Tidal swimming pool design

A multidisciplinary project in Lüderitz, Namibia

Team tidal pool
Noa Elbers
Raja Kambazembi
Bernice van der Kooij
Amenenge Shatilwe
Jeremy Trotereau
Nadine van Westerop

Supervisors
Erastus Ashipala
Hugo Stam
Ferdi Knoester

Delft, September 30, 2022

Preface

Before you lies the design report 'Tidal swimming pool design, preliminary report'. The conclusions of the report are based on public consultations, findings of site surveys, and calculations performed in Lüderitz, Namibia. It has been written to fulfill the requirements of a multidisciplinary project at the University of technology, Delft. The project team was engaged in writing this report from July to September 2022.

The team consists of two BSc civil and environmental engineering honours students at NUST: Amenenge Shatilwe and Raja Kambazembi, two MSc Hydraulic Engineering students at TU Delft: Noa Elbers and Bernice van der Kooij, one MSc Geoscience and Remote Sensing student at TU Delft: Jeremy Trotereau, and one MSc Building Technology student at TU Delft: Nadine van Westerop.

The tidal swimming pool project is initiated by the Kelp Forest foundation and Kelp Blue. The goal and the deliverables of the project are formulated by us: the tidal swimming pool team, our supervisor: Erastus Ashipala, and the CEO of Kelp Blue: Daniel Hooft. During the design process, we have encountered several challenges, each with their own difficulties. However, by conducting extensive investigation we have been able to create the requested design.

We would like to express our thanks for the excellent guidance during this project from our supervisor: Erastus Ashipala (Kelp Blue), and Dr. R.E.M. Riva, Prof. dr. ir. S.G.J. Aarninkhof, and Ir. R. Crielaard (TU Delft). We also wish to thank the members of the Lüderitz community, without whose feedback and enthusiasm for the project and its realization the design process and the final product would not have been the same.

To our other colleagues at Kelp Blue: we would like to thank you for your wonderful cooperation as well. Whenever we needed help, either it concerned setting up site survey, a brainstorm session, or driving to the project location, we could always rely on your help.

Noa Elbers, Raja Kambazembi, Bernice van der Kooij, Amenenge Shatilwe, Jeremy Trotereau, Nadine van Westerop

Lüderitz, August 26, 2022

Figure 0.1: Team: tidal swimming pool design

Contents

1 | Introduction

"We all know the boys with the plastic bottles around their waist, going into the water, thinking that they are safe.", the opening words of Protasius Mutjida at the public consultation for the community project: tidal swimming pool in Luderitz, Namibia (15 July 2022).

Lüderitz, a harbor town at the Atlantic Coast in Namibia, will get a new impulse. The combination of high temperatures, a dynamic wave climate, and only very few people that know how to swim, has resulted in a strong need for a safe place to acquire swimming skills. The cooperation between the Kelp Forest Foundation, Kelp Blue, and Rotary International is enabling the implementation of a tidal swimming pool at Aeroplane Bay which will provide exactly that: a safe and self-sustainable swimming area for the community.

The tidal swimming pool will be a man-made version of the well-known existing rock pools; small pockets of water that naturally occur along rocky areas where the ocean meets the land. The tidal pool concept uses the tide as the main mechanism for water replenishment during every high-water (both neap and spring tide periods). Figure 1.1 visualizes the concept of a tidal swimming pool.

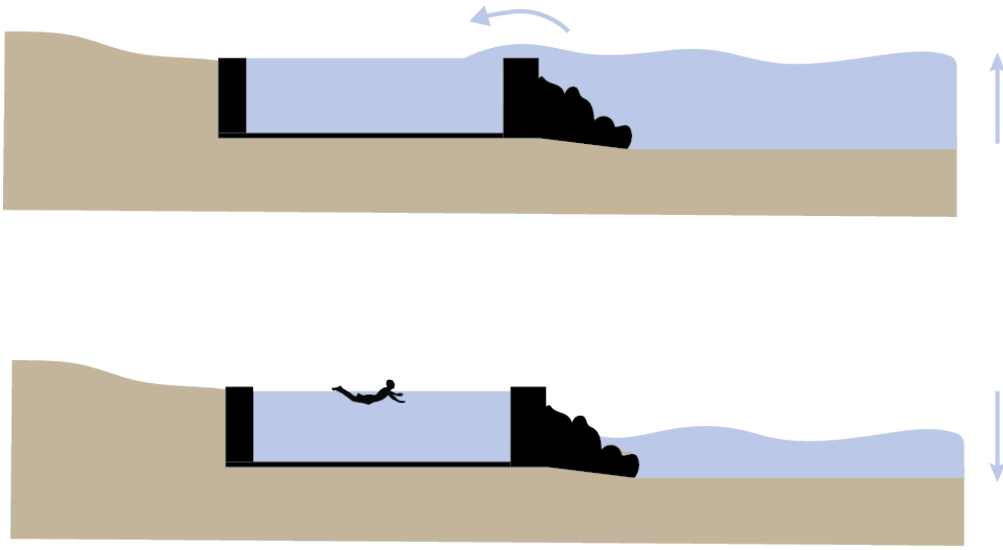


Figure 1.1: Tidal swimming pool concept making use of the tide. High water situation during which the pool is being (re)filled and refreshed (upper image) and low water situation during which people are allowed to make use of the swimming pool (lower image).

Though a great need for more tidal swimming pools along the South African coast exists, tidal swimming pool design criteria could not be found. Therefore, reports of already existing tidal swimming pools have been examined to identify factors that should be considered to ensure a well-designed swimming pool. According to Bosman and Scholtz, 1982, a successful design must provide safe swimming conditions and an effective operation of the pool. It should also minimize the impact on the environment and the required maintenance. Compliance between the design of the tidal swimming pool, its facilities, and these 'success-factors' is found by following a variety of hydraulic and structural engineering guidelines and manuals. The self-sustainability of the tidal swimming pool is optimized by including recreational facilities of which the profit will contribute to the maintenance costs.

This report takes you step by step through the design of both the tidal pool and its onshore facilities delivering a conceptual design for the community project 'Tidal swimming pool, preliminary report'. The report roughly follows the timeline of our project, starting with questions concerning the initiation of the project and narrowing down to a final conceptual design.

It should be noted that the multidisciplinary nature of our project means that not all parts are relevant to all of our fields, and to our supervisors. To keep the report legible, we have striven to write a 'core' report relevant to everyone. The appendixes to this report add further technical depth for each of our fields.

The report is structured in four parts, distributed as follows. In Part I, the project scope is described, a site selection is made and site investigations are described. Part II, elucidates the tidal pool design. Part III is dedicated to the design of the onshore facilities Finally, Part IV discusses and concludes our main findings

Additionally, appendixes provide more in-depth information about the measurements and calculations performed during the project. They are referred to in the chapters to which they pertain.

Part I
General

2 | Project Outline

2.1 | Project Scope

Project description
Kelp Blue, together with the Kelp Forest Foundation and Rotary International is proposing on building a Lüderitz community tidal pool, the purpose of which would be to provide a sizable, safe swimming area for the community.

To meet this proposal in all its facets, the 5 W's methodology is used. This analysis method, often used in project-management, is chosen as it covers several stages that question the fundamental characteristics of a situation: what, where, why, who, and when. The results of the analysis are found below. Note that the 'when' is left out of consideration as it primarily depends on the tender and the financing stakeholders. Also, the 'who' is divided into 'for whom' and 'with whom'.

What: A self-sustainable facility

- where organized swimming lessons and lifeguard training courses are provided.
- that allows community to swim safely under the supervision of a lifeguard.
- for professional swimming and the organisation of swimming competitions.
- to create more tourist attraction in Lüderitz and enhance recreational activities for the community.

Where

- Lüderitz, Namibia [1]

Why:

- There is no safe space to learn swimming
- It provides additional recreation for the local community
- It boosts local tourism

A particular problem in Namibia is that public schools finish at 1 PM. Resulting in a group of children that have nothing to do in the afternoon. Lüderitz offers these children very few opportunities to fill this time. As a consequence, many of them tend to get bored and start using drugs and/ or alcohol. The swimming pool could thus function as an opportunity to spend their time constructively.

For whom

- the local community (particularly children)
- local entrepreneurs

The emphasis was and is very much on the children. However, financial advantages for the local entrepreneurs in Lüderitz due to a possible boost of tourism has been a "nice to have" from the start.

With whom:[2]

- Kelp Blue and Kelp Forest Foundation
- Rotary International
- Design team of the tidal swimming pool

In conclusion: The goal of the project is to design a natural looking tidal swimming pool that will provide a safe place for the local community (especially the children) of Lüderitz to learn how to swim safely. The project will be initiated, managed and financed by Kelp Blue, the Kelp Forest Foundation and Rotary International.

[1] After evaluation of the multi-criteria analysis elaborated in chapter 3, Aeroplane Bay has been chosen as the final project location.

[2] This list only covers the primary stakeholders involved with the initiation of the project. For a complete overview of the stakeholders, see chapter 3.1 and appendix A.

2.2 | Requirements

At the start of the project, the following requirements have been posed by the client (Kelp Blue):

- Size of the swimming pool should enable the implementation of a swimming area for children (shallow water) as well as adults (deep water). Resulting in in swimming pool dimensions of approximately: 40 by 20 metres.
- The design lifetime of the swimming pool must comply with a user-phase of 25 years.
- The swimming pool must be accessible. Hence, a connection must be created between the (already existing) car road, the tidal swimming pool and its facilities.
- The total costs for the project should be within the available budget of 250.000 euros.

2.3 | Deliverables

To support and enable the educational and recreational uses of the pool, onshore facilities will be needed. The deliverables have therefore been divided into three categories: overall design, tidal swimming pool design and recreational facilities design (see figure 2.1).

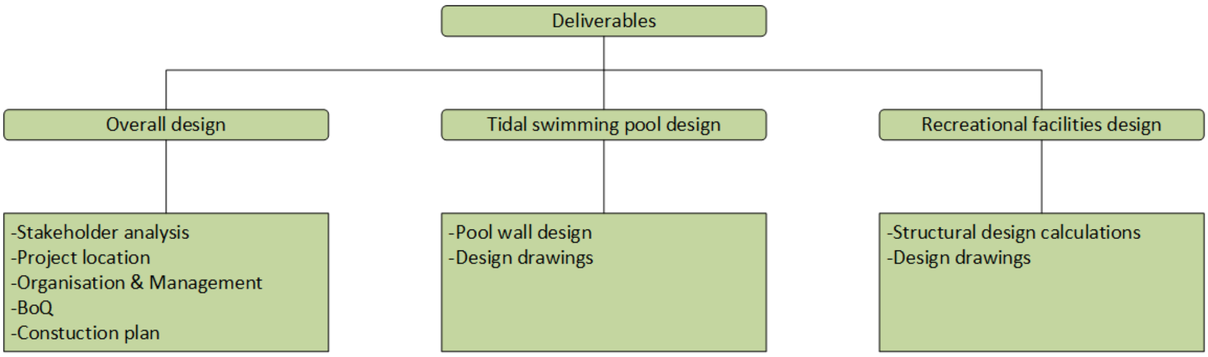


Figure 2.1: Project deliverables

3 | System analysis

This chapter analyses the components that are inextricably linked with the system behind the project. It also includes interpretations of the collected facts, and identification of potential problems. The components are categorized into and discussed in the following order: social context, location context, and legislation context. Note that this chapter only provides an overview of the most import elements that are considered for the system analysis. For more 'in-depth' information each subsection refers to the corresponding section and/or appendix.

3.1 | Social context

A successful project is one that meets the expectations of its stake holders. The past has often proven the importance of engaging the stake holders to the project, understanding their needs, and delivering on the promises made. This section will provide an overview of the projects stakeholders, their interests and how they will be impacted by the project. Appendix A provides an extensive analysis with respect to these stakeholders and outlines their possible interests with respect to and their impact and influence on the project. It also discusses how each of the stakeholders is expected to either delay or contribute to the project and how they should be engaged with in order to ensure that they have been given voice and transparency throughout the project.

Primary stakeholders
In this project the primary stakeholders are defined as the internal and external parties that have a direct influence on the project outcome and that are crucial to the success of the project.

- Kelp blue
- Kelp Forest Foundation
- Community of Lüderitz
- Lüderitz Rotery Club & Rotary International
- Local Authorities; i.e Town Council, municipality
- Ministry of Environment and Tourism, Ministry of Fisheries and Marine Resources
- Primary schools
- Secondary schools
- Contractor and construction companies

Secondary stakeholders
In this project the secondary stakeholders are defined as the internal and external parties that don't normally affect project implementation and operations unless they actively involve themselves and become vocal about their positions.

- Local business owners
- Local clubs
- Potential sponsors

Kelp Blue, Local authorities and the people of Lüderitz have most interest in the project. Kelp Blue, as main funder, is primarily involved during the design and construction phase of the project. The local authorities contribute by reviewing the project's EAP, assessing the EIA application and ultimately awarding a clearance certificate. The people of Luderitz are the ones making use of the facility and provide the project team with feedback during the design phase. The latter being of importance as we want the stakeholders to be able to express their concerns on the project without hindering the completion of the project.

Due to the differences between the roles of the stakeholders it is crucial that the project finds balance between adhering to Kelp Blues' objectives, complying with existing regulations and legislation of Lüderitz, and delivering a tidal swimming pool design that is both safe, sustainable, and simple to maintain for the community.

3.2 | Location context

One of the first steps of the tidal swimming pool design is identifying suitable project locations. The characteristics that are taken into consideration for the determination of the project location are given below. See chapter 6 for insights on the methodologies used to acquire the data needed to assess each of the mentioned characteristics.

- Accessibility
- Reputation of the location
- Hydrodynamics (tide, waves, currents)
- Bathymetry
- Elevation
- Geotechnics (soil composition, bearing capacity)
- Water quality
- Ecology
- Climate (wind)

Accessibility
The accessibility is defined by the walking distance from (elementary) schools to the location of the tidal swimming pool[3].

Reputation of the location
Reputation of the location embodies the associations of the local community with respect to the location. Awareness of negative reputations of considered project locations has been acquired by performance of public consultation. Examples of concerns expressed by the community are: believe of danger of sharks [4] and negative atmosphere.

Tide
The tide is key for designing the levels of the tidal swimming pool. At the Lüderitz Bay area, the tide is semidiurnal; the tidal period is 12 hours and 25 minutes. Furthermore, the tidal range varies and has a maximum value of 1.67 meters. Tidal predictions (available 30 days in advance) are found on the internet (Snow-Forecast.com Ltd., 2022) for the location Luderitz Bay (Lat Long: 26.65°S 15.15°E). In theory, only the minimum tidal range (during neap tide), maximum tidal range (during spring tide) and the average tidal range need to be known. For the month July (month of investigation), the spring and neap tidal range are 1.5 and 1.0 m, respectively.

Waves
Information about the wave conditions is, amongst others, needed to determine the structural stability of the design. In absence of any nearshore data, the offshore significant wave height provided by the report "Environmental data for the Kelp Blue Project" is used as input for the project. Appendix E presents the methodology (including the calculations) of the wave transformations (offshore wave data into nearshore wave data).

Currents
The current is determined by using a Tilt Current Meter (TCM-1), which uses the drag-tilt principle. The obtained data, stored in the built-in data logger, is to be configured by Domino software for Windows.Though information about the currents could be important with respect to both sediment transport and prevention of unwanted sedimentation within the tidal pool, it will not be discussed in this report. [5]

Bathymetry
The bathymetry will be retrieved from the website (Navionics, 2022). The bathymetry is needed to find the wave celerity which is needed for the nearshore wave transformations. More details can be found in chapter 6.

Elevation map
Photogrammetry is used as a quick and simple way of gathering the needed height data. Photogrammety infers the 3D positions of objects in the same way as our eyes do; overlapping images of the same object taken from

[3]The distance is determined by using Google Maps.
[4]According to the local authority, sharks have not been detected over the last decade.
[5]This decision was made after the TCM-1 got stolen during its measuring time during which it was already discovered that the seabed consists of bedrock; eliminating the need for site specific current information.

different positions shift closer objects more than farther objects. Using this phenomenon (called parallax), both our brains and a computer can situate points in space. The photogrammetry is performed by using a software called COLMAP. For the exact methodology, see chapter 6.

Soil composition
The under water soil composition is important for the feasibility of the tidal pool. The soil composition determines the bearing capacity of the soil layer that will have to carry the water- and soil retaining walls. It also influences the type of failure mechanisms that need to be taken into account, i.e. scour and overturning. The methodology for this survey can be found in chapter 6.

Water quality
As people will be swimming in the tidal pool, the water must comply with certain water quality standards. The water quality is measured daily in Lüderitz harbour by the Ministry of Fisheries and Maritime Affairs. Hence, for locations inside the Lüderitz bay, it is assumed that the water is well mixed and that the readings of the ministry (which include turbidity, oxygenation and pH) are representative for the tidal pool [6]. Also, it is assumed that the water at locations outside of Redford Harbour, can safely be assumed as being unpolluted. This is because the waters off Namibia are known to be relatively clean, as can be seen in figure 3.1. Furthermore, the entrance to the Lüderitz bay is large enough to dilute waste coming from Redford Harbour.

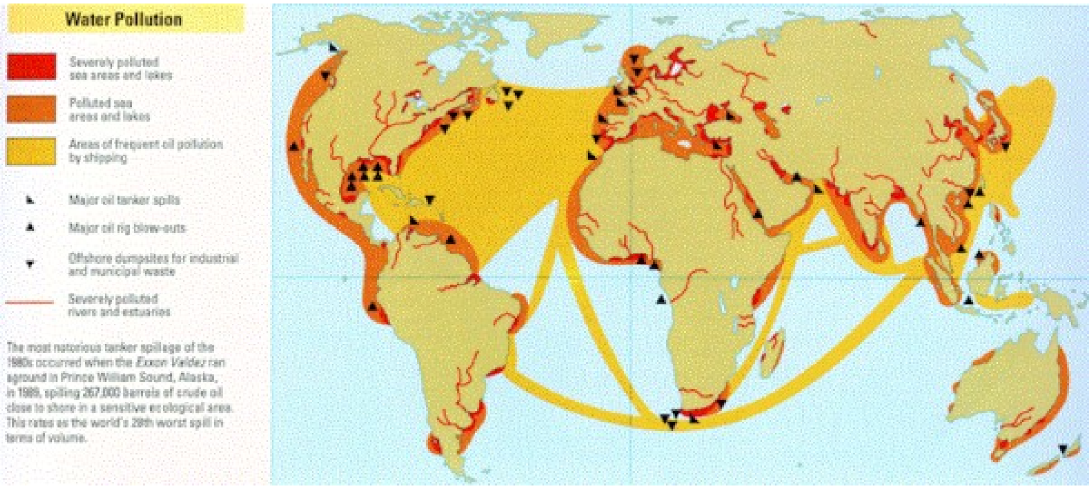


Figure 3.1: Global maritime pollution map

Ecology
For a fast analysis of the ecology, literature study based on existing Environmental Impact Assessments (EIAs) and Environmental Management Plans (EMPs) are performed. EIAs and EMPs used as reference for the project are:

- Lüderitz Mariculture environmental scoping report for the introduction of non-native scallop
- Port of Lüderitz EMP
- Novanam's EMP for expanded seawater treatment
- Kelp Blue Pilot plot EIA

Wind climate
The climatology is most important for considering the built environment. The WSP report (Smith, 2021) is used to determine the dominant wind direction (S.E).

[6] In addition to assessment of reports provided by the Ministry of Fisheries, a visual survey is performed to check for the presence of any plastic pollution. This is done at low tide, so that a differentiation can be made between plastic that was already deposited and plastic that is being washed up in the present tidal cycle.

3.3 | Legislation

Any type of project should comply with the law. The following legislation is considered for determining the project area:

- Environmental Management Act no 7 of 2007
- EMA Regulations GN 28-30 (GG 4878) (February 2012)
- Labour Act no 11 of 2007
- Water Act no 54 of 1956
- Soil Conservation Act no 76 of 1969
- Public Health Act no 36 of 1919
- Water Resources Management Act no 24 of 2004
- National Heritage Act 27 of 2004
- Forestry Act 12 of 2001

From assessment of the legislation mentioned above, it is concluded that the Shark Island, Shearwater Bay, Guano Bay and Grosse Bucht are off-limits due to legal protection.

