $\mu = 0.3$, by reducing the dry concrete-concrete friction factor by 25% [11].

7.1.4 Forces

The forces acting on the construction are calculated in Appendix E. The wave period is set to eight seconds calculated in Section 6.2.4 resulting in a four second applied force.

- **Horizontal wave force**
  The horizontal wave force is equal to 10.5 kN per block. The force-time diagram is shown in Fig. 7.27. In the model, this force is represented by a pressure on the surface where the wave hits the structure. The pressure acts in the positive x-direction.
- **Uplifting wave force**
  The uplifting wave force on the top left block is equal to 10.5 kN and acts on the block where the wave hits the structure. The uplifting force is assumed to gradually decrease to half of the uplifting force at the top right block. The shape of the force-time diagram is the same as the horizontal wave force (see Fig. 7.27. The force acts in negative z-direction.

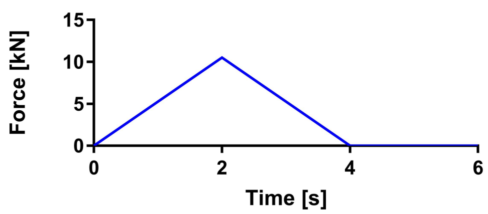


Figure 7.27: Force-time diagram of the horizontal and the uplifting wave force.

- **Gravitational force**
  The gravitational force acts from t=0 and is constant over time. The force acts in positive z-direction.
- **Hydrostatic pressure**
  The blocks are analysed in seawater with a water level of 0.5m above the top of the structure. The density

of seawater is set to $\rho_{\text{seawater}} = 1025\ \text{kg}\,\text{m}^{-3}$. Thy hydrostatic pressure is maximum at the seabed and decreases linearly to zero at the water surface.

7.2 Structural analysis

In this part, the structural behaviour is analysed. First, the triangular and hexagonal structure are compared and their behaviour is shown for the hurricane conditions. The most stable structure is chosen and a sensitivity analysis for this structure is performed. The sensitivity analysis will be performed for the friction coefficient μ, the force (amplitude and shape) and for placement errors and dimension deviations.

7.2.1 Comparison Triangular and Hexagonal construction

Strength

The stress in the structures remains well below the maximum tensile and compressive stress. The highest stress during wave impact occurs at the triangular structure (See Fig. 7.28). As the stress during waves is low, the strength of the blocks will be designed for lifting. In Section 7.4 , this is done for the chosen concept.

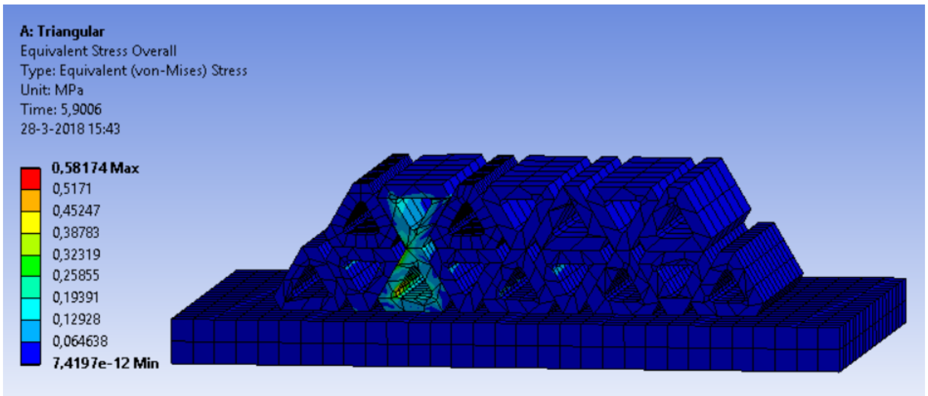


Figure 7.28: Maximum stress in concrete blocks during wave impact.

Stability

The stability of the structure can be analysed by looking at the displacement of the complete structure and of the individual blocks. The displacement of the hexagonal structure over time is displayed in Fig. 7.30. As can be seen, the displacements are small and the structure is stable.

The displacement of the triangular structure over time is displayed in Fig. 7.31. This construction behaves less stable compared to the hexagonal construction. This is also shown by a comparison of the displacement of the top left block in x-direction (see Fig. 7.29).

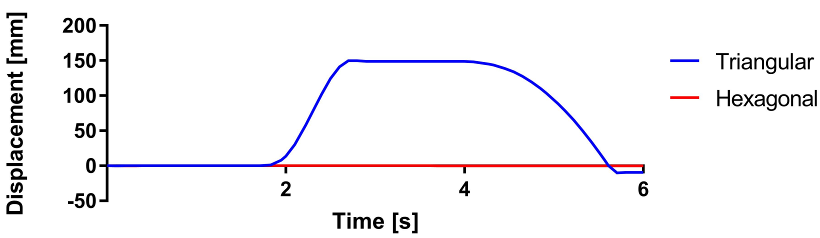
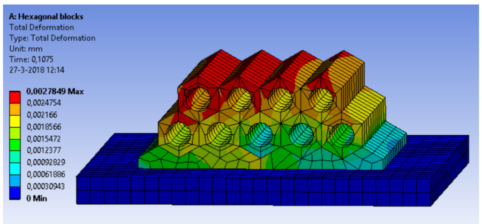
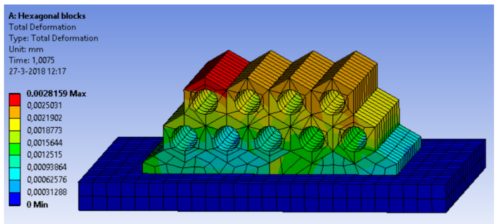


Figure 7.29: Horizontal displacement of the top left block for both the triangular and the hexagonal structure. The force-time diagram is shown in Fig. 7.27.

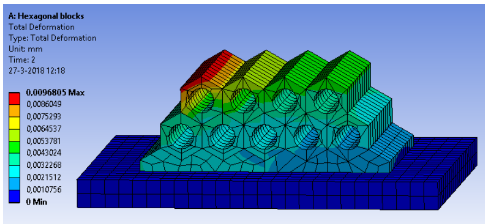
Although both structures remain intact, the triangular structure shows a bigger displacement. The structures have also been analysed with a higher force. For this force, the difference in behaviour became more clear. As the hexagonal structure behaves more stable, this is the preferred concept.



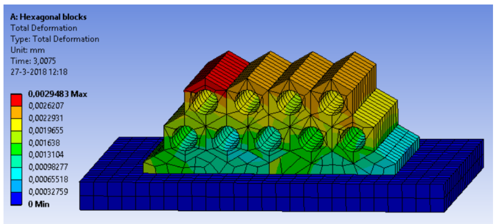
(a) Displacement at t=0 s.



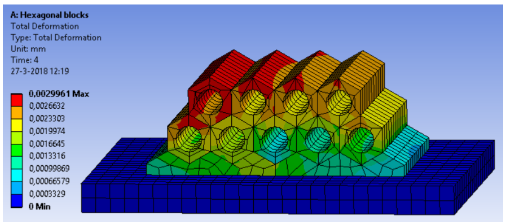
(b) Displacement at t=1 s.



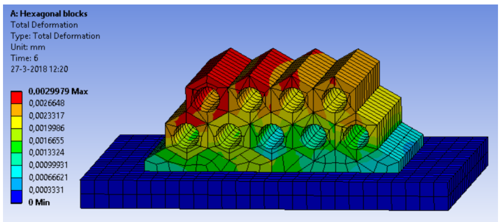
(c) Displacement at t=2 s.



(d) Displacement at t=3 s.

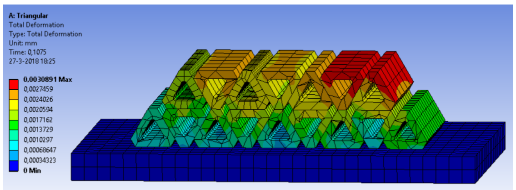


(e) Displacement at t=4 s.

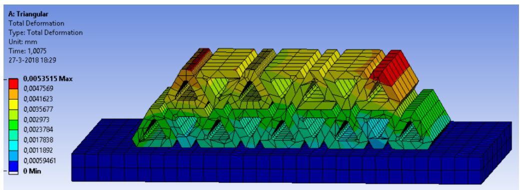


(f) Displacement at t=6 s.

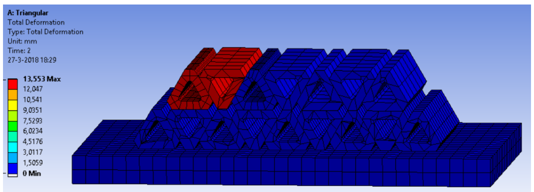
Figure 7.30: Displacement of hexagonal structure during loading. Forces: 10.5 kN horizontal wave force per block, 10.5 kN uplifting force per block, hydrostatic pressure (water level 0.5 m above structure) and gravity. As can be seen, the structure is stable and no failure mode can be seen.



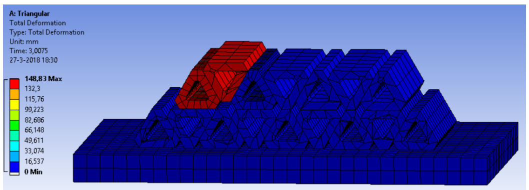
(a) Displacement at t=0 s.



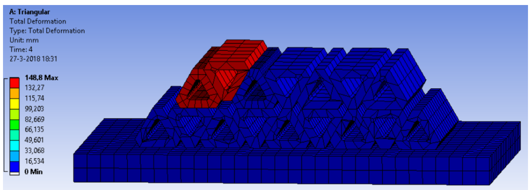
(b) Displacement at t=1 s.



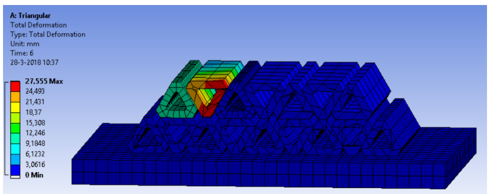
(c) Displacement at t=2 s.



(d) Displacement at t=3 s.



(e) Displacement at t=4 s.



(f) Displacement at t=6 s.

Figure 7.31: Displacement of triangular structure during loading. Forces: 10.5 kN horizontal wave force per block, 10.5 kN uplifting force per block (top left), hydrostatic pressure (water level 0.5 m above structure) and gravity. As can be seen, the structure is stable and does not fail. However, the failure mode becomes clear. The second left top block is pushed up.

7.3 Sensitivity analysis

Sensitivity analysis is performed to look at the behaviour of the hexagonal structure for different input parameters. As the value of the friction coefficient μ is not exactly known, the influence of different friction factors is analysed first. Also, the wave force could differ. The force-time function could be different and the amplitude of the force may be different. The last sensitivity analyses covers the influence of placement errors and dimension deviations. The analysed construction in the previous section assumes perfect stacking and dimensions, while this may not be the case in reality.

7.3.1 Influence of friction factor μ

To look at the influence of the friction factor, the hexagonal structure is analysed for four different friction factors. For friction factors of $\mu \geq 0.3$, the construction does not displace visually. The influence can clearly be seen by the horizontal displacement of the bottom right block (see Fig. 7.32). In this analysis the friction factor of block-block and base-seabed is changed together.

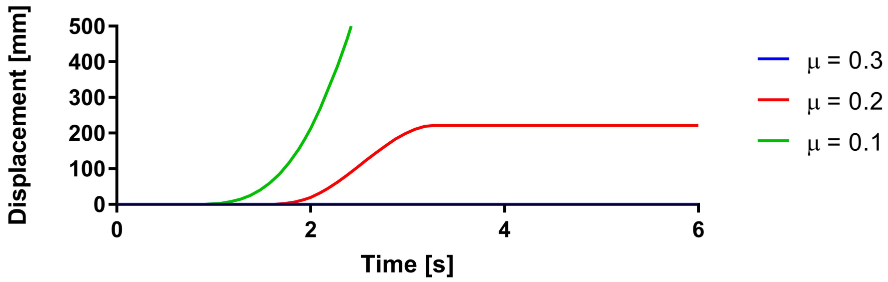


Figure 7.32: The sensitivity of the hexagonal construction for friction factor μ. The block-block and the base-seabed friction factor are changed together.

For both the $\mu = 0.1$ and $\mu = 0.2$, the failure mode is sliding of the total construction over the seabed. For $\mu = 0.2$, the structure displaces by 200mm, before finding a new equilibrium. Also, the behaviour of the construction is analysed for a friction factor of $\mu = 0.3$ for base-seabed, and $\mu = 0.2$ and $\mu = 0.1$ for block-block. For this friction coefficient values, the construction does not slide and the hexagonal blocks stay in place. Thus the friction coefficient between the blocks is not governing when the blocks are stacked perfectly.

7.3.2 Influence of force time function

The force-time function may be the most important and least known parameter. To look at the influence of a different force-time function, two different sensitivities are looked at. First, the shape of the function is changed to an impulse-like wave. Second, the wave profile is maintained, but the values are changed.

Shape of function

In section 7.2.1, the wave force was modelled with a symmetric wave profile. In reality, the wave may have a more impulse-like shape. The different force-time profiles and their influence on the deformations are visualised in Fig. 7.33. As can be seen, the wave profile has no influence on the displacements.

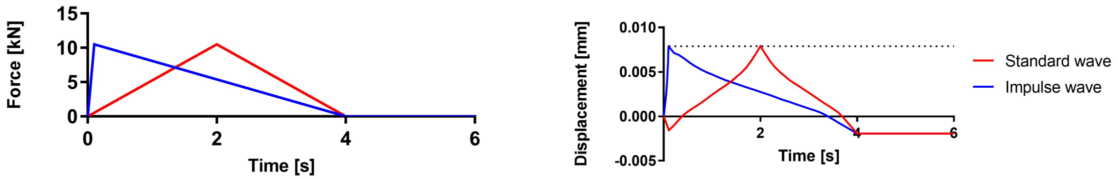


Figure 7.33: Horizontal displacement of top left block for wave different wave profiles. The maximum and final displacement are similar for both waves.

Amplitude of function

The amplitude of the force, influences the behaviour of the structure. When the wave forces are multiplied by a factor two, large displacements of the blocks can be seen and the structure is not stable. The stability loss, caused by the uplifting forces, is also found by simple hand calculations. The submerged gravity force of a hexagonal block is calculated by $F_g = V \cdot (\rho_{\text{concrete}} - \rho_{\text{seawater}}) \cdot g = 0.94 \cdot (2400 - 1025) \cdot 9.81 = 12.7\text{kN}$. When the uplifting force is multiplied by a factor 2, the maximum uplifting force is equal to $10.5 \cdot 2 = 21.0\text{kN}$. As the uplifting force is higher than the downward relative gravity force, the block will lift up. When the uplifting force is equal to 12.7kN, stability is still possible. The friction force caused by friction between the vertical surfaces, equal to $F_{\text{fric}} = N_{\text{horizontal}} \cdot \mu_{\text{concrete-concrete}}$, will add an extra downward force component. This helps to keep stability, but this contribution is to small to prevent instability for higher uplifting forces.

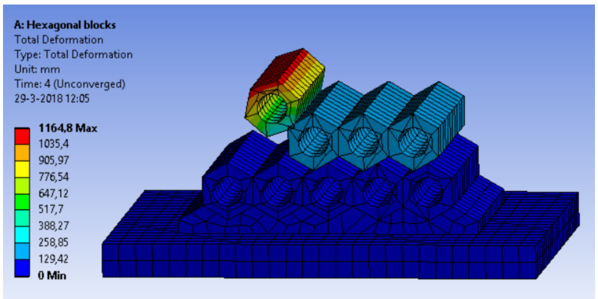


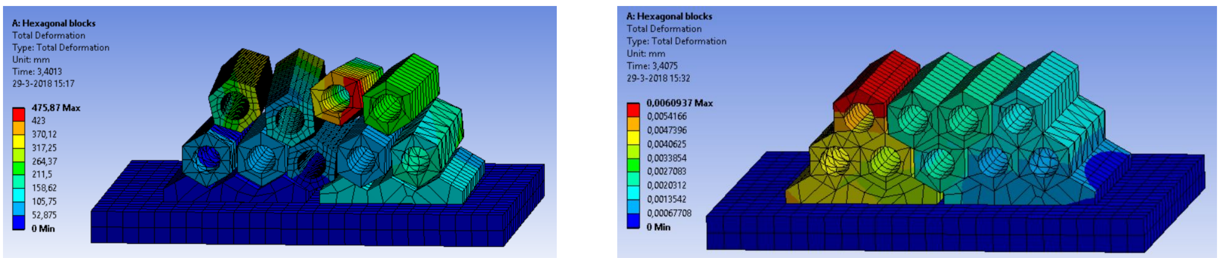
Figure 7.34: Failure mode of hexagonal construction with a force-time diagram of Fig. 7.27, with a doubled force value of both the horizontal and the vertical uplifting force.

7.3.3 Sensitivity to placement accuracy and dimension deviation

In the previous models, the blocks were perfectly stacked and no gaps between the blocks were present. Also, all blocks had exactly the same dimensions. This perfect situation will not be the case in reality.

Placement accuracy

To look at the influence of imperfect stacking, we modelled a situation with a (horizontal) gap of 100 mm between the base blocks. For this situation, the system becomes unstable and the structure will fall apart (see Fig. 7.35a). This instability problem can be solved by tying the top left block to the base (see Fig. 7.35b). In reality, this can be done with a cable. Chapter 9 will go more into detail about the connection.



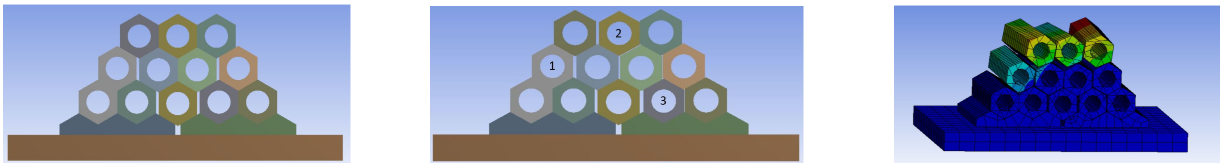
(a) No connection between the blocks. **(b)** Connection between top left and base block.

Figure 7.35: The difference between a structure with loose blocks and a structure with a connection between the top left and left base block. Both structures have an initial horizontal displacement of 100 mm between the base blocks.

The connection is modelled by making small bonded contact surfaces between the top left block, the one below and the base. After running the model, the stresses in the connection between the top left block and the one below is measured and multiplied by the contact surface area. This results in an force of 6.5 kN in the cable when two cables are applied from the top block to the base. This force is reasonably low and can easily be taken by a cable. For indication: an steel cable of 6 mm diameter would be sufficient ($A = \frac{F}{\sigma} = \frac{6500}{235} \approx 28\text{mm}^2 \rightarrow \varnothing = 6\text{mm}$).

Dimension deviation

Besides the placement accuracy the blocks can also differ in dimensions. There will be deviations of the blocks after casting. This will give shifts of the blocks and therefore open spaces between the blocks. In Fig. 7.36b a final lay-out is given with a displacement error of the base and blocks 1,2 and 3 have a total size deviation of 3 %. To show that in practice a loose top block is inevitable an extra layer of blocks is added which normally connects the top blocks if only base displacement is applied Fig. 7.36a. When displacement and deviation error occurs (see Fig. 7.36b) the top block becomes loose again. As shown in the analysis, this will reduce the stability of the structure. The tension connection is of great importance to overcome practical errors and to create a stable structure. The same failure mode appears as in Fig, 7.35a.



(a) Base dimension deviation. **(b)** Base + block dimension deviation. **(c)** Failure of Fig. 7.36b.

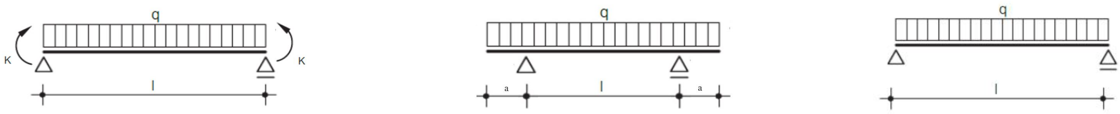
Figure 7.36: Fig. 7.36a shows a stable top layer, while Fig. 7.36b shows a loose top layer. This configuration leads to the failure mode of Fig. 7.36c.

7.4 Structural Design

For the final design, the reinforcement of the hexagonal blocks and base will be calculated. The calculation procedure of Eurocode 2 is used for the bending and shear check.

7.4.1 Forces

The maximum forces will be present during transport/lifting of the blocks and base. Checks for bending and shear are done in the critical cross-section shown in Fig. 7.39. The blocks and base are assumed to be lifted with a cable. The angle of the cable, taken as 60 °, is also taken into account for the blocks because of the large eccentricity. This gives an additional bending moment. To represent the load on the block, it is modelled as a beam on two supports loaded by its dead weight (q) and the external moment (k) at the support shown in Fig. 7.37a. The base is modelled as a plate on four point supports with a representative thickness in a finite element program. To check the outcome of the program, half of the base is also modelled as a beam on two supports (see Fig. 7.37b). For the base, a unit width of 1 m will be used in the calculations.



(a) Hexagonal block. **(b)** Base block X-direction. **(c)** Base block Y-direction.

Figure 7.37: Schematic of load on hexagonal blocks and base.

To take into account the dynamic/shocking behaviour during lifting, a dynamic factor γ_{dyn} of 1.6 is taken from Fig. 7.38. The safety factor γ_g for constant loading comes from the Eurocode and is equal to 1.35 for dead weight.

The value of q_{block}, $q_{halfbase}$ and k_{block} are given in Tab. 7.15. These values are calculated from the final designed cross-section and are determined in an iterative manner.

Table 7.15: Loads on model.

q_{block}:	7.4	$\mathrm{kN\,m^{-1}}$
k_{block}	4.81	$\mathrm{kN\,m}$
q_{base}	6.9	$\mathrm{kN\,m^{-1}\,m^{-1}}$

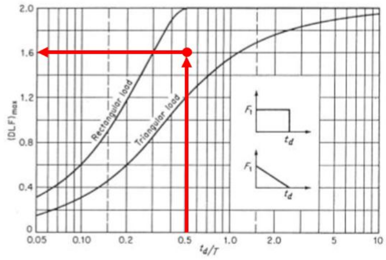


Figure 7.38: Dynamic load factor.

7.4.2 Hexagonal blocks

The critical cross-section for the shear force is A-A and the bending moment is critical in B-B shown in Fig. 7.39.

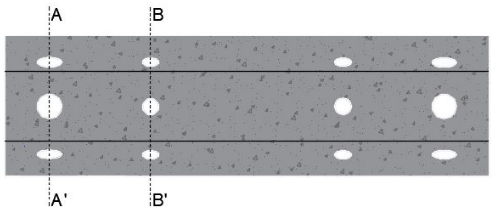


Figure 7.39: Critical cross-sections.

The maximum forces will not occur in the critical cross-section but to be conservative, the checks will use the calculated maximum values. The forces are determined with the following equations:

$$V_{ed} = \frac{q_{block} \cdot l}{2} \gamma_g \cdot \gamma_{dyn} \tag{7.3}$$

$$M_{ed} = (M_q + k_{block}) \gamma_g \cdot \gamma_{dyn} \tag{7.4}$$

$$\text{with} \tag{7.5}$$

$$M_q = \frac{q_{block} \cdot l^2}{8} \tag{7.6}$$

$$k_{block} = V_{ed} \tan 60 \cdot e \tag{7.7}$$

First the bending moment will be checked. This will give a value for the required reinforcement.

Bending moment check

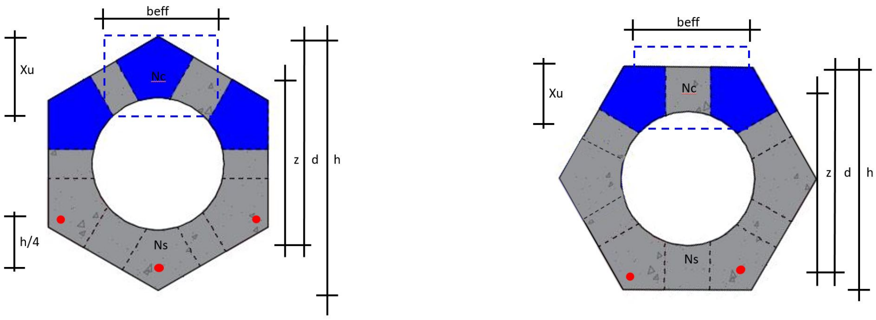
For calculation of the cross-section the height of the compression zone x_u is taken as 30 percent of the total height. Because of the holes the compression zone is reduced. The area of the potential compression zone is indicated as blue in Fig. 7.40a. For the calculation an effective area is calculated with the x_u and a b_{eff} representing the concrete compression zone during bending. The forces in the cross-section follow from the equations:

$$M_{ed} = (M_q + M_{ec}) \gamma_g \cdot \gamma_{dyn} \tag{7.8}$$

$$M_q = \frac{q \cdot l^2}{8} \tag{7.9}$$

$$M_{ec} = \frac{q \cdot l}{2 \tan 60} e \tag{7.10}$$

The formulas used for the calculation of the required reinforcement are given in Eq. 7.11. An overview of the cross-section calculation is given in Fig. 7.40 with all dimensions indicated. When the internal level arm is calculated, the minimum required amount of reinforcement can be determined by taking M_{ed} equal to $M_{rd,s}$. The reinforcement will be placed in every corner so that the blocks can be lifted in any orientation.



(a) During placement.

(b) During transport.

Figure 7.40: Lifting method.

$$d = h - c_{nom} - h_{gp,s} \tag{7.11}$$

$$x_u = 0.3 \cdot h \tag{7.12}$$

$$z = d - (x_u \cdot \beta) \qquad \text{with} \qquad \beta = 0.39 \tag{7.13}$$

$$M_{rd,s} = z \cdot A_s \cdot f_{yd} \tag{7.14}$$

$$M_{rd,s} \geq M_{ed} \tag{7.15}$$

$$A_{s,req} = \frac{M_{ed}}{f_{yd} \cdot z} \tag{7.16}$$

$$\rho_l = \frac{A_{s,tot}}{A_{c,block}} \tag{7.17}$$

The total reinforcement is calculated from de required $\varnothing$with a total of six longitudinal rebars in the block. The input and outcome of the final calculation for both cross-sections are given in Tab. 7.16. The checks gives that a minimal reinforcement $\varnothing$ of 9 mm is needed. This gives a practical $\varnothing$ of 10 mm which is used for the calculation of $A_{s,tot}$. The unity check with the final reinforcement plan is 0.71.

Table 7.16: Results bending check.

	Lifting-a	**Lifting-b**	
c_{nom}	65	65	mm
h	866	750	mm
d	657	685	mm
x_u	260	225	mm
b_{eff}	228	264	mm
z	556	597	mm
e	433	375	mm
M_{ed}	30.83	29.1	kNm
$A_{s,req}$	127	112	mm^2
$\varnothing_{min,s}$	8	9	mm
$A_{s,tot}$	302	471	mm^2
ρ_l	0.13	0.20	-
$M_{rd,s\varnothing10}$	56.96	40.81	kNm
UC	0.54	0.71	-

To check the hand calculations and the influences of the holes, a 3-D model analysis is performed (see Fig. 7.41). The maximum tensile stresses are very low. In theory the tensile concrete capacity can take up the stresses if the concrete has no cracks or imperfections. For safety, the calculated minimum reinforcement is applied.

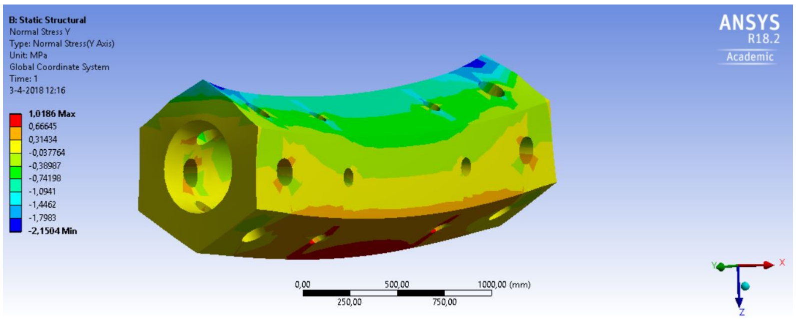


Figure 7.41: Normal stresses Y-direction.

Shear force check

First the capacity of the block without shear reinforcement will be checked with the formula's from Eurocode 2 (Eq. 7.18-7.24). Also for the shear check an effective area is assumed. The shear forces have to be taken by the web of the blocks. To look at the influence of the holes, the maximum shear stresses will be calculated in a static analysis in ANSYS.

$$V_{rd,c} = C_{Rd,c} \cdot k(100 \cdot \rho_l \cdot f_{ck})^{\frac{1}{3}} b_w \cdot d \tag{7.18}$$

$$V_{min,rd,c} = v_{min} \cdot b_w \cdot d \tag{7.19}$$

$$\tag{7.20}$$

with

$$C_{Rd,c} = \frac{0.18}{\gamma_c} \tag{7.21}$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \tag{7.22}$$

$$\rho_l = \frac{A_{s,tot}}{b_w \cdot d} \leq 0.02 \tag{7.23}$$

$$v_{min} = 0.0035 \cdot k^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} \tag{7.24}$$

The final results of the calculations are given in the Tab. 7.17.

Table 7.17: Results shear check.

V_{ed}	24	kN
d	433	mm
b_w	295	mm
$C_{Rd,c}$	0.12	-
k	1.68	-
ρ_l	0.00236	-
v_{min}	0.34	MPa
$V_{rd,c}$	43.20	kN
$V_{min,rd,c}$	43.54	kN
UC	0.56	-

The minimum shear capacity is given by $V_{min,rd,c}$ which gives a unity check value of 0.56 which is safe, so no shear reinforcement is needed. For practical reasons there will be stirrups added to the blocks. The finite element model gives shear stresses around the edges of the holes. The forces on the block are the design values from Section 7.4.2. In Fig. 7.42 it can be seen that the maximum shear stress is 0.44 MPa locally around the holes and boundary conditions. Over the complete block the maximum shear stress does not exceed the shear capacity of the concrete. To take into account the local shear stresses, the stirrups will be placed next to the holes to redistribute the local shear stresses away from the holes.