4 | Vision for Management and Maintenance

One of the great pitfalls of infrastructure projects in developing nations, such as Namibia, is that the infrastructure gets built thanks to an important initial investment but that maintenance is often forgotten about (see page 10, Bhattacharya et al., 2012). Therefore it is important that the project goals will not be accomplished by merely building the physical infrastructure. Instead the project should also foresee in what this project termed "social infrastructure", i.e. the system of incentives and responsibilities that will keep the tidal swimming pool in working order: clean, safe, well-maintained, and operational.

The sections below provide suggestions for how to ensure that the pool is self-sustaining during its designed operational phase.

4.1 | Swimming instructors and life guards

Typically, lifeguarding and swimming instructions will be provided on a volunteer basis. In case of leaving volunteers or a need of expansion of the volunteer corps, the corps will be replenished by new recruits stemming from recently accomplished (swimming) students.

The first lifeguards and instructors will have to be chosen with care as they will set the cultural norms and values of the volunteer corps. Preferably the lifeguard is a community member; one that the children can look up to.

In order to attract this first group of instructors and lifeguards more easily, these people could be offered money for there services. A schematic representation of the instructions above is given in figure 4.1.

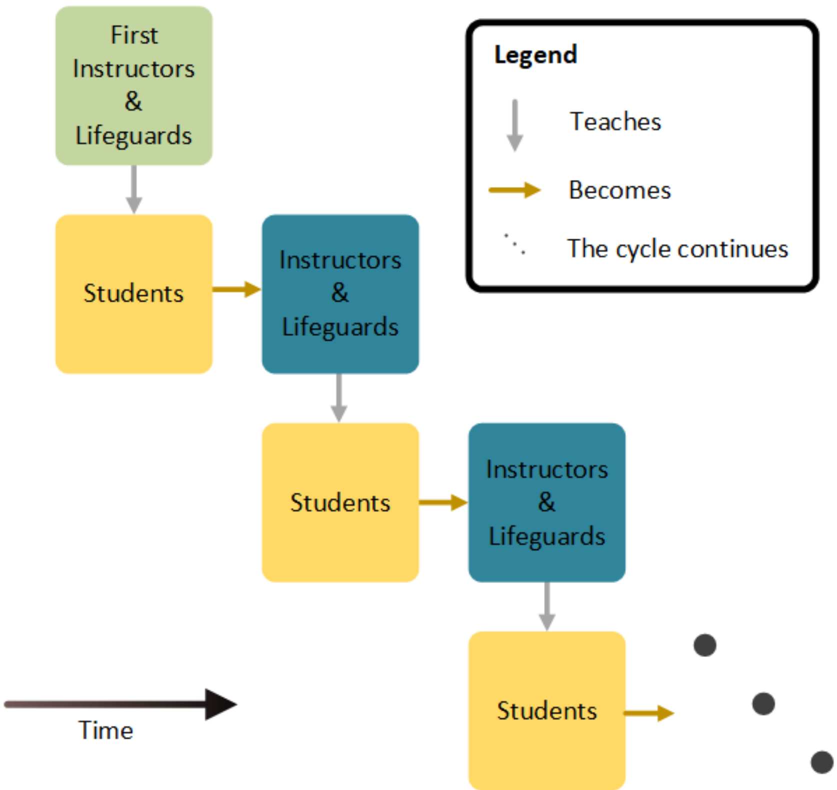


Figure 4.1: Recruitment and continuity of swimming instructors and lifeguards

In addition, the first lifeguards and swimming instructors will have to be trained. For the swimming instructors,NASFED, the Namibian Swimming Federation, might be interested to contribute to the project by offering their services. However, sponsors will have to be found to pay for the training of the lifeguards.

Furthermore, we envision that there will be a core team that is in charge of the tidal swimming pool and its facilities. This core team will be paid and consists of 4 functions:

- Management and maintenance: in charge of the state of the facility and finances
- Head of teaching: in charge of the training of swimming instructors and the swimming lesson schedule
- Head of life guards: in charge of the training of life guards and schedule for life guards

- PR and recruitment: in charge of relations with school, sponsors and other parties, and recruitment of volunteers

4.2 | External relations

As mentioned above, PR and recruitment will be in charge of external relations. They are free to pursue a policy of their own making, of course, but we have a suggestion for what these relations might look like.

The cafe inside the building will be rented out to an external party. The cafe would reserve the rights to sell food and drinks within the pool bounds. In return, the cafe provides payment in the form of money or services, such as cleaning the facilities.

As one of the main target groups are school-aged children, coordination between the schools and swimming pool is beneficial. For instance, for instance, the youngest children might be accompanied to the pool by school teachers.

In the medium term, fans of (competitive) swimming might want to form a swimming club. Swimming clubs are still rare in Namibia, but there are some examples in Windhoek and in Swakopmund. They are federated together by the Namibian Swimming Federation, NASFED.

This is more of an off-hand suggestion, but the organization of an open swimming competition would bring a great deal of enthusiasm and attention for the swimming pool and all its activities.

4.3 | Finances

Finally, the PR and recruitment officer will be in charge of sponsorships. Of course, for the initial construction, the Kelp Forest Foundation and Rotary International are the main sponsors for the project. But the officer will be in charge of long-term sponsorships. Again, here are a number of suggestions we have.

Especially in the beginning, the NASFED might be able to provide monetary or material support, or support in the form of instructors.

Swimming instructors can also be found through international NGOs. One example of such an NGO would be SwimTayka SwimTayka, 2022.

Yet another source of swimming instructors would be local diamond divers. They would do this on an individual basis. The advantage of using diamond divers is that they are an integral part of local culture and folklore. It would be a great start to have the first instructors be people that are looked up to.

For monetary sponsors, we suggest looking toward the local businesses. These would include supermarkets big and small, local industries (particularily those with links to the sea).

Finally, some amicable deal might be found between the pool and the municipality, particularly if the municipal council finds the pool a good of public interest.

5 | Location alternatives

Several locations can be considered for the tidal swimming pool. After eliminating industrial areas and protected coastlines, we consider two locations that are within walkable distances of Lüderitz, namely the point west of Aeroplane bay, and the coastal area south of the Nest Hotel, west of Radford Bay.

Figure 5.1: Map showing location and accessibility of Location Alternatives

Based on the brief meetings with project supervisors, a summary of the locations' characteristics was provided. The team carried out their own visual observations of the two locations. Figure 5.1 shows the areas available for possible location of the pool. Areas occupied for or near industrial activities are marked in the red zone. The orange zone on the map shows Shark Island which is a nature reserve, and therefore off limits.

From the remaining coastline, two rocky bays remain within reasonable distance to Lüderitz proper. As can be seen in the map shown above, both bays are close to residential neighbourhoods in Lüderitz. It should be noted that the town's population is economically segregated. In the absence of recent census data, the build environment gives a rough indication of household wealth by area. A map of the architecture is given in figure 5.2. The old town is the area in which the wealthy typically live. The formal communities is where the middle class live. The informal settlements (known locally as "the location") are typically host to the poorest inhabitants.

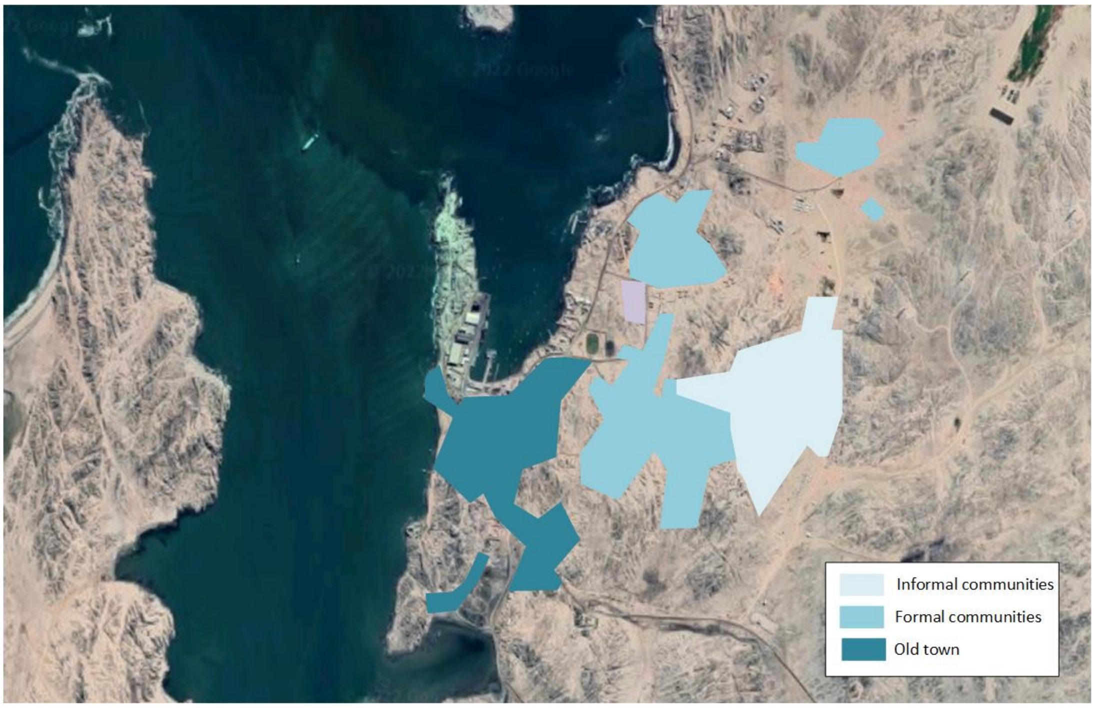


Figure 5.2: City plan of Lüderitz by building type

The two options are discussed below.

5.0.1 | Aeroplane Bay

Of the two potential locations, Aeroplane bay is closest to the poorer middle class neighbourhoods. The goal of the swimming pool is enable the teaching of swimming to all of Lüderitz's inhabitants. Given that we can reasonably expect the richer neighbourhood's inhabitants to have greater access to transportation, this is a significant argument for Aeroplane Bay, as it would be easily accessible for a larger portion of the population. Additionally, Aeroplane Bay is closer to the primary schools, which is where most of the population targeted for the pool attend. Aeroplane bay is also home to a popular swimming beach and has facilities for barbeque (referred locally to as "braai")

5.0.2 | Radford bay

Another area free/available to consider is west of Radford bay. This area is most accessible to the well off community of Lüderitz, and situated far from the schools.

On the south of the Nest Hotel, the waves from the of the bay would have a perpendicular impact on the tidal pool, requiring a stronger ocean barrier. This can be done either natural by locating the pool further land-inward making water refreshment more difficult, or artificially, which would require more materials than if the wave impact of the location was more minimal.

5.1 | Criteria for multi-criteria analysis

The choice of the tidal pool location will be based on a multi-criteria analysis (MCA). The criteria are defined and have been assigned a weight of importance. Each optional location is given a rating according to the following criteria.

- Wave impact: is preferably small, to avoid the needs for a lot of material. We want to avoid impulsive waves. Furthermore, wave impact should not pose a danger for the users of the tidal pool.
- We would preferably avoid net erosion or accretion
- We want an area that is accessible by foot, with a small distance from built-up area centre and primary schools
- The tidal pool location should have land nearby to construct recreational facilities
- The area should be accessible by road for construction vehicles
- If possible, locate the pool in a good-looking area

- Avoid obvious or dangerous water pollution
- Rocky or sandy soil is necessary for foundation
- Wind weather pattern
- Strength of the soil, in terms of its CBR value, plasticity etc.
- Availability of nearest connection point for electrical, water and sanitary services.

5.2 | Multi Criteria Analysis

Table 5.1: MCA for the location

	Weights	Aeroplane Bay	Radford Bay
Accessibility by foot	3	3	2
Accessibility by road	3	2	1
Proximity to amenities	2	2	1
Reputation	1	3	3
Tide	3	2	2
Waves	3	2	3
Currents	1	2	2
Bathymetry	2	3	3
Elevation	2	3	3
Geotechnics	2	2	2
Water quality	2	2	2
Ecology	1	2	3
Wind	1	3	2
Score	-	61	56

The performed MCA is tabulated in table 5.1. The scores are briefly elucidated below. Both bays are accessible by foot, but the target audience is closer to Aeroplane Bay, which therefore gets a higher score. Both bays have earthen paths to them, Radfords is both longer and steeper, making it poorly accessible by road. Aeroplane bay has a sewage pumping station nearby, whereas Radford does not. Both have a reputation as locations to swim (despite rumours of sharks) Both have medium wave conditions. Aeroplane bay has large swell waves that are diffracted before they hit the wall. Radford bay is more sheltered, but waves would hit a wall head-on. The shape of Radford bay also funnels the waves onto the wall. Neither wall suffer from strong currents Both have bathymetry suitable for building swimming pools. Both have an elevated coastline close to the pool location for amenities to be left dry. Both have a suitable substrate to work with; Radford is sand (in the near shore area) and Aeroplane is rock. As discussed above, water quality is unlikely to be an issue in both locations. Aeroplane bay is home to muscles that are a popular source of food for the locals, as well as a number of tidal pools, whereas Radford bay is mostly lifeless. Aeroplane bay coast is on the leeward side the land, the coast at Radford bay runs parallel to the coast.

6 | Site Investigation

Once Aeroplane Bay was chosen as the location, necessary information about the site had to be acquired. Where possible, existing data from the internet or local government were used. Where necessary, however, a site physical investigation was performed.

6.1 | Site description

Figure 6.12 shows Aeroplane Bay and the surrounding location. Moving along the coast, starting from the North, we see an industrial area (that is currently not in use). The beach then transitions from a rocky to a sandy beach, which is a popular local swimming destinations. The two crescent shapes near the beach are soil retaining walls that create a popular barbeque ("braai") location.

The coast then changes to rocky again and abruptly moves westward. The white circle indicates the rocky bay selected as the tidal pool location. The point to the west acts as a natural breakwater for incoming swell waves. There is a smaller rock formation to the east. One more useful detail: the two small buildings just above the road Southeast from the tidal pool location are sewage pumping stations, providing a simple connection for the pool facilities to the local sewage system. Continuing from the tidal pool, the coast goes southwest and remains rocky, with small rock pools along the way.

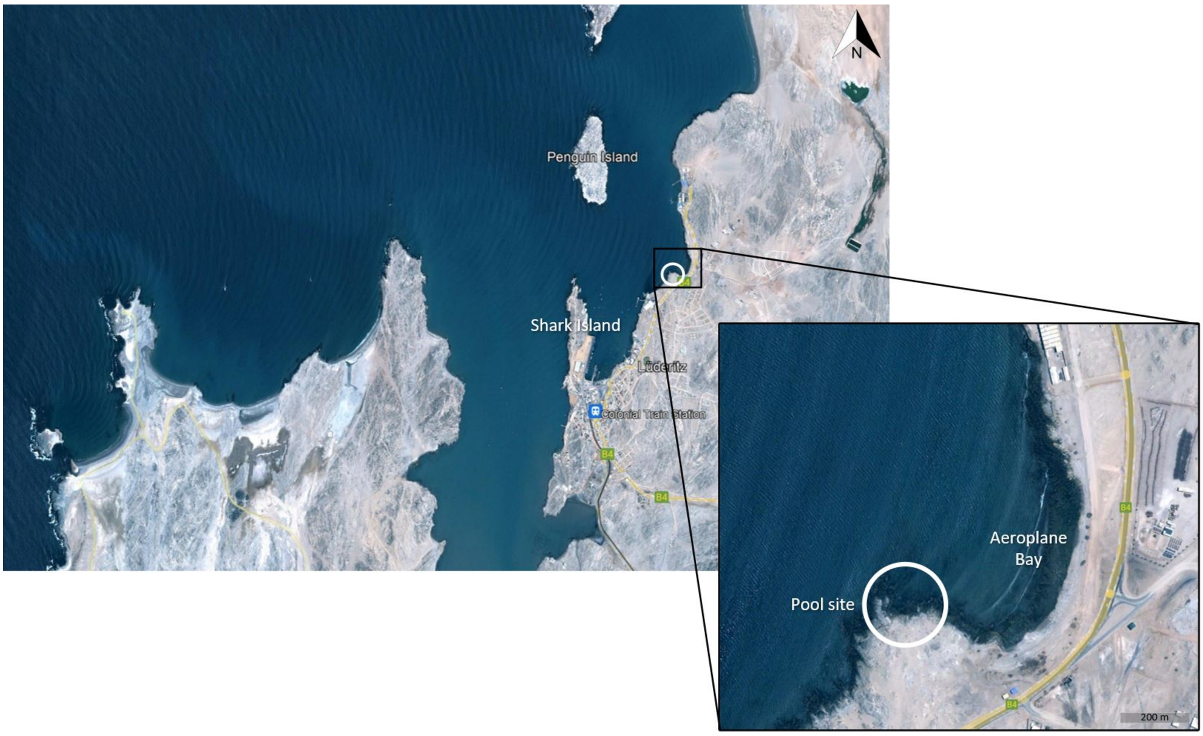


Figure 6.1: Satellite imagery of Lüderitz bay area Aeroplane Bay

6.2 | Physical site investigation

The physical site investigations that have been performed are:

- Photogrammetry
- Photo/video documentation
- Water level measurement
- Offshore geotechnical investigations
- Bathymetry measurements
- Area/length measurements

Each of these has been discussed briefly in following paragraphs.

6.2.1 | Photogrammetry

The goal of the photogrammetry was to provide a geolocated 1 m spatial resolution Digital Elevation Model (DEM) of the area. This resolution was chosen as it would be helpful as it was easily attainable and would enable the setting of the design in the landscape and generate.

The plan was to take pictures of target location, and geolocate certain points in the pictures with an external GPS. Then the figures would be postprocessed using COLMAP, a free and open-source Structure-from-Motion (SfM) software (Schoenberger, 2022).

Pictures of the site were taken twice: once using a phone camera and a GPS receiver for geolocalisation. Another set was taken with a Go-Pro that has an internal GPS system.

For the first photo session, markers were laid down and their positions measured with the GPS receiver such that the resulting point cloud could be geolocated. This was not possible, however, as the GPS receiver malfunctioned and kept drifting, as can be seen in appendix C.4.

Instead, to orient the mesh, points on the shoreline were chosen as the photogrammetry does not measure points on the sea (as can be seen in figure 6.2). This was made more accurate by using points on the hill opposite to the location, which is far away, thereby reducing the error in the angle.

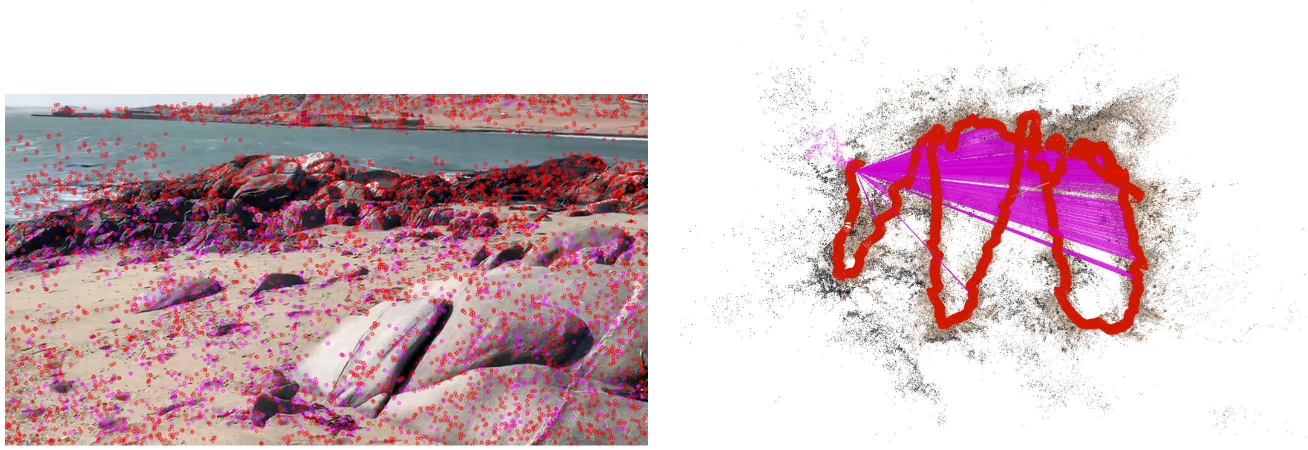


Figure 6.2: On the left: pictures as analysed by COLMAP. Red points are extracted features, pink points are features that were successfully matched, creating a point in the point cloud. On the right the resulting point cloud, with the camera positions in red.

The scale of the point cloud was given by measuring the distance between two recognizable points in google maps, and then scaling the distance between the two corresponding points.

For the second photo session, a kite with a Go-Pro was used to capture images every 2 seconds. The geotagged images were constructed into a mesh and then rotated using COLMAP's model_aligner function (only available in the command line interface). The output, which was in Earth-Centered Earth-Fixed (ECEF) coordinates, was then transformed back into WGS84 coordinates (standard for GPS). This was then used to place the points in the project.

The final height map resulting from the photogrammetry is shown in figure 6.3. Unfortunately, the photogrammetry could not be used in the Revit 3D modeling of the site elevation. More on this in chapter 13.

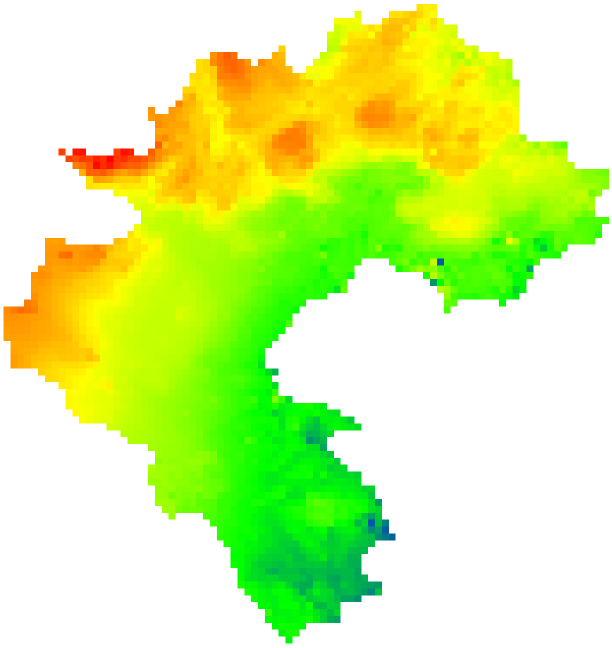


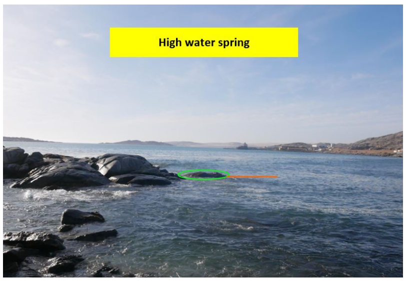
Figure 6.3: Final height map from photogrammetry

6.2.2 | Photo/video documentation

During the project, several visits to the Aeroplane Bay were made to document the characteristics of the location. This documentation was done visually via photos and videos. This paragraph descibes the findings from the inspection of the site, which can be subdivided in pool-related and building-related site descriptions.

Pool-related

The visual documentation for the pool was done over various moments, to capture the different tidal levels at the pool location and to observe various wave conditions. In figure 6.4, various tidal levels are illustrated. In combination with water level measurements (as described in paragraph 6.2.3), the tidal range was determined. Waves were observed and always seem to arrive at the site with a similar direction, this direction is illustrated in figure 6.5. Figure 6.6 shows some heavy swell conditions, waves overtop the natural breakwater on the left side.



(a) High water spring

(b) Low water spring

(c) Mean sea level

Figure 6.4: Different tidal levels captured (orange line illustrates high water spring level)

Figure 6.5: Swell wave direction

Figure 6.6: Heavy swell waves

Figure 6.7: Algae in a secluded area between rocks on the left side of the pool area

The documentation also captures the layout out of the various rock formations and the locations of fauna and flora. For example, between the rocks on the left, some algae are established as illustrated in figure 6.7.

Finally, a kite with a camera was deployed to capture the pool site from the top, resulting in figure 6.8. More pictures of the site are included in appendix C.

Figure 6.8: Picture taken with a camera on a kite

Building-related

For the on-shore facilities of the tidal pool, site investigations were performed through measuring the vertical distances with measuring tape, and documenting the elevations through photography. Photo's showing the layout of the on-shore site are shown in figure 6.9, and photo's showing the method of measuring the site dimensions are shown in figure 6.10. The photo documentation and measurements were used to minimize excavation and to let the on-shore facilities fit into the landscape. This is further documented in part IV of this report.

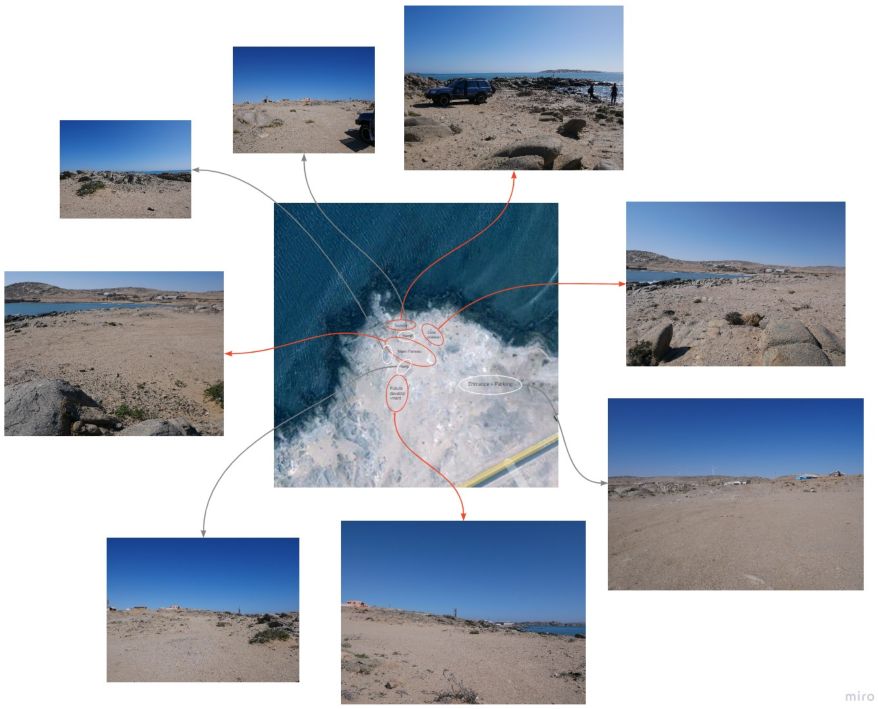


Figure 6.9: Photographic documentation, on-shore.

Figure 6.10: On-shore site measurements

6.2.3 | Water level measurement

The goal of the water level measurements was to estimate the tidal range at the tidal pool location. This is necessary as tidal ranges can vary strongly per location.

The tidal range was estimated from 2 measurements, taken on the 29th of July, 2022. This was a spring tide with calm weather and low swell. Measurements were taken at high and low tide, to find the spring tidal range. The findings were compared to tidal predictions from the Snow-Forecast.com Ltd., 2022. The values from these sources were scaled such that the spring tidal range would correspond to the measurement, which resulted in a scaling factor of 0.85. Next, this factor was applied to the average tidal levels according to The South African National Hydrographer (SA Navy), 2022. This resulted in the tidal levels in table 6.1.

Of course, this methodology implicitly makes the important assumption that the tidal level at the project site is well approximated by a linear scaling of the measuring point at Robert Harbour or offshore. This means no time lag, or any deviation from the tidal curve. These seem like reasonable assumptions when we take into account that the tidal gauge and pool location are separated by about 2 km of water.

The measurements were performed using a measuring stick, with respect to a fixed reference point on the rock. From the sea to the reference, points were heightened using a horizontal stick. A phone or the horizon over the sea were used to keep the horizontal stick horizontal.

Systematic flaws in the measurement include the measuring location, which is very rocky, which tends to retain retreating water. Particularly at low tide, this might lead to an overestimation of the sea level elevation, therefore decreasing the tidal amplitude.

Furthermore, the measurement method is quite inexact. We do not have an exact error budget for this measurement, but we estimate that the error in estimating the sea level (which is of course perturbed by waves, $\sigma \approx 5$ cm) is larger than the heighting error ($\sigma \approx 2$ cm). It must be said that again ebb water level was measured more poorly, as heighting had to be performed over a considerably longer distance than with flood.

Table 6.1: Tidal levels at Lüderitz offshore and local

Tidal level	Offshore (SA Navy) [m CD]	[m MSL]	Project site (calculated) [m MSL]
Highest Astronomical Tide (HAT)	1.99	1.05	0.89
Mean High Water Springs (MHWS)	1.65	0.71	0.60
Mean High Water Neaps (MHWN)	1.22	0.28	0.24
Mean Sea Level (MSL)	0.94	0	0
Mean Low Water Neaps (MLWN)	0.65	-0.29	-0.25
Mean Low Water Springs (MLWS)	0.23	-0.71	-0.60
Lowest Astronomical Tide (LAT)	0.0	-0.94	-0.80

Figure 6.11: Measuring water levels in the field

6.2.4 | Offshore geotechnical investigations

One of the important unknowns within the project was the local geology. The beach was known to have a granite bedrock, possibly covered by sand or shingle. The question was to know whether there was shingle, and at what depth the bedrock was present.

To do this, on the 4th of August, metal stakes were driven by hand into the ground to find the depth of the overlaying material. This was done by divers from Kelp Blue, both for safety reasons and because they have more experience in underwater geology (This is because one of Lüderitz's main industries is diving for diamonds off the coast, which these divers have a background in).

The staking methodology is simple, but also has its limitations. Firstly, the stakes were only about 50 cm long, and anyway, driving the stakes any further by hand whilst submerged is unrealistic. This means that in the event of a thick sand layer, we would still not know at what point bedrock starts.

Secondly, it is impossible to drive a metal stake by hand through shingle, so we would not know the thickness of any shingle layer.

Thirdly, if you do hit rock, there is no guarantee that it is the bedrock. That is just assumed. That is a risky assumption as it is possible (though not very likely) that we would hit a loose boulder.

Fortunately, the divers came back telling us that we can safely assume that there is bedrock everywhere at the surface, and that they did not find any shingle. This result is fortunate as it avoids many of the limitations mentioned above.

6.2.5 | Bathymetry measurements

Although a very precise bathymetry may not be necessary, an indication is useful when dimensioning a structure. In absence of sophisticated tools, a very basic method was applied to measure the depth profile. The focus was particularly on the depth between the ends of the two rock formations protruding seaward.

Two surveys were performed, one more general round for a rough indication, the other more specific, measuring the depth at the location of the structure.

This second measurement was performed during mean sea level on the 10th of August. The plan was to use a long rope marked every 2 meters held by the land crew (2 people) on one end of the wall. The sea crew (2 people), after entering the water in wetsuits, would then keep the rope tight and measure at the markings. The land crew would take notes and communicate with the sea crew about their lateral deviation.

The measurement was performed under difficult conditions with high swell, and was aborted when getting close to the rocks on the east side, where taking measurements was considered too dangerous.

The findings of this survey are as follows:

- Going from shallow to deeper water, there are many rocks just below the water surface (the visibility is very low). It was impossible to safely measure the depth from shallow to deep (transect B-B' in figure C.4 in appendix C).
- Because of the many rocks, the bottom profile is very non-uniform in general. Within a meter width, the bathymetry may vary 0.5 m.
- The water depth between the two rock formations (transect A-A' in figure C.4 in appendix C) fluctuates around 1.8 m.

All measurements are provided in table C.2 in appendix C.

Figure 6.12: Sea crew in the water. Note that the rope is not yet tightened

6.2.6 | Area/length measurements

To find the dimensions of the site and possible structures, multiple measurements were made. First approximations were made using Google Earth, these were later checked by using a tape measure on the beach. The slope of the beach has also been approximated with these field measurements and a slope of 11:1 was found (see figure 6.13).

Figure 6.13: Slope of the beach

Figure 6.14: Some measurements at the site

Part II

Pool design

7 | Tidal pool concept

In this chapter the design concept is briefly explained. The design concept is mainly based on reference projects, since neither of the students has experience with designing such a structure. Therefore, a brief summary of the literature and reference project study is given first.

7.1 | Literature and reference project study

Although numerous tidal swimming pool have been created in the past, surprisingly little documentation is available. A few important questions at the start of this project were, what does a tidal swimming pool look like and how is it designed?

A collection of images of tidal pool reference projects are included in appendix B. Several conclusions are drawn from said collection. Tidal swimming pools can be located on a sandy beach or in a rocky environment. Natural tidal pools (also called rock pools) are often created by a natural formation of rock. Artificial (man-made) tidal pools are typically created by introducing a concrete barrier between the sea and pool, either standing alone or connecting naturally present rock. These pools can be partly or fully enclosed by walls. The extent of a tidal swimming pools varies, both in size and design complexity.

Although little information about individual projects is found, some sources provide a summary of findings for existing tidal pools. For example the report "A survey of man-made tidal swimming pools along the South African coast" (Bosman & Scholtz, 1982), which provides an analysis of physical characteristics, operations, maintenance and problems of tidal swimming pools along the South African coast. And "Coastal Engineering advice for Ballina Ocean Pool" (Carley et al., 2018) which investigates existing ocean pools in New South Wales (NSW), Australia and provides design considerations for a new ocean pool.

7.2 | Requirements

Some pool specific requirements given by Kelp Blue are:

- The pool should provide a safe swimming space
- The pool should be low-maintenance
- The pool size should be in the order of 40 x 30 m
- The pool should have a natural look
- The pool should not fill up with sand (as has happened with a small local tidal pool, see reference project in appendix B)
- The lifetime of the pool is 25 years

7.3 | Design concept

Based on literature and reference project study, requirements and physical characteristics of the site, a rough idea for the design was created: The two natural rock formations will be used as pool walls, and will connected by a concrete wall, which closes the pool. To increase the length of the pool, a soil retaining wall can be placed at the land side of the pool.

Two alternatives wall constructions were considered, namely a rubble mound or caisson. The caisson structure was quickly rejected, since construction would not be possible in the area, and because of the very non-uniform shape of the pool area. An in-situ construction method has clear preference. A rubble mound has been briefly considered, it was rejected because the structure would not be watertight. In addition, rubble mounds are locally less conventional, therefore there is limited expertise in the construction of these types of structures. However, looking back, more attention could have been spend on this design alternative.

Later in report several design choices are considered (see chapter 10), such as the wall crest height, the floor type, any internal separation. In order to be able to dimension the pool, design values and boundary conditions are investigated as described in chapters 8 and 9. The resulting preliminary design is presented in chapter 11.

8 | Failure mechanisms and design values

As mentioned before, the tidal swimming pool will have a lifetime of 25 years. Based on the lifetime and risks related to failure, the accepted probabilities (and corresponding return periods) are found for the serviceability limit state (SLS) and ultimate limit state (ULS). The maximum probability of admissible damage are quantified by the Italian guidelines for breakwater design, which are presented in table 8.1. (PIANC, 2016) The guidelines for breakwaters are used here, since concrete wall separating the pool from the ocean can be considered as a type of breakwater, hence the use of breakwater guidelines.
The function of this "breakwater" is only to create a recreation area and not protect any structures or economic valuable areas, therefore the economic repercussion and risk to human life are low/limited in the case of damage or total destruction. This leads to the probability of incipient damage of 0.50 and 0.20 for total destruction. Using equation 8.1. These can be translated to return periods.

$$P_f = 1 - \exp{(T/R)} \tag{8.1}$$

In which P_f is the failure probability, T the design lifetime of the structure and R the return period of the design event. The return periods for the SLS and ULS are 36 and 112 (rounded to 100) year respectively.

Incipient damage	**Risk to human life**	
Economic repercussion	**Limited**	**High**
Low	0.50	0.30
Medium	0.30	0.20
High	0.25	0.15
Total destruction	**Risk to human life**	
Economic repercussion	**Limited**	**High**
Low	0.20	0.15
Medium	0.15	0.10
High	0.10	0.05

Table 8.1: Maximum probability of admissible damage in lifetime. Italian guidelines (1996) (PIANC, 2016)

The failure mechanisms that are considered for the stability of the pool wall are sliding and overturning. As discovered during site surveying, the bottom of the pool area and further offshore is made of bedrock. Therefore, no soil related failure mechanisms are considered.

Both sliding and overturning are considered as ultimate limit states (with return period 100 year for the design event).The resistance against sliding and overturning is quantified with a safety factor design value: $SF \geq 1.2$.

Some special attention should be paid to the overtopping "failure mechanism". The requirements state that the pool must be self-sustaining and the pool water should be of sufficient quality. The environment is rich in algae and nutrients, making it a priority to prevent extended periods of stagnant water in the pool. Therefore overtopping is, in contrast to the many hydraulic structures, a desired design element of a tidal pool. On the contrary, it would be considered a failure is no sufficient refreshment (and thus overtopping) takes place. This can be considered a servicibility limit state.
The required overtopping volumes are based on literature research, since no guidelines exist for this. Carley et al., 2018 found that a certain ocean pool in NSW, the South Curl Curl pool, has favourable conditions: sufficient water flushing and acceptable safety. During high water spring tide, the following overtopping discharges are observed at South Curl Curl:

- 50% exceedance wave condition: 10-50 l/m/s
- 10% exceedance wave condition: 40–120 l/m/s
- 1 year return period wave condition: 100-250 l/m/s

These discharges will be considered as design values for overtopping.

9 | Boundary conditions

Finding the right boundary conditions has been a serious challenge. Unlike in the Netherlands, there is little data available for this area and no sophisticated measuring tools were available. One important source for deriving the boundary conditions has been a wave analysis report by WSP created for several offshore locations for Kelp Blue's kelp farms. (Smith, 2021) The contents of this report are summarized in paragraph 9.1. Based on the information provided in the WSP report, a more local climate for the tidal pool location is determined, as is described in paragraph 9.2.

9.1 | Offshore environment

The offshore (potential) locations of kelp farms are presented in figure E.1. For these location waves, water levels and wind climate have been analysed. The pool site is marked in figure E.1 as well. It is clear that a significant difference in environment is to be expected, especially regarding the wave climate.

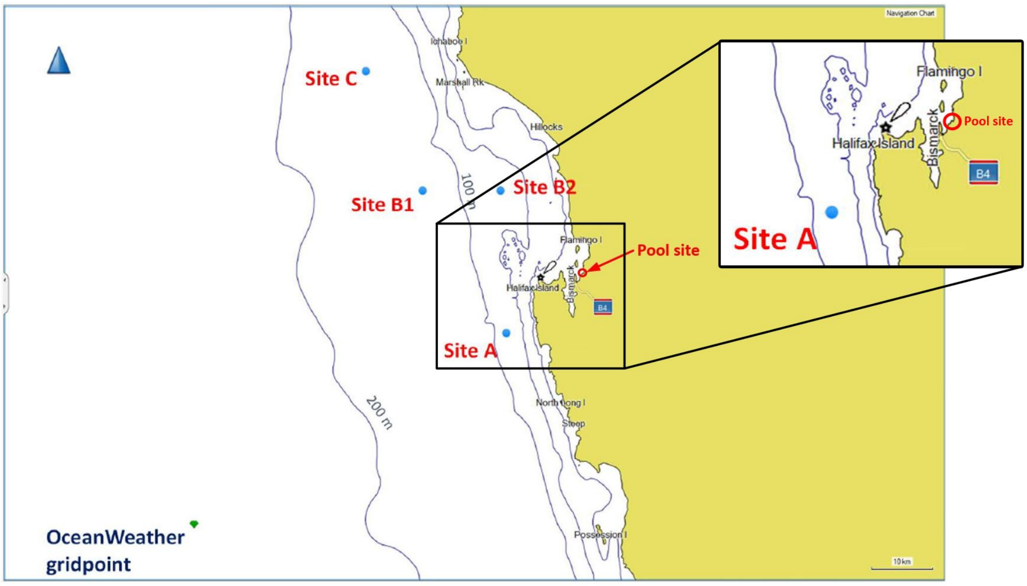


Figure 9.1: Overview offshore locations wave analysis report

9.1.1 | Waves

The site location is considered to be a west coast swell environment. The wave climate near Lüderitz originates from two primary sources:

- Cold front systems that originate from around 40-60 degrees south. These low-pressure systems generate significant winds that blow over a significant fetch often for multiple days. Resulting from these systems are long waves with peak periods around 12-16 s and a south-westerly direction. These systems are a source of destructive storm waves and are associated with erosion or damage on the African west coast.
- The South Atlantic High Pressure systems which circles the southern hemisphere at around 30 degrees south. This system generated smaller waves that typically have a peak period of 6-10 s and are a southerly direction.

The closest available offshore hindcast data set near Lüderitz is the OceanWeather Inc gridpoint, located approximately 70 km offshore. This data set covers a period from 1979 to 2019 and comprises of wind and wave parameters, including 2-dimensional spectral data. From this point the wave climate for the four locations in figure E.1 is derived using the numerical model SWAN applied within a Delft3D environment.