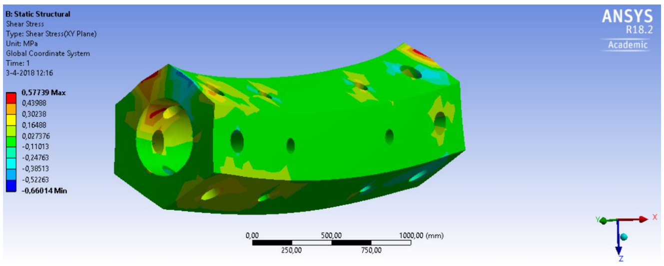


Figure 7.42: Shear stress.

7.4.3 Base

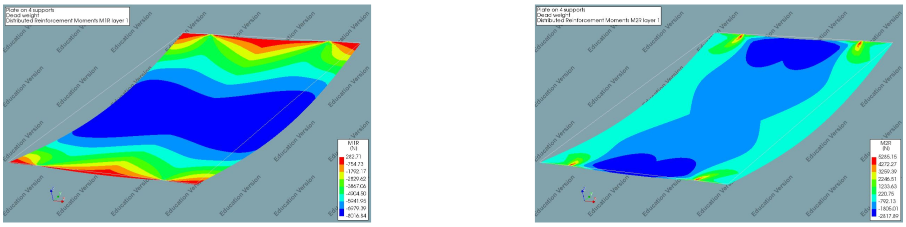
The calculation of the base is based on the K-method from the British Standard BS 8110 [12]. This method is used to design the reinforcement in plates. The lifted base is represented by a plate on 4 supports. The forces are extracted from finite element calculations in DIANA.

Bending moment check

The design value of the bending moment in plates is a combination of the bending and twisting moments and can be calculated with Eq. 7.25 and Eq. 7.26. Finite element program DIANA gives these values as reinforcement moments $M1R$ and $M2R$. The maximum moment in the two directions is taken to keep the reinforcement plan easy to construct. To check the output of the finite elements program, quick hand calculations are done. To take into account the sawtooth edges an overall rectangular cross-section with the same area is used. The holes are of little influence on the total weight and thus neglected. The effective height h_{eff} of the base is determined as 293.25 mm. The same concrete properties are used in DIANA and ANSYS.

$$M_{ed,x} = M1R = (M_{xx} + |M_{xy}|)\gamma_g\gamma_{dyn} \tag{7.25}$$

$$M_{ed,y} = M2R = (M_{yy} + |M_{xy}|)\gamma_g\gamma_{dyn} \tag{7.26}$$



(a) M1R in DIANA. **(b)** M2R in DIANA.

Figure 7.43: Design moments in base.

In Fig. 7.43b there is a singularity at the point support. The model gives a high peak bending moment which can be redistributed over a certain length. To check, a hand calculation is done which gives a design bending moment of 1.05 kNm. With an iterative method the height of the smallest cross-section of the base is calculated. For calculation, the general formula's for the K-method are used and given below.

$$d = h - c_{min} - \frac{3\varnothing}{2} \tag{7.27}$$

$$K = \frac{M_{ed}}{b \cdot d^2 \cdot f_{ck}} \tag{7.28}$$

$$K' = 0.49\delta - 0.14\delta^2 - 0.18 \qquad \text{with} \qquad \delta = 1.0 \tag{7.29}$$

$$z = \frac{d}{2}(1 + \sqrt{1 - 3K}) \tag{7.30}$$

$$A_s = \frac{M_{ed}}{f_{yd} \cdot z} \tag{7.31}$$

$$A_{s,min1} = \frac{0.26 f_{ctm} \cdot b \cdot d}{f_{yk}} \tag{7.32}$$

$$A_{s,min2} = 1.25 A_s \tag{7.33}$$

$$A_{s,max} = 0.04 A_c \tag{7.34}$$

$$A_{s,req} = \min(A_{s,min1}, A_{s,min2}) \leq A_{s,max} \tag{7.35}$$

$$M_{rd,s} = z \cdot A_{tot} \cdot f_{yd} \tag{7.36}$$

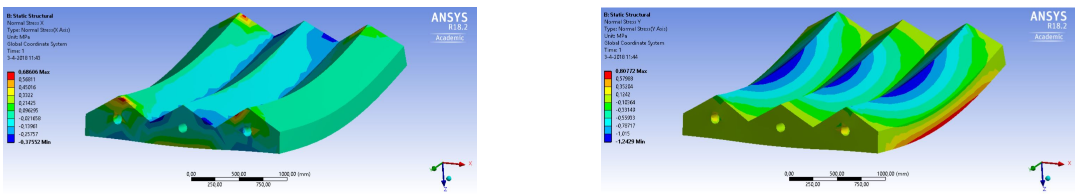
$$\tag{7.37}$$

The final results from the check are given in Tab. 7.18. The values are per meter width.

Table 7.18: Results bending moment check

	X bottom layer	**Y bottom layer**	**Y top layer**	
b	1000	1000	1000	mm
d	80	80	80	mm
M_{ed}	17.28	6.26	1.05	kNm
δ	0.9	0.9	0.9	-
K	0.135	0.049	0.008	-
K'	0.159	0.159	0.159	-
z	70.85	74.06	79.50	mm
$A_{s,req}$	701	234	105	mm^2
$\varnothing_{min,s}$	10	10	10	mm
$Spacing$	100	205	410	mm
$A_{s,tot}$	785	393	196	mm^2
ρ_l	0.268	0.134	0.067	-
$M_{rd,s}$	24.21	13.14	6.79	kNm
UC	0.74	0.48	0.15	-

The base is reinforced in a conservative manner with a maximum unity check of 0.74. During transport or lifting is it unlikely that the base will fail. The strains in the reinforcement will be low which is positive for the cracking of the concrete. The reinforcement to take over the positive bending moments in the top layer is the same as the stirrup reinforcement. An overview of the normal stress distribution in X-direction (see Fig. 7.44a) and Y-direction (see Fig. 7.44b) shows that the stresses are low when cross-section shape is taken into account.



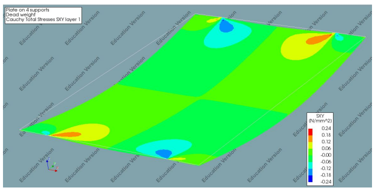
(a) Normal stress X-direction.

(b) Normal stress Y-direction.

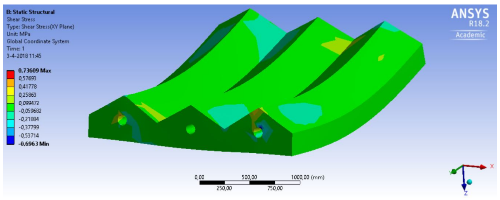
Figure 7.44: Design stresses in base.

Shear force check

Generally shear forces are not governing in slabs. The large cross section combined with the shear capacity of concrete shows no shear reinforcement is necessary. For placement of the bending rebars, practical reinforcement will be placed which also acts as stirrups and can distribute shear forces. Even with the singularities in the model the maximum shear stress do not exceed over 0.24 MPa shown in Fig. 7.45a. The minimum shear strength of concrete calculated in Section 7.4.2 is 0.34 MPa. The assumption of no shear reinforcement is correct. Only locally next to the hole high shear stresses are present, which arise due to the modelling. The stresses from both models are equal which is an extra check to verify the results of the model.



(a) DIANA.



(b) ANSYS.

Figure 7.45: Shear stresses in base.

7.4.4 Crack width

The construction is not checked for cracking. The reinforcement is only required during lifting. If after many years the reinforcement is affected due to cracking of the concrete, the structural integrity will be sufficient because reinforcement in not required when it is placed. Besides, the concrete cover is taken maximal to increase the lifetime of the structure. Also the permeability of the concrete is reduced to take durability into account. For all these reasons detailed crack width calculation is disregarded in this report.

Chapter 8. Concrete mixture design

For every concrete structure a special mixture have to be designed with special properties. For this concept several aspects such as environmental classes and concrete properties influences the mixture. A brief explanation about those aspects is given in this chapter. Extra information about the topic can been found in appendix G.

8.1 Environmental Classes Criteria

The construction will be placed in seawater. Therefore environmental classes **XC** and **XS** have to be taken into account. **XC** is **C**arbonation Initiated Corrosion and **XS** stands for Chloride Induced Corrosion by **S**ea water. The classes results in maximum criteria for the concrete mixture. The most important is the concrete cover which should be minimal 65 mm. The water/cement (W/C) ratio can be maximum 0.45 and 0.55 when air entraining agents are used for reinforced concrete. A lower value of 5% for the W/C ratio is advised for the sake of safety. The minimum amount of cement/binder recommended for a seawater environment (class 4) is 280 kg/m3. The amount of air entrainment agents depends on the aggregate size. With the known aggregate size the minimum amount of fine material can be calculated. Values can be found in Appendix G.1.1.

8.2 Special Requirements

From the survey done in Chapter 4 some benefits are found which can be implemented into the concept. By adjusting the concrete mixture some of those benefits can be added. Also a short notice is given regarding the climate with respect to the casting procedure. At last a research is done about the use of non-corroding reinforced fibres.

8.2.1 Casting in warm climate

The hardening process of the concrete is critical for the level of quality. Warm climate can have a large influence on it. Therefore special consideration are of great importance. Plastic shrinkage and hot steel formwork due to the sun can give temperature difference inside the mixture which can lead to cracks. Additives can overcome some problems like evaporation of the water which effects the workability of the mixture.

8.2.2 Concrete surface

The surface of the concrete is an important factor for settlement of the coral larvae. It should be rough and have small voids. This can be managed by using retarders on the molds and using air entrainments. By rinsing of the unhardened concrete layer after de-molding, which can be done because of the retarders on the formwork, the aggregates and voids from the air entrainments will be exposed. This gives the blocks a rough surface. When a rough surface is created new coral can settle more easier on the concrete blocks which enhances the coral growth.

8.2.3 Low pH

The pH-value of the concrete is an important property which can make or break the ecosystem enhancement of the structure. The pH after hardening is around thirteen. Because of the degradation processes, this will decrease at the surfaces. When a hardened structure is placed in the sea directly after casting, the surface still has a high pH-value. Carbonation will take place and the calcium hydroxide will slowly leach out in the seawater. This changes the pH of seawater surrounding the structure. To many types of marine life, the high pH-value is toxic and settlement of coral on the structure is retarded. Species which are resistant to pH changes like barnacles, will settle on the structure. If after a couple of weeks the concrete surface is fully carbonated and the pH drops to the seawater value of 8.3-8.9, the coral can not settle any more on the surface because the places are taken by unwanted species. In Fig. 8.46 the carbonation process is shown of the surface. In the first days the process is the fastest and therefore proper handling in the first days is important.

Dropping the pH-value can be done in two ways. Adding Silica fume and 'curing' the blocks. By adding silica

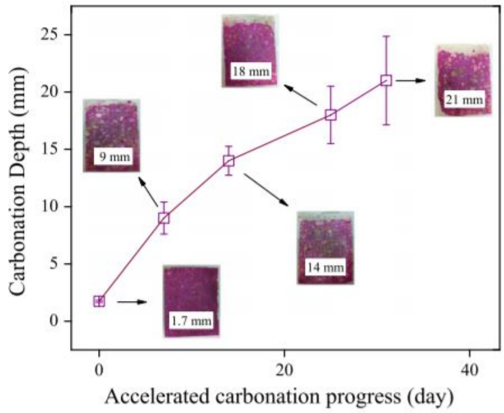


Figure 8.46: Carbonation concrete surface.

fume the pH-value of your total mixtures drops with 0.4 when 10% of cement is replaced with silica fume [13]. This is not sufficient and when reinforcement is used a low pH-value leads to corrosion of the reinforcement. The benefits of adding 10% of silica fume is that it increases the strength with 25% and give the concrete a significantly lower permeability. The curing of the blocks is done by storing the blocks in a humid area. This accelerates the carbonation process at the surface. Advised is to have a carbonated surface layer of 20 mm to make sure most of the toxic lye is gone. This can be achieved by storing the blocks for approx. 40 days preferably in a windy environment with a relatively humidity of 58%. Simple checks can be performed to see in the concrete surface has a natural pH-value.

8.3 Fibre Reinforcement

During the reinforcement calculation, it became clear that the forces on the blocks are relatively low. To be safe the blocks are reinforced. Non-corroding fibre reinforcement, found in the Reef Balls, can be interesting to be used for the blocks. Replacing the reinforcement with polymer fibres can have great benefits itemized below.

- Concrete thickness can be reduced, because no cover is needed. The total amount of material will be lower which can reduce the cost of the block. Advised is that the submerged weight is larger than the maximum uplifting force of 4.67 kN/m2.
- No reinforcement have to be fabricated and placed, which saves labour hours and material use.
- Casting the blocks without reinforcement will be easier. For example the vibration needle can easier be lowered into the formwork. Also no major errors can occur like failure/displacement of the reinforcement basket and cover.
- Durability of blocks will increase. Fibre reinforced concrete has more resistance to impact loads, abrasion and cracking. The tensile and bending strength increases 2.5 times when 3-4% of the total volume is fibres. Corrosion of the reinforcement is not possible.
- The cost will be reduced. The additional cost of the fibres is approx. 80 dollar per cube of concrete when 3% of fibres are used [14]. Assumed is that this is lower than fabricating and placing a reinforcement basket in the concrete.
- More holes can be added to the blocks because there is no limitation due to the cover of the stirrups. Extra calculation has to be done to check the maximum stresses do not exceed the fibre reinforced concrete capacity.
- Reduce the cracking of the concrete during the green stage as well as the final stage.
- PH-value can be decreased in the total concrete mixture. Curing time can be reduced when more silica fume is added.

Before using fibre reinforced concrete, special attention should be given to the stresses in the blocks. Especially for the lifting and the introduced stresses at the connections. For the lifting a different method can be used. When the blocks are lifted with a rigid bar/pipe through the inner hole, no bending stresses will occur. The stresses in the block with this lifting method are very low (see Fig. 8.47). The maximum von Mises stress is 0.078 MPa due to standard gravity. Multiplied by the safety and dynamic factor, the maximum design stress is 0.17 MPa which can be taken easily by fibre reinforced concrete. Because the stresses in the blocks during wave impact are also low and the fibres improve the impact resistance, it is looks like the blocks with fibre reinforcement will be structurally sufficient. Although the stresses from the connections can be governing. Further research is advised in the local behaviour when the final connections are designed.

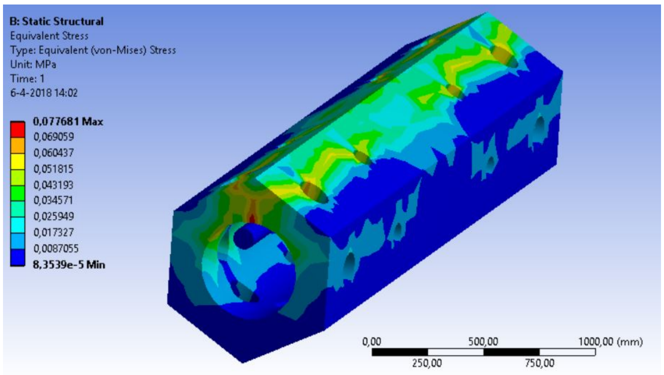


Figure 8.47: Von Mises stresses of block with the beam lifting method.

Final design of fibre reinforced concrete will not be done. It can be an alternative for the reinforcement. In this report the final structural design is done for normal reinforced concrete.

8.4 Conclusion

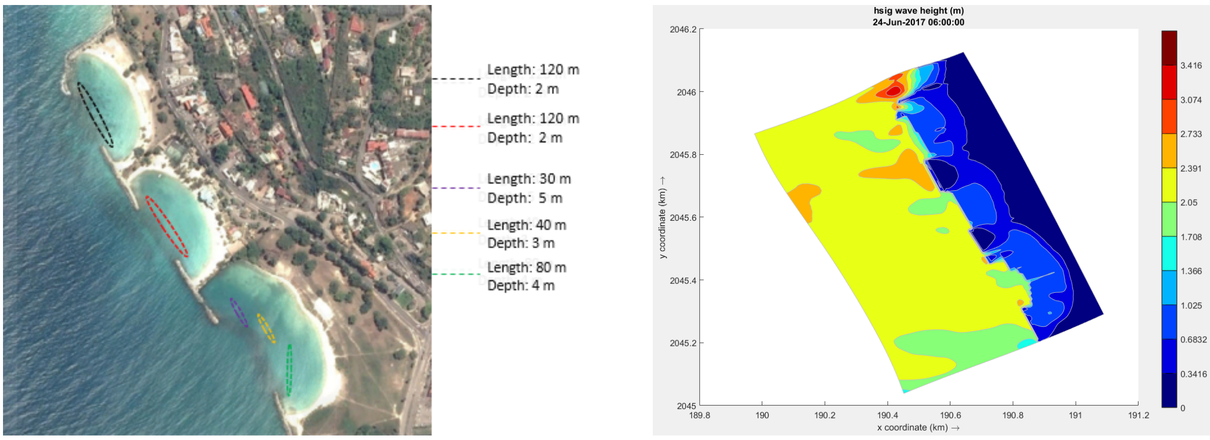
By taking into account specific measurements for the concrete mixture, the durability of the structure can be enhanced. Decreasing the permeability is critical for structures in seawater to resist the environmental attacks. This can be done by replacing 10% of cement with silica fume and by adding air entrainment agents. To make settlement possible for new coral colonies the surface is important. By curing the element, the concrete surface will carbonate and reduce in pH-value close to the seawater. This reduces the leaching significantly. By creating a rough surface with sugar water(retarder) and also air entrainment agents, coral larvae attachments is more likely on the elements. The mixture design and treatment of the concrete after casting is important to make the structure a success for enhancement of the aquatic life. Also fibre reinforced concrete can be interesting for the blocks.

Chapter 9. Final design

The structural analysis (see Chapter 7) showed the hexagonal concept is most promising, seen from a stability perspective. Because of that, this chapter will cover this design more into depth. The overall layout of the structure in Montego Bay and the influence on the wave heights and erosion will be shown. Next, the number of blocks and the reinforcement design of the blocks and base will be given. Also, the formwork, casting and connections will be elaborated on. The chapter ends with an analysis of the construction method and the total cost of the project.

9.1 Overall plan

Based on the results of the different submerged breakwater configurations, a final lay-out is made. This combines al the positive outcomes of the three lay-outs described in Section 6.2.4. It was observed that for the north and middle bay the best lay-out is with a continuous submerged breakwater over the opening just behind the existing sill (lay-out 1). The depth is pretty constant over the length, which gives little complications during construction. For the southern bay at Dump-Up Beach, a combination of lay-out 2 and 3 is used. The most southern breakwater is placed perpendicular to waves coming from the west and extended to just behind the middle breakwater. Fig. 9.48a shows the complete final lay-out. Fig. 9.48b shows the resulting wave heights with this lay-out. Compared to the original situation, the waves at the beach are reduced with 0.75-1.0 meter.

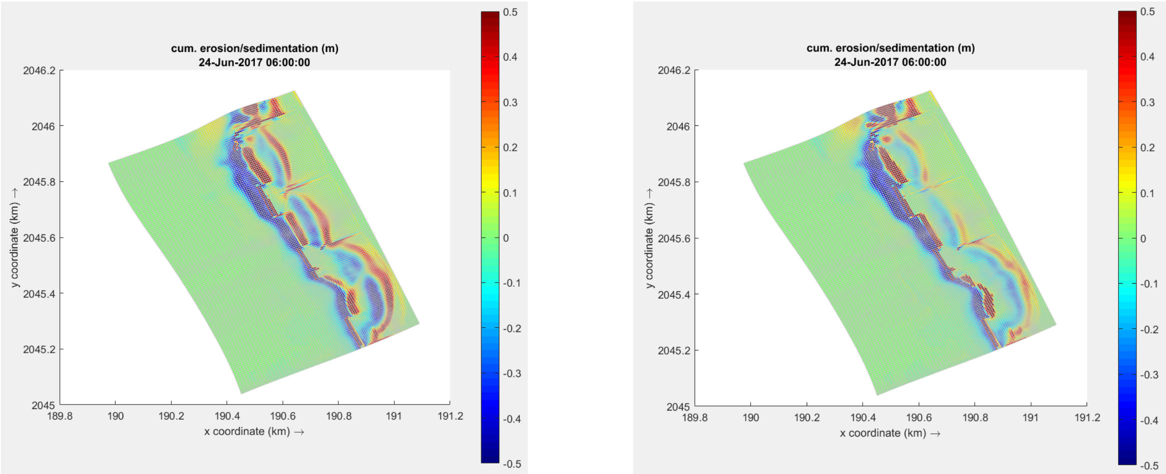


(a) Final lay-out [Google 2018].

(b) Wave heights final lay-out.

Figure 9.48: Final lay-out of the breakwaters and the resulting wave heights.

The new breakwaters do impact the total erosion during storms. Fig. 9.49 shows the erosion after six hours of storm. It shows that the erosion is reduced when the breakwaters are present. It should be noted that the figures do not give accurate values, but it does give a feeling of the relative erosion and accretion. The second point of attention is that the erosion and accretion are storm related, which means that the beach will return to its original shape given that the sediment stays inside the bay.



(a) Erosion & accretion without breakwaters.

(b) Erosion & accretion with breakwaters.

Figure 9.49: Comparison of erosion & accretion without breakwaters and with final breakwater lay-out.

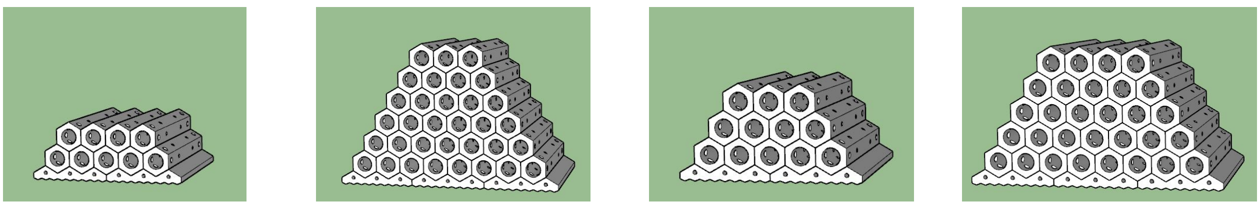
9.1.1 Final submerged breakwater design

The final design of the north and middle bay have the same dimensions. The south bay has 3 different breakwater configurations due to the changing depth. This means, in total 5 different submerged breakwaters designs are present, from now on referred to as sill 1-5 where 1 is the most northern and 5 the most southern submerged breakwater. Analysis showed that the height is governing for the wave attenuation factor and the crest width has minimal effect. Therefore, to keep the design as economic as possible, a height-width ratio has to be found in which the width is small. Using Fig. D.11 the dimensions are chosen such that the wave attenuation factor K_t is around 0.5. In Tab. 9.19 the final outer dimensions can be seen along with the number of needed blocks.

Table 9.19: Dimensions, wave attenuation factor and number of blocks per submerged breakwater.

	Depth [m]	**Height** [m]	**Crest width** [m]	**Length** [m]	$\mathbf{K_t}$ [-]	**No. of base blocks**	**No. of hexa-blocks**
Sill 1 & 2	2.1	1.7	3	120	0.45	160	720
Sill 3	5	4.3	2.25	30	0.54	30	330
Sill 4	3	2.4	2.25	40	0.54	28	168
Sill 5	4	3.65	3	80	0.46	81	810
Total						**299**	**2028**

In Fig. 9.50 one row of each sill is shown. It can easily be seen that for larger depths the amount of blocks increases rapidly.



(a) Cross section sill 1 & 2.

(b) Cross section sill 3.

(c) Cross section sill 4.

(d) Cross section sill 5.

Figure 9.50: Different lay-outs of submerged breakwaters for the project site at Montego Bay.

To get the final concept, an electrified steel frame is added. This frame will support the coral and enhance their growth and health. Also, the electric field will attract and support marine life. For now the electrified steel frame will only be placed on the beach side, but it is a possible to place the frame on both sides. An artist impression of the final concept can be seen in Fig. 9.51. Because of the treatment of the concrete blocks and the curing process, the coral can also naturally grow on the surface of the submerged breakwaters.

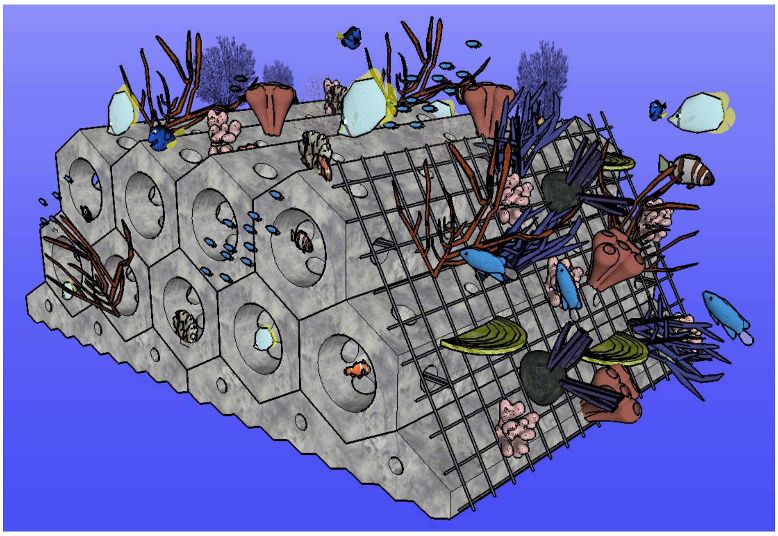


Figure 9.51: Artist impression of final concept with corals.

9.1.2 Dimensions hexagonal block & reinforcement

The final design and dimensions of the hexagonal block can be seen in Fig. 9.52. Two different sized holes are used, as shelter for fishes (so-called fish condos). They are aligned in such a way that they always connect to a hole in the adjacent block. The largest holes are located near the edges of the block, because the bending moment is largest in the middle. Diameters of 101.6 mm and 152.4 mm are used, so inch sized pipes of 4" and 6" can be used. For the reinforcement, standard bars of ⌀10 mm and for the stirrups in the blocks ⌀6 mm is used.

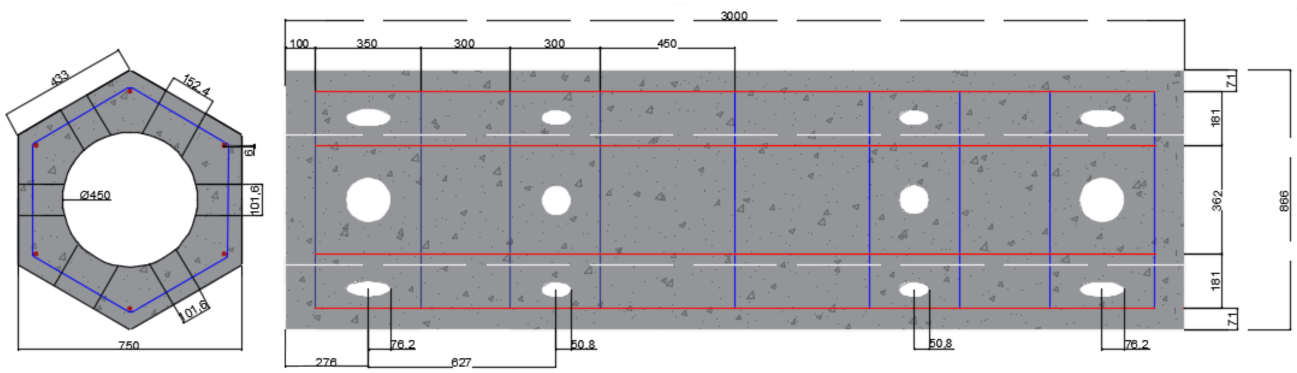


Figure 9.52: Dimensions of hexagonal block in mm.

9.1.3 Dimensions base block & reinforcement

The final design and dimensions of the base block can be seen in Fig. 9.53 and Fig. 9.54.The base has a saw toothed bottom to increase the friction with the seabed. For transportation and placing, reinforcement in two directions is needed. The diameter of the bars is ∅10 mm and for the stirrups ∅10 mm is used. The holes are 101.6 mm (4").

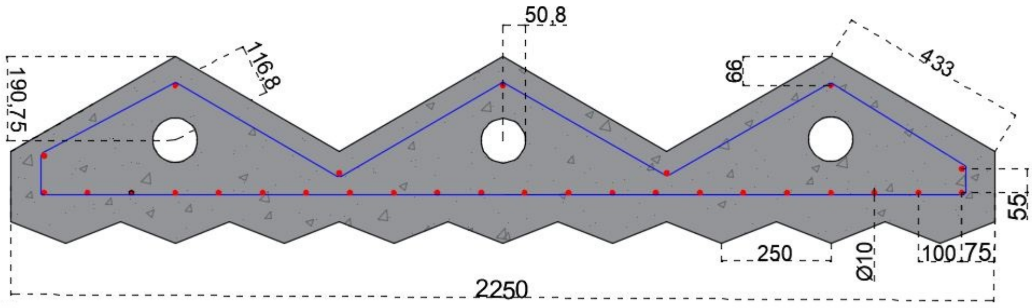


Figure 9.53: Dimensions cross-section base block in mm.