Since the wave are southerly/south-westerly directed (see wave roses in appendix D), a logical choice of location to derive the local climate from is Site A (water depth is 73 m). From Site A, the wave path refracts around the peninsula, reaching the more sheltered bay area.

The resulting wave heights and periods are retrieved from the report and are summarized in table 9.1. For he 50% and 10% exceedance wave, no peak period is given. This period is estimated based on the occurrence distribution of wave height versus peak periods, which leads to a peak period of 10 seconds.

Table 9.1: Total wave height for Site A (Smith, 2021)

Return period	Wave height [m]	Peak period [s]
50% exceedance	2.2	10
10% exceedance	3.2	10
1 year	4.7	12.6
100 year	6.0	16.0

9.1.2 | Water levels

Variation of water levels in Lüderitz are dominated by tidal influences, however wind and atmospheric pressure can also attribute. On the long term, sea level rise (SLR) will play a role.

Tides
The site location has a semidiurnal tidal environment. The tidal range is in the order of 1 meter, various tidal levels are provided in table 9.2.

Table 9.2: Tidal levels at Lüderitz (The South African National Hydrographer (SA Navy), 2022)

Tidal level	Value [m CD]	Value [m MSL]
Highest Astronomical Tide (HAT)	1.99	1.05
MHWS	1.65	0.71
MHWN	1.22	0.28
MSL	0.94	0
MLWN	0.65	-0.29
MLWS	0.23	-0.71
LAT	0.0	-0.94

Non-tidal variability
Several other factors influencing the water levels are the atmospheric pressure (inverse barometer effect), storm surge caused by winds and seasonal variability. These mechanisms can counteract each other, for example in winter there is a higher amplitude storm surge variability, but seasonal water levels are at the lowest. The various mechanisms add up to an extreme high water level of $\sim$ +10 cm to MSL and an extreme low of $\sim$ -20 cm below MSL.

Sea level rise
Localized sea level rise depend on a number of factors. The three major influences analyzed here are: Geological subsidence, the location on the globe, and humanity's future global carbon emissions.

We will assume geological subsidence is negligible. The other two sources of variability are captured by Iturbide et al., 2021. The former is captured by giving regional sea level rise. The latter is captured in the SSP's (Shared Socioeconomic Pathways). For our analysis, the SSP3-7.0 was used. This is a conservative scenario. This scenario still has internal variability. Of this variability, the median probability is taken. This gives us a value for the local sea level rise of 0.25 m by the year 2050.

9.1.3 | Wind

Prevailing winds in Lüderitz are directed from south east south, see figure D.2 in appendix D.

9.2 | Local environment

The local environment is determined based on the knowledge provided by the WSP report and low-tech observations at the site. This paragraph summarized the local wave and water level conditions and is supported by appendices C and E in which observations and methodology are elaborated.

9.2.1 | Waves

Unfortunately, the SWAN/Delft3D model used by WSP could not be extended to the site of interested (due to the lack of an accurate bathymetry among others). Therefore a different method is applied to determine local wave conditions. The transformation the offshore waves at Site A to the local wave characteristic was done as follows:

- The wave refraction around the peninsula is derived from Google Earth imagery and simple hand calculations were made to determine the wave height between Shark Island and Penguin Island (point B, see figure 9.2) after refraction and shoaling.
- A diffraction factor is determined from Goda, 2000.
- A SwanOne model is applied to determine the transformation in the last section (orange track in figure 9.2)

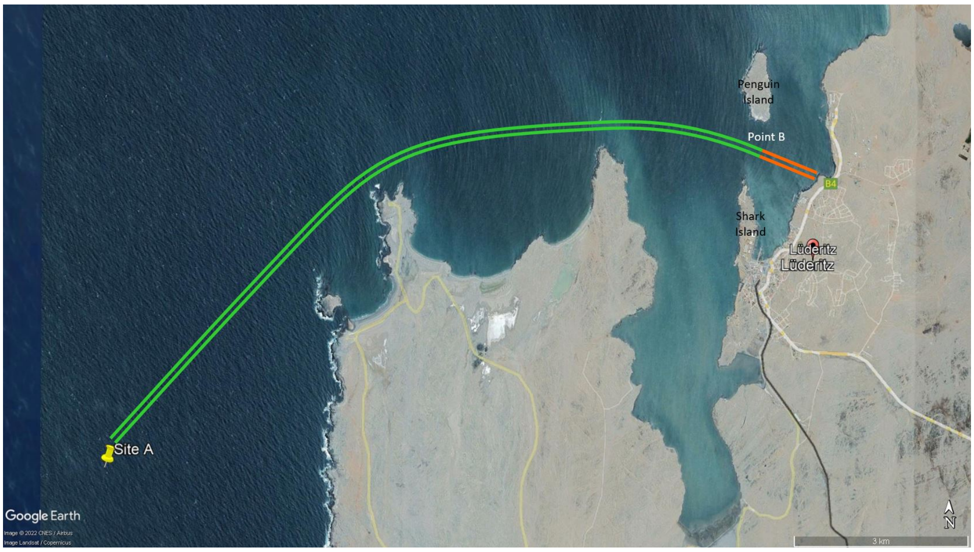


Figure 9.2: Wave path from Site A to pool site

In figure 9.2, the wave angle at Site A is considered to be south-westerly. This is the direction of swell waves (see figure D.1 in appendix D), which are the type of waves most important at the local site. The method as summarized above is elaborated in detail in appendix E. Table 9.3 shows the resulting wave characteristics at te pool site.

It is important to note that this method is not of the accuracy than one hopes to have in a design process. Several assumptions are made that may in fact be far from reality, for example the assumption of parallel depth contours and a uniform coastline in refraction calculations. For stability calculations a conservative approach can be used, however this isn't applicable for the failure mechanism of overtopping (refreshment versus safety). The unreliability of the wave transformations should be acknowledged and the authors suggest strongly to deploy a measuring tool (e.g. a wave buoy) to confirm these findings.

Table 9.3: Local wave characteristics

Description	H_{m0} [m]	T_p [s]	T_{m-1} [s]
MHWS + 50% exceedance wave	1.03	10.0	5.2
MHWS + 10% exceedance wave	1.19		5.2
MHWS + 1 year wave	1.17		6.1
MHWS + 100 year wave	1.17	15.5	7.3
MHWN + 50% exceedance wave	0.76	10.0	4.5
MHWN + 10% exceedance wave	0.85		4.3
MHWN + 1 year wave	0.88		5.7
MHWN + 100 year wave	0.84	8.0	6.3
SLR + MHWS + 100 year wave	1.44	15.5	

9.2.2 | Water levels

For the local water levels, only the tides are considered and any effect of non-tidal variability are neglected. Wind set-up/set-down is considered to be insignificant because of the prevailing wind (SWS) direction and the shape of the bay. Locally the wind is directed offshore, but since the bay mouth is very wide, set-down (as in a closed basin) is unlikely.

The local tidal range may differ from the offshore tidal range, because of the complex bathymetry of the bay area. Observations were done to determine the tidal range, this has been described in paragraph 6.2.3.

The resulting tidal levels are presented in table 9.4.

Table 9.4: Local tidal levels at project site

Tide level	Value [m MSL]
HAT	0.89
MHWS	0.60
MHWN	0.24
MSL	0
MLWN	-0.25
MLWS	-0.60
LAT	-0.80

9.2.3 | Currents

To quantify any currents, a measuring device was put in place on the outside of the west breakwater. Unfortunately, this device was lost and no data was collected. However, since the bottom material is bedrock and the structure is a concrete wall, it is assumed that currents are of negligible importance to the structure.

10 | Design considerations

In this chapter several components of the pool design are discussed, design alternatives are introduced and motivations are explained.

10.1 | Pool layout

Although the pool design is strongly bound to the dimensions of given surroundings, there are still some choices that can be made. Several ideas were considered for the layout of the pool, in which special attention was paid to making use of the natural "services" of the system. For example the natural breakwater, the waves that enter the pool area over the west breakwater or the (intertidal) flora and fauna that is currently present.

Some initial ideas try to conserve some of the local ecosystem. Species are likely to be strongly dependent on the natural tide, which means they may not thrive in a when separated from the tide (by being build in withing the pool). Two design alternatives are illustrated in figure 10.1. The tidal motion is preserved between the east rock formation and the smaller row of (bed)rock a few meters to the left. In this small area, flora and fauna is present, for example the shellfish attached to the rocks. A concrete wall would close the pool on the left of this channel, and the channel area could be used to install a (wooden) deck as walkway at the side of the pool. The area in front could be used as children's pool or to form an entrance to the pool.
This design tries conserve nature at the pool area and makes as much use of the natural rocky features. However, some disadvantages are that the size of the pool is quite limited. Also, the deck over the water may be a risky structure because of strong swell waves that propagate underneath and push the deck upward. Since the size of the pool is an important requirement, this layout idea is rejected.

Another idea includes the wooden deck, but closes the sea end completely between the east and west rock formations, which reduces the issue with upward pressure under the deck. But again, a lot of swimming space would be lost.

Figure 10.1: Pool layout idea: tidal inlet between rocks (picture taken during low tide

A layout that maximizes the swimming space consists of a pool wall which reaches from the east until the west rock formation and the removal of all of the rocks within the enclosed basin. Apart from the size being an advantage, safety is also easier to control in this design, since swimmers are not at risk to be hurt by sharp edges of rocks.

For the placement of the pool wall, the wave breaking abilities of the west rocks are considered. Since we are dealing with a vertical monolithic structure, there is a risk of sudden and complete failure. Impulsive waves should be avoided, to limit this risk. If the wall is placed landward of the break point, the impact with which the waves hit the wall is reduced since the a significant part of the wave energy is dissipated. This concept limits the length of the pool. However, the pool can be extended landward as well, by placing a small soil retaining wall on the land end. One aspect that may be compromised is the natural look of the pool: the east and west rocky features are used, but rock in the middle are removed. A sketch of this pool layout is given in figure 10.2

This final idea satisfies the most important requirement, which is a large swimming area (of ∼ 30 x 40 m). The pool may be less natural looking, but still uses some natural features.

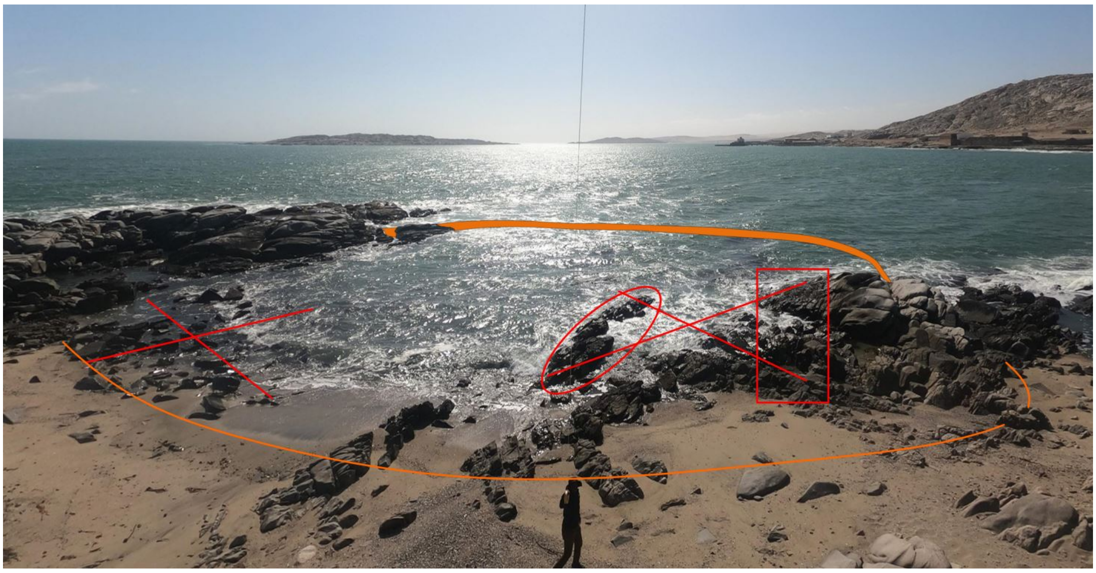


Figure 10.2: Pool layout idea: large swimming area (red: rock to be removed, orange: indication of concrete wall placement)

10.2 | Pool depth and floor

The pool floor can be made from various materials, for example sand, shingle or concrete. For environmental, durability and maintenance reasons, we advise against the use of concrete. Concrete will deteriorate over time, and should be kept clean and not be covered with other materials (that may arrive naturally over time). The best method is to use the natural bottom after removal and excavation of rock. We predict that the bottom material will be sand, and perhaps shingle. Both these materials are appropriate bottom materials, are low-cost, low-maintenance and natural looking.

The pool depth should accommodate both kids and adults. The bottom of the pool can either have a gradual slope or be separated into different depths with an abrupt change.

If two depth levels were to be chosen, this could be realised by creating a containing structure for bottom material and sand. This structure can be made from rock (as a rubble mound) or of concrete. A rubble mound would take up a significant width and introduce danger to swimmers, because of exposed (sharp) rocks, and is therefore not seriously considered.

A concrete wall (as illustrated in figure 10.3) will take up less space, but introduces the use of more concrete. Some practical issues concern the filling material, which may be lost from the shallow area into the deeper area and lead to more maintenance. And when sand is displaced, sharp edges of the wall may cause a danger for swimmers.

A gradual slope would be more favourable. It requires less effort to create and maintain, no unnecessary dangers are introduced and it looks most natural. The pool depth can start at 0.5 m at the soil retaining wall and gradually increase to 1.8 m at the pool wall. A depth of 1.8 m is deep enough to learn swimming without touching the floor and has limited risk of drowning.

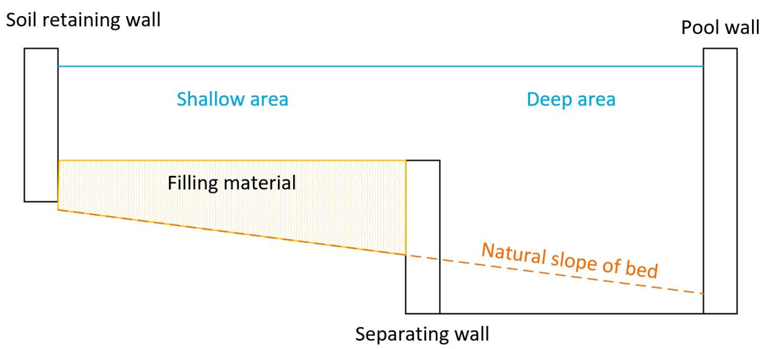


Figure 10.3: Pool stairs placed between rocky features

10.3 | Pool entrance

The pool entrance should allow easy access for both kids and adults. A ladder maybe difficult to use for certain individuals, which is why regular stairs are favourable. The stairs can be placed anywhere along the land wall, but integrating it with the rocks would improve the natural look. Figure 10.4 illustrates a potential location for the stairs.

Figure 10.4: Pool stairs placed between rocky features

10.4 | Side pool walkway

A walkway to be able to walk along the pool and reach the end is very useful for swimming instructors, lifeguards and swimmers. The best location for a walkway is on the right side of the pool, because of the swell wave coming in on the left side.

The first idea for a walkway is a wooden deck between the rocks (as described before) or over the rocks on the right side. This environmental impact of this design is small and it has a natural look. However, the pool water may not be safely accessible if the wooden decks follows the contour of the rocks, which may rise several decimeters above the water surface. On top of that, the wood could be slippery because of spray, which creates a hazard as well.

Another concept is creating a concrete wall attached to the right rock formation. This provides a sturdy and usable design, but requires a large volume of concrete.

A third concept is the use of a floating walkway. The floating deck can be locally constructed out of plastic barrels and a wooden frame. Some advantages are the this design is modular, low-tech, easy maintainable, repairable and replaceable. The deck material can be wood, which may be covered in a material that reduces slipperiness. This concept is not a civil structure, it can be developed locally and is therefore not further detailed.

10.5 | Drainage and maintenance

According to a survey of South African tidal pools, many pools are drained and cleaned fortnightly. These pools are equipped with drain pipes at the lowest position in the pool. However, they state that some well-sited pool with low crest-levels are self maintaining because of frequent and sufficient inflow. (Bosman & Scholtz, 1982)

Since the pool should be low-maintenance, the second situation is favourable. But, it cannot guaranteed that that is never any maintenance needed, so a drainage method should be implemented. Drainage pipe in the wall can be part of the solution, however the main challenge is the opening and closure of these pipes. The majority of the time, water needs to be retained which means water should not escape through the pipes. The method that facilitates opening and closing of drainage pipes, should be non-moving and low-maintenance, easy accessible for maintenance personnel, but not operable for pool users. This resulted in several ideas, all increasing the complexity of the design in various measures. After having discussed with the contractor, we came to the conclusion any complexity should be avoided. It is very likely that very limited maintenance will be executed and when something breaks it will probably stay broken for a long time.

Therefore, the drainage method will not be integrated in the wall design. Instead, a pump will be used to drain the pool. The pool is separate from the pool and can be operated whenever the personnel thinks needed.

10.6 | Wall cross-section

Two rival suggestions were made for wall construction method, and subsequently the cross section. The first is a caisson. It has a number of advantages. First of all, it is more cost-effective to construct. Secondly, it relatively easy to float into place. The downsides of a caisson is that the substrate has to be prepared, as a prebuilt caisson does not match the rocky substrate, and that the location is difficult to deliver and sink a caisson to.

To match the substrate, one could used the second alternative: an in-situ poured retaining wall. This method is simple. The major advantage of this method is that this is well within the technical abilities of local contractors. The main disadvantage is that the contractor will have to build an expensive temporary sheet pile wall to enable the pouring of the wall.

Nevertheless, we decide to go with the second option, as it does not require too much external expertise, and generally requires less technical finesse. To be on the safe side, we decide to make the wall a gravity wall. This implicitly means that the wall does not need to have a strong bond with the soil below, which, in light of our rudimentary geotechnical investigation, was seen as a necessary precaution.

10.6.1 | Inlets

For the tidal swimming pool design two cases, with respect to water refreshment, are considered. The first being a swimming pool design with in/outlets. The second one being the design of an infinity pool. As both design options come with pros and cons, a MCA (see table 10.1)is performed to determine which of the two design options is most suited for this project. Note that the weight-factor is based on the preferences of the project initiators Kelp Blue and the Kelp Forest Foundation.

Table 10.1: MCA swimming pool with inlet vs infinity pool

Objectives	Weights	Inlets	Infinity Pool
Safety	3	3	1
Overall accessibility	1	1	1
Circulation	3	3	2
Operational hours	2	3	2
Aesthetics	2	1	3
Constructability	2	2	3
Maintenance	1	1	3
Overall score	-	32	29

According to the MCA a swimming pool including inlets has the preference. The inclusion of inlets in the design has the advantage of improvement of the water circulation within the pool which will minimize the growth of algae and the occurrence of smelly water due to stagnant water. Most importantly, the implementation of inlets increases the overall safety of the swimming pool as it minimizes overwash and it improves the visibility of the end-walls. These advantages outweigh the downsides of a less aesthetic swimming pool.

10.6.2 | Concrete mix and reinforcement

As we have made the decision to design a gravity wall, the main function of the concrete is to add weight to the structure. However, the cement must also protect the rebar. This means that it will have a high enough cement content to increase alkalinity. As such, a medium-strength concrete class (C30) was chosen for calculations. A good choice of cement type would be a CEM-II cement, as the pozzolan additive strengthens and densifies the concrete over time, reducing the permeability. Higher salt resistance is indicated by SR (Sulphate Resistant, Betoniek, 2012).

In contrast to other developing nations, no concerns need to be raised about the way concrete is poured, or the cleanliness of the water used to create the concrete.

The rebar, on the other hand, was determined to need special attention. As the tidal pool wall will necessarily be continuous contact with saline water, corrosion is a real concern. In addition to the concrete specifications named above, the rebar must also be sufficiently covered by the concrete. The US Army core recommends 3.5 inches, which has been translated to 100 mm of concrete cover over the structure.

10.7 | Handrails

According to Hallet Cove Ocean Feasibility report (Carley et al., 2018), handrails are often included in tidal swimming pool designs in case fall-hazard (more than 1 meter). As this is one of the situations, occurring at low tide when the pool is completely filled, the swimming pool design will also include handrails at the endwall. It must be noted that handrails increase the costs of maintenance as practice shows that they often get damaged by boulder impacts during storm events (Carley et al., 2018). Besides the fencing the endwall, fences will also be places at the sides of the swimming pool, this to control the paid entrance of the pool.

10.8 | Lane separation

As the floor of the tidal swimming pool will consist of the naturally present sand layer, no lane separation lines can be painted on the floor. Instead lane ropes will be used. These ropes, on which buoys are attached, will float on the water surface, clearly marking the different sections of the swimming pool. This will improve the overall user-experience as adults and children are able to swim in a water depth (either the shallow or the deep water section) suitable for each. The lane ropes will also facilitate to the the function of location for swimming competitions.

11 | Preliminary design

11.1 | Wall placement and dimensions

The wall has been designed with very strong safety margins in mind. This is in part because of the uncertainties in the data, and in part because of local design methods and construction methods. For example, it is usual to anchor structures to the local bedrock, even if the structure is auto-stable.

The chosen structure is a gravity wall. This structure was chosen for its relative simplicity in both design and implementation, and reliability.

The height was dictated by the wall bathymetry and sea level measurements explained in subsections 6.2.3 and 6.2.5. Using these heights we came up with the design that can be seen in figure 11.1.

Working from top to bottom, we have a wall which is relatively wide to resist the powerful winter swells that occur along the Namibian coast. To reduce wave impact the sea-facing wall was inclined with a slope of 1:3. The strong shear forces are also resisted by the relatively thick rebar cage, which was calculated using a software called Prokon ("Prokon 4.0", n.d.). The footing was chosen very wide to prevent overturning.

As can be seen in figure 11.1, the foot of the wall is dug in. This presents two advantages. The first is that it increases the resistance of the wall to sliding. The second is that it is safer not to have a depth discontinuity in the pool. We must remember that the sea water in Lüderitz is turbid, so such a discontinuity might be dangerous. It must be noted that in practice, the foot may well not be neatly rectangular as in figure 11.1. This is because we would want the concrete to be poured directly into the rocky excavation. This ensures the best cohesion between the rock and the concrete.

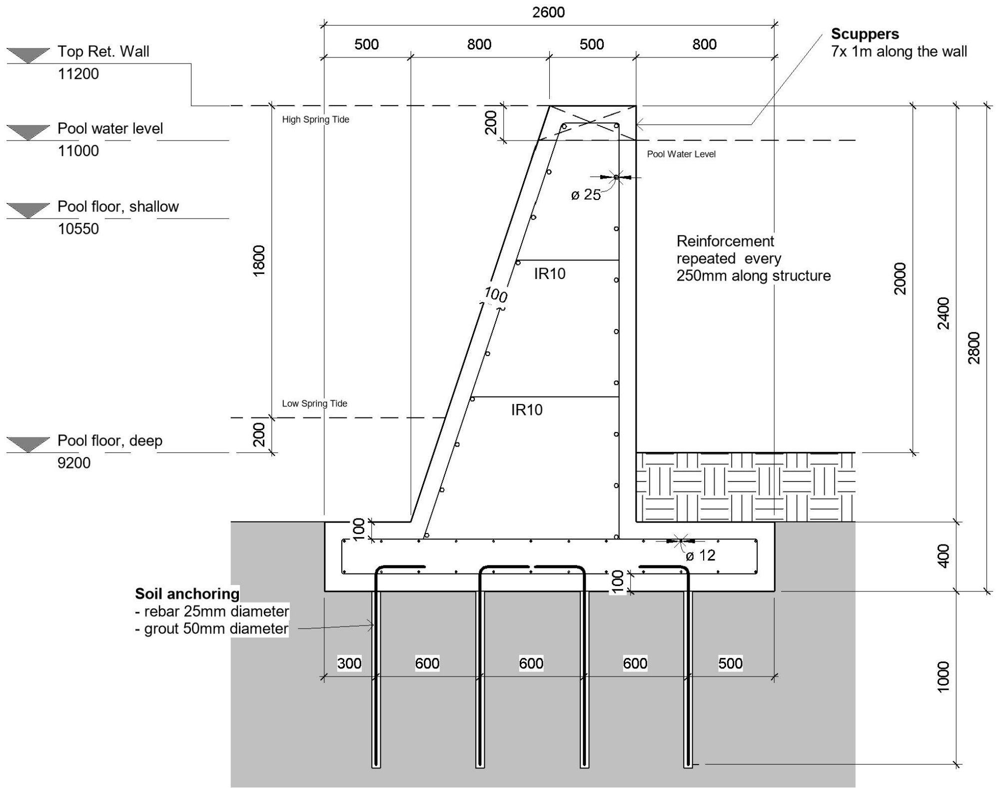


Figure 11.1: Final design of the tidal pool wall, with the sea on the left and the pool on the right

In the horizontal plane, the wall is designed to be curved. This can be seen on the pool wall plan in figure 11.2. Benefits of a curved wall instead of a straight one include that the curvature of the wall strengthens it and protects the base from any scour. It also increases the overall volume of the pool.

In discussions with our Namibian supervisors, concern was raised about the implementation of the curved wall. This will have to be discussed with the contractor. A wall made out of several sections (like an irregular polygon) confers many of the same benefits of curved wall whilst making it easier to implement, so long as the sections are not too long. The only benefit that is somewhat lost is scour protection. This is of little concern as the wall will be anchored in bedrock, leaving little to be scoured away.

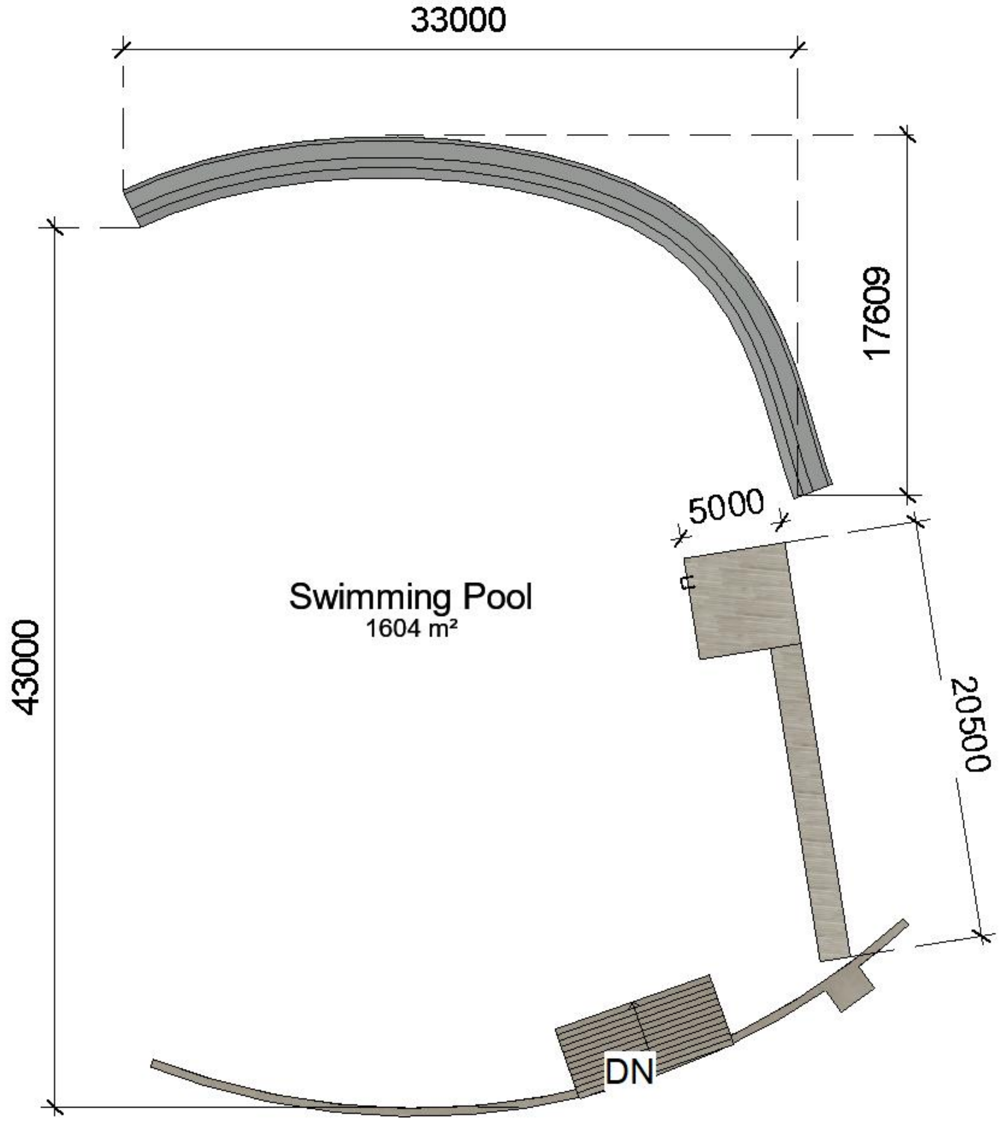


Figure 11.2: Map of the pool walls

In figure 11.2, we can see the wall in the top. At the bottom is the soil retaining wall with stairs, and on the right a walkway into the pool.

11.2 | Wall anchoring

Figure 11.1 also shows that the wall is anchored in the bedrock on which it sits. As was explained in the previous section, this is local practice when building such structures. As such, the structure provides supplementary resistance to shear and overturning. The design team is confident that the wall will stand by itself and is very much aware that it is a very conservative precaution to take. That being said, the team designed the anchors as was requested of them.

The anchors are one metre long to spread the load and to bind eventual rock layers firmly together. The anchor strength was calculated using an ultimate bond strength between rock and grout of 1000 kPa (see table 6 of Leventakis, 2016, original values from H. and T., 2002). The ultimate limit strength of the grouted anchors was calculated using the guidelines found in Miltenberger, 2001. This includes a shear safety factor of 0.65 and a tension safety factor of 0.75.

The same rebar was used in the ground anchoring as in the stem of the wall. This ensures that the structure is well tied to the bedrock, and simplifies the construction of the wall.

One last important note: The placement of the rebar within the concrete is purely indicative, as we were told to draw it in, but that the contractor would know how to place it.

11.3 | Overtopping

For the tidal pool to be self maintaining, it is important that the height of the wall crest allows frequent and sufficient inflow as this will minimize algae growth and sand accumulation. As a rule of thumb it could be stated that inflow should occur at least during high water neap tides with dominant wave conditions (Bosman & Scholtz, 1982)

The overtopping discharge is directly related to the impulsiveness of the waves in front of the wall, the water level (WL), the incident significant wave height (H_{m0}), the spectral period (T_{m-1}), and the crest freeboard (i.e. crestheight of the structure, relative to the water level.).

Calculations for the determination of the wave characteristics can be found in Appendix E. The procedure, including the calculations, for finding the design crest freeboard and its overtopping discharge corresponding to the wave characteristics is provided in Appendix F.1.

Table F.2 shows that a crest freeboard of 0.06 [m] will result in an overtopping discharge that ensures both sufficient water refreshment during HWS and HWN (50% and 10% exceedance wave conditions). This value should be considered to be the maximum value as it will be easier to raise the walls if this is afterwards found to be necessary than to lower them.

11.4 | Inlets

The design of the inlets follows the principles of a fully contracted weir, i.e. a weir which has an approach channel whose bed and walls are sufficiently remote from the weir crest and sides for the channel boundaries to have no significant influence on the contraction of the nappe. For the limitations on the design of a rectangular sharp-crested fully contracted weir, see table F.4.

Appendix F.3.2 includes the calculation for the filling time of the tidal pool after complete drainage. Several combinations of the amount and width of the inlets result in the desired filling time. For the final design of the water retaining wall, the available filling time of approximately 100 minutes is held governing. This wish complies with a wall design including 7 inlets with a width of 1.0 [m] and a filling time of 78 minutes.

11.5 | Management guidelines

The pool has three major guidelines that are summarized below.

- The pool needs to be emptied every two weeks to prevent algal growth
- The pool must not be empty during high swell episodes
- When the waves overtop the wall, the pool must be closed for safety

11.6 | Soil retaining wall

As can be seen on the map in figure 11.2 there is also a soil retaining wall. Its purpose is to maintain the depth of the pool at the shore side at around 0.45 m, the depth used to teach children to swim in the Netherlands.

The soil along the line where the soil retaining wall will be placed is made up of a layer of sand with rock underneath. With a preliminary design of 1 m deep (dug downward from the surface), the soil wall will likely overlap with the rock layer. In this case, the wall can be placed on top of the rock layer by pouring it in-situ. For the design, however, the conservative assumption was made that there was only sand.

The limiting factor for the design is salt intrusion into the concrete, which might cause corrosion. As before, we use 100mm of cover. This causes the soil retaining wall to be thick. The resulting design can be seen in figure 11.3.

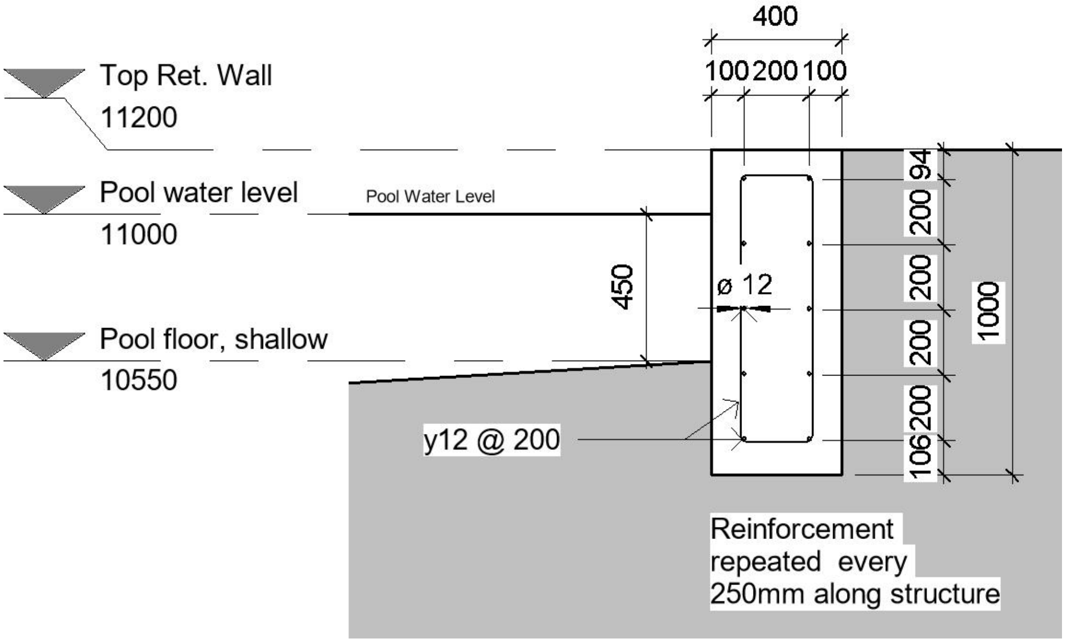


Figure 11.3: Preliminary soil retaining wall design

12 | Preliminary construction plan

This chapter describes some aspects of the construction plan for the pool.

12.1 | Cofferdam

It has been decided to use a cofferdam surrounding the project area, so that concrete elements can be cast in-situ. There was limited time to detail the cofferdam structure. However, we have consulted with the contractor, who has experience with cofferdams and assured us that the installation would be complex but possible. Therefore, we don't elaborate further on the cofferdam in this report.

12.2 | Excavation plan

Besides the installation of the concrete, the excavation of rocks is an important element in this project. The pool area is characterized by many rocks and bedrock on the sides and the bottom. An aerial view is provided in figure 12.1. This view is decisive in the excavating areas, however other pictures from a lower angle may provide a better view on the area, see figures **??** and 12.2. The excavation areas are tagged with letters. The next points provide comments for each section:

- In general, the rocks in the center of the pool (below the water level in figure 12.1) need to be excavated. Several specific regions where excavation is also needed will be described here.
- Section A: most rocks here are interfering with the pool use and have to be removed. The natural channel formation marked in green (3) is to be preserved.
- Section B: on the right side of the pool: a floating walkway will be placed along the breakwater. (Bed)rock that interferes with the placement of this walkway should be removed.
- Section C: the bedrock in the center of the pool should be removed. The rocks of the green areas (4) should be preserved, since they form the sides of the concrete pool stairs.
- Section D: removal is optional if the rock interferes with installation of the land wall.

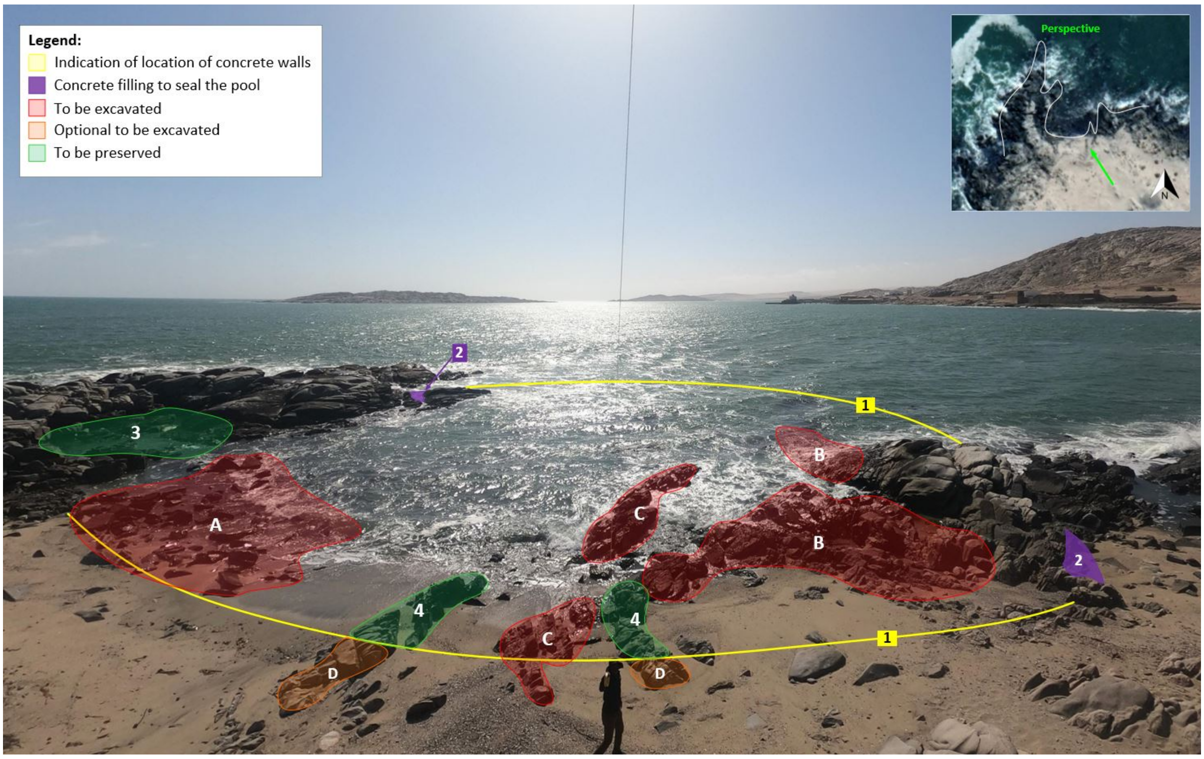
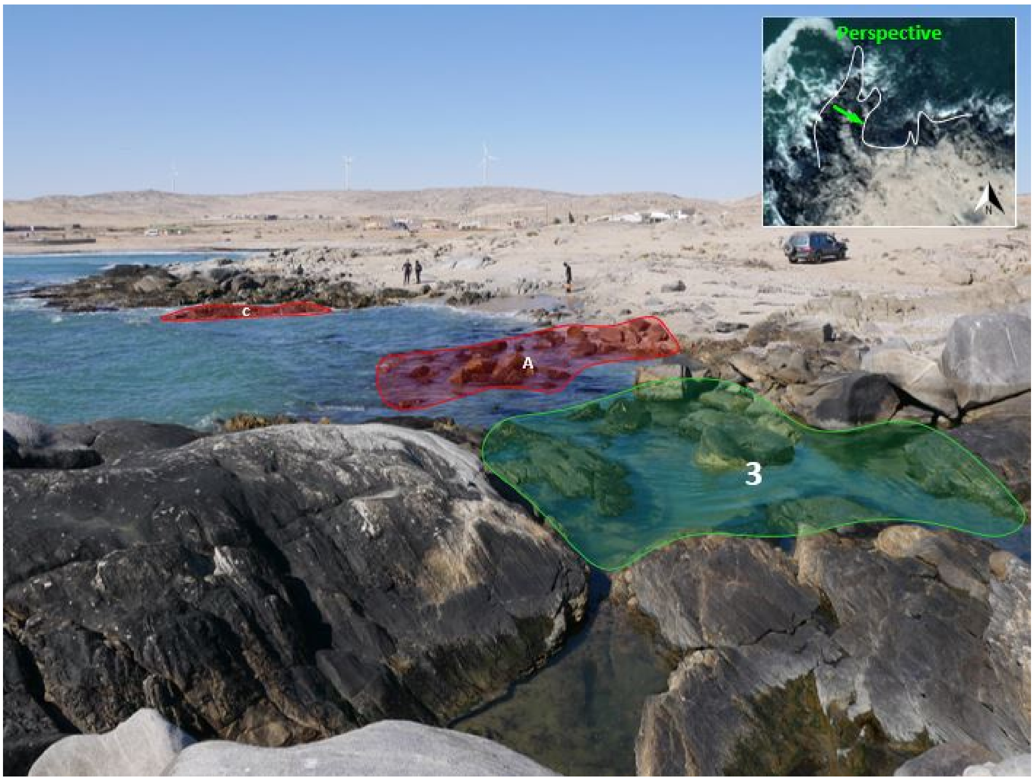
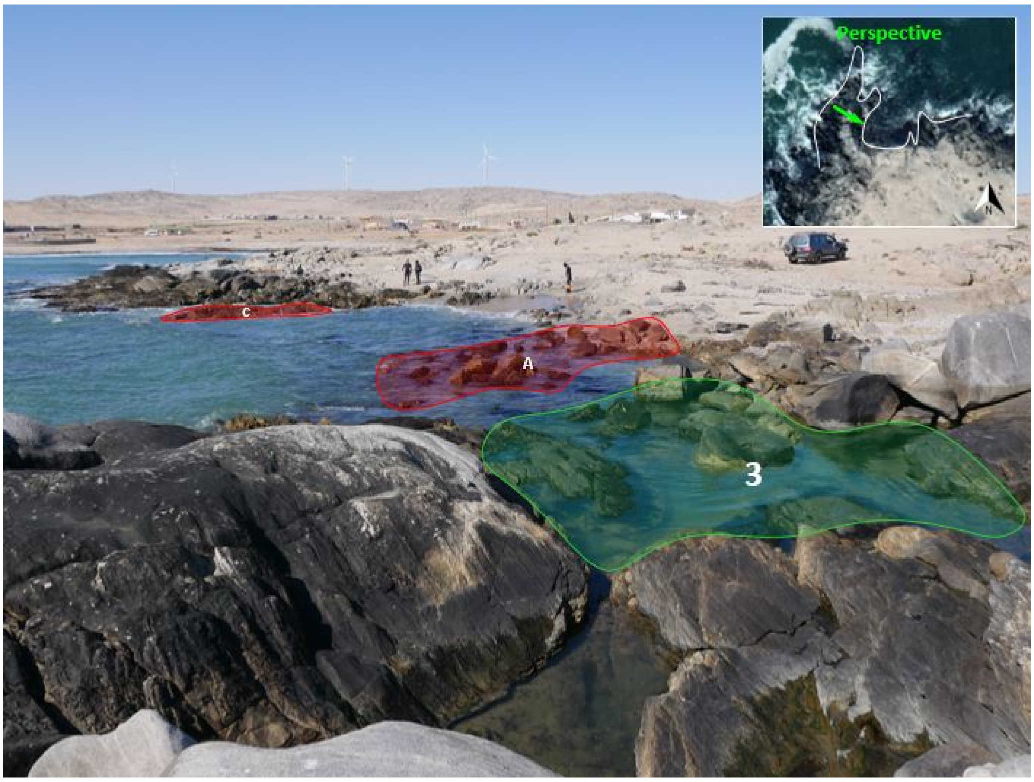