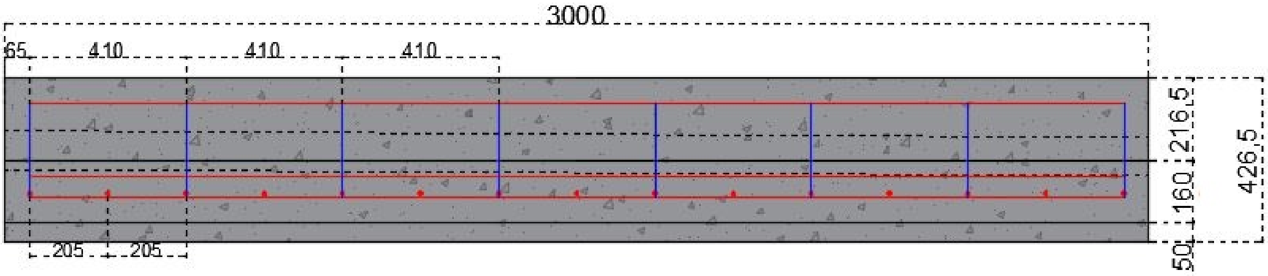
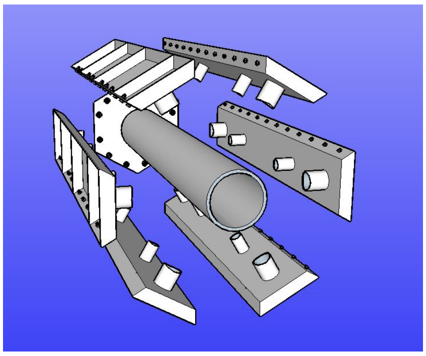


Figure 9.54: Dimensions base block.

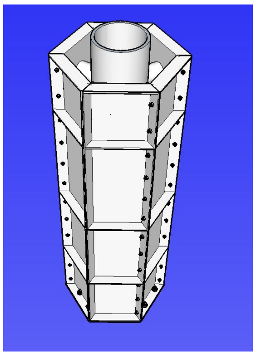
9.1.4 Formwork & casting procedure

Looking at the amount of blocks, multiple steel formworks are required to reduce the production time. Steel formwork will also decrease the casting errors and dimensions deviations, which are important for the integrity of the structure. Because of the shape of the base, the casting is likely to be done on the side. Therefore a large scaffolding is needed on site. For the casting process an excavator is needed to, among other things, tilt the base from a vertical to a horizontal orientation. When the elements are casted upwards, the concrete will flow in the mould at a height greater than 3 meters. To prevent segregation of the mixture, a funnel most be used during casting. The formwork of the block is designed as 6 equal plate frames, 2 end frames and a HDPE tube which can be bolted together (see Fig. 9.55a). The inner hole can be made with an HDPE tube too. For horizontal casting, three plates frames with an end plate can be screwed together. Afterwards, the reinforcement basket can be placed inside. Next, the HDPE tube can be put inside. To keep it all in place, some connections on the end plates are needed. At last, the two top edges can be attached. The final view is given in Fig. 9.55c. For vertical casting, six edges and one end plate are needed (see Fig. 9.55b). The concrete mixture needs to have a good workability to flow around the tubes. Also greasing of the tube can be of importance to remove the tube after hardening. For the blocks and base extra attention needs to be paid to the removal of the tubes. Local stresses can damage the elements when not fully hardened. The elements have to be handled with care.

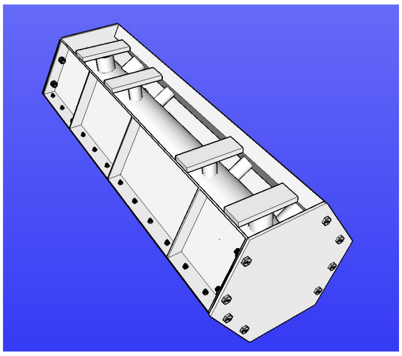
Demoulding can be done when the concrete has a minimal compressive strength. Calculations done with the method in Section 7.4.2 shows that the block can theoretically be lifted when the concrete has a f_{cd} of 1.33 MPa. The bending stresses are also calculated in a 3-D model and are given in Fig. 7.41. This gives a local maximum compressive stress of 2.15 MPa due to the distribution of the eccentric force. After one day the concrete has 30% of its final strength [15]. This will be 4 MPa for the assumed concrete so the formwork can be removed after 1 day of hardening. An extra day of hardening will be more conservative.



(a) Steel frames.



(b) Standing casting method.



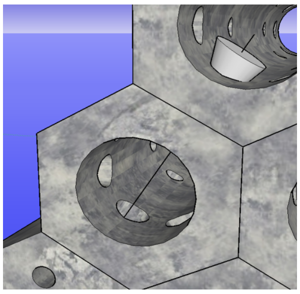
(c) Flat casting method.

Figure 9.55: Steel formwork for hexagonal blocks.

Because of the large amount of concrete, production on site is advised. The project site has enough space to accommodate a mixing station. Storage of the materials can also be done on site. The concrete mixture can easily be adapted during the execution if needed. The process will take more time in the beginning and errors are likely to be made. Extra attention is needed at the beginning of the project to guaranty a high quality end product. The procedure is repetitive so a learning curve will be present, which leads to an increased production speed and smaller error margins.

9.2 Connections

Under 1 in 50 year hurricane conditions, some sliding of the top left blocks occurred. To prevent this and to make the structure more stable, a tension connection between the top left block and the left base is necessary. Multiple solutions are possible for this connection (see Fig. 9.56). Other possibilities are a cable around the structure, clamps between the blocks or a cable between the block and the base as shown in Fig. 9.56a. This 'plug' connection is inspired from a prestressing fixation method of the cable. In the base a precasted hole with a steel bar can be designed where the cable can be hooked on. For every method it is important that the local tension stresses are checked. Extra reinforcement may be needed to redistribute the (tensile) stresses in the concrete. Also for the steel frame connection, an option is given. When a loop is attached on the steel frame they can be put through the holes in the blocks. With a wedge, the frame can be easily fastened to the blocks. This options can been seen in Fig. 9.56b.



(a) Cable connection.

(b) Steel frame connection.

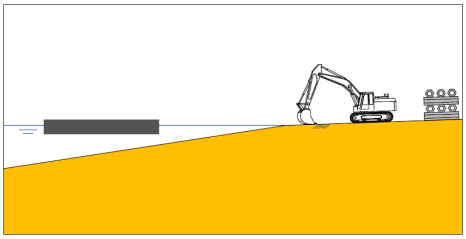
Figure 9.56: Connections of block-base and steel frame-blocks.

9.3 Construction

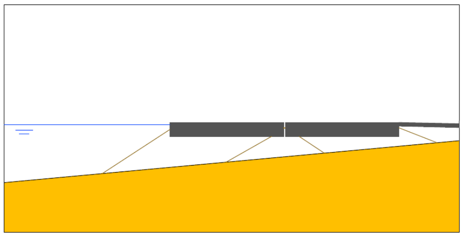
To get a clear overview of the construction of the hexagonal submerged breakwater, the construction steps are explained one-by-one. Firstly, the construction steps are given and listed in chronological order. Extra textual explanation and elaboration will be given for some construction steps in the coming sections which are substantiated with cartoons in Fig. 9.57 and Fig. 9.58.

Table 9.20: Construction steps modular submerged breakwater.

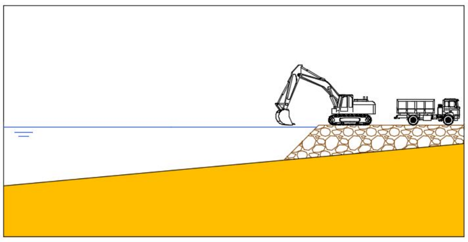
Step	Activity
1a	Transport material and formwork for concrete blocks and base to construction site.
1b	Manufacturing of the base & hexagonal blocks on site.
2	Prepare construction site by moving equipment (excavator, pontoons, etc.) to site.
3a	Move and anchor pontoon at correct position.
3b	Attach ramp to pontoon to enable excavator to drive onto pontoon from the beach.
3c	Build walkway road with four meters crest width.
4	Prepare seabed by flattening gradient and/or placing geotextile with ballast (if necessary).
5	Place base blocks on (prepared) seabed.
6	Place (if required) additional base blocks in cross-shore direction to get required width.
7	Place first row of hexagonal blocks on base blocks.
8a	Top up the hexagonal blocks until the required submerged breakwater height.
8b	Secure critical block against uplift.
9	Repeat steps 4-11 till submerged breakwater has the desired length in alongshore direction.
10a	Install electrified frames at the beach-side of the submerged breakwater.
10b	Attach power cable to the frames and a power source on shore.



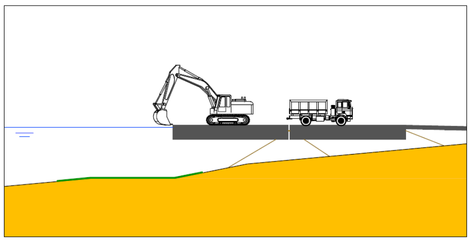
(a) Construction step 2.



(b) Construction steps 3a & 3b.

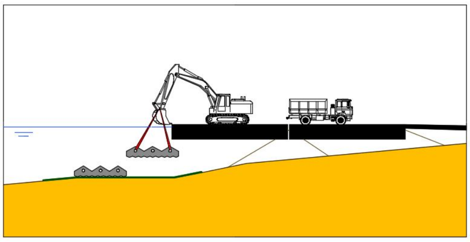


(c) Construction step 3c.

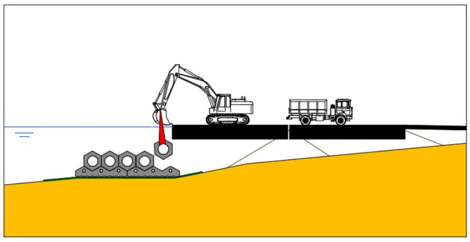


(d) Construction step 4.

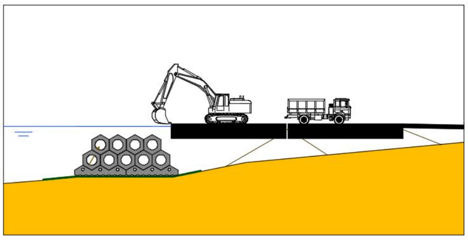
Figure 9.57: Cartoon construction steps 2-4 (not on scale).



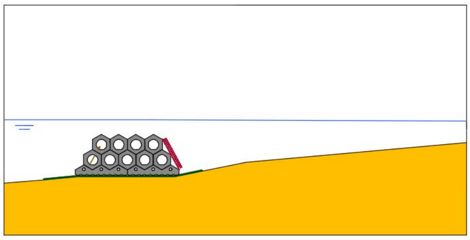
(a) Construction steps 5 & 6.



(b) Construction step 7.



(c) Construction step 8.



(d) Final structure with electrified frame.

Figure 9.58: Cartoon construction steps 5-8 (not on scale).

Construction step 2
When the manufacturing of the modular blocks has started, the construction equipment can be moved to the site. This includes a excavator, truck, pontoons and any other required equipment.

Construction step 3
When using pontoons for construction they need to be secured, so they don't float away. Furthermore, the pontoons have to be checked for their rotation resistance, so excavators and other equipment can drive and stand on the pontoons safely, see Fig. H.3. Lastly, a ramp has to be constructed/attached to enable the excavator and truck to drive up the pontoons.

An alternative, what is used more often, is to built an access road to the location where the structure will be located. As it seems to include more effort than with the pontoon, it should only be considered if using a pontoon is not possible. Fig. 9.57c shows an impression of such a road. For the following construction steps the pontoon-alternative is used.

A third alternative is installing the blocks with a floating device. This alternative looks very promising, but is not shown in the construction steps. This construction alternative needs to worked out further to see if it is feasible.

Construction step 4
Before the base blocks can be placed, it might be necessary to prepare the seabed. When the gradient of the seabed is too large (so the blocks may slide down slope) or when the seabed is not flat, the seabed needs to be excavated to ensure a flatter bed. When scour might be a failure mechanism, a geotextile with ballast is needed as well.

Construction steps 5 & 6
After the seabed has been prepared, the base blocks can be put in place. This is done by a excavator, which lowers the blocks slowly into the water. The lifting of the base blocks is done by lacing an hoist cable through the outer two holes of the blocks. This way the chains can easily be removed after placement. In this step, divers will be in the water to help to maneuver the block into place. This may be done by attaching ropes to the outer edges of blocks which the dives can pull to rotate the blocks. The width of the breakwater can be adjusted by adding additional base blocks after one another.

Construction step 7
With the base blocks in place, the hexagonal blocks can be placed on top of the base blocks. This can, again, be done with the help of divers to help positioning the blocks correctly. The blocks will be lifted by adding a chain through the inner hole of the hexagonal blocks. Due to gravity, the blocks will hang in the right orientation to ensure proper placing.

Construction step 8
During construction step 8 more layers of hexagonal blocks are stacked on top of the existing ones. In this step the required height of the submerged breakwater is reached.

While placing the first block per row on the sea side of the breakwater, a cable is laced through the blocks. When the top layer is reached, the cable will be tensioned by the force of a excavator or a power tool. A wedge is used to ensure the cable stays tight (see Section 9.2). Note, no prestressing has to be performed, as this induced extra stress in the concrete anchorage and does not contribute to stability.

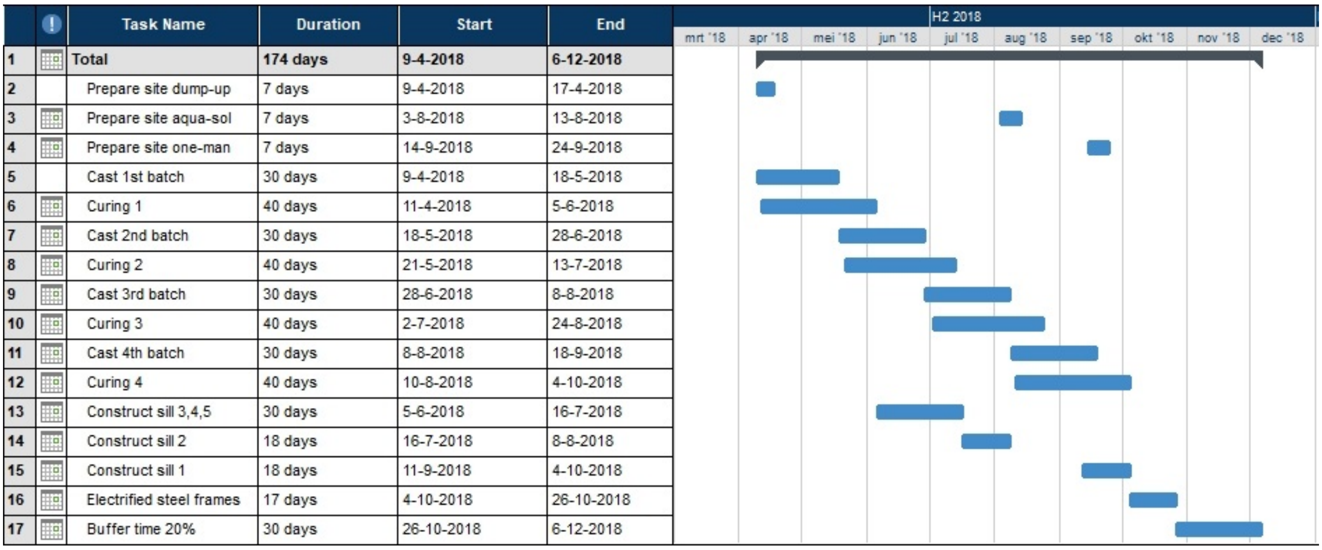
Completion
After going through all construction steps listed in Tab. 9.20 the structure will be ready. This means that the electrified frame is added to structure on which coral will start growing. Fig. 9.58d shows the completed stage of the modular submerged breakwater.

9.3.1 Construction time

For each phase of the project, the construction time is given. The construction time is based on typical construction values. The construction process is divided in four parts:

- Preparing the site for construction.
- Casting & curing of the concrete.
- Construction of the submerged breakwaters.
- Placement of the electrified steel frames.

Preparation of the site at each bay is expected to take one week. For casting of the concrete 20 sets of formwork are used. It takes two days for one piece to reach enough strength to be lifted out (conservatively) and another 40 days before it is cured and ready for placement. In total 2 400 blocks are needed, which this is divided into four batches, 600 each. In this way one batch takes 30 days. The placement speed is taken as three blocks per hour. All values are implemented in a Gantt char (see Fig. 9.59). The expected construction time is 190 weekdays (weekends are taken as non-working days), including a buffer of 30 days. Some of the work can be done simultaneously, reducing the construction time.



	!	Task Name	Duration	Start	End
1		Total	174 days	9-4-2018	6-12-2018
2		Prepare site dump-up	7 days	9-4-2018	17-4-2018
3		Prepare site aqua-sol	7 days	3-8-2018	13-8-2018
4		Prepare site one-man	7 days	14-9-2018	24-9-2018
5		Cast 1st batch	30 days	9-4-2018	18-5-2018
6		Curing 1	40 days	11-4-2018	5-6-2018
7		Cast 2nd batch	30 days	18-5-2018	28-6-2018
8		Curing 2	40 days	21-5-2018	13-7-2018
9		Cast 3rd batch	30 days	28-6-2018	8-8-2018
10		Curing 3	40 days	2-7-2018	24-8-2018
11		Cast 4th batch	30 days	8-8-2018	18-9-2018
12		Curing 4	40 days	10-8-2018	4-10-2018
13		Construct sill 3,4,5	30 days	5-6-2018	16-7-2018
14		Construct sill 2	18 days	16-7-2018	8-8-2018
15		Construct sill 1	18 days	11-9-2018	4-10-2018
16		Electrified steel frames	17 days	4-10-2018	26-10-2018
17		Buffer time 20%	30 days	26-10-2018	6-12-2018

Figure 9.59: Gantt chart for construction time.

9.4 Costs

The final costs of the project are estimated by splitting it up in four main categories:

- Concrete elements.
- Electrified steel frame.
- Transport.
- Construction.

For each category a cost break down will be done, before combining the costs to the total cost.

The costs for all activities are listed in Tab. 9.21. These values are retrieved from a construction expert at Smith Warner and are expected to give a good representation of the real costs.

Table 9.21: Costs per activity.

Activity	Costs
Casting concrete blocks (incl. reinforcement, formwork, etc)	700 $/m^3$
Transport per truck load (15 m^3)	200 $/trip
Excavator	750 $/day
Barge	2 000 $/day
Diver	200 $/day
Supervisor	70 $/day
Labourer	45 $/day
Road fill	20 $/m^3$
Armour stone	100 $/m^3$

9.4.1 Concrete blocks

The costs for the concrete blocks are determined using the total volume and the price per meter cubed, which is 700 $/m^3$. This price includes manufacturing, reinforcement and formwork. The blocks can be casted on site which saves transport costs. Only the necessary ingredients (aggregate, reinforcement) need to be transported to the site.

Table 9.22: Costs concrete blocks.

Type	Number of blocks	Volume per block [m^3]	Total volume [m^3]	Costs US$
Hexagonal block	2028	0.94	1906.3	1 334 424
Base block	299	1.91	571.1	399 763
Total	-	-	**2477.4**	**1 734 000**

It is interesting to know the cost per block in case different lay-outs at other locations are made. For each block the price is as follows:

Hexagonal block: $660 ; **Base block**: $1 334

Compared to a typical rubble mound breakwater these costs are a factor 7 higher. 100 $/m^3$ for armour stones compared to 700 $/m^3$ for blocks of concrete. However there is probably some money to be won in terms of transport, since all the armour stone has to be transported from the quarry and the concrete blocks can be made on-site. However, as will be told later, these blocks have other benefits and large armour blocks are sometimes hard to get.

9.4.2 Electrified steel frame

The costs for the electrified steel frame are calculated based on a reference project done by Smith Warner for Royalton Negril [16] (see Fig. H.1 and Fig. H.2) and was carried out by Coralive. The surface area covered in this project is compared to the reference project and the corresponding costs are scaled to that ratio. The costs are built-up in two parts: materials and project costs. The steel frames are only placed on the beach side of the submerged breakwaters, this results in a total covered area of 1 077 m^2 over the 5 structures which translates to a total of 7 m^3 of steel bars. Now the cost evaluation can be made.

In Tab. 9.24 the cost breakdown can be seen. The combined cost evaluation for both materials and projects costs is $**32 765**. In case an electrified steel frame is used on both sides the costs will be higher.

Table 9.23: Deliverables steel frame electrification.

Deliverable	**Duration**
Inspection of site, knowledge transfer, meeting with staff.	2 days
Analysis of structures, environmental factors. Calculations and determinations of needs.	2 days
Preparations for electrification by connecting structures to a grid and allocating the anodes.	7 days
Testing, gathering data, make adjustments.	3 days
Finishing up reef work, training staff, hand over project.	2 days
Manuals, media and social media preparation.	1 days
Total	**17 days**

Table 9.24: Costs steel frame electrification.

Materials	**Costs (US$)**	**Project costs**	**Costs (US$)**
Variable power supply units (5x)	1075	Food and accommodation	3400
Special electric cable (500m)	1640	Flights for 2 people	2000
Anodes (20x)	4800	Per diem for 2 people for 17 days	7050
Waterproof connectors and steel hose clamps	1600		
General material and parts	4000		
Total	**20 315**		**12 450**

9.4.3 Transport

The transport cost consist of the prices for transport of the materials needed for the concrete blocks, such as aggregate and reinforcement. The concrete blocks are manufactured on-site which reduces transportation costs. In Section 9.4.1 it was determined that a total amount of 2 478 m^3 of concrete is needed. Because of the weight of steel 1 m^3 per truckload is assumed.

Table 9.25: Transportation costs.

Material	**Volume (m^3)**	**Number of truck loads**	**Costs (US$)**
Aggregate etc. for concrete	2478	166	33 200
Steel reinforcement	9	9	1 800
Total			**35 000**

9.4.4 Construction

The costs for construction depend on the expected needs, such as the number of excavators that will be used and man-hours. At this point a trade-off must be made between using either a road or a pontoon. The exact costs for a pontoon are not known, therefore the costs for a barge are used. The disadvantage of a road system is that first the material has to be bought, brought and placed, then removed an relocated to a different location. Thus not only material costs, but also costs for placement with an excavator and transport have to be taken into account, while a barge can be used only during the actual construction phase. After a quick evaluation it turned out that a road system would cost more than a pontoon so the pontoon is chosen.

The use of excavators is limited to two. One will be used during the entire construction phase for lifting concrete blocks out of their formwork and assisting in the construction phase. The second will only be used during placement.

Finally it is assumed that 10 labourers will be present during the entire construction time and one supervisor in addition to 2 divers and machine operators. With use of Fig. 9.59 this results in the cost of Tab. 9.26.

Table 9.26: Construction costs.

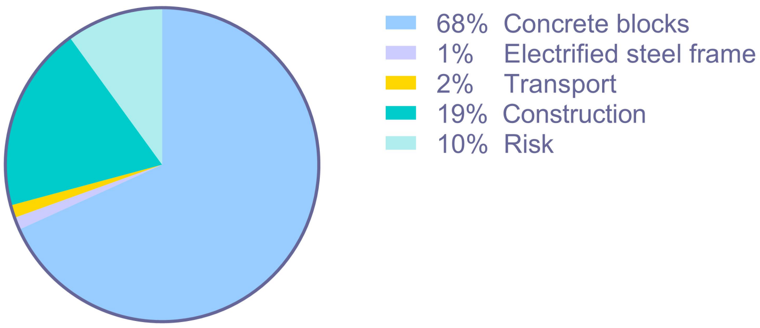
Activity	Duration (days)	Costs per activity (US$/day)	Total costs (US$)
Excavators	247	750	186 000
Divers	182	250	45 500
Supervisor	145	70	11 000
Labourers	1440	45	64 800
Pontoon	80	2000	160 000
Mobilization			20 000
Total			**487 300**

9.4.5 Combined total costs

Now that the costs for each category are determined, they can be combined to get the total costs. Tab. 9.27 gives an overview of the different categories and the total cost. In the total cost an extra 10% is included to take the risk into account. It can be seen that production of the concrete blocks holds the largest part of the costs. Since the number of blocks increases rapidly for larger depths, it can be concluded that this design is less economical for larger depths.

Table 9.27: Total cost of hexagonal protection structure for the Hip-Strip in Montego Bay.

Category	Costs (US$)
Concrete blocks	1 734 000
Electrified steel frame	32 765
Transport	35 000
Construction	487 300
Total (including 10% risk)	**2 517 760**



Chapter 10. Conclusions & recommendations

In this final chapter, conclusions are drawn and recommendations are given regarding the project. The conclusion will also contain an overview of the advantages and disadvantages of the Honeycomb (Hexagonal) system compared to a traditional rubble mound breakwater. In the recommendations further improvements to the design will be mentioned.

10.1 Conclusions

The results from the numerical models (Delft3D and ANSYS) showed that the design is able to reduce the wave height by a factor 2 and that it remains stable under hurricane conditions, when a cable system is present. Combined with the literature study on coral and marine enhancement it can be concluded that the blocks show best potential:

- In high wave energy climate in which larger waves are present on a daily basis.
- For locations at which marine life enhancement is of importance.
- For clients interested in a durable construction which stays stable during hurricanes.

For larger depths the blocks are less economically feasible. Since the number of blocks increases rapidly with a larger depth, the costs also increase rapidly. The costs per block are as follows:

Hexagonal block: \$660 **Base block**: \$1334

Cost analysis showed that the concrete blocks account for 68% of the total costs (\$ 2 517 760) and 65% of this portion is due to the breakwater in a water depth of 4-5 meter. Placing the hexagonal structure in larger depths, makes it more expensive and harder to construct. It will be a challenge for the operator of the excavator to place the blocks within the tolerances. Also, production of a large amount of blocks will increase the production time significantly.

For deeper water there are a few other solutions. The first possibility is to fill-up the depth some meters and afterwards create the structure on top of the filling material. A second solution is a hybrid structure, where the hexagonal blocks are put in shallower depths and the armour stones in the deeper part.

10.1.1 Hexagonal blocks vs Rubble mound: Pro's and Con's

In order to make a good comparison between the designed blocks and a traditional rubble mound breakwater, all the pro's and con's of the Honeycomb blocks in relation to the rubble mound alternative are listed below.

Pro's Honeycomb blocks

- With a tension cable, the design shows high stability. Once the base layer is in place the hexagonal blocks can easily be stacked. On long-term this will results in less inspection and maintenance.
- The structure will enhance coral and marine life. Besides the positive effects on the marine ecosystem, this enhancement is beneficial for recreational (tourist) activities, like diving and snorkelling.
- The structure is a state-of-the-art design. Dumping rocks is considered as old fashioned and not innovative. The Honeycomb blocks blend with the marine environment and will encounter less resistance by stakeholders.
- The final structure will precisely agree with the designed dimensions. This is not possible for rubble mound breakwaters, leading to less predictable results.
- Material for large rubble mound blocks is not easily available everywhere. Importing a large amount will increase the costs, while the production materials of concrete are easily available.

Con's Honeycomb blocks

- The costs of the hexagonal blocks are considerably higher compared to standard armour blocks; \$700/m^3 for hexagonal blocks vs \$100/m^3 for the armour blocks. Reduced placement and transportation blocks can not make up this difference.
- The seabed needs to be flat or seabed preparation must be performed.
- Fabrication of the blocks must done with care. Complex formwork is required and handling has to be done carefully.

10.2 Recommendations

The pilot site at Montego Bay doesn't have the perfect conditions for the designed concept; the Honeycomb blocks. Design options are limited because of the existing structures and the daily wave conditions are not really significant. From the site survey, pollution on the sills and algea's in the water next to Dump-up beach was observed. When marine enhancement techniques are applied the local water quality should be taken into account. A possible solution are mangroves discussed in the Appendix B.6.5. General recommendations regarding the concept are the following:

Water depth
The number of blocks increases rapidly with depth. The designed concept is therefore most viable in shallow waters (2-3 meters) with a moderate to heavy wave climate.

Stakeholders
The combined properties of wave attenuation and marine life enhancement are particularly interesting for hotels and resorts. The beaches are protected against erosion and the marine life is an attraction for tourists.

Calculated values
The wave attenuation factor K_t was based on Eq. 3.2 by Friebel & Harris, an indicative formula based on several studies only. Wave flume tests or large scale field tests should be done to verify the formula and to better predict the behaviour. In addition, the forces acting on the structure should be investigated in more detail. The formula by Morison (see Eq. E.2) was used to give an indication.

Fibre reinforced concrete
Polymer fibre reinforced concrete can be a very good alternative for reinforcement. No steel is needed and because of the complex formwork, the construction will be easier. Further research is needed to show the feasibility and the reduction in cost.

Lifting method
The calculations are based on lifting with a cable through the blocks. To reduce stress, lifting of the block with a rigid pipe through the hole can be done. Take into account the removal of the pipe after placement.

Seawall
One of the requests by Smith Warner, was the option to extend the design into a seawall. Because time was limited, a detailed design is not worked out. However a first concept was made up, which can be seen in Fig. 10.60. For heavy wave attack the diameter of the holes can be reduced to increase the weight and resistance against the waves. No sloping angle is applied to limit the number of blocks. Further research must be done to find the acting forces and modelling must be performed to check the behaviour and stability.

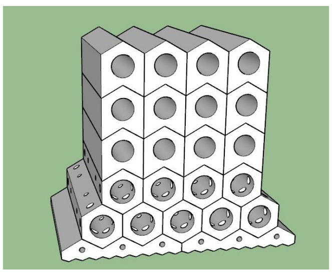


Figure 10.60: Concept for an emerged breakwater.

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Appendix A

Site investigation Montego Bay - 26 & 27 February 2018

To get a better insight in the conditions and dimensions of the project, a site investigation was done during 26th and 27th of February. A survey at Aquasol Theme Park was not possible, because the park was closed off. During the site survey at One Man and Dump Up beach the breakwaters were documented. Also the area's with coral reefs, seagrass, pollution and stockpiling of sand were mapped. The coral and marine life on and around the sills is also reported. A short visit at the river outflow was also done. The findings are discussed and pictures are shown to give a good impression of the project site. (see Fig. A.1). *The survey is to give an impression. Expressing the outcome in numbers and percentages is not executed.*

Figure A.1: Overview Hip-Strip survey.