Figure 12.1: Top view excavation plan



(a) Perspective from right rock formation



(b) Perspective from left formation

Figure 12.2: Side view excavation plan

Part III
Building design

13 | Site design

After choosing the location of Aeroplane Bay for the design of the Tidal Swimming pool, the site was further analysed as described in chapter 6, Site investigation, and conceptual site plans were made. The on-shore facilities needed to support the tidal swimming pool design consist of:

- **Clubhouse:** Shared by the lifeguards, swimming club, and as an educational room to teach children and other visitors.
- **Café**: Operated by an external party, which takes care of daily maintenance through a financial incentive of selling drinks.
- **Showers:** Public outdoor showers prevent the need for individual shower cubicles.
- **Toilets:** Three toilets are needed. We decided that one of them must be wheelchair accessible.
- **Changing rooms:** There must be two changing rooms for male and female visitors.
- **Entrance:**

13.1 | User interaction

From the café, visitors should have a view to the north, so the sightlines from both inside the café and the terrace should reach both the swimming pool and the islands on the north-west horizon. For the design, this means that the cafe needs to be angled in the right direction, and the facade of the visitors should be maximally transparent.

13.2 | Concept scheme

First, a configuration of spaces is laid out based on the traffic flows between the different facilities, as seen in figure 13.1. Then, the surface area of each function in the scheme is displayed to scale with rectangles, and multiple configurations were considered, two of which are shown in figure 13.2.

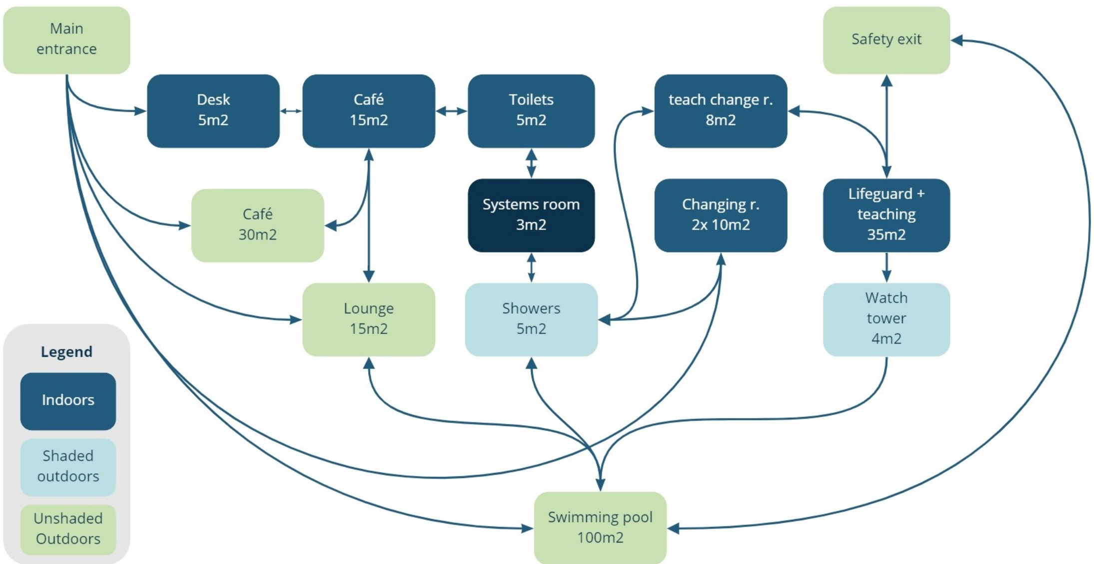


Figure 13.1: Circulation scheme

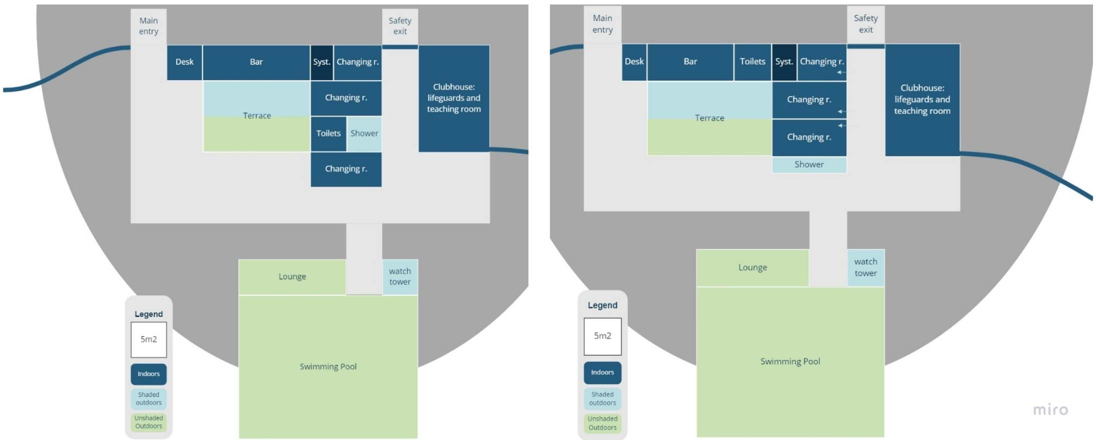


Figure 13.2: Schematic site plans

13.3 | Context integration

In order for the design to fit into the local context, the routing, climate and height elevation were mapped and photographed as described in chapter 6, Site investigation.

Site measurements

The on-shore site measurements as shown in figure 6.10 in chapter 6, Site investigation, were quality checked with the on-scale image taken from google earth, and found to be matching. The satellite image chosen for this quality check is taken from 2004, since in that year the rocks on the location were most clearly defined.

Elevation implementation

Simultaneously with the development of a schematic site plan, the height maps of the location were created, using CADmapper and photogrammetry for the on-land elevations, as seen in figure 13.3.

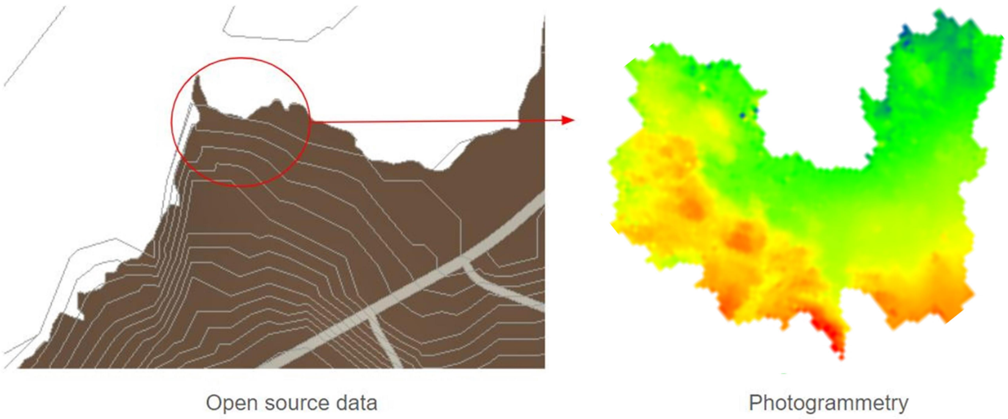


Figure 13.3: Elevation maps made with CADmapper (left) and the height map (right)

This height map was then imported into Revit to use for the 3D modelling of the existing site elevation. Three points from the photogrammetry were geo-located and matched to locations of the satellite footage, but since the coordinates did not match with each other, unfortunately, the photogrammetry could not be used in the Revit 3D modeling of the site elevation.

Local climate

The climate conditions have been documented in chapter 3.2. These conditions impact the building design in a few primary ways. First, due to the wind from the south, it was decided to use the buildings themselves as a wind shield for visitors. The openings and entrances are on the north side, as well as the terrace areas where people can sit outdoors. Second, due to the intensity of the sun during summer time, glazed surfaces must be shaded to prevent direct irradiance. Lastly, if photovoltaic solar panels are implemented on the roof, the ideal angle to optimise PV efficiency would be 20-30 degrees.

13.4 | Site Plan

Café view From the café, visitors should have a view to the north, so the sightlines from both inside the café and the terrace should reach both the swimming pool and the islands on the north-west horizon. For the design, this means that the cafe needs to be angled in the right direction, and the facade of the visitors should be maximally transparent.

Landscape integration
Integrating the site analysis and the conceptual schemes resulted in the schematic site plan shown in figure 13.4. In the top image, the on-shore facilities have been placed on the relatively flat area further up the hill. On the image below, the buildings have been integrated with the landscape and curve around the elevation of rocks to the south. After feedback from Kelp Blue, the principle of a curved landscape integration was chosen, as shown on the right side of the image.

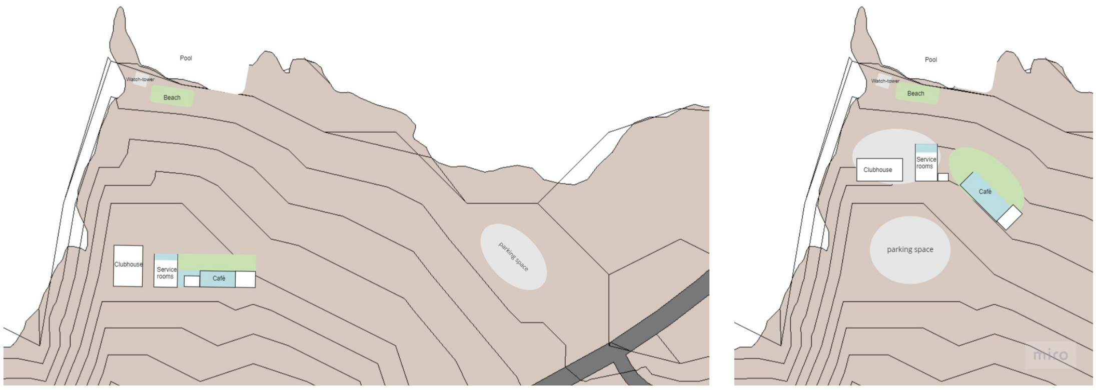


Figure 13.4: On-shore facilities placed to minimize excavation: on the flat plateau (left) or along the curvature of the hill (right)

Joined or separate
Initially, each cluster would be given their own separate building, so that the different functions could be constructed and installed over a larger span of time and so that the construction of each function would be independent from each other. We initially divided these separate functions between the clubhouse, the service rooms (with the toilets, showers and changing rooms), and the bar with the entrance desk. However, as the design developed further, we decided to keep the functions joined under one roof.

Rotation of the building. Following the principles of landscape integration first, and view to the surroundings second, the resulting building orientation would be as shown on the left image of figure 13.5, whereas an optimal view of the ocean and the islands at the horizon would result in a building rotation as shown on the right image. After presenting the benefits and drawbacks of these choices, it was decided that an optimal view was more important than the increased amount of excavation that would be needed. Finally, the rotation as shown on the right was chosen, with grid line 7 pointing to the north, and the east and west wings of the building angled 17,5 degrees towards the south.

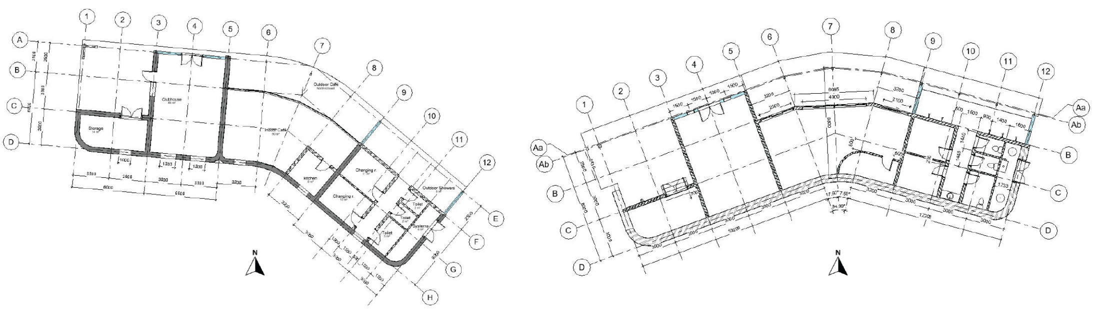


Figure 13.5: Optimal landscape integration (left) and optimal view from the café (right)

Site conclusion

The resulting site design is shown in the site plan of figure 13.6, the north-south site section of figure 13.7 and the aerial view of figure 13.8. The building design will be further explained in the next chapters.

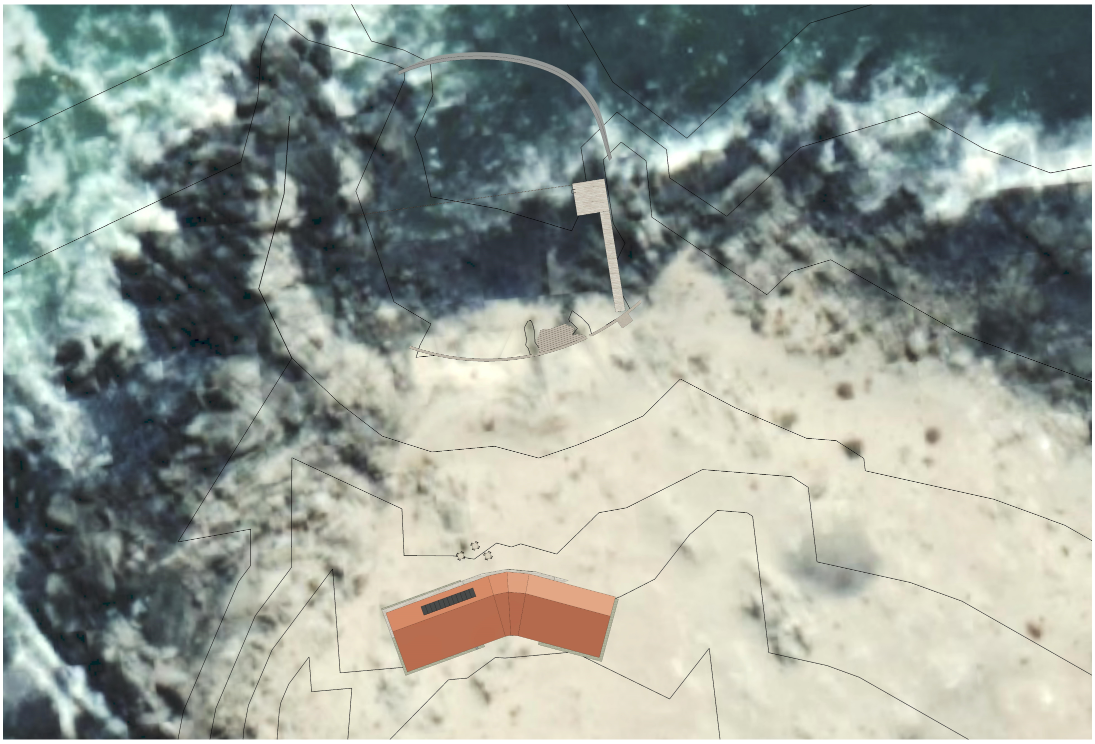


Figure 13.6: Site plan

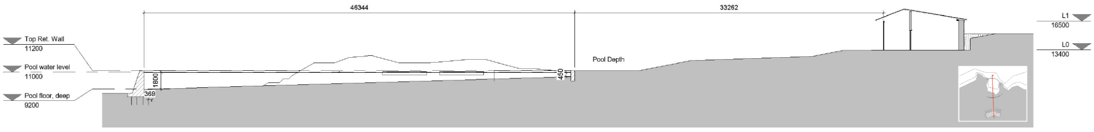


Figure 13.7: Site section

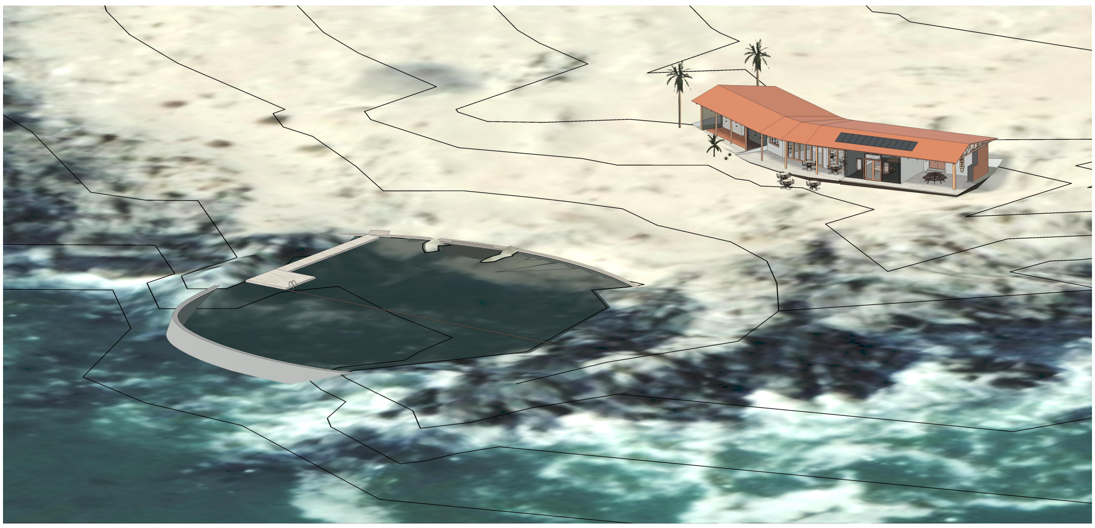


Figure 13.8: Site aerial view

14 | Design Concept

The main two principles for the design of the on-shore facilities are sustainability and photogenic attraction. Since the facilities will be used for educational purposes, sustainability itself can be used as the photogenic attraction. The building can be used to educate visitors about sustainability through the use of recycled materials, and passive climate systems. By visiting the building, both local people and tourists can be reminded of the importance of recycling and become inspired to rethink conventional building practices.

14.1 | Earthships

One of the main inspirations for the building typology comes from the earthship buildings. Mainly built in desert climates, these buildings are mainly made from natural or recycled materials and (mostly) self-sufficient in their own water and energy needs. Although the on-shore facilities for the tidal swimming pool will be connected to the urban grid for energy, fresh water and sewage, principles applied by Earthship buildings can be applied to reduce the building's effect on the environment. The figures below show several Earthship concepts and several exteriors and interiors of building which were used per inspiration for the project.

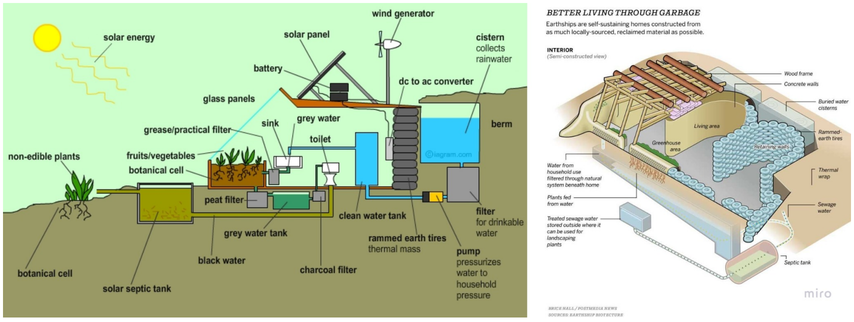


Figure 14.1: The Earthship Concept

Figure 14.2: Earthship exterior reference projects

Figure 14.3: Earthship Interiors

14.2 | Materials

Car tires: structural and climate regulation
Similar to how Earthship buildings are constructed, the building that facilitates the on-shore facilities for the tidal pool is mainly constructed with recycled car-tires, rammed with local soil. The wall is then used to retain the loads from the hill south of the wall indicated in chapter 13.

Together with the hill, the southern tire wall acts as a large thermal mass storage. By storing the heat of the day, and slowly releasing it during the night, the southern wall act as a buffer to fluctuating air temperatures. This increases the passive climate comfort of the building. The location of the car-tire wall is indicated in figure X.

Additionally to the retaining wall, recycled car tires can be used as outdoor furniture as shown in figure 14.5.

Figure 14.4: Examples of tire walls in Earthship buildings

Figure 14.5: Outdoor furniture created from recycled car tires

Glass bottles: photogenic and sustainable
For the blue wall indicated on figure X, the surface mainly consisting of recycled glass bottles. The bottles can be collected by activating the local community. Then the tapering end of the bottle has to be cut with a circle-saw, and two bottom halves of a glass bottle can be glued together so that a cylinder with the same length as the wall thickness is created. The glass bottle wall is held together with a mixture of sand, clay and cement.

Figure 14.6: Glass bottle walls

Timber: durable and bio-based After weighing the pros and cons of timber against a steel roof structure, a timber structure was chosen mainly from the perspective of sustainability. In initial stages of the roof design, a minimalist webbing of diagonal members was chosen, shown in figure 14.7 on the left. However, after consulting national regulations, and after analysis of the timber members was performed, a different type of structure is chosen, as shown on the right. Instead of a hardy wood type with a larger dimension, the timber members can be constructed with a dimension as small as 46x111mm for the rafters and 46x73mm for the bottom chord and diagonals, reducing the costs of construction. The material chosen for these structural calculations was South African Pine.

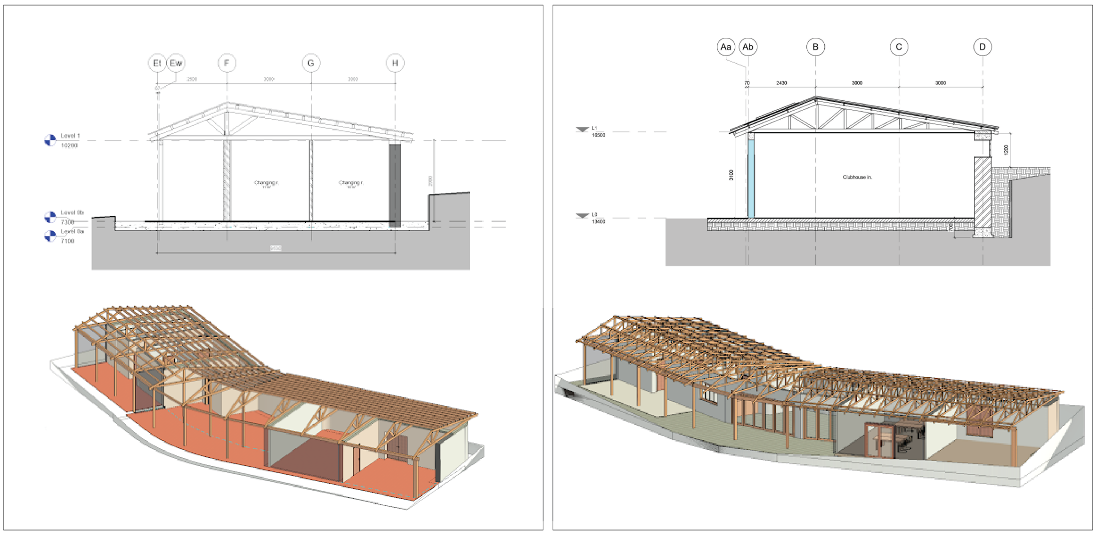


Figure 14.7: Development of timer trusses

15 | Building Design

The drawings provided in this chapter can be found on scale in the appendix 'tidal pool building drawings' accompanying this report.

15.1 | Floor Plans

In figure 15.1, the room schedule and surface areas are shown. The clubhouse has it's own sheltered outdoor area on the west. The indoor area of the clubhouse has large glazing doors on its northern facade built into a wall of recycled glass bottles. This was has not been placed on a set-back from the roof like the rest of the northern facade so that the sunlight can fall directly onto the glass bottles. The set-back of the café area makes sure that this direct sunlight is prevented from entering the indoor café area, which means a reduction of temperature fluctuations due to solar irradiance. The café is equiped with a small kitchen area and a bar on the east side of the cafe, and in the wall north of the café area, a sliding window gives the possibility of selling drinks and snacks to visitors without the visitors needing to enter the café directly. Then on the eastern section of the building, the outdoor showers are placed between two glass-bottle walls, on which the light can fall directly during early and later hours of the day, when the sun is lower. Behind the showers are the changing rooms and toilets, and the system room where an electricity closet, water boiler, and water tank with recycled water caught by the shower drain can be stored.

Floor Surfaces

The idea behind the floor surfaces is to give the building a natural appearance. A seamless transition between the indoor and outdoor areas of the café is created by grouping the indoor timber of this place together with the recycled timber floorboards that are used on the outside deck that connects the different indoor building elements. The wet-areas such as the toilets and showers will have ceramic tiling on the floor.

Figure 15.1: Floor plan with room schedule

Figure 15.2 shows the structural grid lines of the building. With the center-lines of the walls placed on the axis of the grids. The thick outer walls are the ones consisting of locally collected recycled 14" and 15" car tires, and the room separation walls can be placed by conventional masonry bricks. The angle of the east and west wing of the building, could be adjusted based on more precise site measurements.

Figure 15.3 shows the grid lines of the roof structure, with a truss always placed maximally 1.2 meters apart, as required by Namibian building codes.

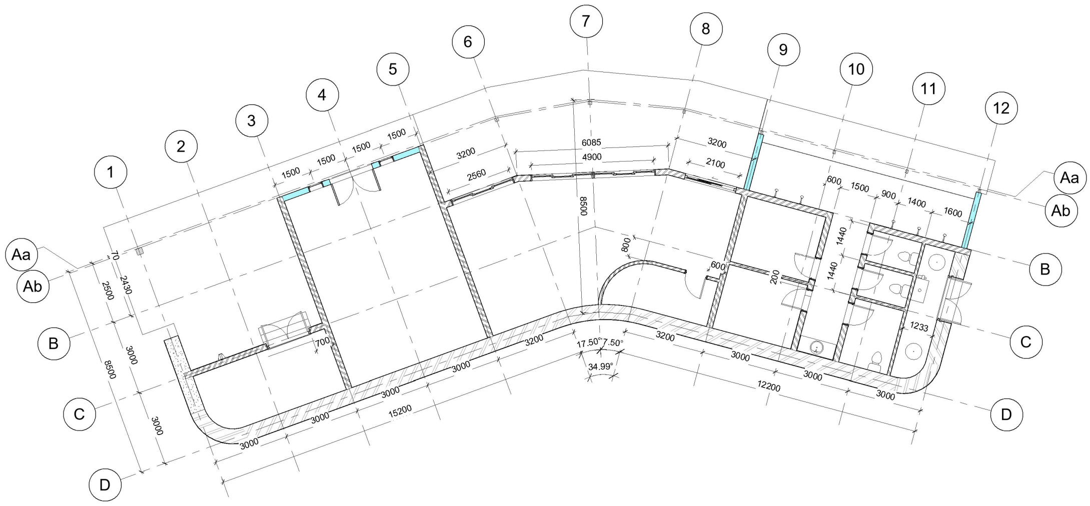


Figure 15.2: Floor plan with grid lines and dimensions

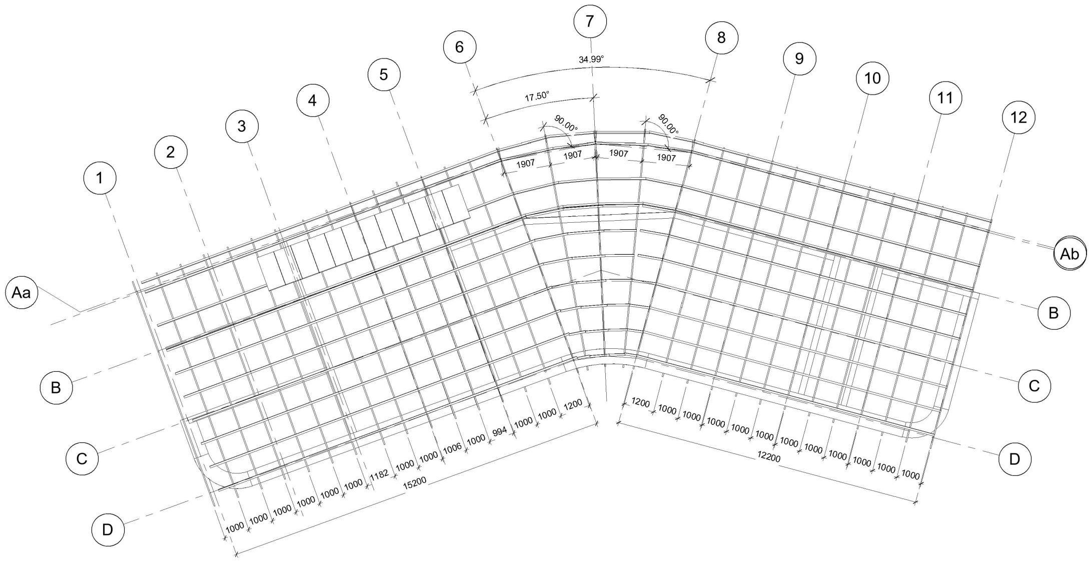


Figure 15.3: Roof plan with grid lines and dimensions

15.2 | Elevations

The northern facades of the building are the most transparent elevations. They are designed to let in the natural light while shading against direct solar irradiation on the glazed surfaces, and the northern roof is angled so that pv-panels are most efficient in their generation of solar energy. The off-white plaster sets this facade apart from the rusty red plaster as seen in figures , which is more resistant against soiling from the hill on the south.

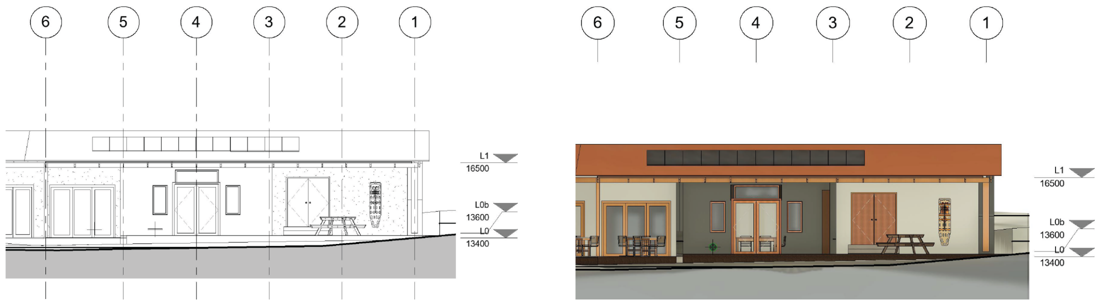


Figure 15.4: Building elevation north, Grid 1-6

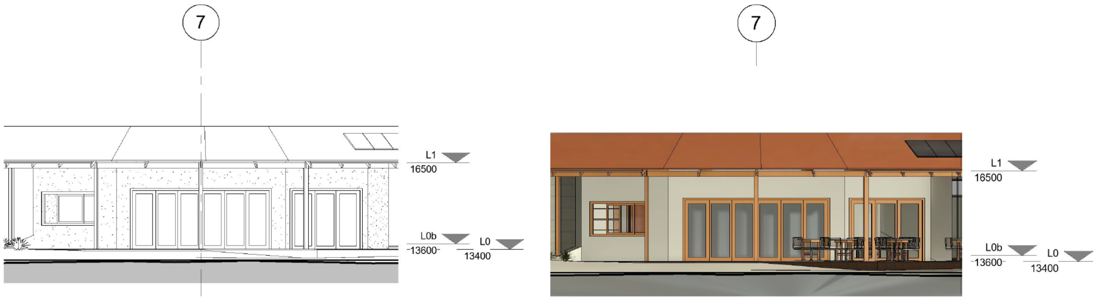


Figure 15.5: Building elevation north, Grid 5-9

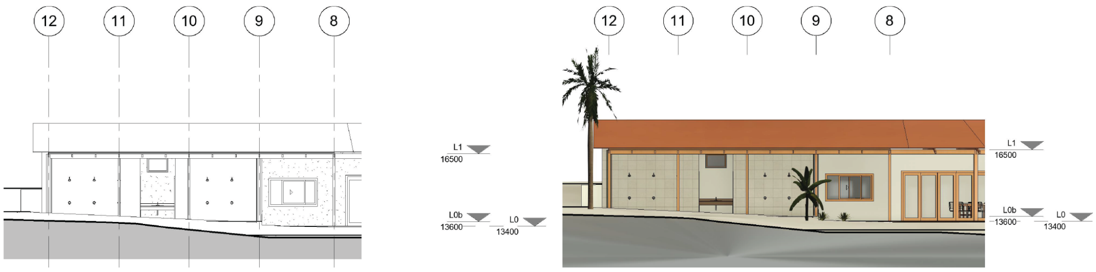


Figure 15.6: Building elevation north, Grid 8-12

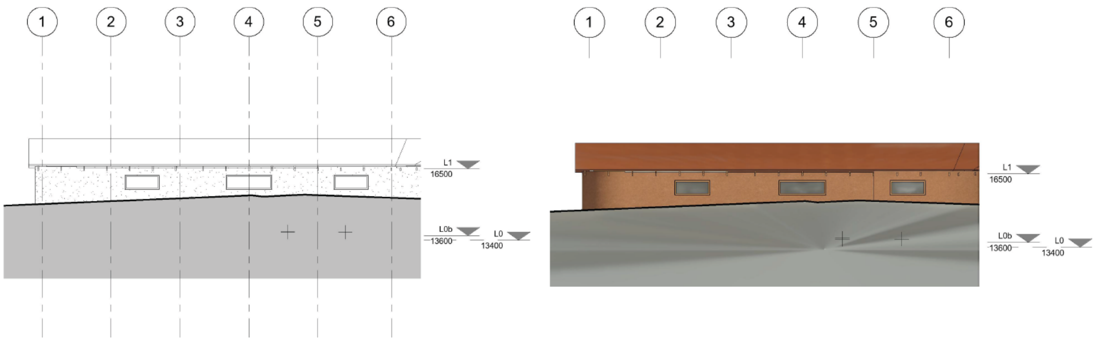


Figure 15.7: Building elevation south, Grid 1-6

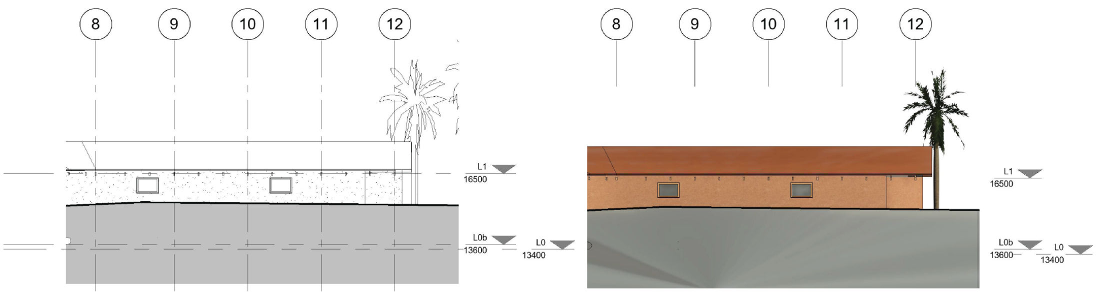


Figure 15.8: Building elevation south, Grid 8-12

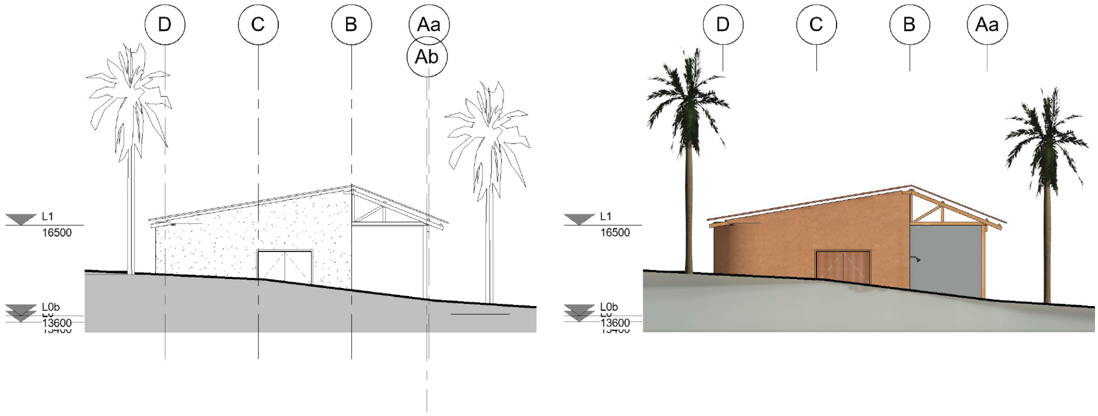


Figure 15.9: Building elevation east

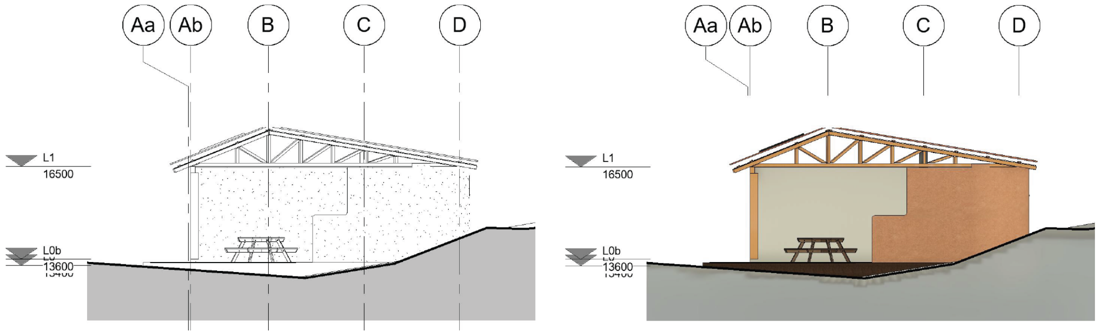


Figure 15.10: Building elevation west

15.3 | Sections

In the following four sections, the conceptual building design is annotated and dimensions are given. These sections are the most typical for each of the areas in the building, and the more detailed construction drawings such as 1:20 building sections or 1:5 details did not fall within the scope of the design assignment. These will have to be developed further in a later stage of the building design.