A.1 One Man's Beach

The northern beach of the three public beaches is nowadays the most popular one. Both locals and tourist are visiting the beach. Locals have started small souvenir and beverage shops to create some job opportunity for themselves. They keep the beach clean and give the beach a nice vibe by playing music. The conditions of the beach and water are good. No excessive amount of algae is visible. The cross-section of the beach is over the complete circumference constant and wide.

Figure A.2: Photo One Man's beach.

Northern groyne

The northern groyne is severely damaged as can be seen in the Fig. A.3. Over a large length the elevation above mean sea level is almost 0. Due to big storm surges and the lack of maintenance a breakwater can encounter significant damage with the result of losing its workability. Actions have to be taken to restore the safety of the beach, specially during storms.

Figure A.3: Present state of the northern groyne.

Submerged sill 1

For the survey of the sill a snorkelling expediting was executed. By photographing the rocks, coral and marine life an indication of the sill is made. The beach is not fully eroded during the lifetime of the sill. However locals mentioned that the beach was bigger years ago. The true efficiency of the sill in the current state is difficult to determine.

Condition

After extensive inspection of the submerged sill it can be concluded that the sill is in a good condition. The rocks are laying stable in the sand bed and close to each other. The height of the sill is constant over the width and the freeboard above the sill is high enough for recreational uses. An overview of the sill is given in Fig. A.4.

Figure A.4: Overview sill One Man beach.

Coral & marine life

On and around the sill diverse coral colonies are observed. Despite the lack of expertise in zoology, an estimation of 10 coral species is made (see Fig. A.5). Mainly stony coral is living on the sill. The coral is not flourishing. Colonies are not fully developed and the colors are bleached. The most common types were [17]:

- Box Fire Coral, *Millepora squarrosa.*
- Massive Starlet Coral, *Siderastrea siderea*
- Boulder Brain, *Colpophyllia natans*
- Thin Finger Coral, *Porites divericata*

Also an invertebrates were spotted like the Giant Anemone, *Condylactis Gigantea.* Different grass types were also established on the sills. No extensive mud or pollutions on the rocks and coral were noticed. Large amount of the rocks were covered with seaweeds. Some large colonies like the Box Fire and Boulder Brain corals are in the area. Also there were some sea grasses located in the bay.

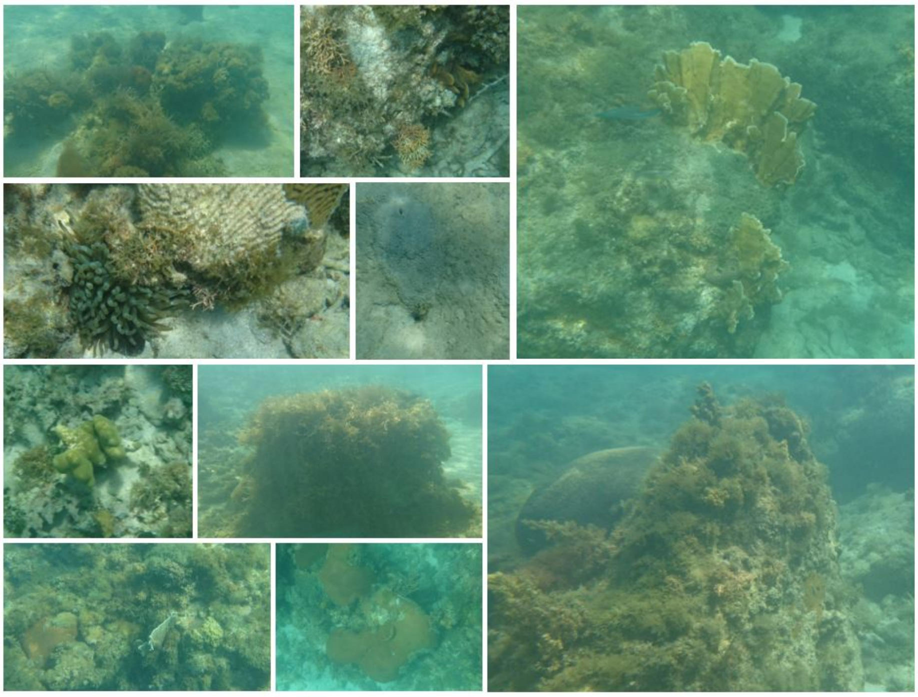


Figure A.5: Overview coral One Man's beach.

The fish populations is divers but the amount is low. No large groups are captured. The estimated species spotted in the area are ten shown in Fig. A.6. The special types were [17]:

- Blue Tang, *Acanthurus coeruleus.*
- Web Burrfish, *Chilomycterus antillarum*
- Banded Butterflyfish, *Chaetodon striatus*
- Bluestriped Lizardfish, *Synodus saurus*

The largest group located were in the range of ten fishes. This represent not a very large marine life. Enhancement of the ecosystem will be a good thing for this site.

Groyne 2

The groyne is still fully functioning and has no severe damage (see Fig. A.7). No actions have to be taken on groyne 2.

Figure A.6: Fishes at One Man's beach.

Figure A.7: Present state of groyne 2.

A.2 Aquasol Theme Park

The Aquasol Theme Park was closed. Locals told us that the park was closed months ago and that the government wants to revive the area by restoring the beaches and buildings. In Fig. A.8 the forgotten Aqua Theme Park is shown. The Aquasol Theme Park can be a potential hotspot in Montego Bay. Further investigation of the site is not executed.

Figure A.8: Photo's Aquasol Theme Park.

A.3 Dump up beach

During the survey we were immediately warned by a local that is was unsafe for us to swim at dump up beach. He recommended One Man beach instead. Dump up beach is further away and less visible from the road. Only a handful of locals walk on the site. The water contains large spots with algae shown in Fig. A.9a. The abundance of nutrients comes from the river outflow behind the southern groyne. The flow direction in de bay looks different which can come from the harder wind. This results in stockpiling of the sand at both sides of the bay (see Fig. A.9b).

(a) Pollution from algae.

(b) Stockpilling of sand.

Figure A.9: Dump Up Beach.

Groyne 5

Groyne 5 is also severely damaged as can be seen in Fig. A.10. The overall height is reduced and in the middle the level is close to the mean sea level. Restoration of the breakwater is required. Dump-Up beach is currently not well protected during storm surges.

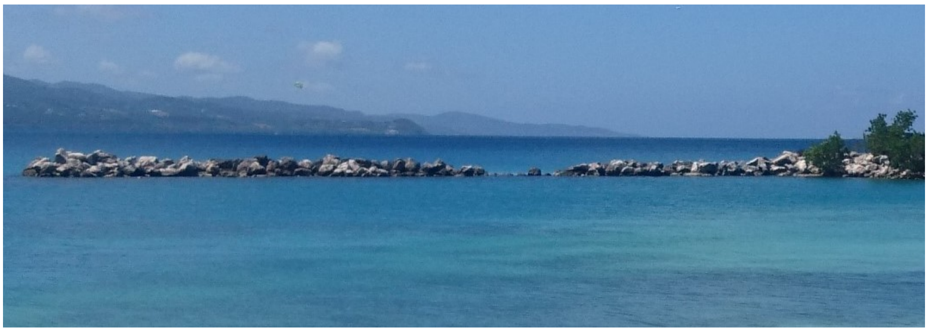


Figure A.10: Present state of groyne 5.

Submerged sill 3

The sill at Dump Up is wider than at One Man. The beach is also not fully eroded so it is still functioning.

Condition

The wider sill can come from displacement of the rocks during storms. The sill has more open places than the sill 1. Also the sand surrounding the rocks for stability looks washed away. The height of the sill is not constant over the width. The sill encountered more damage during its lifetime.

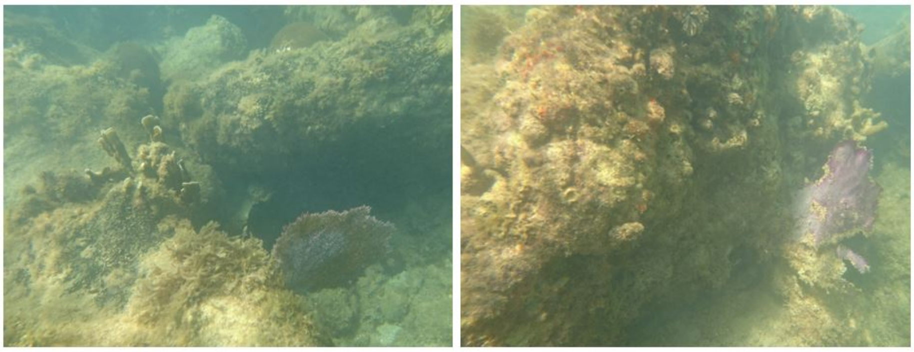
Coral & Marine life

The coral and marine life are not in a good condition. Due to river outflow the amount of algae and mud on the coral is large (see Fig. A.12a). The pollution results in a bad environment for the marine life. The fish and

Figure A.11: Overview sill 3.

coral population are very low. Only a handful fishes were spotted. Two different coral species were detected. [17]

- Yellow Tube Sponge, *Aplysina fistularis.*A.12b
- Common Sea Fan, *Gorgonia ventalina*A.12c



(a) Pollution on coral.

(b) Yellow Tube Sponge.

(c) Common Sea Plan.

Figure A.12: Corals on sill 3.

Southern groyne

The southern groyne is still functioning and has no severe damage. In the middle the breakwater has some small damage. This has to be repaired by adding new rocks to stabilise the construction (see Fig. A.13). No immediate actions have to be taken on the southern groyne.

Figure A.13: Present state of the southern groyne.

A.4 River

The river have a big influence on the water quality of the Dump Up bay. The river transports excessive rain and waste water from higher ground. This water contains a lot of sediments, nutrients and waste. To increase the water quality, measures are obligated by the river outflow. Otherwise enhancement of the ecosystem at the bay's is difficult. The measures which will be taken at Dump Up beach will not be effective and a waste of resource. Controlling the river outflow and water quality is outside the scope of this research. In the reference projects, the north Java solution can be found, which can be success full to overcome this water quality problem.

A.5 Overview Montego Bay

The Reef Check Foundation tries to help and preserve the ocean and reefs. By mapping the ecosystems all over the world they contribute to the scientific research on ecosystems. They perform reef surveys to map the coral and fish species. For the benthic survey a modified Reef Check Method is used to asses the substrate in each area. Substrate type are categorized as Hard Coral (HC), Soft Coral (SC), Recently Killed Coral (RKC), Nutrient Indicating Algae (NIA), Sponge (SP), Rock (RC), Rubble (RB), Sand (SD), Silt/clay (SI) and Other (OT).[18]

For the fish survey the modified Visual Fish Census of Atlantic and Gulf Rapid Reef Assessment (AGRRA) is employed to capture fish data. All fishes identified are given a frequency rating of Single (S = single individual), Few (F = 2-10 individuals), Many (M = 11-100 individuals), or Abundant (A = >100 individuals).

The results of two surveys at the left and right side of the project site give an indication of the species and the distribution of the marine life. The result of this professional survey is shown in Fig. A.14.

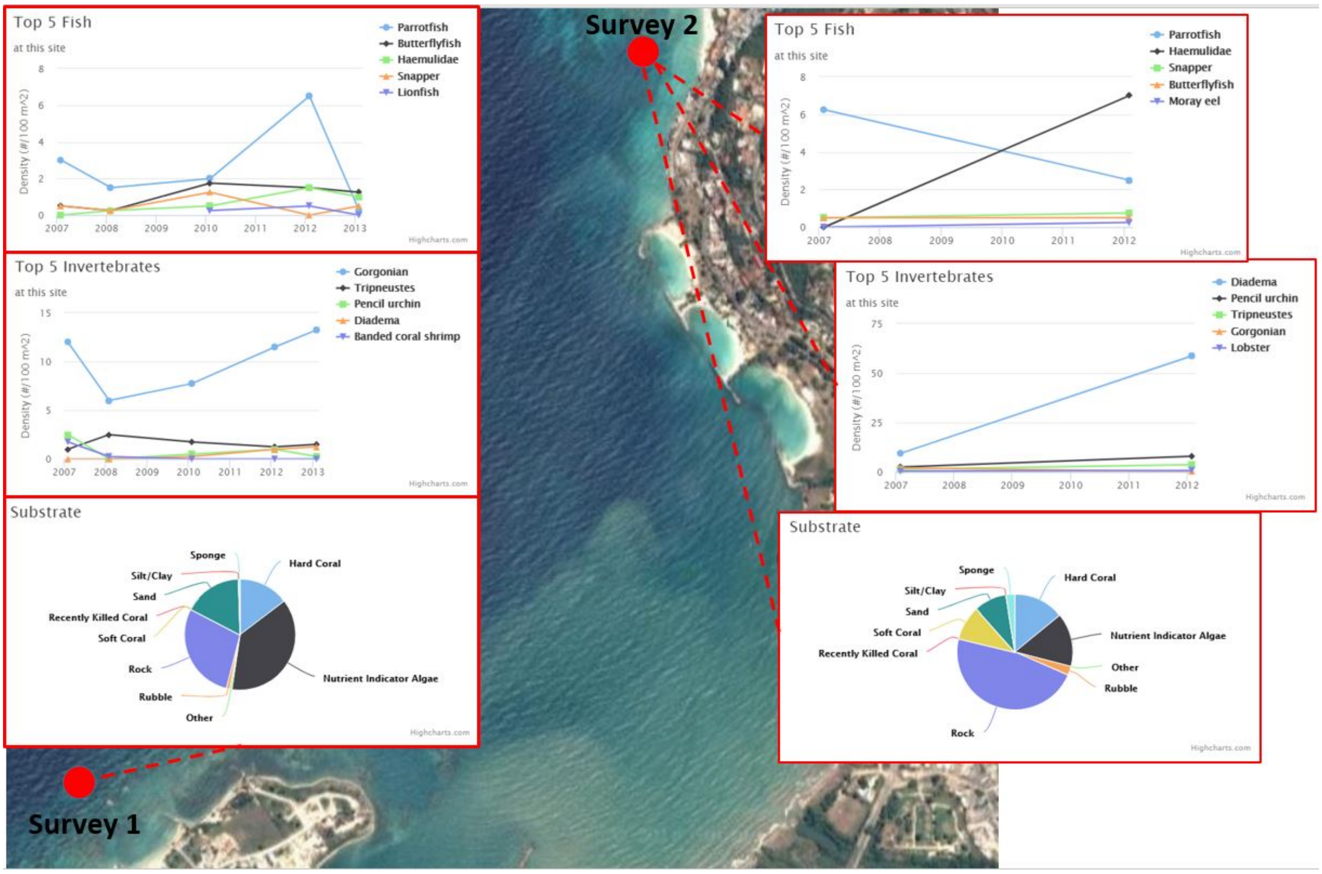


Figure A.14: Overview Reef Check around project site.

Appendix B

Coastal management techniques

B.1 Coastal management techniques

In this section an overview is given of coastal defence solutions. First, the different categories of solutions are explained and a division between them is made. Next, some widely used eco-friendly products are elaborated on to show their effectiveness. In the following section, multiple coastal projects are discussed in which different hybrid coastal strategies are used. Projects like Biorock, Reef Balls and modified concrete structures are used as a solutions. The well-known solutions Biorock, Reef Balls and Wave attenuation devices(WAD) will be discussed more comprehensive. The project in Northern Java perfectly shows the application of Building with Nature and the Negril project shows a similar project as the one addressed in this report; Montego Bay. In the final part of this section, conclusions are drawn with the respect to the different coastal management strategies.

B.2 Biorock®

B.2.1 General

Biorock is a product for creating artificial reefs. By placing a relative simple steel structure in the sea combined with the Biorock technology its provides great benefits to the local marine ecosystem. The Biorock technology has been developed by a team led by Professor Wolf Hilbertz and Dr. Tom Goreau. The development in the past 30 years resolved in a product which is applied all over the world to solve the environmental problems .The concept initially was to naturally ‘’mine” minerals in seawater as an alternative construction material. Hilberts soon realized the great potential for marine applications. The Biorock method is now used for many marine applications such as construction and restoration of coral reef habitat, mariculture, erosion control, shoreline enrichment, wave barriers and other specialized uses. According to Biorock it is the most cost effective and environmental friendly solution available for protection coastal resources from the effect of global warming and sea level rise [19].

Figure B.1: Biorock frame in ocean.

The method of Biorock is based on mineral accretion technology. Due to an electric current in the steel frame, ionisation causes the precipitation of a layer of calcified minerals on the steel frame. The calcified material is natural limestone and this is the ideal base for the settlement of the coral reefs. To apply the technology an anode and a cathode is needed. The steel frame acts as an cathode and breaks down the seawater into hydrogen. An anode is placed in the seawater which brakes it down to oxygen. The most effective current for creating limestone is 1.23 V which is also the required minimum. Projects published in Goreau and Trench (2012) show the use of 3.8-17 V. The amount of ampere in not clear. Borrel et al. (2010) uses 12 V and 2.8 +-0.1 A/m^2. Even with a live current of 1 μA/cm^2 inhibit of coral settlement is shown, Benedetti et al (2005).[20] Decreasing of the seawater acidity causing new limestone minerals to grow on the steel. Studies have shown that coral larvae prefer to settle on clean limestone surfaces over any other naturally occurring or artificial construct [21].

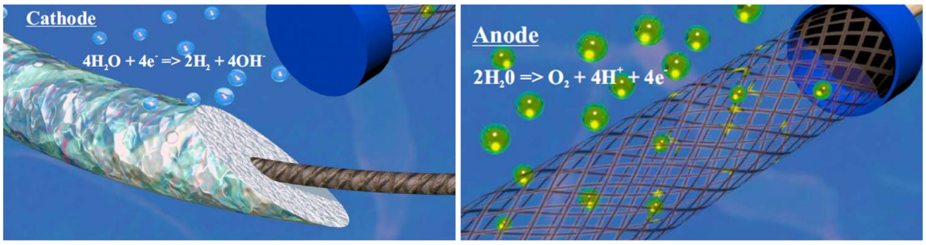


Figure B.2: Reaction process electric field.

The electric field around the steel frame also increases the resilience of the reef. The electric field creates an active ecosystem existing in a state of constant flux. There are different explanations for this phenomenon. The first theory argues that the polyps' exoskeletons might be affected by the electricity. This effect is similar to human accelerated bone fracture healing by electrical stimulation, which is a well-known theory in human medical science. The other explanation is that the electric field changes the properties of the surrounding seawater. Due to the electric current and the crystallization, limestone will also form close to the steel frame. On the crystallized limestone coral skeleton will grow. Normally the coral polyps uses their own energy for creating this base. Now the Biorock frame will create this base so the coral polyps can devote their energy to growth, reproduction and resisting environmental stresses.

The other advantage is the elevated position of the coral. In this position the polyps gain more metabolic energy. Other important benefits due to the elevated position are the reduced sedimentation stress and more exposure to stronger current which provides more zoo-plankton food. Biorock coral will built up more energy reserves from all the advantages to resist the adverse environmental changes [22].

The benefits from the Biorock can be expressed in numbers. By recovering the coral reef in the Maldives, a new beach of 15 meter wide grew behind the Biorock reef in 2-3 years and has been stable for more than 15 years. Biorock structures can grow upwards at around 20 mm/year. Taken into account the rising sea-level of 3-4 mm/year, the Biorock can keep up with the rising sea-level compared to conventional protection structures. The corals on Biorock grow up to 8 times faster and the reefs recover 20 times faster from physical damage. The resistance of the coral to environmental stresses like ocean acidification and global warming is 50 times higher in the electric field surrounding the Biorock [4]. Also the habitat of the fisheries are restored by the Biorock method. The Biorock creates a hiding place for juvenile fishes and turning the mortality rate of >95% into a survival rate of >95% [23].

The Biorock is a good solution to enhance the local ecosystems. Biorock can also absorb some wave energy. The structure will act as a permeable barrier to attenuate the waves. From the pilot projects the workability of the Biorock process has been shown. But for coastal protection the projects are limited and on a small scale. Therefore the effectiveness for coast protection is unsure.

B.2.2 Constructibility

The Biorock design is not restricted to certain shapes. In Indonesia local artists design the most creative structures to give the Biorock more aesthetic value for the tourist. Due to the this adaptability is it applicable for a wide range of situations. It can also be implemented on structures to create suitable holes for the marine life. This leads to higher population densities than natural reefs where the safe places for marine life are limited. Most of the projects creates only one Biorock frame. The Biorock frame is often welded on the beach close to the final location. After fabrication the frame is transported by a boat. For larger frames, buoy's are attached to float the construction to the final destination. When it is placed divers can attached 'loose' coral on the frame to enhance the coral colonization.

(a) Creative design.

(b) Coral placement

(c) Transport of frame.

Figure B.3: Biorock® frames in water and on shore.

B.3 Reef Balls

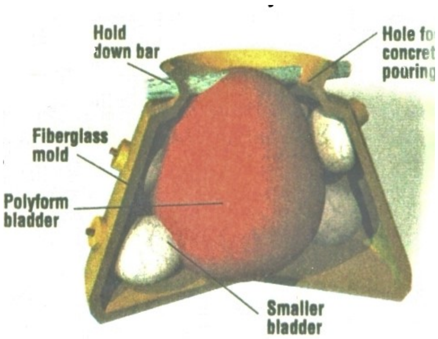
B.3.1 General

The mission of te Reef Ball Foundation is to rehabilitate and protect the ocean and natural reef ecosystems. The Reef Balls are the most commonly used structures in reef restoration projects. There are over 3500 projects conducted with a total of 0.5 million deployed Reef Balls. Despite the many projects, only 10 specifically used the Reef Balls for coastal protection purposes according to the survey [5]. The reef balls can only be deployed in shallow waters <2m and can be used as semicircular breakwaters (Armono and Hall 2002). Reef Balls are modules ranging in types and sizes. The concrete elements are hollow to provide a safe habitat for the marine life. To attenuate waves the element is made permeable by adding multiple holes. This also creates a specific water flow through the elements. This current provides extra stability and distribution of valuable nutrients for the marine life [24]. The modules can also be modified to accommodate seagrass and mangrove plants.

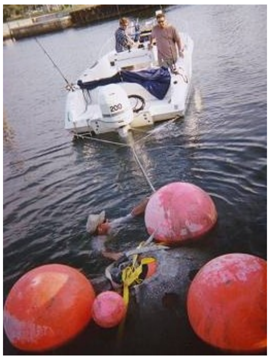
Figure B.4: Reef Balls in ocean.

B.3.2 Constructibility

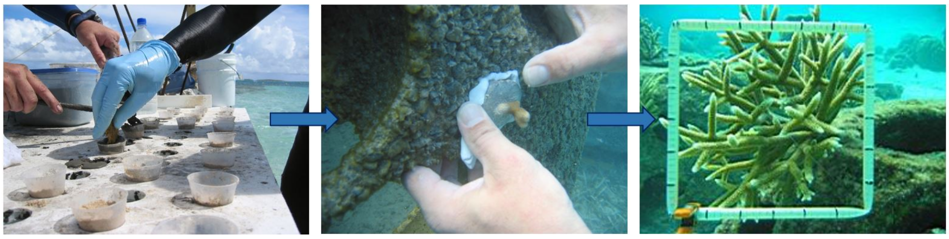
The Reef Balls are made of concrete mixed with microsilica, to match the pH of seawater, and non-corroding reinforced fibre. End of day concrete can be used when the additives are added to the mixture. Spraying the form-work with sugar water (delays hardening process of the concrete surface) and using admixtures for air voids in the concrete are two methods to created a textured surface. This is to promote settlement of the coral larvae [24]. The formwork is an easy to use portable fibreglass mold system. The holes are created with inflatable balls shown in Fig. B.5a. After deployment of the Reef Balls, attachment of the coral can be done. For the transplantation of the coral reef a special plugging process has to be followed. Corals have to be rescued, fragmented, plugged, and planted within a few hours. In only 20 months small coral plugs can results in a fully developed coral colony (see Fig. B.5c) [25].



(a) Formwork.



(b) Floating deployment.



(c) Coral attachment.

Figure B.5: Reef Balls during construction and in use.

The deployment of the elements can be by crane on a barge or by a floating device. For the floating deployment, a buoy will be attached to the element. The Reef Balls will be towed to the final place by a boat (see Fig. B.5b). Special attention is required to the safety of the operation. For barge deployment the water depth and available crane capacity have to be taken into account. For extra stability the Reef Balls can be anchored to the seabed.

B.4 Wave attenuation devices

B.4.1 General

Wave Attenuation Devices (WADs) or Coast Havens are methodically designed with the principles of hydrodynamics to reduce the wave energy near the shore. WADs will protect and restore the shoreline. Because of the engineering approach the WADs are extremely stable and effective. A study shows that the WADs are capable of reducing both wave height and wave energy substantially over 80%. The configuration and design of the structure is strongly influencing the functioning of the WAD [26]. Reference projects show that WAD systems remain in place during category 5 hurricanes. The systems, depending on the size, can be placed in water depths ranging from 1 to 25 feet. Generally the modules emerge a little from the water.

Figure B.6: Wave attenuation devices in ocean.

The holes in the structure provide essential fish habitat. Therefore the WADs are also referred to as fish havens. They can also be applied as oyster havens. From endorsement letters it is shown that the fish havens truly increase the marine life [27].

B.4.2 Constructibility

The WADs are specially made for every project. They are constructed in a patented process using tested, marine grade, reinforced concrete. The minimum strength is 34 MPa. The modules are cast in a welded steel formwork. The pattern of deployment is critical for the effectiveness of the structures. They are placed by crane in a sawtooth pattern (see Fig. B.6).

B.5 Building with nature

Coastal management techniques can be divided into two categories; *hard engineering* and *soft engineering*. Hard engineering options are efficient in wave reduction and tend to be reliable short term options. However they are usually more expensive and they could have a high impact on the landscape or environment. Examples of such constructions are sea walls, groynes and rock armour barriers. Soft engineering solutions are often less expensive than hard engineering solutions. Also, they are usually more sustainable and long-term. This category of solutions has less impact on the environment. Soft engineering solutions can be divided in two categories; *beach management* and *managed retreat*. Beach management is done by replacing beach of cliff material that has been removed by erosion or longshore drift; beach nourishment. Managed retreat is a coastal management technique that allows certain low value areas to erode and overflow. [28]

Building with Nature is a relatively new philosophy in hydraulic engineering that utilizes the forces of nature, thereby strengthening nature, economy and society [6]. Building with Nature solutions generally consist of a combination of green (soft) and grey (hard) engineering measures. Depending on the situation an ideal combination can be found.

B.6 Reference projects

Over the years many coastal challenges are tackled. For some of them, a Building with Nature solution is used, or can be used as a durable and long term strategy. The project in Java shows an effective and successful application and the project in Negril shows a potential site for the application of Building with Nature.

B.6.1 Biorock projects

A coral restoration project named 'The Necklace' in Ihuru, N Male Atoll is a 45 m long, 4-8 m wide, 1.5m high steel structure transplanted with nearby loose coral. The Biorock was constructed in 1997. The beach grew 15 meter in 3 years and was not affected by erosion during the 2004 tsunami. The beaches and reef perform well based on the photos taken in 2012.

The Biorock Antiwave project in Gili Trawangan (Lombok, Indonesia) had a positive effect on the beach within the first months. The beach grew 1 m/year and did not erode during the monsoon season. Because of bad maintenance, the structure started to rust and a storm in 2012 eroded the shoreline. The effect of the Biorock on the beaches were small and not sufficient to defend the beach from a storm [29].

B.6.2 Reef Balls Projects

The project at Gran Dominicus Resort (Dominican Republic) was the first Reef Ball breakwater project. During the project a lot of modification had to be done to the keep the stability of the structure. The breakwater could not resist the storm surges. After the changes, the sand volumes increased in the first 2 years but the control profile showed no change in the beach width. The Reef Balls did show an increase in colonized massive corals [30].

At the project site at Seven Miles Beach (Grand Cayman) also a Reef Balls breakwater is realized. At Seven Miles Beach, the beach grew by 15 meters in the first 3 months and reached 25 meters after 4 years. The beach erosion was before the project 3.2m/year and after the installation it accreted 3.4m/year. The breakwater reduced wave height by 60%. During the hurricane Ivan the beach experienced some erosion but this was mitigated. The coral was damaged during the hurricane and repaired after. No information is available about the state of the coral and beaches since [31].

The largest Reef Ball breakwater was placed at Maiden Island (Antigua). 3500 Reef Balls were deployed. The main goal was to improve recreational value of the beach by attenuating wave energy. No data is available regarding the shoreline response. It is not known if the breakwater works well. The coral that was planted on the reef balls had a 95% survival rate in the first 3 years [32].

B.6.3 Wave Attenuation Devices Projects

For accretion of the beach in Negril 160 WAD units were placed. The 4 mile breakwater was placed from land without disturbance to the marine ecosystem. After deployment marine life was observed in and on the barrier reef system. After the first year an estimated 1.857 metric tons of additional marine life biomass was measurable. After twenty-one weeks a survey was done for the accretion of the beach. A mean sand accretion of 45 feet was measured. During the second survey (44 weeks) te maximum sand accretions was 74 feet. The WAD created a newly stabilized beach in less than a year. Effects on the surrounding beaches is not known.

B.6.4 Concrete structure projects

There has been some pilot projects which tried to create a living breakwater from existing concrete structures like reef-blocks (modified SHED's), mattresses and rock piles. The focus on all of those projects were to restore the reef. There is no data available over the workability as a breakwater regarding to the coastal protection.

B.6.5 Building with Nature - Strongly eroded coastline Northern Java

The coastline in northern Java, Indonesia has retreated locally more than 1.5 km in the last 10 years. The strongly increased population, led to an increased need for food. For this purpose, the mangroves along the shoreline where cut down in favour to fishponds. As a result, the natural coastal defence weakened and erosion took place. In order to prevent further erosion, authorities decided to built concrete sea walls. However, as a consequence of the unstable foundation, the turbulence of the waves increased and erosion was enhanced instead of decreased. [33]

Figure B.7: Building of the permeable dams close to Demak, the pilot site for the Building with Nature solution [INFIS].

The Ecoshape consortium tried tackle the problem with a combination of hard and soft engineering solutions. Semipermeable dams (consisting of bamboo piles and brush wood) in front of the shoreline were used which took up part of the wave energy, while allowing the slib to pass and settle behind the dams (see Fig. B.7). As slib is the breeding ground for mangroves, the mangroves are restored and the (natural) coastal defence is enhanced. To increase the support from the local residents and to show them the value of this solutions, local people are building the dams and education is given about maintenance works of the dams. [33].

B.6.6 Building with Nature - Negril beach erosion

At the coast of Negril, specifically Long Bay and Bloody bay, severe erosion is taking place. The southern shoreline of Long Bay is retreated by 15 meters over the last 50 years [34]. The current rate of erosion has been established at 1 meter per year at Long Bay and 0.5 meter per year at Bloody Bay. To prevent tourism decrease, finding a long term solution for the erosion is of great importance.

The main cause for the erosion is the degradation of sea grasses[35]. Seagrasses tackle waves and currents and capture sand. In one area on Long Bay beach, the lack of seagrass beds could explain 41% of shoreline recession. Another cause can be found in the eroded (coral) reefs, resulting in more wave energy reaching the coast. The third cause is the decreased area of wetlands. In the 1950's wetlands have been converted into cropland. Wetlands, like mangroves, play an important role in the sediment process, wave attenuation and quality of the water.

Domestic sewage discharge pollution, agricultural fertilizer run-off and overfishing are an important factor in the decrease of the natural coastal protection. Not only at Negril, but in the whole Caribbean excessive fertilization of coastal waters is found [35]. The increased amount of nutrient fuels the grow of free-floating microscopic plants. Once these phytoplankton die, they sink and decompose, leading to a reduction in the available oxygen content at the bottom of the ocean. Combined with the decreased light penetration through the ocean, the amount of vegetation at the bottom of the ocean goes down.

Because of the reduced amount of vegetation, measures should be taken to defend the coast against excessive erosion. Hard measures can be applied as a mean to tackle the erosion, however long-term effects of these measures (i.e. groynes, seawalls and breakwaters) can be adverse on the landscape and coastal ecosystem [36].

Another option is the application of beach nourishment. Sediments are repetitively brought to the site to elevate and extend the beach seaward. However, there are uncertainties about ecological impacts on both mining and target sites.

More sustainable solutions are based on enhancing the coastal ecosystem. Coastal vegetation, seagrass beds, coral reefs, and wetlands are natural protective buffers and barriers against storm surges, flooding and landslides. To recover them, it is of great importance to look at the relationship between marine, estuarine and wetland processes. Those ecosystems are tightly connected and only if all ecosystems will be enhanced, success can be expected.

Coral transplantation can be used to grow artificial reefs with relative ease. The recovery of transplanted coral normally takes 2 to 5 years [35]. Tests done offshore Gran Dominnicus with 450 reef balls showed promising results. Unscathed by hurricanes and the beach gained 12 meters in 3 years. Also, sea grass planting can be done successfully. However, hurricanes are able to wipe out a vast amount.

It is important to treat waste water as they lead to eutrophication and to coral diseases. Wetlands can keep nutrients from polluted water, and so lessen the impact on marine ecosystems. Functioning marine ecosystems provide protection for erosion, storms, beach sedimentation and land-based pollution.

B.7 Conclusions

For the Montego Bay site, it is wise to choose a long term solution which enhances the total ecosystem. In this way the nature will help the future protection of the coast. As in the case of Java, the use of permeable dams could be a successful sub-solution for the place where polluted water is entering the ocean. Putting a mangrove in front of the beaches is off course not sensible, as tourists want a white beach with a view on clear water. Therefore, for the sandy shoreline, a hard (grey) solution will be proposed. However, this construction will be carefully designed to enable marine life to flourish. By the use of an open structure with electrified frames and permeable concrete, nature will help to protect the coast. Enhancing the marine life will be of interest of most of the stakeholders, as it will increase snorkelling opportunities. From the survey it became clear that the biggest project challenges where the lack of funding for maintenance(63.4%) and effective management/solving political problems(43.9%) [5]. The other projects challenges where due to global warming like bleaching(34.1%) and sedimentation(41.5%). The costs were also difficult to predict because of rarely available cost documentation and of the large amount of volunteering work in the projects. An estimation of the project cost for coastal defence is done and gives a cost indication ranging from US$ 2 000-1 000 000/ha.[5] Also monitoring of the beaches to require data is not extensively applied in this relative new field of engineering. In general there are limited hybrid coastal protection projects. The majority of the artificial reef project's focus only on one aspect. An effective modular systems which will take into account the erosion problem, enhancement of the marine life and restoration of the coral reef is not developed yet. Biorock shows that the resistance of coral can be increased but the effect of the structure on the shoreline is small, specially during storm surges. The mostly used hybrid protection structure is the Reef Ball. Reef Balls can attenuate the wave energy but the effectiveness to resist storm surges is also very low. The cost of the Reef Balls are relatively high compared to conventional breakwaters. The WAD unit are very effective water breakers and increase the marine life but do not enhance the coral reef. Further research in a hybrid and cost-effective coastal protection structure is recommended.

Table B.1: Overview effects coastal management techniques.

	Coral	**Fish**	**Wave attenuation**	**Modular**	**Costs**	**Hurricane stability**
BioRock	++	o	-	o	+	-
Reef balls	+	+	+	+	o	o
Fish haven	-	+	++	+	-	+

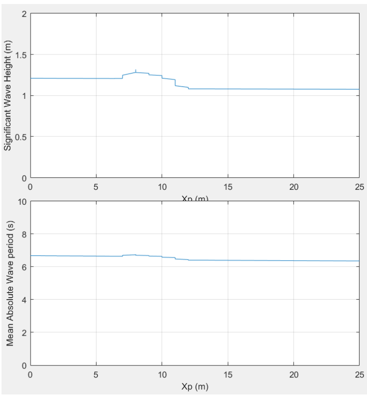
Appendix C

Results one-dimensional SWAN model

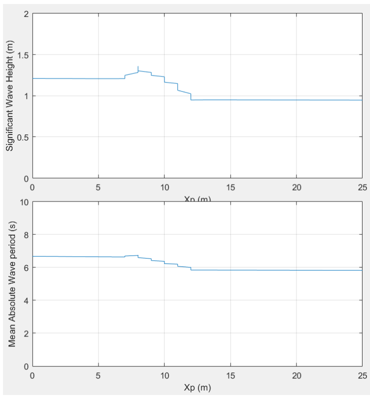
In Section 3.3, Eq. 3.2 is validated using a SWAN one-dimensional model. In this model different submerged breakwaters are tested on their effect on the significant wave height. In these models no wind and currents are included. Firstly, Tab. C.1 shows an short overview of the dimensions of the tested submerged breakwaters. After that several figures are shown with the results which are also given in Section 3.3.

Table C.1: Dimensions different submerged breakwaters.

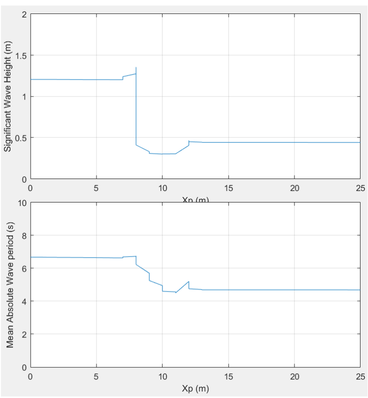
Breakwater type	Height [m]	Berm width [m]
Type 1	1.5	3
Type 2	2.0	3
Type 3	2.5	3
Type 4	2.0	2
Type 5	2.0	4
Type 6	2.0	6



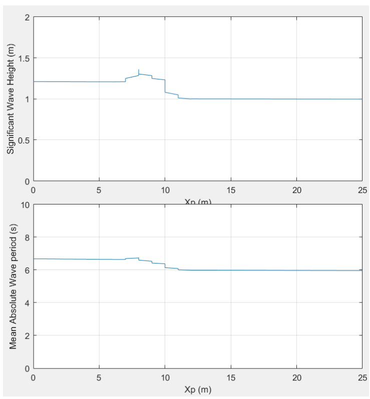
(a) Results SWAN submerged breakwater type 1.



(b) Results SWAN submerged breakwater type 2.

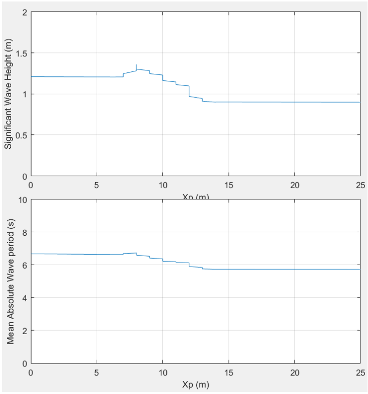


(c) Results SWAN submerged breakwater type 3.

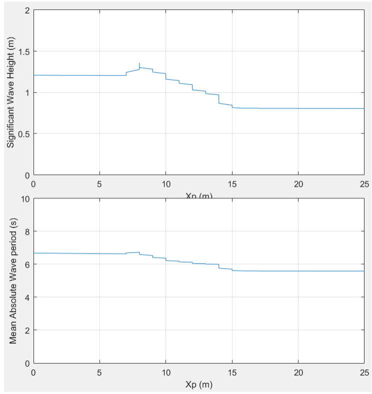


(d) Results SWAN submerged breakwater type 4.

Figure C.1: SWAN results breakwater type 1-4.



(a) Results SWAN submerged breakwater type 5.



(b) Results SWAN submerged breakwater type 6.

Figure C.2: SWAN results breakwater 5-6.

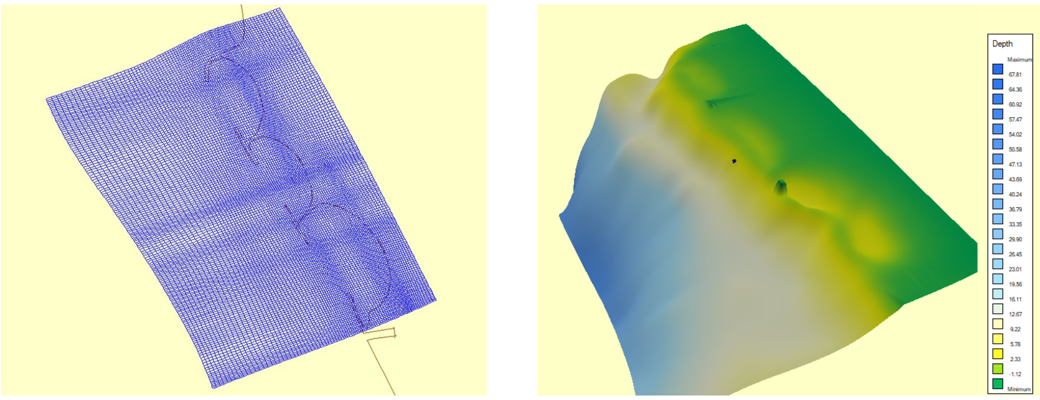
Appendix D

Delft3D model

In this report a Delft3D model is used to study flow- and wave patterns. In this Appendix an extensive overview is given about how the model came to be. Firstly the base model, Delft3D Flow module and its components are elaborated on. Afterwards the part of Delft3D Wave and its model are explained.

D.1 Grid & bathymetry

The base grid for the model is the whole area around the project site. This is done with the use of splines in the RFGrid program integrated in Delft3D. On this grid the depth is projected by using samples supplied by Smith & Warner International Ltd.. The resulting grid and bathymetry are given in Fig. D.1a and Fig. D.1b respectively. The approximated minimal grid sizes are $\Delta x = 4$ m and $\Delta y = 4$ m. The grid is loaded into Delft3D at a latitude of 18.5°. Furthermore the model is split into two layers.



(a) Grid for Delft3D Flow. **(b)** 3D view bathymetry.

Figure D.1: Grid and bathymetry of project site.

The area and beaches are protected by breakwaters. To model this correctly the choice is made to use thin dams to prevent flow velocities through breakwaters to model their impermeable core. Also thin dams have been placed in between the three beaches to prevent water flow from one to another. Furthermore the breakwaters are made to stop the waves. They are set to be non-reflective. The heights of the breakwaters are already mentioned in the Introduction.

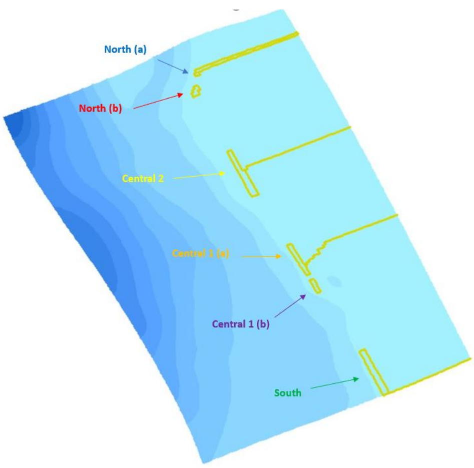
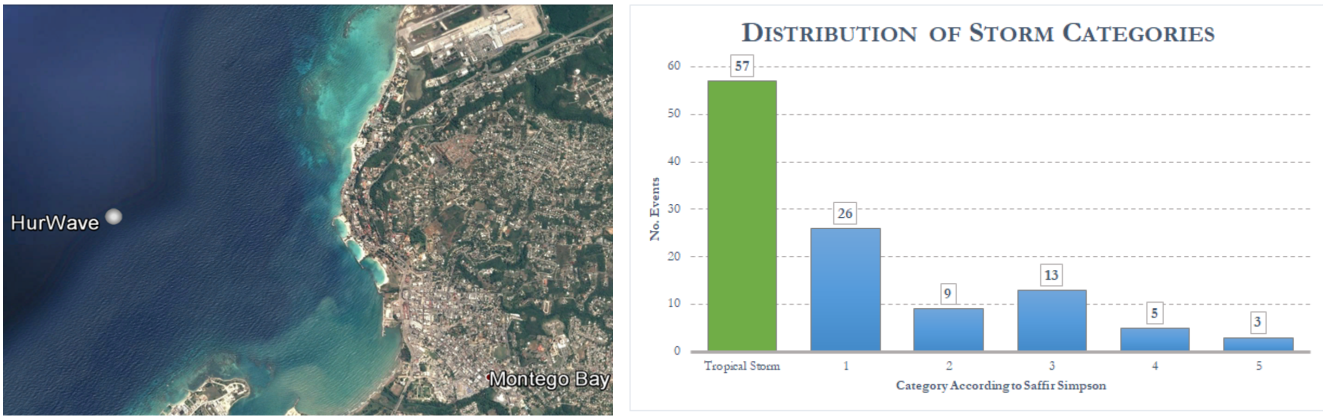


Figure D.2: Overview of thin dams in model area.

D.2 HurWave

In order to simulate the area under storm conditions the program HurWave is used. HurWave is a program that gives statistics of hurricane frequencies and occurrences in the North Atlantic Basin and performs external analysis to find extreme offshore conditions from parametric wave models [37]. In Fig. D.3b the number of storms and category from which the data is used in HurWave is shown. Fig. D.3a shows the location at which the modelled data is presented. HurWave collects data from storms passing in a radius of 300 kilometres.



(a) HurWave extraction point.

(b) Storm class data.

Figure D.3: Wind & wave data Montego Bay.

When a HurWave simulation is finished it returns the windspeed, wave height and the wave period for different directions. HurWave gives these values for 4 different return periods, 25 years, 50 years, 100 years, 200 years. In Fig. D.4 the results for a return period of 50 and 100 years are shown.

In Delft3D, the 50 year return period is chosen and simulated for different directions to see how the waves affect the location.

50 Year Return Period Simulations

Direction	Direction (deg)	Windspeed (m/s)	FACTOR Ws for MIKE21	Wave height (m)	Wave period (s)
North	0	30,91	18,55	6,48	10,71
North East	45	38,12	22,87	10,93	14,89
East	90	36,13	21,68	11,58	15,44
South East	135	34,60	20,76	10,19	14,24
South	180	29,19	17,51	8,30	12,52
South West	225	28,06	16,84	6,76	11,00
West	270	26,76	16,06	7,00	11,24
North West	315	24,49	14,69	6,71	10,95

Directional Spreading Factor	8,00
IBR (m)	0,34
Highest Astronomical Tide (m)	0,22
Sea level rise (m)	0,38
Storm Surge (m)	0,93

100 Year Return Period Simulations

Direction	Direction (deg)	Windspeed (m/s)	FACTOR Ws for MIKE21	Wave height (m)	Wave period (s)
North	0	35,00	21,00	7,82	12,05
North East	45	42,18	25,31	12,93	16,55
East	90	39,95	23,97	13,15	16,72
South East	135	38,38	23,03	11,39	15,28
South	180	32,89	19,73	9,66	13,77
South West	225	32,04	19,22	7,89	12,12
West	270	30,31	18,19	8,06	12,28
North West	315	28,15	16,89	7,57	11,81

Directional Spreading Factor	8,00
IBR (m)	0,42
Highest Astronomical Tide (m)	0,22
Sea level rise (m)	0,75
Storm Surge (m)	1,39

Figure D.4: Data from HurWave simulation for 1/50 and 1/100 years.

The model simulation also returns the increase in water level due to the IBR (reduced atmospheric pressure) as a result of a hurricane and wind set-up. Along with predicted sea level rise and the storm surge. Combined, these components will induce additional water level rise.

D.3 Yearly storm data

To find the yearly storm values the data from WaveWatch 3 was analysed. In Fig. D.5 all available wave data from 1999 to 2007 is plotted. Data till 2018 was available but the dates were missing, making it hard to create a fitting plot of the correct wave height at the right date. Since the numbers were quite similar, the period between 1999-2007 can still be used as a reliable reference.

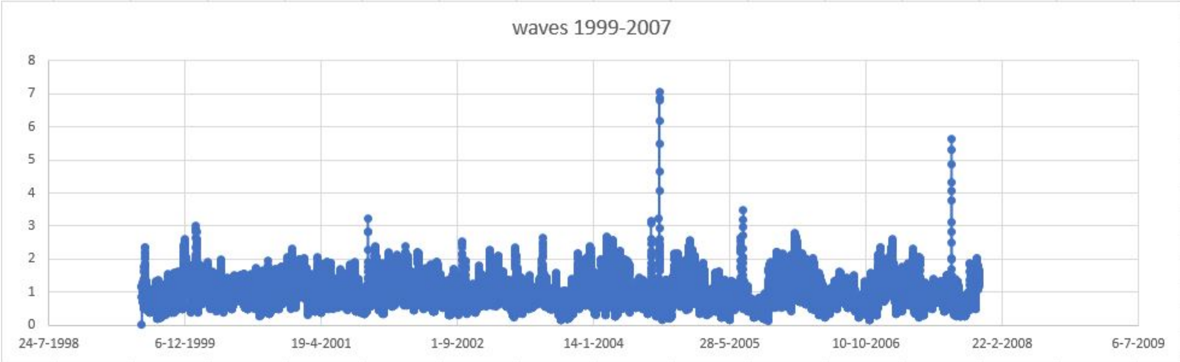
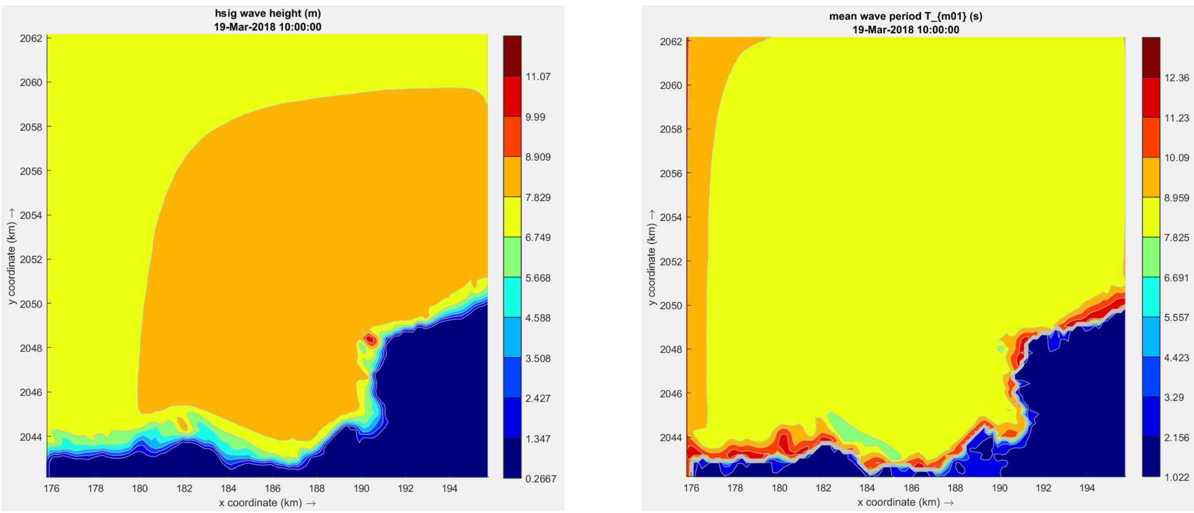


Figure D.5: Plot wave heights 1999-2007.

The data includes wave direction, wave height, wind speed and wind direction. Looking at the plotted wave heights, it became clear that storm conditions apply roughly for wave heights higher than 2.5 meters. From the WaveWatch 3 data, a Western storm was selected (which is governing for the area). This resulted in a significant wave height of 2.5 meters and a mean wave period of 8 seconds.

D.4 Remaining hurricane model results

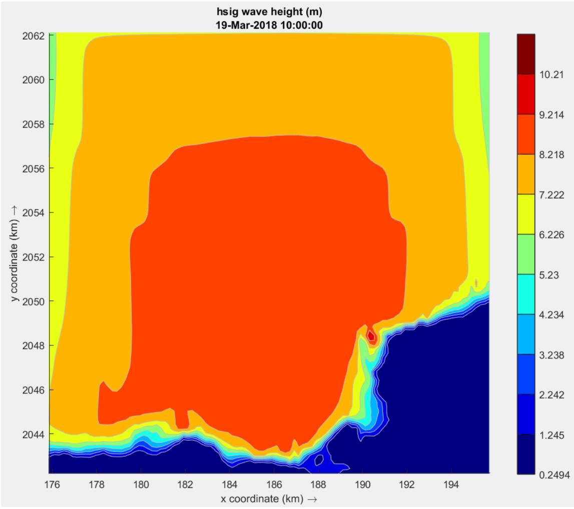
In the figure below the wave-heights and wave periods for hurricane conditions are simulated in Delft3D. A large area is used to allow deep-water to shallow water transition. In this section, the other results from Section 6.2.2 are shown.



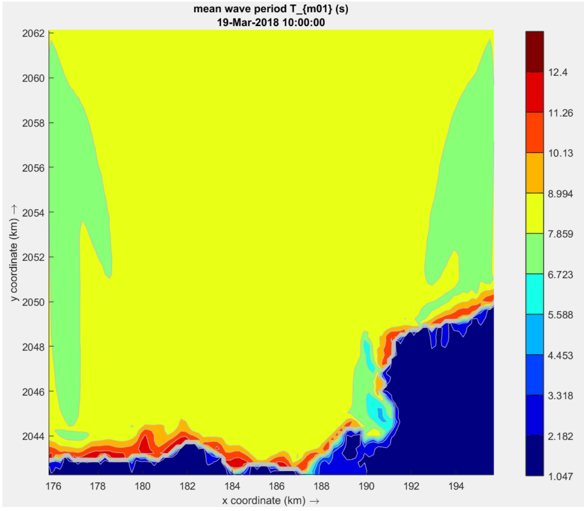
(a) Waveheight Nort-West 1/50 years.

(b) Wave period North-West 1/50 years.

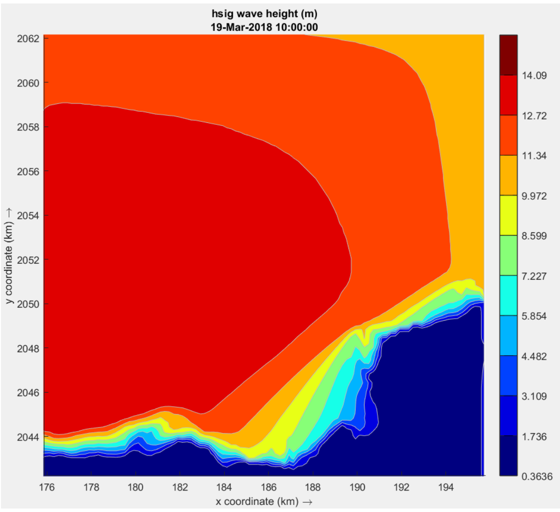
Figure D.6: Wave data hurricane.



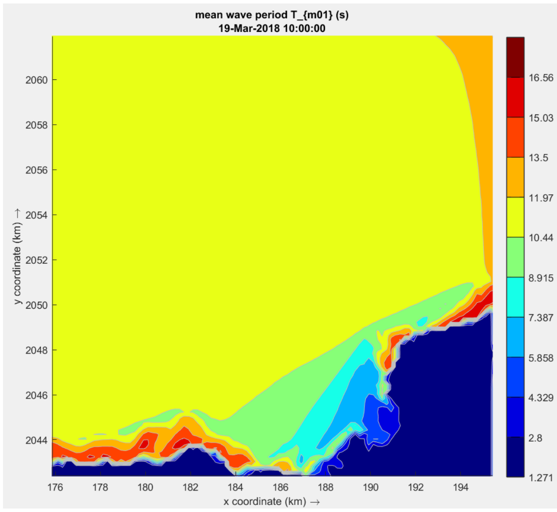
(a) Waveheight North 1/50 years.



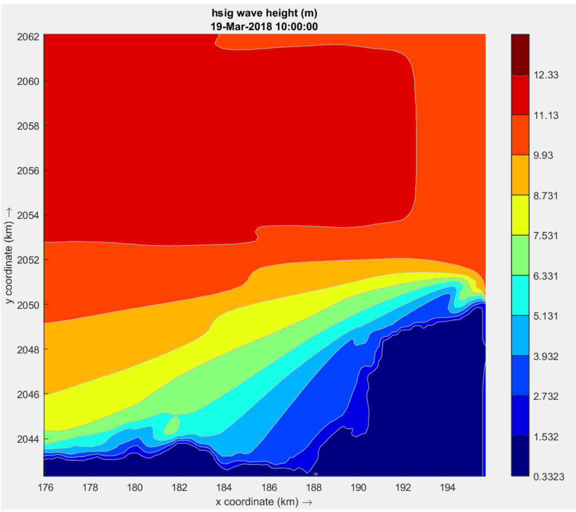
(b) Wave period North 1/50 years.



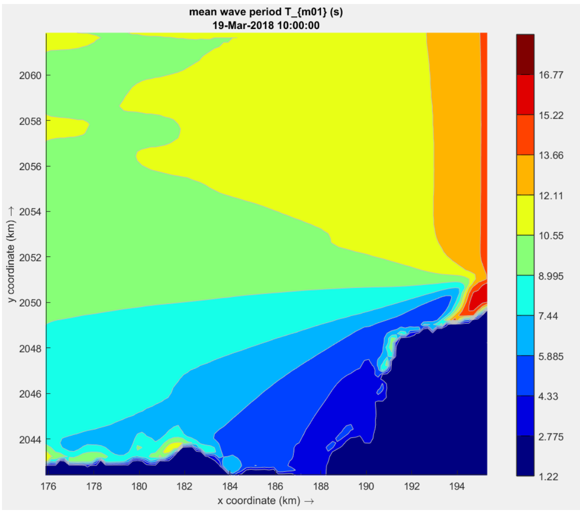
(c) Waveheight North-East 1/50 years.



(d) Wave period North-East 1/50 years.



(e) Waveheight East 1/50 years.

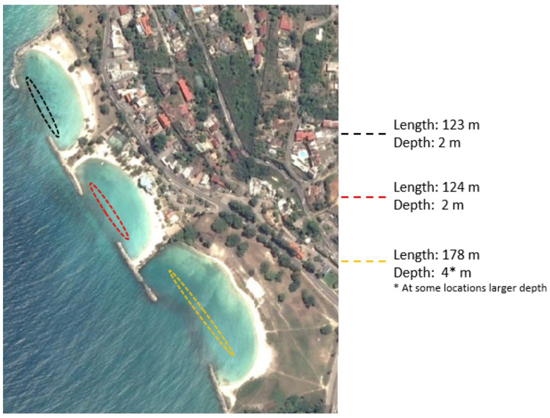


(f) Wave period East 1/50 years.

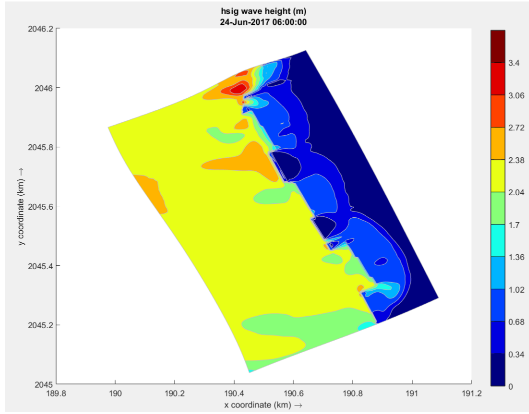
Figure D.7: Wave data hurricane.

D.5 Submerged breakwater lay-outs

The first configuration consists out of 3 long breakwaters over the length of the opening. For the top and the middle bay the water depths over this length is pretty constant at 2 meters. The bottom bay has more gradient over the length of the breakwater. At the north side the depth can increase to 5 meters, while in the middle the depth is only 3 meters. The south side is at 4 meters depth.



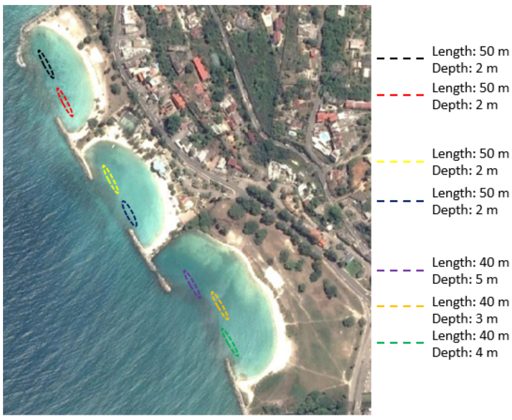
(a) Lay-out 1 [Google 2018].



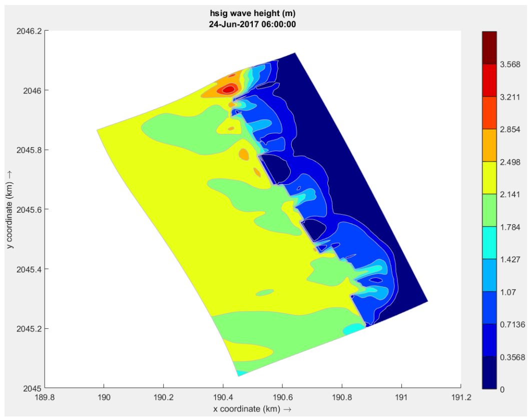
(b) Wave heights breakwater lay-out 1.

Figure D.8: Breakwater lay-out 1.

The second configuration consists out of several smaller submerged breakwaters that are aligned parallel to each other. The main reason to split up in several smaller parts is to avoid the problem of uniform depth. At the separate locations the depth is more constant.



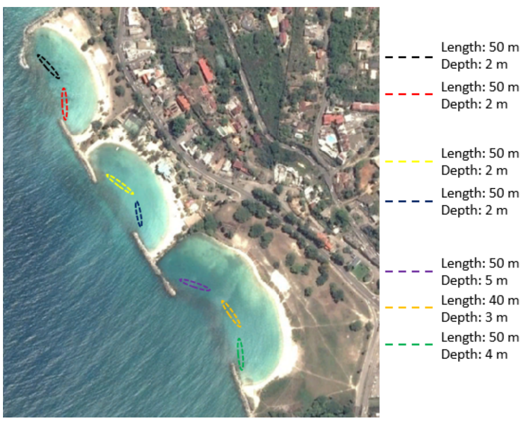
(a) Lay-out 2 [Google 2018].



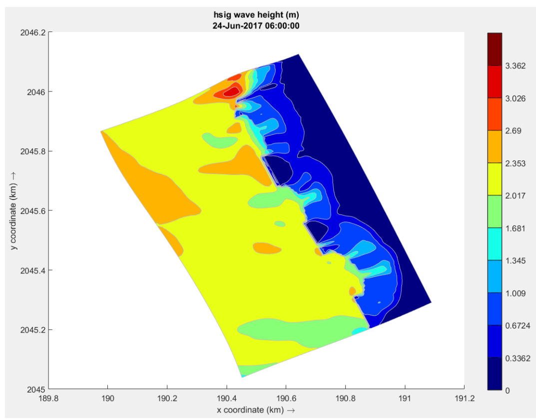
(b) Wave heights breakwater lay-out 2.

Figure D.9: Breakwater lay-out 2.

The third configuration is similar to the second with several smaller breakwaters. The main difference is that the outer breakwaters are turned in an angle. In this way it is perpendicular orientated to the west where the waves are coming from.



(a) Lay-out 3 [Google 2018].



(b) Wave heights breakwater lay-out 3.

Figure D.10: Breakwater lay-out 3.

Now for every sill several designs are compared with different heights and crest widths. The resulting attenuation factor K_t and transmitted wave height can be seen in Tab. D.11. Also the area for each design is listed. A slightly better attenuation for a much higher volume would not be economical.

	Hs (m) Incoming wave	Depth (m)	Structure height (m)	Crest width (m)			Kt			Ht (m)			Area (m2/m)		
							Attenuation factor			Transmitted wave			Area of structure		
Sill 1 & 2	1.8	2	1.4	2	3	4	0.57	0.54	0.51	1	0.97	0.92	3,9	5,3	6,7
			1.5	2	3	4	0.53	0.5	0.47	0.96	0.9	0.85	4,3	5,8	7,3
			1.6	2	3	4	0.49	0.46	0.43	0.9	0.83	0.78	4,7	6,3	7,9
Sill 3	2	5	3,5	2	3	4	0,67	0,66	0,65	1,34	1,33	1,32	14,1	17,6	21,1
			4	2	3	4	0,59	0,58	0,57	1,19	1,16	1,13	17,2	21,2	25,2
			4,5	2	3	4	0,5	0,48	0,46	1	0,96	0,91	20,7	25,2	29,7
Sill 4	2	3	2	2	3	4	0,64	0,63	0,61	1,28	1,25	1,22	6,3	8,3	10,3
			2,3	2	3	4	0,57	0,55	0,53	1,14	1,1	1,05	7,7	9,7	12,3
			2,5	2	3	4	0,52	0,49	0,47	1,04	0,99	0,94	8,6	11,1	13,6
Sill 5	2	4	2.5	2	3	4	0.7	0.7	0.69	1.4	1.39	1.37	8,6	11,1	13,6
			3	2	3	4	0.61	0.6	0.58	1.22	1.20	1.16	11,2	14,2	17,2
			3.5	2	3	4	0,51	0,49	0.46	1	0.96	0.92	14,1	17,6	21,1

Figure D.11: Wave attenuation for different configurations.

D.6 Grain size distribution Montego Bay

In the sediment transport model runs, a nominal grain size diameter of 300 μm is used. This is approximately the average of the grain sizes determined by Smith & Warner International Ltd. at the north coast near Montego Bay. It is assumed that a similar distribution is located at this project site.

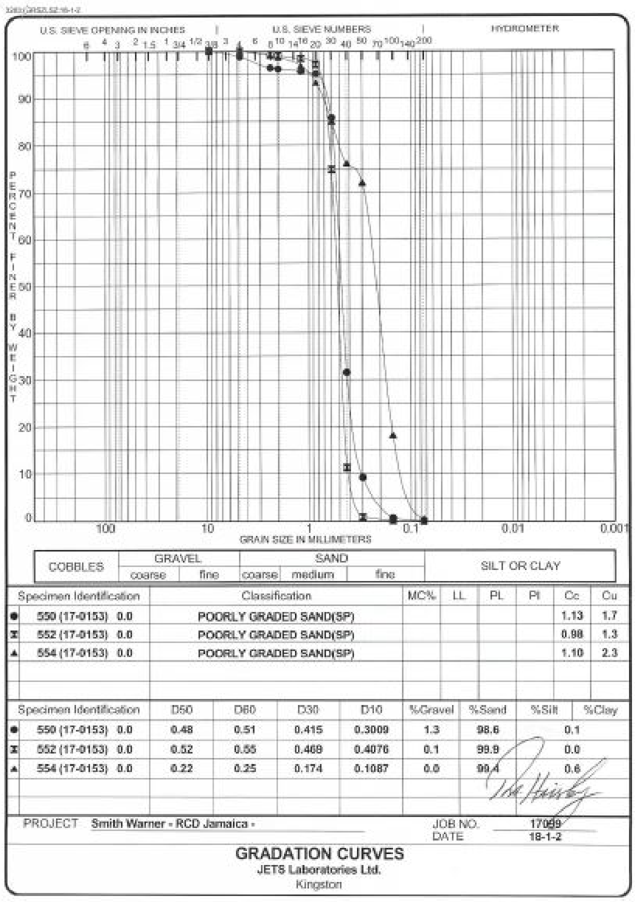


Figure D.12: Sediment grain size distribution [Hard Rock deliverable, Smith & Warner International Ltd.].

Appendix E

Wave force

E.1 Drag & lift Forces

In order to determine the forces acting on the submerged structure the following formulas have been used [38]:

$$F_L = \frac{1}{2} C_l \rho_w A u^2 \tag{E.1}$$

$$F_x = \frac{1}{2} C_D \rho_w A u^2 + \rho_w C_m V \frac{du}{dt} \tag{E.2}$$

with

$$\begin{aligned}
C_l &= \text{Lift coefficient } [-] \\
C_D &= \text{Drag coefficient } [-] \\
C_m &= \text{Inertia coefficient } [-] \\
\rho_w &= \text{Water density } [\mathrm{kg\,m^{-3}}] \\
A &= \text{Reference area of the body } [\mathrm{m^2}] \\
V &= \text{Volume of the body } [\mathrm{m^3}] \\
u &= \text{Flow velocity } [\mathrm{m\,s^{-1}}] \\
\frac{du}{dt} &= \text{Flow acceleration } [\mathrm{m\,s^{-2}}]
\end{aligned}$$

Eq. E.1 represents the uplift force due to water flow around the body. The force parallel to the flow direction is determined with the Morison equation (see Eq. E.2) and is the sum of two components: an inertia force in phase with the local flow acceleration and the drag force. The Morison equation contains two empirical coefficients: a drag coefficient C_D and an inertia coefficient C_m. Both are determined from experimental data.

E.1.1 Input

In the last section, Section E.1, the force formula's were introduced. In these formula's several constants are included. The values corresponding to the constant are given below.

Table E.1: Constants for equations E.1 & E.2

Parameter	Value
$\mathbf{C_l}$ [−]	0.8
$\mathbf{C_D}$ [−]	0.8
$\mathbf{C_m}$ [−]	2.0
$\rho_\mathbf{w}$ [kg m^{-3}]	1025

The horizontal flow velocity in this model has a maximum constant component u_0 and a time-varying harmonic component due to the waves. The expressions for the horizontal flow velocity and acceleration are defined by the following equations (Eq. E.3 and Eq. E.4).

$$u = u_0 + \omega \cdot a \frac{\cosh k(d+z)}{\sinh kd} \sin \omega t \tag{E.3}$$

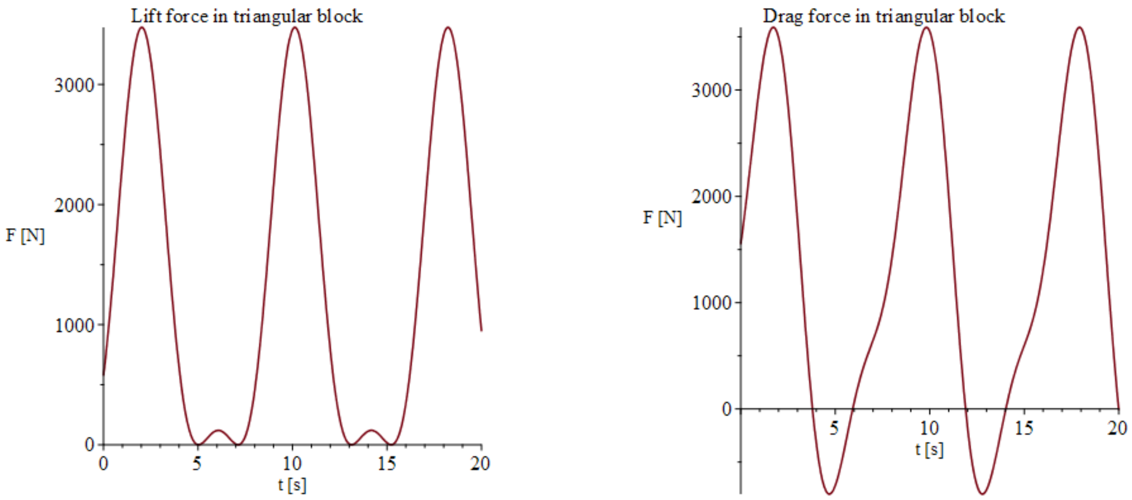
$$\frac{du}{dt} = \omega^2 \cdot a \frac{\cosh k(d+z)}{\sinh kd} \cos \omega t \tag{E.4}$$

With:

$$\begin{aligned}
u_0 &= 1.4 \quad \mathrm{m\,s^{-1}} \\
T_{m0} &= 8.1 \quad \mathrm{s} \\
H_s &= 3 \quad \mathrm{m} \\
d &= 4 \quad \mathrm{m} \\
\omega &= \frac{2\pi}{T_{m0}} = 0.78 \quad \mathrm{rad\,s^{-1}} \\
L &= \sqrt{9.81 d}\, T_{m0} = 56 \quad \mathrm{m} \\
a &= \frac{1}{2} H_s = 1.5 \quad \mathrm{m} \\
z &= -1 \quad \mathrm{m}
\end{aligned}$$

E.1.2 Resulting forces

The lift and drag forces are calculated for the two types of blocks, the triangular and hexagonal blocks. The resulting forces are a function of time as can be seen in Fig. E.1 & Fig. E.2. The maxima of these forces are used in the stability models in ANSYS.

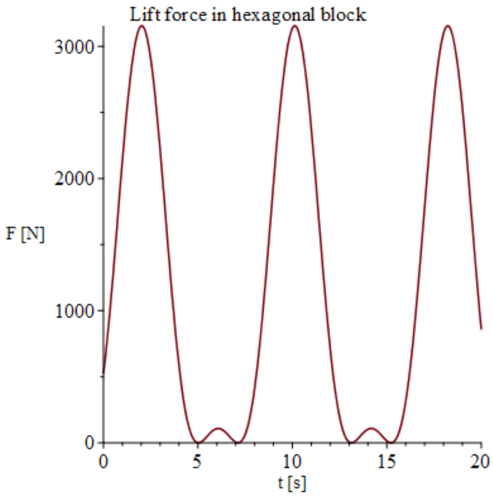


(a) Lift force on triangular block.

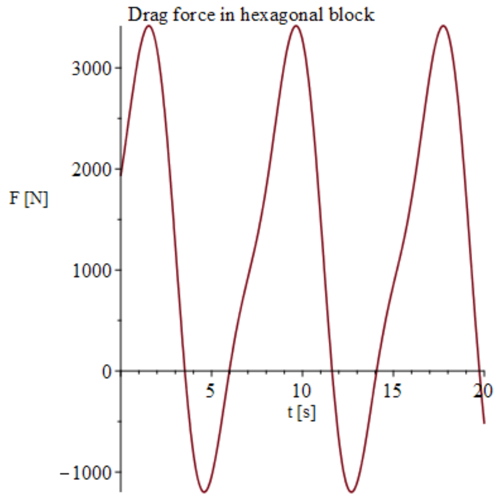
(b) Drag force on triangular block.

Figure E.1: Forces on triangular block.

The difference in forces for the two blocks is the results of the different area facing the flow and the volume of the blocks. The values which are used for the triangular blocks are: $A_t = 0.72\,\mathrm{m}^2$ and $V_t = 0.30\,\mathrm{m}^3$. The hexagonal block values are $A_h = 0.65\,\mathrm{m}^2$ and $V_h = 0.43\,\mathrm{m}^3$ respectively. It must noted that the resulting forces shown in Fig. E.1 & Fig. E.2 are per running meter.



(a) Lift force on hexagonal block.



(b) Drag force on hexagonal block.

Figure E.2: Forces on hexagonal block.

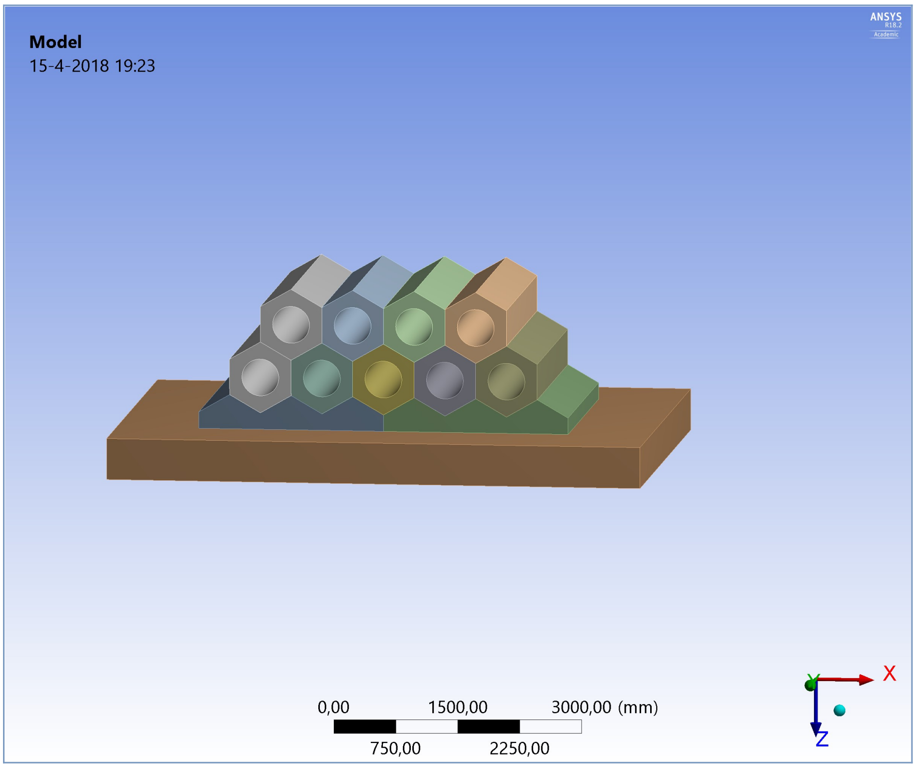
Appendix F

ANSYS report hexagonal structure

In this chapter, the settings of the ANSYS model are shown. The report is from the hexagonal structure, with the impact wave force profile. Other details can be found on the following pages.

Project

First Saved	Tuesday, March 20, 2018
Last Saved	Wednesday, March 28, 2018
Product Version	18.2 Release
Save Project Before Solution	No
Save Project After Solution	No



Contents

Report Not Finalized

Not all objects described below are in a finalized state. As a result, data may be incomplete, obsolete or in error. View first state problem. To finalize this report, edit objects as needed and solve the analyses.

Units

TABLE 1

Unit System	Metric (mm, t, N, s, mV, mA) Degrees rad/s Celsius
Angle	Degrees
Rotational Velocity	rad/s
Temperature	Celsius

Model (A4)

Geometry

TABLE 2
Model (A4) > Geometry

Object Name	*Geometry*

State	Fully Defined
Definition	
Source	C:\Users\Marc\Documents\TU Delft_MASTER - SE_ANSYS FILES_MARC\Maik\Hexa_AllforceNewwave_Fric0,3_impactWave0.1sec_Totcon_Lightblocks_files\dp0 \SYS-1\DM\SYS-1.scdoc
Type	SpaceClaim
Length Unit	Meters
Element Control	Program Controlled
Display Style	Body Color
Bounding Box	
Length X	6500, mm
Length Y	5000, mm
Length Z	2215,5 mm
Properties	
Volume	2,927e+010 mm³
Mass	68,623 t
Scale Factor Value	1,
Statistics	
Bodies	12
Active Bodies	12
Nodes	15990
Elements	2750
Mesh Metric	None
Basic Geometry Options	
Solid Bodies	Yes
Surface Bodies	Yes
Line Bodies	Yes
Parameters	Independent
Parameter Key	
Attributes	Yes
Attribute Key	
Named Selections	Yes
Named Selection Key	
Material Properties	Yes
Advanced Geometry Options	
Use Associativity	Yes
Coordinate Systems	Yes
Coordinate System Key	
Reader Mode Saves Updated File	No
Use Instances	Yes
Smart CAD Update	Yes
Compare Parts On Update	No
Analysis Type	3-D
Mixed Import	None

Resolution	
Decompose Disjoint Geometry	Yes
Enclosure and Symmetry Processing	Yes

TABLE 3
Model (A4) > Geometry > Parts

Object Name	*SYS-1 Light blocks\Solid*	*SYS-1 Light blocks\Solid*	*SYS-1 Light blocks\Solid*	*SYS-1 Bonded\Soil*	*SYS-1 Light blocks\Solid*
State	Fully Defined				
Graphics Properties					
Visible	Yes				
Transparency	1				
Definition					
Suppressed	No				
Stiffness Behavior	Flexible				
Coordinate System	Default Coordinate System				
Reference Temperature	By Environment				
Behavior	None				
Material					
Assignment	Concrete			Limestone (NL)	Concrete
Nonlinear Effects	No			Yes	No
Thermal Strain Effects	No				
Bounding Box					
Length X	750, mm	2250, mm		6500, mm	750, mm
Length Y	3000, mm			5000, mm	3000, mm
Length Z	866,03 mm	416,51 mm		500, mm	866,03 mm
Properties					
Volume	9,843e+008 mm^3	2,0807e+009 mm^3		1,625e+010 mm^3	9,843e+008 mm^3
Mass	2,3623 t	4,9937 t		37,375 t	2,3623 t
Centroid X	-1724,8 mm	-1349,8 mm	900,24 mm	-224,76 mm	-974,76 mm
Centroid Y	1500, mm				
Centroid Z	-2025,2 mm	-1552,7 mm		-1142,2 mm	-2025,2 mm
Moment of Inertia Ip1	1,8983e+006 t·mm^2	3,7944e+006 t·mm^2		7,8643e+007 t·mm^2	1,8983e+006 t·mm^2
Moment of Inertia Ip2	2,4529e+005 t·mm^2	2,1147e+006 t·mm^2		1,3237e+008 t·mm^2	2,4529e+005 t·mm^2
Moment of Inertia Ip3	1,8983e+006 t·mm^2	5,8109e+006 t·mm^2		2,0946e+008 t·mm^2	1,8983e+006 t·mm^2
Statistics					
Nodes	809	1501	1323	6093	809
Elements	132	240	216	1040	132
Mesh Metric	None				
CAD Attributes					
PartTolerance:	0,00000001				
Color:143.175.143					

TABLE 4
Model (A4) > Geometry > Parts

Object Name	*SYS-1 Light blocks\Solid*	*SYS-1 Light blocks\Solid*	*SYS-1 Light blocks\Solid*	*SYS-1 Light blocks\Solid*	*SYS-1 Light blocks\Solid*
State	Fully Defined				
Graphics Properties					
Visible	Yes				
Transparency	1				
Definition					
Suppressed	No				

Stiffness Behavior	Flexible				
Coordinate System	Default Coordinate System				
Reference Temperature	By Environment				
Behavior	None				
Material					
Assignment	Concrete				
Nonlinear Effects	No				
Thermal Strain Effects	No				
Bounding Box					
Length X	750, mm				
Length Y	3000, mm				
Length Z	866,03 mm				
Properties					
Volume	9,843e+008 mm³				
Mass	2,3623 t				
Centroid X	-224,76 mm	525,24 mm	1275,2 mm	-1349,8 mm	-599,76 mm
Centroid Y	1500, mm				
Centroid Z	-2025,2 mm			-2674,8 mm	
Moment of Inertia Ip1	1,8983e+006 t·mm²				
Moment of Inertia Ip2	2,4529e+005 t·mm²				
Moment of Inertia Ip3	1,8983e+006 t·mm²				
Statistics					
Nodes	809	751	821		751
Elements	132	143			
Mesh Metric	None				
CAD Attributes					
PartTolerance:	0,00000001				
Color:143.175.143					

TABLE 5
Model (A4) > Geometry > Parts

Object Name	*SYS-1 Light blocks\Solid*	*SYS-1 Light blocks\Solid*
State	Fully Defined	
Graphics Properties		
Visible	Yes	
Transparency	1	
Definition		
Suppressed	No	
Stiffness Behavior	Flexible	
Coordinate System	Default Coordinate System	
Reference Temperature	By Environment	
Behavior	None	
Material		
Assignment	Concrete	
Nonlinear Effects	No	
Thermal Strain Effects	No	
Bounding Box		
Length X	750, mm	
Length Y	3000, mm	
Length Z	866,03 mm	
Properties		
Volume	9,843e+008 mm³	
Mass	2,3623 t	
Centroid X	150,24 mm	900,24 mm
Centroid Y	1500, mm	
Centroid Z	-2674,8 mm	
Moment of Inertia Ip1	1,8983e+006 t·mm²	
Moment of Inertia Ip2	2,4529e+005 t·mm²	

Moment of Inertia Ip3	1,8983e+006 t·mm²
Statistics	
Nodes	751
Elements	143
Mesh Metric	None
CAD Attributes	
PartTolerance:	0,00000001
Color:143.175.143	

Coordinate Systems

TABLE 6
Model (A4) > Coordinate Systems > Coordinate System

Object Name	*Global Coordinate System*
State	Fully Defined
Definition	
Type	Cartesian
Coordinate System ID	0,
Origin	
Origin X	0, mm
Origin Y	0, mm
Origin Z	0, mm
Directional Vectors	
X Axis Data	[1, 0, 0,]
Y Axis Data	[0, 1, 0,]
Z Axis Data	[0, 0, 1,]

Connections

TABLE 7
Model (A4) > Connections

Object Name	*Connections*
State	Fully Defined
Auto Detection	
Generate Automatic Connection On Refresh	Yes
Transparency	
Enabled	Yes

TABLE 8
Model (A4) > Connections > Contacts

Object Name	*Contacts*	*Contacts 2*
State	Fully Defined	
Definition		
Connection Type	Contact	
Scope		
Scoping Method	Geometry Selection	
Geometry	All Bodies	
Auto Detection		
Tolerance Type	Slider	
Tolerance Slider	0,	
Tolerance Value	21,237 mm	
Use Range	No	
Face/Face	Yes	
Face Overlap Tolerance	Off	
Cylindrical Faces	Include	
Face/Edge	No	
Edge/Edge	No	
Priority	Include All	
Group By	Bodies	
Search Across	Bodies	
Statistics		
Connections	24	1

Active Connections	24	1

TABLE 9
Model (A4) > Connections > Contacts > Contact Regions

Object Name	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Bonded\Soil*
State	Fully Defined				
Scope					
Scoping Method	Geometry Selection				
Contact	2 Faces	1 Face			
Target	2 Faces	1 Face			
Contact Bodies	SYS-1 Light blocks\Solid				
Target Bodies	SYS-1 Light blocks\Solid				SYS-1 Bonded\Soil
Definition					
Type	Frictional				
Friction Coefficient	0,3				
Scope Mode	Automatic				
Behavior	Program Controlled				
Trim Contact	Program Controlled				
Trim Tolerance	21,237 mm				
Suppressed	No				
Advanced					
Formulation	Augmented Lagrange				
Small Sliding	Program Controlled				
Detection Method	Program Controlled				
Penetration Tolerance	Program Controlled				
Elastic Slip Tolerance	Program Controlled				
Normal Stiffness	Program Controlled				
Update Stiffness	Program Controlled				
Stabilization Damping Factor	0,				
Pinball Region	Program Controlled				
Time Step Controls	None				
Geometric Modification					
Interface Treatment	Add Offset, No Ramping				
Offset	0, mm				
Contact Geometry Correction	None				
Target Geometry Correction	None				

TABLE 10
Model (A4) > Connections > Contacts > Contact Regions

Object Name	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Bonded\Soil*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*

State	Fully Defined				
Scope					
Scoping Method	Geometry Selection				
Contact	2 Faces	1 Face			2 Faces
Target	2 Faces	1 Face			2 Faces
Contact Bodies	SYS-1 Light blocks\Solid				
Target Bodies	SYS-1 Light blocks\Solid		SYS-1 Bonded\Soil	SYS-1 Light blocks\Solid	
Definition					
Type	Frictional				
Friction Coefficient	0,3				
Scope Mode	Automatic				
Behavior	Program Controlled				
Trim Contact	Program Controlled				
Trim Tolerance	21,237 mm				
Suppressed	No				
Advanced					
Formulation	Augmented Lagrange				
Small Sliding	Program Controlled				
Detection Method	Program Controlled				
Penetration Tolerance	Program Controlled				
Elastic Slip Tolerance	Program Controlled				
Normal Stiffness	Program Controlled				
Update Stiffness	Program Controlled				
Stabilization Damping Factor	0,				
Pinball Region	Program Controlled				
Time Step Controls	None				
Geometric Modification					
Interface Treatment	Add Offset, No Ramping				
Offset	0, mm				
Contact Geometry Correction	None				
Target Geometry Correction	None				

TABLE 11
Model (A4) > Connections > Contacts > Contact Regions

Object Name	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*
State	Fully Defined				
Scope					
Scoping Method	Geometry Selection				
Contact	2 Faces	1 Face			
Target	2 Faces	1 Face			
Contact Bodies	SYS-1 Light blocks\Solid				

Target Bodies	SYS-1 Light blocks\Solid
Definition	
Type	Frictional
Friction Coefficient	0,3
Scope Mode	Automatic
Behavior	Program Controlled
Trim Contact	Program Controlled
Trim Tolerance	21,237 mm
Suppressed	No
Advanced	
Formulation	Augmented Lagrange
Small Sliding	Program Controlled
Detection Method	Program Controlled
Penetration Tolerance	Program Controlled
Elastic Slip Tolerance	Program Controlled
Normal Stiffness	Program Controlled
Update Stiffness	Program Controlled
Stabilization Damping Factor	0,
Pinball Region	Program Controlled
Time Step Controls	None
Geometric Modification	
Interface Treatment	Add Offset, No Ramping
Offset	0, mm
Contact Geometry Correction	None
Target Geometry Correction	None

TABLE 12
Model (A4) > Connections > Contacts > Contact Regions

Object Name	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*
State	Fully Defined				
Scope					
Scoping Method	Geometry Selection				
Contact	1 Face				
Target	1 Face				
Contact Bodies	SYS-1 Light blocks\Solid				
Target Bodies	SYS-1 Light blocks\Solid				
Definition					
Type	Frictional				
Friction Coefficient	0,3				
Scope Mode	Automatic				
Behavior	Program Controlled				
Trim Contact	Program Controlled				
Trim					

Tolerance	21,237 mm
Suppressed	No
Advanced	
Formulation	Augmented Lagrange
Small Sliding	Program Controlled
Detection Method	Program Controlled
Penetration Tolerance	Program Controlled
Elastic Slip Tolerance	Program Controlled
Normal Stiffness	Program Controlled
Update Stiffness	Program Controlled
Stabilization Damping Factor	0,
Pinball Region	Program Controlled
Time Step Controls	None
Geometric Modification	
Interface Treatment	Add Offset, No Ramping
Offset	0, mm
Contact Geometry Correction	None
Target Geometry Correction	None

TABLE 13
Model (A4) > Connections > Contacts > Contact Regions

Object Name	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*	*Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*
State	Fully Defined			
Scope				
Scoping Method	Geometry Selection			
Contact	1 Face			
Target	1 Face			
Contact Bodies	SYS-1 Light blocks\Solid			
Target Bodies	SYS-1 Light blocks\Solid			
Definition				
Type	Frictional			
Friction Coefficient	0,3			
Scope Mode	Automatic			
Behavior	Program Controlled			
Trim Contact	Program Controlled			
Trim Tolerance	21,237 mm			
Suppressed	No			
Advanced				
Formulation	Augmented Lagrange			
Small Sliding	Program Controlled			
Detection Method	Program Controlled			
Penetration Tolerance	Program Controlled			
Elastic Slip Tolerance	Program Controlled			
Normal				

Stiffness	Program Controlled
Update Stiffness	Program Controlled
Stabilization Damping Factor	0,
Pinball Region	Program Controlled
Time Step Controls	None
Geometric Modification	
Interface Treatment	Add Offset, No Ramping
Offset	0, mm
Contact Geometry Correction	None
Target Geometry Correction	None

TABLE 14
Model (A4) > Connections > Contacts 2 > Contact Regions

Object Name	*Frictionless - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid*
State	Fully Defined
Scope	
Scoping Method	Geometry Selection
Contact	1 Face
Target	2 Faces
Contact Bodies	SYS-1 Light blocks\Solid
Target Bodies	SYS-1 Light blocks\Solid
Definition	
Type	Frictionless
Scope Mode	Manual
Behavior	Program Controlled
Trim Contact	Program Controlled
Suppressed	No
Advanced	
Formulation	Program Controlled
Small Sliding	Program Controlled
Detection Method	Program Controlled
Penetration Tolerance	Program Controlled
Normal Stiffness	Program Controlled
Update Stiffness	Program Controlled
Stabilization Damping Factor	0,
Pinball Region	Program Controlled
Time Step Controls	None
Geometric Modification	
Interface Treatment	Add Offset, No Ramping
Offset	0, mm
Contact Geometry Correction	None
Target Geometry Correction	None

Mesh

TABLE 15
Model (A4) > Mesh

Object Name	*Mesh*
State	Not Solved
Display	
Display Style	Body Color
Defaults	
Physics Preference	Mechanical
Solver Preference	Mechanical APDL
Relevance	0

Element Order	Program Controlled
Sizing	
Size Function	Adaptive
Relevance Center	Coarse
Element Size	Default
Mesh Defeaturing	Yes
Defeature Size	Default
Transition	Fast
Initial Size Seed	Assembly
Span Angle Center	Coarse
Bounding Box Diagonal	8494,60 mm
Minimum Edge Length	200,0 mm
Quality	
Check Mesh Quality	Yes, Errors
Error Limits	Standard Mechanical
Target Quality	Default (0.050000)
Smoothing	Medium
Mesh Metric	None
Inflation	
Use Automatic Inflation	None
Inflation Option	Smooth Transition
Transition Ratio	0,272
Maximum Layers	5
Growth Rate	1,2
Inflation Algorithm	Pre
View Advanced Options	No
Advanced	
Number of CPUs for Parallel Part Meshing	Program Controlled
Straight Sided Elements	No
Number of Retries	Default (4)
Rigid Body Behavior	Dimensionally Reduced
Mesh Morphing	Disabled
Triangle Surface Mesher	Program Controlled
Topology Checking	No
Pinch Tolerance	Please Define
Generate Pinch on Refresh	No
Statistics	
Nodes	15990
Elements	2750

TABLE 16
Model (A4) > Mesh > Mesh Controls

Object Name	*Body Sizing*	*Body Sizing 2*
State	Fully Defined	
Scope		
Scoping Method	Geometry Selection	
Geometry	9 Bodies	3 Bodies
Definition		
Suppressed	No	
Type	Element Size	
Element Size	500,0 mm	250,0 mm
Advanced		
Defeature Size	Default	
Behavior	Soft	

Named Selections

TABLE 17
Model (A4) > Named Selections > Named Selections

Object Name	*Middle back base*	*Block 1 Top Middle*
State	Suppressed	
Scope		

Scoping Method	Geometry Selection
Geometry	No Selection
Definition	
Send to Solver	Yes
Visible	Yes
Program Controlled Inflation	Exclude
Statistics	
Type	Manual
Total Selection	No Selection
Suppressed	0
Used by Mesh Worksheet	No

Transient (A5)

TABLE 18
Model (A4) > Analysis

Object Name	*Transient (A5)*
State	Not Solved
Definition	
Physics Type	Structural
Analysis Type	Transient
Solver Target	Mechanical APDL
Options	
Environment Temperature	22, °C
Generate Input Only	No

TABLE 19
Model (A4) > Transient (A5) > Initial Conditions

Object Name	*Initial Conditions*
State	Fully Defined

TABLE 20
Model (A4) > Transient (A5) > Initial Conditions > Initial Condition

Object Name	*Modal (None)*
State	Fully Defined
Definition	
Pre-Stress Environment	None

TABLE 21
Model (A4) > Transient (A5) > Analysis Settings

Object Name	*Analysis Settings*
State	Fully Defined
Step Controls	
Number Of Steps	3,
Current Step Number	2,
Step End Time	4, s
Auto Time Stepping	On
Define By	Time
Carry Over Time Step	Off
Initial Time Step	5,e-002 s
Minimum Time Step	5,e-004 s
Maximum Time Step	0,1 s
Time Integration	On
Solver Controls	

Solver Type	Program Controlled
Weak Springs	Off
Large Deflection	On
Restart Controls	
Generate Restart Points	Program Controlled
Retain Files After Full Solve	Yes
Combine Restart Files	Program Controlled
Nonlinear Controls	
Newton-Raphson Option	Program Controlled
Force Convergence	Program Controlled
Moment Convergence	Program Controlled
Displacement Convergence	Program Controlled
Rotation Convergence	Program Controlled
Line Search	Program Controlled
Stabilization	Off
Output Controls	
Stress	Yes
Strain	Yes
Nodal Forces	No
Contact Miscellaneous	No
General Miscellaneous	No
Store Results At	Equally Spaced Points
--- Value	40,
Damping Controls	
Stiffness Coefficient Define By	Direct Input
Stiffness Coefficient	0,
Mass Coefficient	0,
Numerical Damping	Program Controlled
Numerical Damping Value	0,1
Analysis Data Management	
Solver Files Directory	C:\Users\Marc\Documents\TU Delft_MASTER - SE_ANSYS FILES_MARC\Maik\Hexa_AllforceNewwave_Fric0,3_impactWave0.1sec_Totcon_Lightblocks_files\dp0 \SYS-1\MECH\
Future Analysis	None
Scratch Solver Files Directory	
Save MAPDL db	No
Delete Unneeded Files	Yes
Nonlinear Solution	Yes

Solver Units	Active System
Solver Unit System	nmm

TABLE 22
Model (A4) > Transient (A5) > Analysis Settings
Step-Specific "Step Controls"

Step	Step End Time	Carry Over Time Step
1	0,1 s	
2	4, s	Off
3	6, s	Off

TABLE 23
Model (A4) > Transient (A5) > Analysis Settings
Step-Specific "Output Controls"

Step	--- Value
1	20,
2	40,
3	20,

Real forces

TABLE 24
Model (A4) > Transient (A5) > Real forces > Loads

Object Name	*Wave impact*	*Uplift Block 1*	*Uplift Block 2*	*Uplift Block 3*	*Uplift Block 4*
State	Fully Defined				
Scope					
Scoping Method	Geometry Selection				
Geometry	6 Faces	2 Faces			
Definition					
Type	Pressure				
Define By	Components				
Coordinate System	Global Coordinate System				
X Component	Tabular Data				
Y Component	Tabular Data				
Z Component	Tabular Data				
Suppressed	No				

FIGURE 1
Model (A4) > Transient (A5) > Real forces > Wave impact

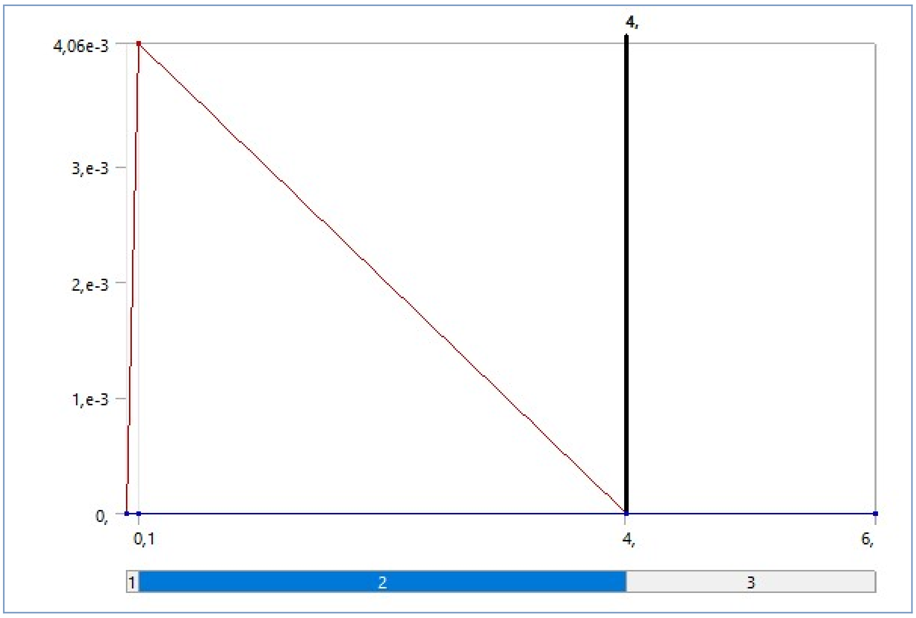


TABLE 25
Model (A4) > Transient (A5) > Real forces > Wave impact

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	0,	0,	0,
	0,1	4,06e-003		
2	4,	0,		
3	6,			
N/A	8,			
	10,			

FIGURE 2
Model (A4) > Transient (A5) > Real forces > Uplift Block 1

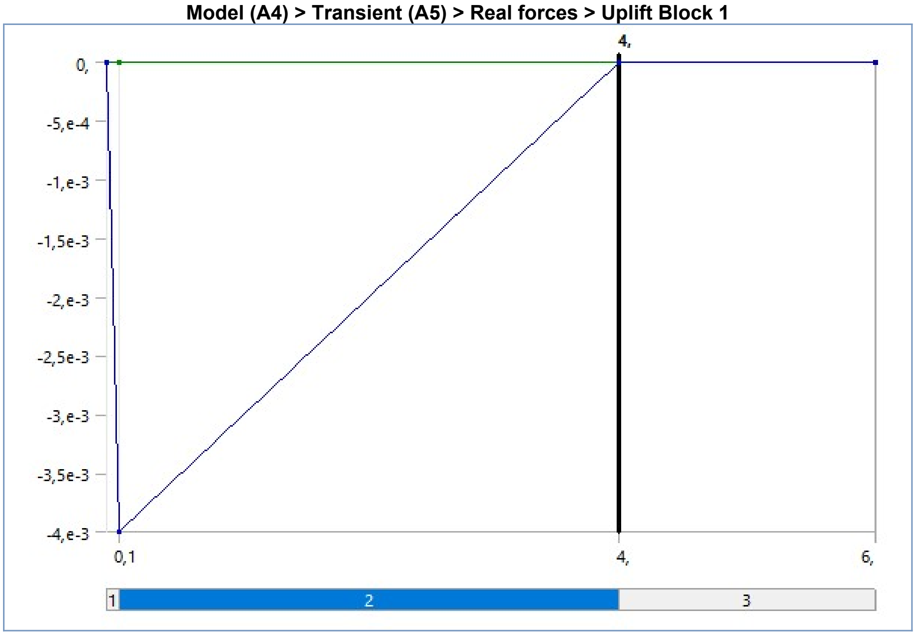


TABLE 26
Model (A4) > Transient (A5) > Real forces > Uplift Block 1

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	0,	0,	0,
	0,1			-4,e-003
2	4,			0,
3	6,			
N/A	8,			

FIGURE 3
Model (A4) > Transient (A5) > Real forces > Uplift Block 2

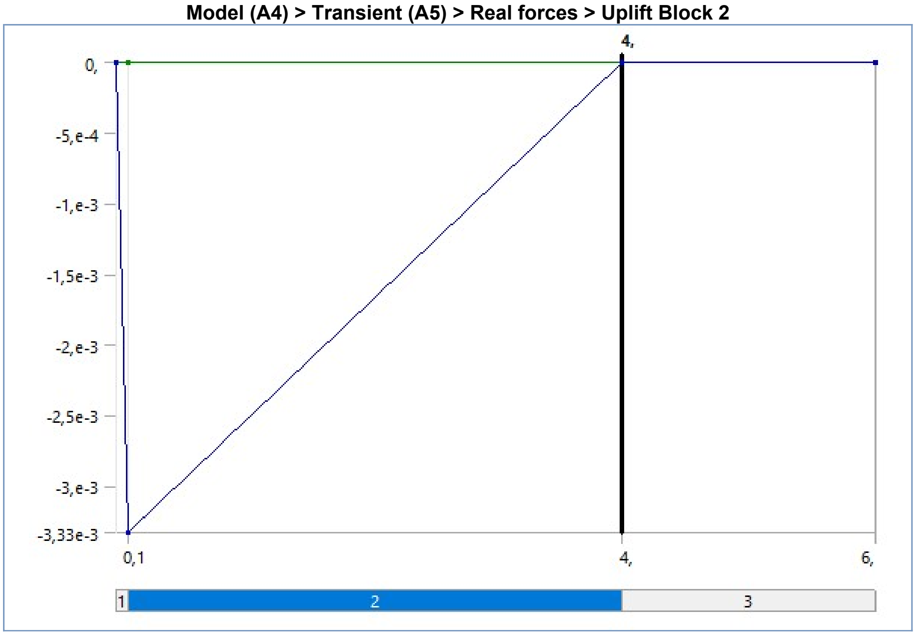


TABLE 27
Model (A4) > Transient (A5) > Real forces > Uplift Block 2

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	0,	0,	0,
	0,1			-3,33e-003
2	4,			0,
3	6,			
N/A	8,			

FIGURE 4
Model (A4) > Transient (A5) > Real forces > Uplift Block 3

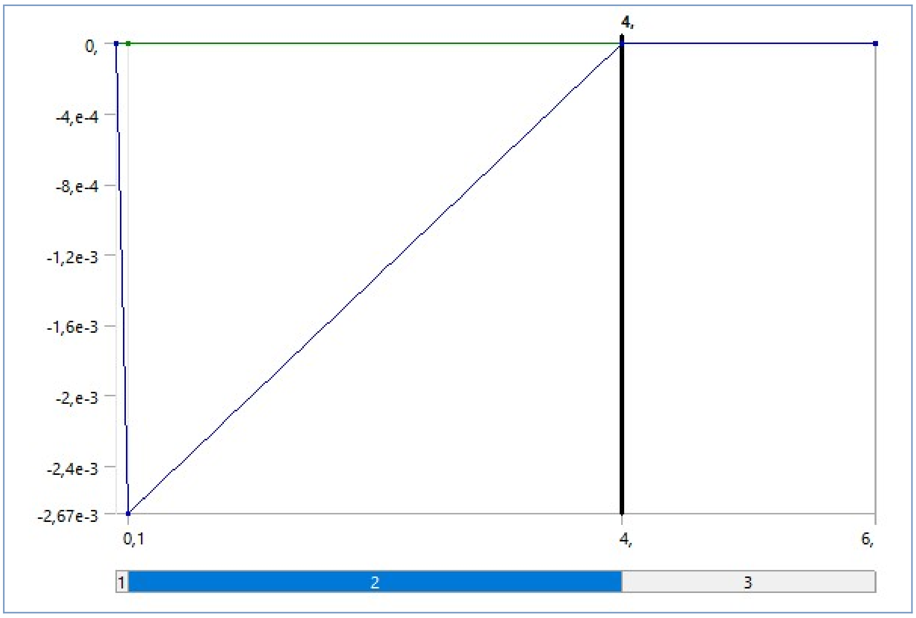


TABLE 28
Model (A4) > Transient (A5) > Real forces > Uplift Block 3

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	0,	0,	0,
	0,1			-2,67e-003
2	4,			0,
3	6,			
N/A	8,			

FIGURE 5
Model (A4) > Transient (A5) > Real forces > Uplift Block 4

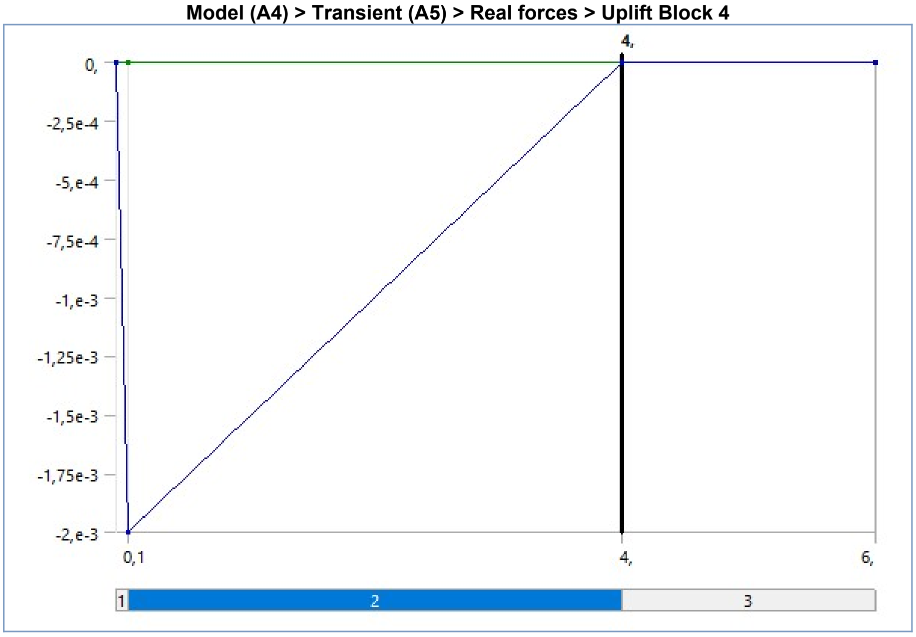


TABLE 29
Model (A4) > Transient (A5) > Real forces > Uplift Block 4

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]

1	0,	0,	0,	0,
	0,1			-2,e-003
2	4,			0,
3	6,			
N/A	8,			

Doubled forces

TABLE 30
Model (A4) > Transient (A5) > Doubled forces > Loads

Object Name	*Uplift Block 1 (x2)*	*Uplift Block 2 (x2)*	*Uplift Block 3 (x2)*	*Uplift Block 4 (x2)*	*Wave impact (x2)*
State	Suppressed				
Scope					
Scoping Method	Geometry Selection				
Geometry	2 Faces				6 Faces
Definition					
Type	Pressure				
Define By	Components				
Coordinate System	Global Coordinate System				
X Component	0, MPa (step applied)				Tabular Data
Y Component	0, MPa (step applied)				
Z Component	Tabular Data				0, MPa (step applied)
Suppressed	Yes				

FIGURE 6
Model (A4) > Transient (A5) > Doubled forces > Uplift Block 1 (x2)

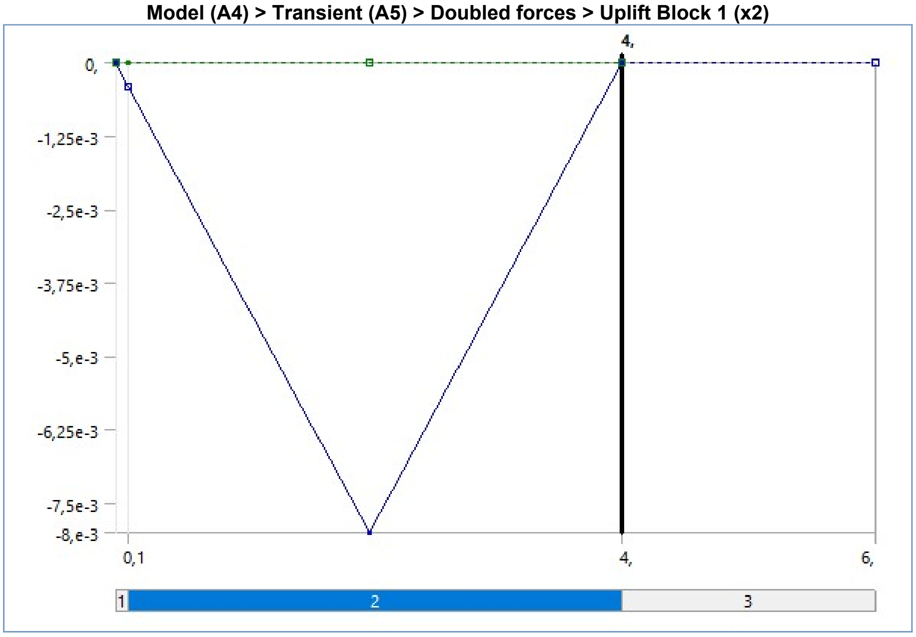


TABLE 31
Model (A4) > Transient (A5) > Doubled forces > Uplift Block 1 (x2)

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	= 0,	= 0,	0,
	0,1	0,	0,	= -4,e-004
2	2,	= 0,	= 0,	-8,e-003
	4,			0,
3	6,			= 0,

FIGURE 7

Model (A4) > Transient (A5) > Doubled forces > Uplift Block 2 (x2)

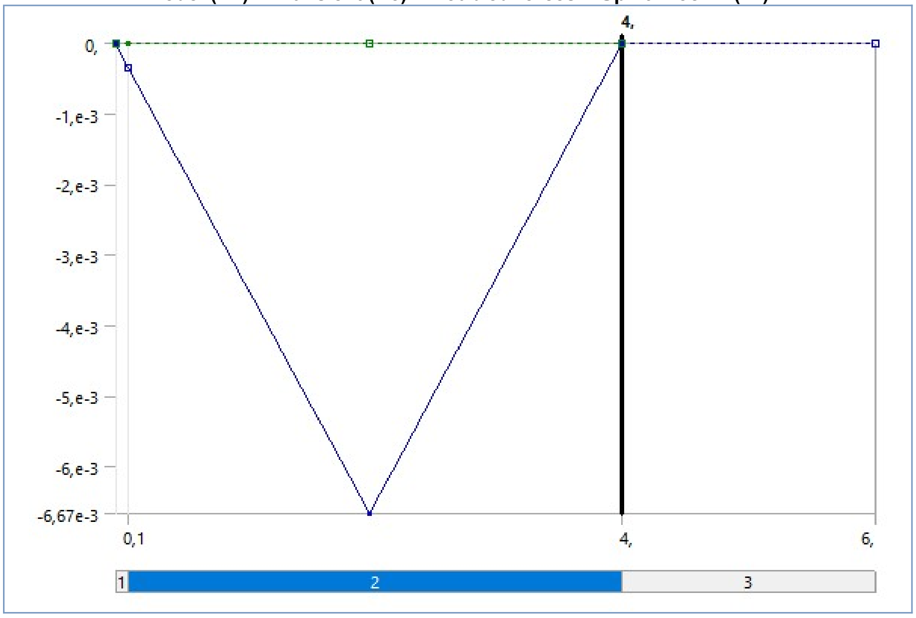


TABLE 32
Model (A4) > Transient (A5) > Doubled forces > Uplift Block 2 (x2)

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	= 0,	= 0,	0,
	0,1	0,	0,	= -3,335e-004
2	2,	= 0,	= 0,	-6,67e-003
	4,			0,
3	6,			= 0,

FIGURE 8
Model (A4) > Transient (A5) > Doubled forces > Uplift Block 3 (x2)

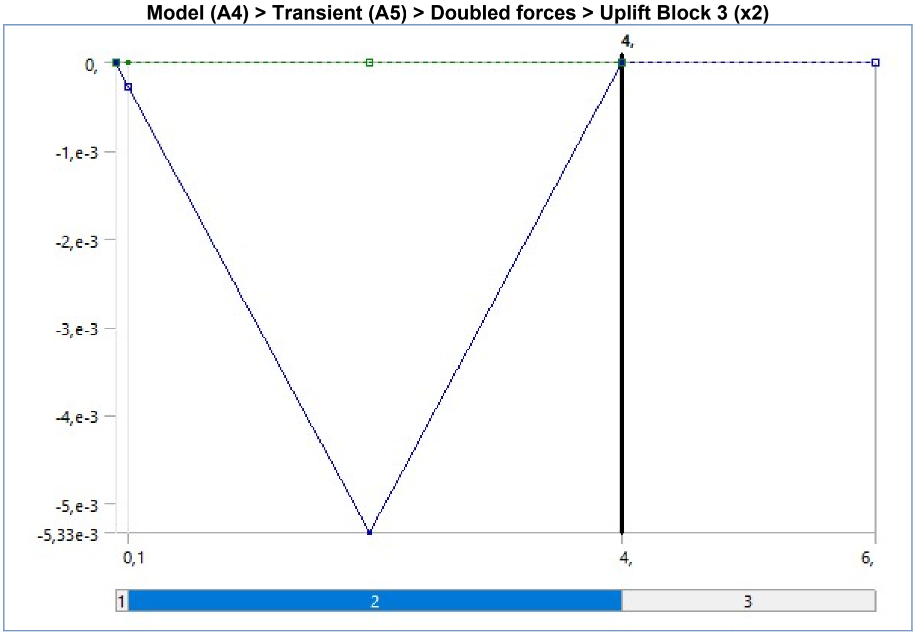


TABLE 33
Model (A4) > Transient (A5) > Doubled forces > Uplift Block 3 (x2)

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	= 0,	= 0,	0,
	0,1	0,	0,	= -2,665e-004
2	2,	= 0,	= 0,	-5,33e-003
	4,			0,
3	6,			= 0,

FIGURE 9
Model (A4) > Transient (A5) > Doubled forces > Uplift Block 4 (x2)

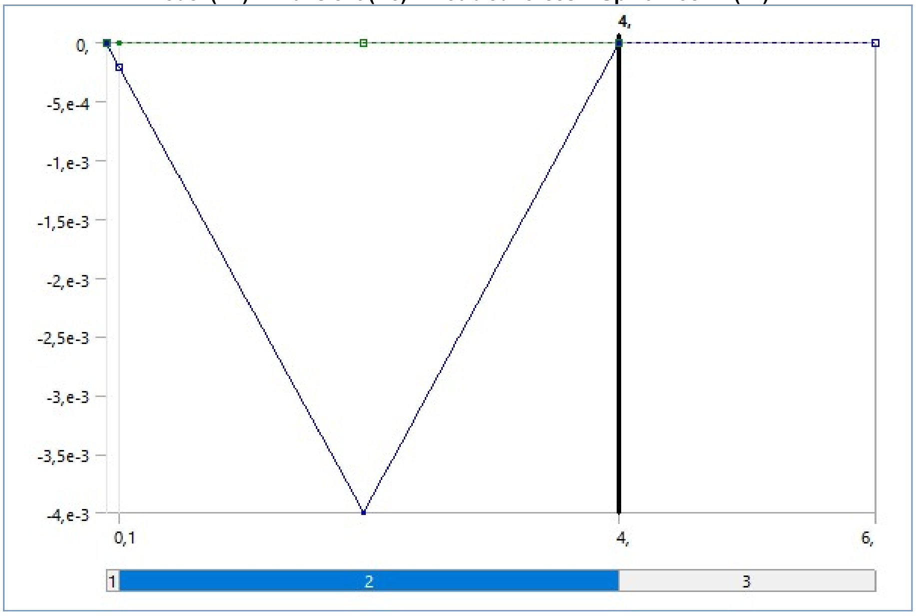


TABLE 34
Model (A4) > Transient (A5) > Doubled forces > Uplift Block 4 (x2)

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	= 0,	= 0,	0,
	0,1	0,	0,	= -2,e-004
2	2,	= 0,	= 0,	-4,e-003
	4,			0,
3	6,			= 0,

FIGURE 10
Model (A4) > Transient (A5) > Doubled forces > Wave impact (x2)

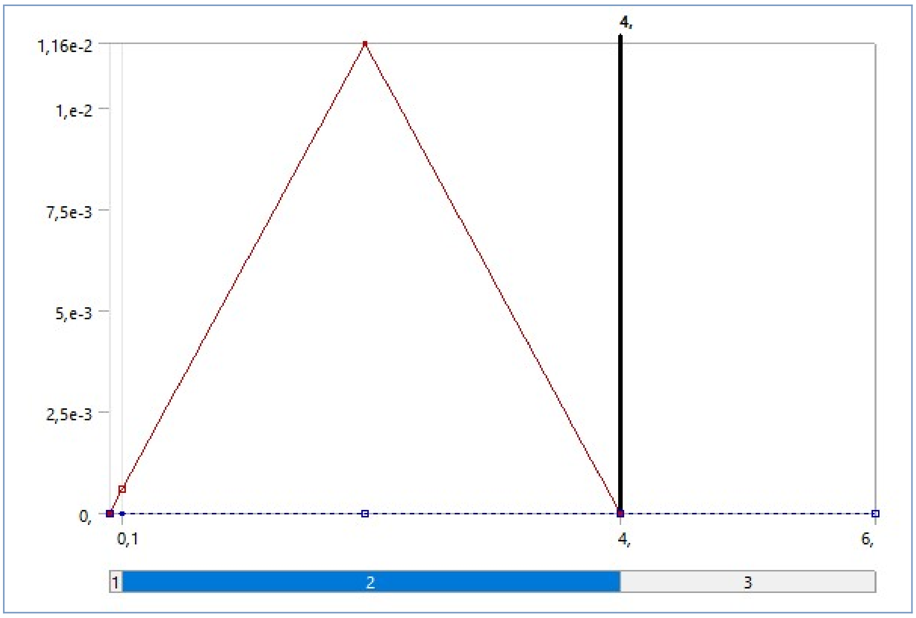


TABLE 35
Model (A4) > Transient (A5) > Doubled forces > Wave impact (x2)

Steps	Time [s]	X [MPa]	Y [MPa]	Z [MPa]
1	0,	0,	= 0,	= 0,
	0,1	= 5,8e-004	0,	0,
2	2,	1,16e-002	= 0,	= 0,
	4,	0,		
3	6,	= 0,		

TABLE 36
Model (A4) > Transient (A5) > Accelerations

Object Name	*Standard Earth Gravity*
State	Fully Defined
Scope	
Geometry	All Bodies
Definition	
Coordinate System	Global Coordinate System
X Component	0, mm/s² (step applied)
Y Component	0, mm/s² (step applied)
Z Component	9806,6 mm/s² (step applied)
Suppressed	No
Direction	+Z Direction

FIGURE 11
Model (A4) > Transient (A5) > Standard Earth Gravity

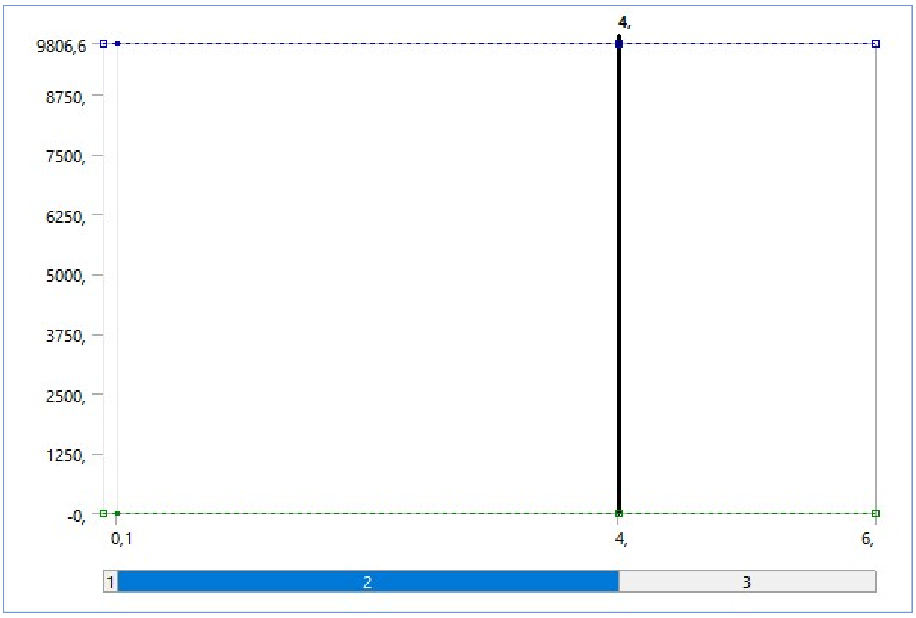


TABLE 37
Model (A4) > Transient (A5) > Loads

Object Name	*Fixed Support*	*Displacement*	*Hydrostatic Pressure*
State	Fully Defined	Suppressed	Fully Defined
Scope			
Scoping Method	Geometry Selection		
Geometry	5 Faces		103 Faces
Definition			
Type	Fixed Support	Displacement	Hydrostatic Pressure
Suppressed	No	Yes	No
Base Excitation		No	
Define By		Components	
Coordinate System		Global Coordinate System	
X Component		Free	
Y Component		0, mm (step applied)	
Z Component		Free	
Fluid Density			1,025e-009 t/mm³
Hydrostatic Acceleration			
Define By			Components
X Component			0, mm/s² (step applied)
Y Component			0, mm/s² (step applied)
Z Component			-9810, mm/s² (step applied)
Free Surface Location			
X Coordinate			0, mm
Y Coordinate			0, mm
Z Coordinate			-3607,8 mm
Location			Defined

FIGURE 12
Model (A4) > Transient (A5) > Displacement

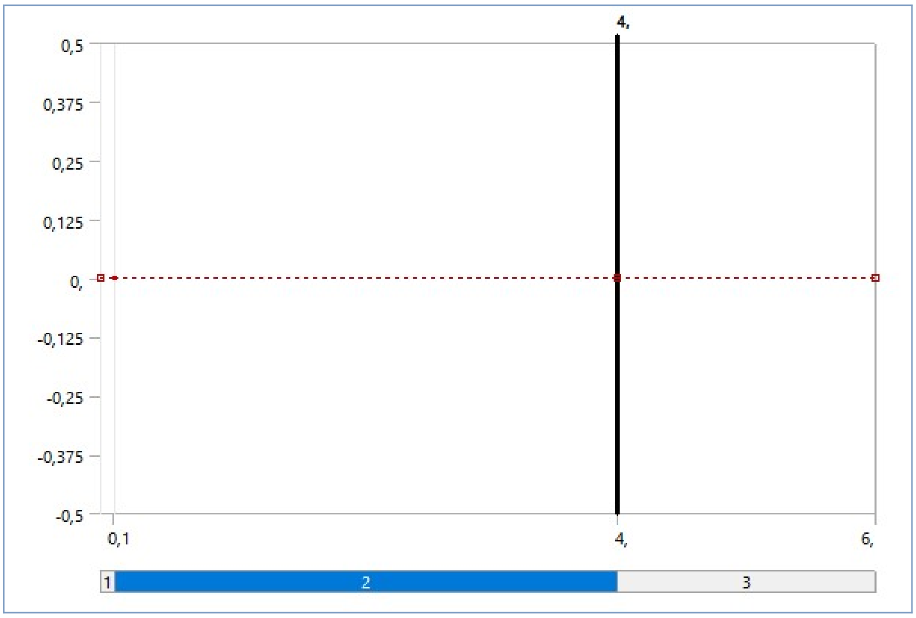
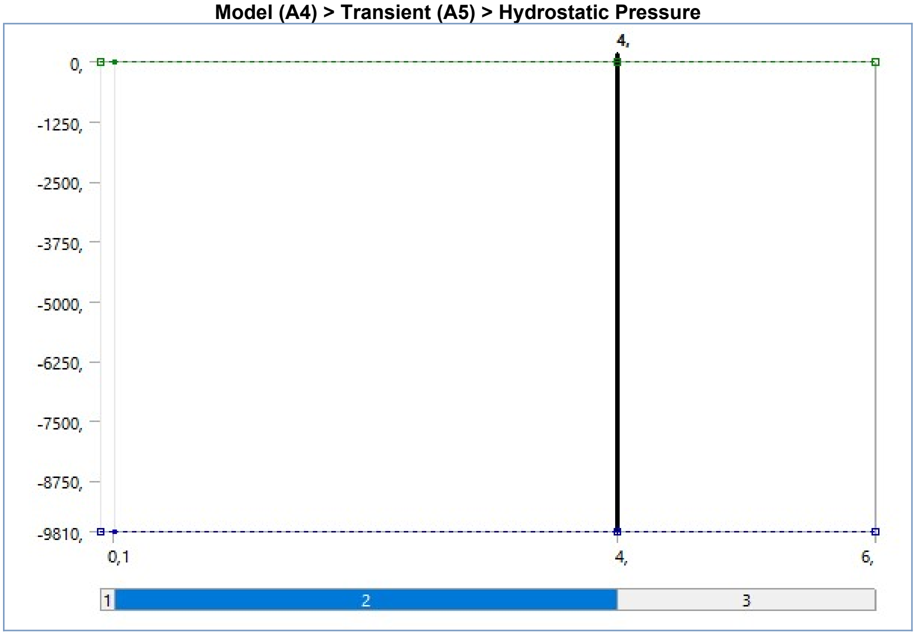


FIGURE 13
Model (A4) > Transient (A5) > Hydrostatic Pressure



Solution (A6)

TABLE 38
Model (A4) > Transient (A5) > Solution

Object Name	*Solution (A6)*
State	Underdefined
Adaptive Mesh Refinement	
Max Refinement Loops	1,
Refinement Depth	2,
Information	
Status	Solve Required

MAPDL Elapsed Time	
MAPDL Memory Used	
MAPDL Result File Size	
Post Processing	
Beam Section Results	No

TABLE 39
Model (A4) > Transient (A5) > Solution (A6) > Solution Information

Object Name	*Solution Information*
State	Not Solved
Solution Information	
Solution Output	Solver Output
Newton-Raphson Residuals	0
Identify Element Violations	0
Update Interval	2,5 s
Display Points	All
FE Connection Visibility	
Activate Visibility	Yes
Display	All FE Connectors
Draw Connections Attached To	All Nodes
Line Color	Connection Type
Visible on Results	No
Line Thickness	Single
Display Type	Lines

TABLE 40
Model (A4) > Transient (A5) > Solution (A6) > Results

Object Name	*Directional Bottom Right X*	*Directional Bottom Right Z*	*Directional Top Left Z*	*Directional Top Left X*	*Equivalent Stress Overall*
State	Underdefined				Not Solved
Scope					
Scoping Method	Named Selection				Geometry Selection
Named Selection	Middle back base		Block 1 Top Middle		
Geometry					All Bodies
Definition					
Type	Directional Deformation				Equivalent (von-Mises) Stress
Orientation	X Axis	Z Axis		X Axis	
By	Time				
Display Time	2, s	Last		5,e-002 s	0,3075 s
Coordinate System	Global Coordinate System				
Calculate Time History	Yes				
Identifier					
Suppressed	No				
Results					
Minimum					
Maximum					
Minimum Occurs On					
Maximum Occurs On					
Information					
Time					
Load Step	0				
Substep	0				
Iteration Number					
Integration Point Results					
Display Option					Averaged
Average Across Bodies					No

TABLE 41
Model (A4) > Transient (A5) > Solution (A6) > Results

Object Name	*Total Deformation*
State	Not Solved
Scope	
Scoping Method	Geometry Selection
Geometry	All Bodies
Definition	
Type	Total Deformation
By	Time
Display Time	1,4586 s
Calculate Time History	Yes
Identifier	
Suppressed	No
Results	
Minimum	
Maximum	
Minimum Occurs On	
Maximum Occurs On	
Information	
Time	
Load Step	0
Substep	0
Iteration Number	

TABLE 42
Model (A4) > Transient (A5) > Solution (A6) > Contact Tools

Object Name	*Contact Blocks*
State	Not Solved
Scope	
Scoping Method	Worksheet

Model (A4) > Transient (A5) > Solution (A6) > Contact Blocks

Name	Contact Side
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Light blocks\Solid	Both

TABLE 43
Model (A4) > Transient (A5) > Solution (A6) > Contact Blocks > Results

Object Name	*Status*	*Gap*
State	Not Solved	
Definition		

Type	Status	Gap
By	Time	
Display Time	Last	
Calculate Time History	Yes	
Identifier		
Suppressed	No	
Integration Point Results		
Display Option	Averaged	
Information		
Time		
Load Step	0	
Substep	0	
Iteration Number		
Results		
Minimum		
Maximum		
Minimum Occurs On		
Maximum Occurs On		

TABLE 44
Model (A4) > Transient (A5) > Solution (A6) > Contact Tools

Object Name	*Contact Tension*
State	Not Solved
Scope	
Scoping Method	Worksheet

Model (A4) > Transient (A5) > Solution (A6) > Contact Tension

Name	Contact Side
Frictional - SYS-1 Light blocks\Solid To SYS-1 Bonded\Soil	Both
Frictional - SYS-1 Light blocks\Solid To SYS-1 Bonded\Soil	Both

TABLE 45
Model (A4) > Transient (A5) > Solution (A6) > Contact Tension > Results

Object Name	*Status*	*Pressure*
State	Not Solved	
Definition		
Type	Status	Pressure
By	Time	
Display Time	Last	2, s
Calculate Time History	Yes	
Identifier		
Suppressed	No	
Integration Point Results		
Display Option	Averaged	
Information		
Time		
Load Step	0	
Substep	0	
Iteration Number		
Results		
Minimum		
Maximum		
Minimum Occurs On		
Maximum Occurs On		

Material Data

Concrete

TABLE 46
Concrete > Constants

Density	2,4e-009 tonne mm^-3
Isotropic Secant Coefficient of Thermal Expansion	1,4e-005 C^-1
Specific Heat Constant Pressure	7,8e+008 mJ tonne^-1 C^-1
Isotropic Thermal Conductivity	7,2e-004 W mm^-1 C^-1

TABLE 47
Concrete > Appearance

Red	Green	Blue
180,	173,	167,

TABLE 48
Concrete > Compressive Ultimate Strength

Compressive Ultimate Strength MPa
28,

TABLE 49
Concrete > Compressive Yield Strength

Compressive Yield Strength MPa
28,

TABLE 50
Concrete > Tensile Yield Strength

Tensile Yield Strength MPa
2,9

TABLE 51
Concrete > Tensile Ultimate Strength

Tensile Ultimate Strength MPa
2,9

TABLE 52
Concrete > Isotropic Secant Coefficient of Thermal Expansion

Zero-Thermal-Strain Reference Temperature C
22,

TABLE 53
Concrete > Isotropic Elasticity

Temperature C	Young's Modulus MPa	Poisson's Ratio	Bulk Modulus MPa	Shear Modulus MPa
	30000	0,15	14286	13043

Limestone (NL)

TABLE 54
Limestone (NL) > Constants

Density	2,3e-009 tonne mm^-3

TABLE 55
Limestone (NL) > Appearance

Red	Green	Blue
222,	222,	222,

TABLE 56
Limestone (NL) > Isotropic Elasticity

Temperature C	Young's Modulus MPa	Poisson's Ratio	Bulk Modulus MPa	Shear Modulus MPa
	37845	0,3077	32800	14470

TABLE 57
Limestone (NL) > Mohr-Coulomb

Temperature C	Initial Inner Friction Angle radian	Initial Cohesion MPa	Dilatancy Angle radian	Residual Inner Friction Angle radian	Residual Cohesion MPa
	1,0472e-002	88,	1,0472e-002	5,236e-003	40,

Appendix G

Concrete mixture design

G.1 Concrete mixture design

In the main report the mixture design is discussed in a brief manner. In this appendix extra information is given about the environmental classes with there processes. The result of these attacks are given and the effect if the climate during casting is explained. The special requirements are also discussed more apprehensive.

G.1.1 Environmental class

The actions on a concrete structure in direct contact with seawater can be larger when no proper measures are taken. The resistance has to be large enough so that the designed service lifetime will be realized. In the Eurocode 2 a concrete structure placed in seawater has the environmental classes **XC** and **XS**. **XC** is **C**arbonation Initiated Corrosion and **XS** stands for Chloride Induced Corrosion by **S**ea water. Assumed is that the structure is submerged and incidentally merged. This results in the classes **XC2** and **XS3** shown in Fig. G.1. The degradation mechanisms will be explained in Section G.1.1 and G.1.1 to get a better understanding of the process.

Class	Description of environment
XS	**Chloride induced corrosion - Sea water**
XS1	Salt containing air
XS2	Constant under water
XS3	Tidal zone, splash zone
XC	**Carbonation initiated corrosion**
XC1	Dry of constant wet
XC2	Wet, rarely dry
XC3	Intermediate moisture condition
XC4	Alternating wet and dry

Figure G.1: Environmental Classes [Eurocode].

The classes result in maximum criteria for the concrete mixture. The water/cement ratio can be maximum 0.45 and 0.55 when air entraining agents are used for reinforced concrete. A lower value of 5% is advised for the sake of safety. The minimum amount of cement/binder recommended for a seawater environment (class 4) is 280 kg/m3. When air entrainment agents are used, a minimum air content % has to be calculated depending on the aggregate diameter shown in Fig. G.2a. Also the minimum amount of fine material is determined by the aggregate sizes (see Fig. G.2b). Fine material is the summation of cement, air bubbles and filler/fine sand smaller than 250 μm.

	Environmental class								
	1	2	3		4		5a	5b	5c,d
			+aea		+aea				
Min. air content [%] for aggr. Diam.									
D = 63 mm	--	--	3.0	--	3.0	--	--	--	--
D = 31.5 mm	--	--	3.5	--	3.5	--	--	--	--
D = 16 mm	--	--	4.0	--	4.0	--	--	--	--
D = 8 mm	--	--	5.0	--	5.0	--	--	--	--
Type of cement					Sulphate resistant BFSC		Sulphate resistant cement (low C_3A content)		

(a) Minimum amount of air content.

Maximum particle D_{max} [mm]	Minimum amount of fine material (< 250 μm) per m^3 concrete [l]
8	140
16	125
31.5	115

(b) Minimum amount of fine material.

Figure G.2: Mixture criteria from maximum aggregate size.

The criteria from the environmental classes and the chosen aggregate size should by used in the procedure, when the concrete mixture is designed.

Carbonation induced corrosion

When concrete structures are placed in a humid environment carbonation of the concrete occurs. The process is explained in this section. During hydration of the cement paste the elements Ca(OH), Na(OH) and K(OH) are formed and solved in the pore water. The high concentration of hydroxide ions (OH^-) make the concrete alkaline. Ordinary concrete has a pH-value in the order of 12.5 á 13.5. This high pH-value keeps the reinforcement steel in a passive state and therefore no corrosion will occur. In a humid environment air and moisture can infiltrated the concrete pores. First carbon-dioxide reacts with water forming carbonic acid (see Eq. G.1). The acid reacts with the basic calcium-hydroxide which is in the pore water of the concrete (see Eq. G.2).

$$H_2O + CO_2 \rightarrow H_2CO_3 \tag{G.1}$$

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O \tag{G.2}$$

The carbonation process reduces the amount of hydroxide ions (alkaline) in the concrete. This results in a decrease of the pH-Value. When all the available hydroxide ions are reacted, the concrete is stable and has a pH-value of approximately 8.3. At this value, reinforcement steel will corrode. The reaction starts at the surface and over time the carbon acid will penetrate deeper into the concrete towards the steel. Important factors of the penetration speed are the concrete permeability and the relative humidity (RH). The reaction will be the fastest when the RH is 58%. When the structure is submerged the reaction (see Eq. G.1) will go slowly because the CO_2 has to penetrate the water-filled pores. The permeability depends mainly on the water/cement ratio(W/C). A higher W/C creates more voids during the hydration process. Therefore the Eurocode gives a maximum value for the W/C. By adding more fine material (<250 µm) the permeability can also be reduced. The benefit of the carbonation process is that it can also perform as a passive self-healing mechanism.

Chloride induced corrosion

[39] Next to the carbonation of the concrete the structure will also be under 'attack' by chlorides in humid environments. The process and hazards of chlorides penetration will be clarified. the most important cause of reinforcement corrosion is the presence of chlorides. The chlorides will breakdown the passive film that initially forms around the steel due to the alkaline pore water. The corrosion of the steel is an electrochemical process with active sites on the bar called the anodic reaction (see Eq. G.3) and a receiving site with the cathodic reaction (see Eq. G.4). To maintain electrical neutrality the ferrous ion combines at the cathodic side to from iron hydroxides or rust (see Eq. G.5).

$$2Fe \rightarrow 2Fe^{2+} + 4e^- \tag{G.3}$$

$$O_2 + 2H_2O + 4e^- \rightarrow 4OH^- \tag{G.4}$$

$$2Fe^{2+} + 4OH^- \rightarrow 2Fe(OH) \tag{G.5}$$

The rate of corrosion is primary due to the availability of oxygen, the electrical resistivity and relative humidity of the concrete, and the pH and temperature. Low permeability, high pH-value and a sufficient cover of the concrete are important to reduce the risk of corrosion. A combination of carbonation and chlorides intrusion is an extra risk, because the carbonations leads to a reduce pH value.

Corrosion of reinforcement

When steel in corroding because of the actions from carbonation and chloride intrusion, two problems are likely to occur [40]:

- Cracking and spalling of the concrete.
- loss of constructive strength of reinforcement.

If the reinforcement is rusting, the steel expands. This results in overpressure in the concrete. When the stresses are to high, cracking and spalling of the concrete will happen. This degrades the structure rapidly and the corroding process of the steel accelerates. During the corrosion process the area of the reinforcement decreases. The forces which the reinforcement can take up will be lower and so the overall constructive value of the complete structure. Therefore measures against these degradations are extremely important for the lifetime of the structures. Besides the mentioned measures such as a maximum W/C(0.45) ratio and minimum amount of fine material, a large concrete cover is very important to expand the lifetime of the structure. In seawater environment a cover of 6.35 cm is recommended [41]. The most important element is a low permeability to reduce the penetration speed.

G.1.2 Warm climate

During casting of concrete elements in warm climates, important factors have to be taken into account to guaranty the quality of the concrete. Hot weather concreting leads to evaporation, larger water requirement, rapid drop of workability and a higher maximum inner temperature. During casting the surface water can rapidly evaporate and give cracks in the green concrete before it has hardened (plastic shrinkage). Proper curing of the surface can overcome this problem. Because of the larger evaporation the mixture requires more water and the workability will drop more rapidly. Adding water to the mixture is not allowed, because it changes the important (long-term) properties of the concrete. Additives have to be used to solve the problem. Another important consideration is that during hardening the maximum temperature in the concrete will be higher in warm climates. This leads to higher temperature drops in the cooling phase. The cooling results in shrinkage of the concrete which leads to tensile stresses. Cracking can occur when the early developed tensile strength is lower than the shrinkage tensile stresses. This should be prevented during concreting of the elements. By performing an adiabatic tests and measuring the temperature during hardening a good estimation can be made if the concrete cracks. The higher temperature of the concrete also accelerates the early hydration. This gives the concrete an early high strength, but a lower long-term strength. Both problems can be countered by cooling of the fresh concrete before casting. The starting temperature and isolation properties of the formwork are important factors as well. Attention should be paid when (steel) formworks are used in warm and sunny climates. High temperatures of the concrete surfaces near the hot formworks can occur. Casting of elements can best be done in controlled area's to reduce the influence of the climate. This means precasting is recommended.

G.1.3 Special requirements

To make the structure enhance the fragile marine ecosystems the properties of the concrete are extremely important. To increase the settlement of corals the following measures should be taken into account.[15]

pH-value

The PH-value of the concrete is an important property, which can make or break the ecosystem enhancement of the structure. As mentioned the pH of hardening is around 13. Because of the degradation processes this will decrease at the surfaces. When a hardened structure is placed directly in the sea, the surface still has a high pH-value. The carbonation will take place and the calcium hydroxide will slowly leach out in the seawater. This changes the pH of seawater surrounding structure. To many types of marine life the high pH value is toxic and settlement of coral on the structure is retarded. Species which are resistant to pH changes like barnacles will settle on the structure. If after a couple of weeks the concrete surface is fully carbonated and the pH drops to the seawater value of 8.3-8.9, the coral can not settle any more on the surface, because the surface is taken by unwanted species. Advised is to store the blocks for around 40 days in a humid area to make sure the most of the toxic lye is gone. Simple checks can be performed to see in the concrete surface has a natural pH value (see Fig. G.3). Now the structure will be fully taken by not favoured species which set up defense to repel settlement of other marine life [42]. The potential of creating new artificial reefs with the structure is then wasted. To enhance te settlement curing of the concrete elements and adding silica fume can be options to implement.

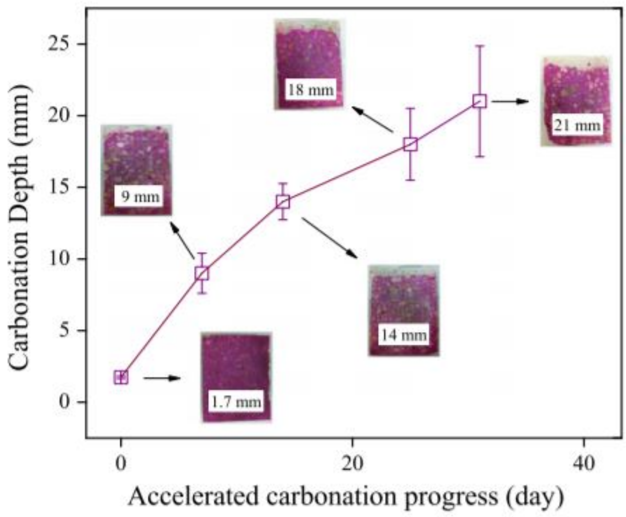


Figure G.3: Carbonation of concrete surface,

Silica Fume

Silica fume is a well known natural pozzolan in the concrete industry. It is a very fine material, 10-100 times smaller than cement particles. Because of the fineness it reacts rapidly with the $Ca(OH)_2$. This reaction produces additional calcium silicate hydrate (CSH) to the mixture. Therefore the long-term strength of the concrete increases. Because of the high blaine fineness, it demands more water. To keep the mixture workable with the same W/C ratio, admixtures are needed. By adding super plasticizers the mixtures can be used. The downside of the this is that the hardening is retarded. To overcome this an accelerator have to be added. The correct combination of admixtures are critical and have to be tested.

Using the silica fume as a cement replacements the carbon footprint of the mixture will reduce which is a great benefits to the environment. The silica fume is effective up to 10 % of the cement amount. This results in a pH drop of 0.4 [13], which is not enough to make the concrete equal to the seawater value. Also a drop of the pH value to 8.3 in the complete concrete structure gives corroding problems to the reinforcement. Then non-corroding fibre reinforcement is obligated.

Using silica fume for dropping the pH value is not very efficient but there are other great benefits for using silica fume. It will decrease the permeability of the structure significantly. The micro-structure of the cement/silica fume paste is very compact. Therefore the abrasion and chemical resistance increases. These benefits are critical for submerged constructions in seawater. Also the extra CSH increases the strength with 25 % when 10 % of silica fume is used [43].

Curing

The silica fume reduces only a small amount of the pH and in the complete structure which is not preferable. To reduce the pH at the surface curing of the element is necessary. By using the carbonation process at the surface the pH-value can be drop quite easy. The process goes fastest in an humid area with a RH of 58 %. After casting the elements should be stored in a humid environment for one month. Recommended is to check the pH-value of the surface during the curing. The surface will be fully carbonated (pH<9) over a depth of 1 cm. Taken into account that the carbonation process will decrease exponentially with time, no large leaching of toxic will occur [44]. The coral can then settle immediately after placements of the structure. Reef Ball foundation recommends to store the structure 3-9 months, depending on the amount of silica fume [45]. When no reinforcement is used, a longer curing period can be preferable to reduce the risk of intoxicating the seawater.

Surface preparation

The surface of the concrete is an important factor for settlement of the coral larvae. It should be rough and have small voids. This can be managed by using retarders on the molds and using air entrainments.

Retarder

A cheap and easy retarder is sugar. Sugar is a 'coating' admixture. This means that sugars is forming a layer around the cement grains and therefore water can not interchanges directly and react with the cement particles. By spraying the mold with sugar water the concrete surfaces will sett slower. Directly after demolding, the green concrete can be rinsed off. This gives a rough surface [46].

Air entrainment

Combining this method with the use of air entrainment, larger voids can be created on the surface. The downside of using air entrainments is that the strength is reducing when more than 3 % air is in the structure. For every % of air the strength decreases with 5 %. Contrary to the expectation, the use of air entrainment agencies reduces the permeability of the concrete [47]. The air voids are not interconnected and reduces bleeding, because the concrete mixture is more resistant to settlement of the aggregates particles. This makes the paste more cohesive.

Appendix H

Final design info

H.1 Steel frame costs

The costs for the steel frames are based on a reference project in Negril. The figures below shows the pricing. These are based on a calculated surface of 150 m^2 and a total steel volume of 0.98 m^3.

Deliverable	Submission
Inspection of site, knowledge transfer about the reef, meeting with staff	2 days
Analysis of structures, Analysis of environmental factors, exact calculations of the existing structures and determination of needs	2 days
Preparations for electrification by connecting the structures to a grid and allocating the anodes, Electrification of the reef	3 days
Testing, gathering data, implementing adjustments	2 days
Finishing up reef work, training staff, hand over project, etc.	2 days
Manuals, media and social media preparation	1 day
TOTAL	**12 days**

Figure H.1: Deliverables steel frame for Royalton Negril [coralive 2016].

Material	Cost
Variable Power Supply Units 3x	US$ 645
Special Electric Cable 200m	US$ 655
Anodes 6x	US$ 1440
Waterproof connectors and steel hose clamps	US$ 430
General material and parts	US$ 250
TOTAL	**US$ 3420**

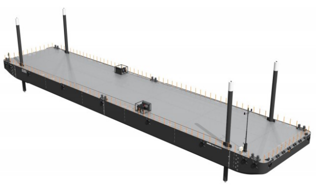
Project costs	Cost
Food and accommodation for 2 people for 14 days	US$ 2450*
2 flights from Europe to Jamaica and back	US$ 2000
Per diem for 2 people for 12 days	US$ 5000
TOTAL	**US$ 9450**

** If not staying at the Resort itself*

Figure H.2: Costs steel frame for Royalton Negril [coralive 2016].

H.2 Construction

Fig. H.3 gives an example of a pontoon from Damen that can be used for construction. Pontoons are available in all kinds of sizes. A pontoon best fitting the local conditions should be looked for.



Property	Value
Draught	2.2 m
Length	60 m
Width	13 m
Deck load	10 $t\,m^{-2}$

Figure H.3 & Table H.1: Pontoon properties [Damen 2018].