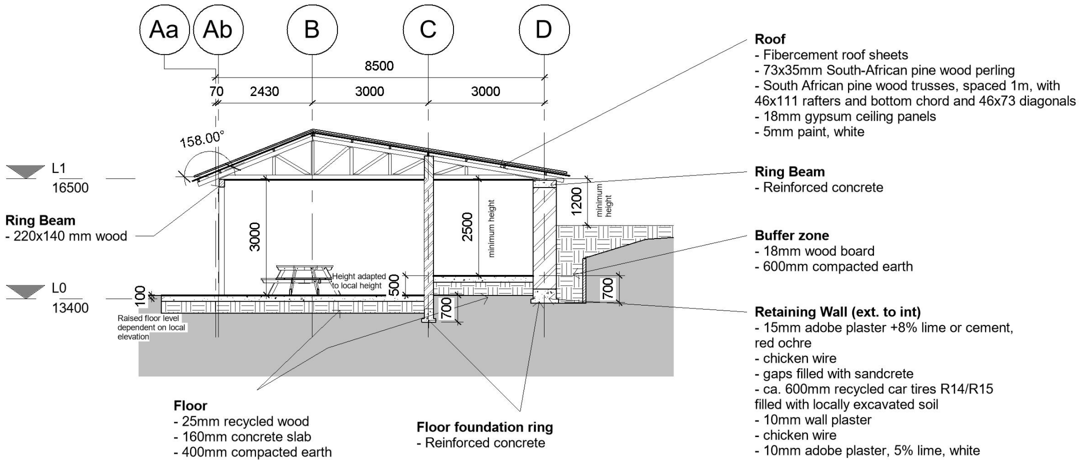


Figure 15.11: Building section: clubhouse outside, original file scale 1:100

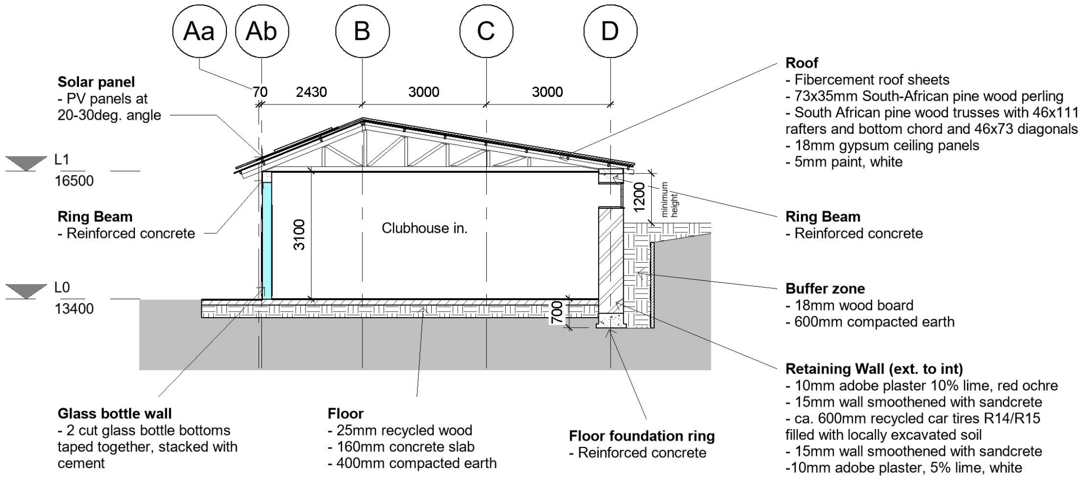


Figure 15.12: Building section: clubhouse inside, original file scale 1:100

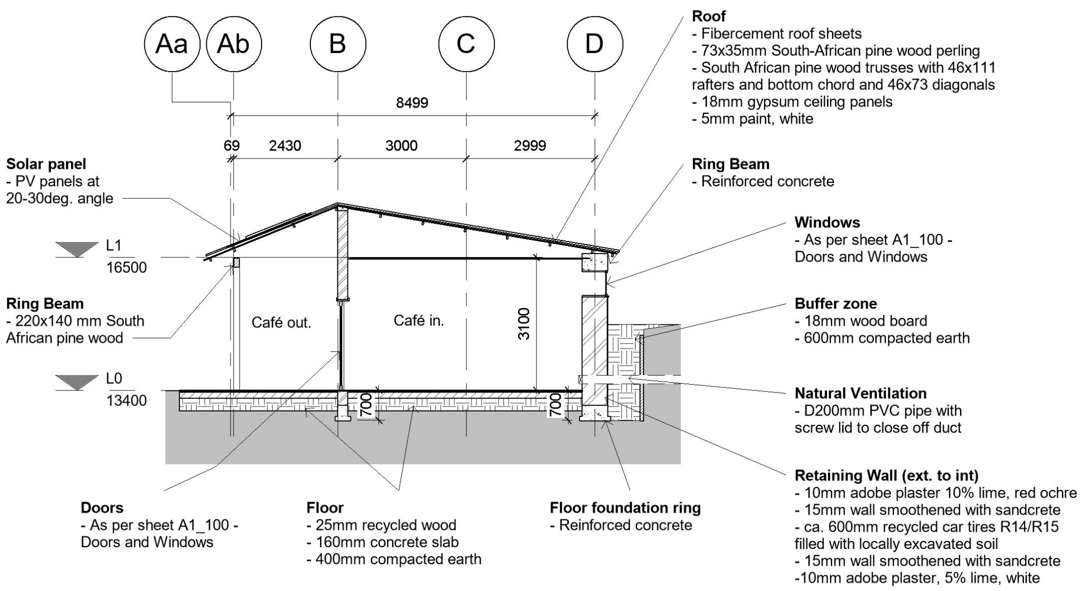


Figure 15.13: Building section: café, original file scale 1:100

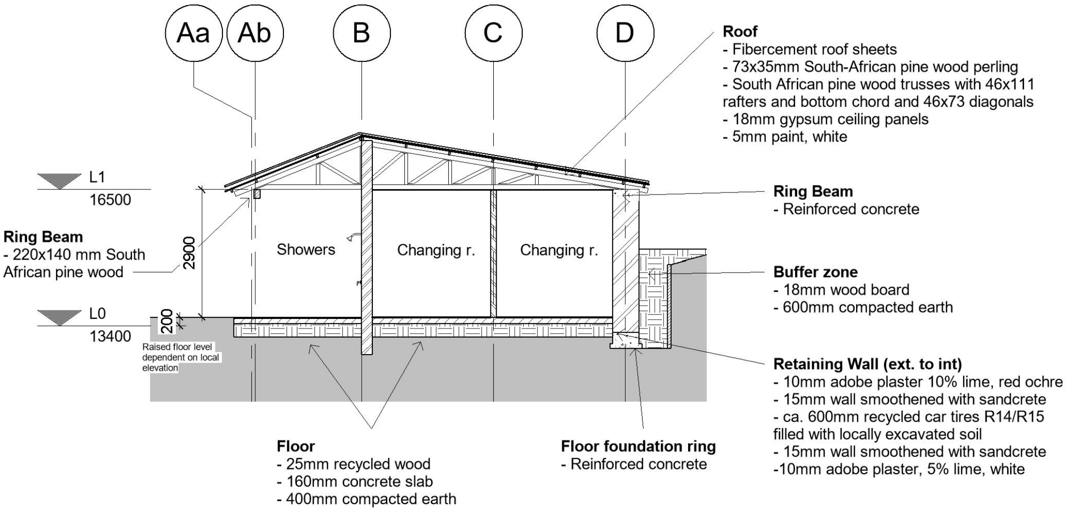


Figure 15.14: Building section: changing rooms and showers, original file scale 1:100

15.4 | 3D impressions

In the following figures, 3D impressions. In figure 15.15, the full tidal pool site is shown as seen from the south-eastern entrance point of the location. Figure 15.16 shows the building as it would be seen from the tidal swimming pool. Figure 15.17 shows the view that visitors of the café will have through the bi-folding doors towards the tidal swimming pool in the north, and finally figure 15.18 shows the full building design from an aerial north-west position.

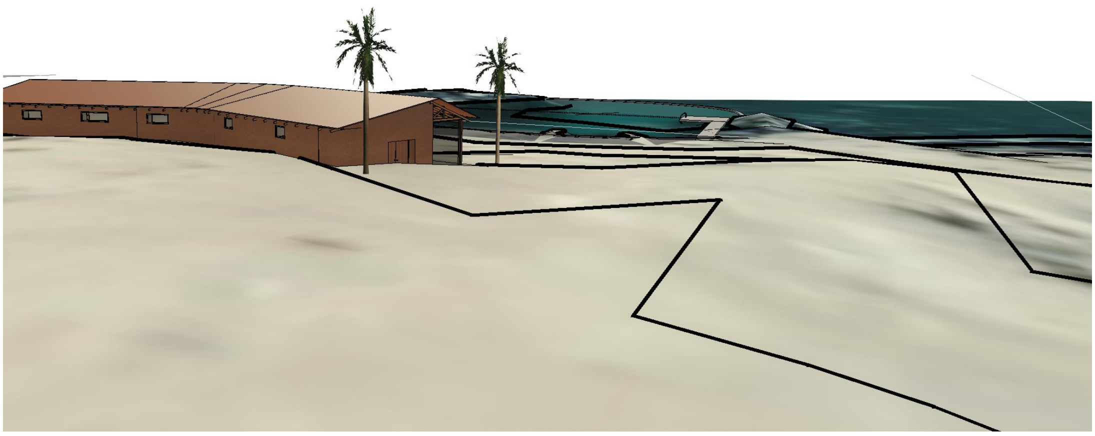


Figure 15.15: View as visitors enter the site area

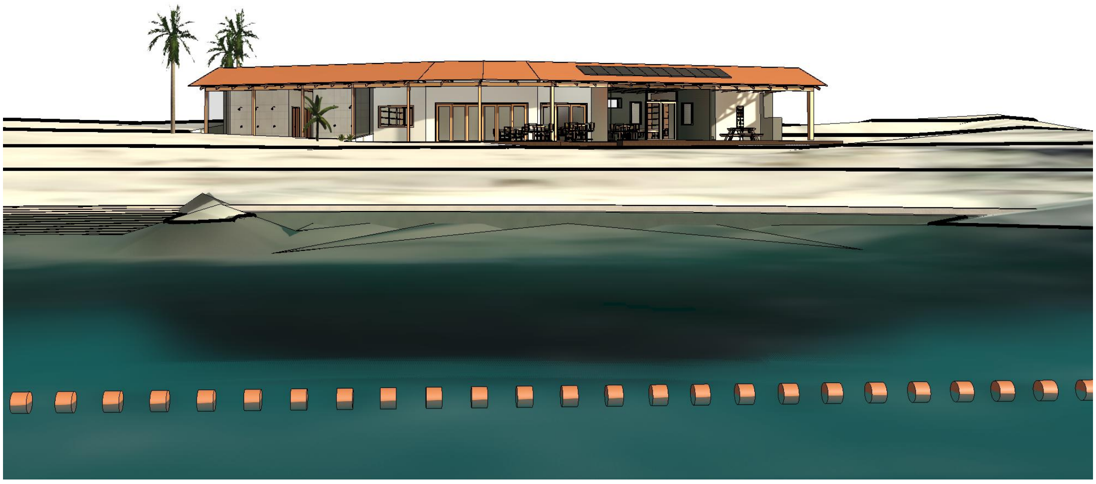


Figure 15.16: View from the tidal pool towards the building

Figure 15.17: View from the café onto the tidal swimming pool

Figure 15.18: 3d impression of the building

Part IV
Conclusion

16 | Conclusion

As was mentioned in the introduction of this report, the tidal swimming pool is suggested by the Kelp Forest Foundation as a means to teach inhabitants of Lüderitz to swim. This idea was brought forward by the Lüderitz Rotary Club, and our design was requested by the Kelp Forest Foundation (KFF). Though the initial investment is made KFF, the goal of the tidal pool is to be self-sustaining. This is an important reason for why the tidal swimming pool also needs on-shore facilities.

For the tidal pool, site surveying has shown that the seabed consists out of bedrock. Therefore, the failure mechanisms that needed to be taken into account were overturning and sliding. Overall stability was guaranteed by implementing a gravity wall. Though based on our calculations, this should suffice, we adhered to local customs by including wall anchors. The resulting tidal pool is slightly enlarged by placing a soil retaining wall. This allows the pool bottom to slope gently downwards toward the sea.

Maitainance to the pool can be performed by the final owner of the pool and on-shore facilites, and daily maintainance can be performed by an external party. To keep the pool free for algae, to prevent water stagnant water becoming smelly and to release built-up sediment, the pool should be drained and refilled every fortnight through the use of an external pump. To ensure the financial self-sustainability of the tidal swimming pool, a bar within the facilities will be rented out to an entrepreneur. The rent, sponsorships and a possible entrance fee will constitute the main incomes that will pay for maintenance.

The facility concept has been inspired by the Earthship concept, and will be mainly built from recycled materials, and the shape adapted to the local landscape. Both the view to the tidal pool and ocean are framed by natural materials such as wood and lime-plaster and are optimally visible from the café with its bi-folding doors and flexible floor plan. There are recycled glass bottle walls situated so that the sun can shine through them, creating a photogenic experience for the visitors, and sustainable energy can be generated through the roof which is angled to maximise solar panel efficiency. Through these methods, the building that accompanies the tidal pool will be an example of recycling in a country still struggling with waste management.

Finally, as is further explained in chapter 17, this report is a preliminary design and can *not* directly be implemented. Further calculations must be made by a licensed professional before the construction process is initialized.

17 | Discussion

The outcomes of the investigations, performed for this project, have provided insight into the social and technical design conditions for the implementation of a tidal swimming pool at Aeroplane Bay, Lüderitz. However, the results should be interpreted with caution due to several limitations experienced throughout the project. This chapter reflects on the design process by discussing these limitations and their corresponding implications for the interpretation of the design outputs. The chapter ends with a recommendation for the finalization of the tidal swimming pool design.

17.1 | Time restrictions

The initial plan was that a second group would come to finalize the design of the tidal swimming pool and its facilities. However, after the first month, we were informed that there would be no second group to continue this project. Thus, the project deliverable was altered from being a conceptual design to being a detailed design. With the scope of the project being significantly enlarged, we were forced to cut back on the amount of design alternatives and the final details of the project.

Below an overview is given of the initial (first three) and the final project deliverables (rest).

- The identification of potential locations for the tidal swimming pool
- Assessment of basic design conditions
- Conceptual design for the tidal pool and its facilities

- Public consultation
- Detailed hydrodynamic and structural design for the tidal swimming pool, including the inlets and outlets
- Detailed design of the facilities
- Organisation and management proposal
- Construction proposal
- Maintenance proposal

17.2 | Scarcity of data

One of the key challenges in this project was the lack of data. Considerable effort was put in gathering data and/or making assumptions to reduce data requirements.

The former, the data gathering, results in low quality data, through a combination of lack of experience, of proper measurement equipment and of long time series. This inaccuracy in the data is solved with relative ease by employing larger safety factors. The main risk is that reasoning behind the safety factors might be erroneous, leading the to be too small still. This we consider unlikely.

The latter, the making of assumptions, may result in large deviations of the correct results. This risk is compounded by our inexperience, which means that we might not recognize such a mistake. Therefore these assumptions will have to be checked by an experienced engineer.

17.3 | Conclusion

In conclusion: this report presents a preliminary design, and therefore does not ensure structural validity. Because of the combination of time restrictions, the experience of the design team, and the availability of site-specific data and modeling software, it is highly recommended that further investigations follow to verify assumptions made concerning the design calculations and to explore other design alternatives, especially regarding the water retaining wall.

18 | References

Betoniek. (2012). *Sr en vhl: Nieuwe aan duidingen in de cementbenaming*. https://www.betonlexicon.nl/C/Cementbenaming

Bhattacharya, A., Romani, M., & Stern, N. (2012). Infrastructure for development: Meeting the challenge. https://policydialogue.org/files/events/PP-infrastructure-for-development-meeting-the-challenge_Romani.pdf

Bosman, E., & Scholtz, P. (1982). A survey of man-made swimming pools along the south african coast. https://doi.org/10.1061/9780872623736.152

Bruce, T., van der Meer, J., Pullen, T., & Allsop, W. (2009). Wave overtopping at vertical and steep structures. https://doi.org/10.1142/9789812819307_0016

Carley, J., Coghlan, I., Miller, B., Drummond, C., & Harrison, A. (2018). *Coastal engineering advice for ballina ocean pool (report no. 2018/23)*. Water Research Laboratory, UNSW Sydney, Manly Vale, NSW.

Goda, Y. (2000). *Random seas and design of maritime structures*. World Scientific.

H., O., & T., B. (2002). *Geotechnical engineering handbook, volume 2: Procedures*. Ernst Sohn Publications.

Iturbide, M., Fernández, J., Gutiérrez, J., Bedia, J., Cimadevilla, E., Díez-Sierra, J., Manzanas, R., Casanueva, A., Baño-Medina, J., Milovac, J., Herrera, S., Cofiño, A., San Martín, D., García-Díez, M., Hauser, M., Huard, D., & Yelekci, Ö. (2021). Repository supporting the implementation of fair principles in the ipcc-wg1 atlas. *Zenodo*. https://doi.org/10.5281/zenodo.3691645

Leventakis, T. (2016). *Ultimate bond strength of anchors in rock*. https://blog.geotechpedia.com/index.php/2016/08/ultimate-bond-strength-of-anchors-in-rock/

LMNO Engineering, Research, and Software. (2014). *Discharge, head, and design calculations. equations and istallation guidelines*. http://www.LMNOeng.com

Miltenberger, M. (2001). Capacity design of grouted anchors. *Transactions SMiRT 16*. https://repository.lib.ncsu.edu/bitstream/handle/1840.20/30697/H1790.pdf?sequence=1&isAllowed=y

Navionics. (2022). *Chartviewer*. https://webapp.navionics.com/#boating@undefined&key=a~o%5C%7BHwxbU

PIANC. (2016). *Criteria for the selection of breakwater types and their related optimum safety levels: Report of working group 196 of the maritime navigation commission*. PIANC Brussels, Belgium.

Prokon 4.0. (n.d.). https://www.prokon.com/

Pullen, T., Allsop, N. W. H., Bruce, T., Kortenhaus, A., Schüttrumpf, H., & van der Meer, J. W. (2007). *Eurotop - wave overtopping of sea defences and related structures: Assessment manual*. EurOtop. http://www.overtopping-manual.com/assets/downloads/EAK-K073_EurOtop_2007.pdf

Schoenberger, J. L. (2022). *Colmap - colmap 3.8 documentation*. https://colmap.github.io/

Smith, G. (2021). *Environmental data for the kelp blue project* [Confidential report]. WSP.

Snow-Forecast.com Ltd. (2022). *Today's tide times for luderitz bay, namibia*. https://www.tide-forecast.com/locations/Luderitz-Bay-Namibia/tides/latest

SwimTayka. (2022). https://swimtayka.org/

The South African National Hydrographer (SA Navy). (2022). *Tide tables for south africa and namibia*. https://www.sanho.co.za/

tide-forecast.com. (2022). *Tide chart for luderitz namibia*. https://www.tide-forecast.com/locations/Luderitz-Bay-Namibia/tides/latest

Van der Meer, J., Allsop, N., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P., & Zanuttigh, B. (2018). *Manual on wave overtopping of sea defences and related structures*. Version 2. EurOtop.

A | Appendix: Stakeholder analysis

Stakeholders Name	Estimated Interest	Estimated Project Impact	Estimated Project Influence	What is important to the stakeholder?	How does the stakeholder contribute to the project?	How could the stakeholder block/delay the project?	Strategies to engage with the stakeholder
Key stakeholders							
Kelp blue (Internal)	High	High	High	To develop a community initiative in accordance with the objectives of Kelp Blue.	To finance the project and accepting the project's final deliverables	when resources are few or insufficient	Weekly update meetings
Lüderitz Rotary Club and Rotary International	High	High	Low	Creating value for the community	It will be responsible for the financing	By denying financial contribution	We will keep the chair of the Lüderitz rotary club updated on the project.
Ministry of Environment and Tourism, Ministry of Fisheries and Marine Resources (External)	Low	High	High – Medium	Project to comply with the Act, regulations, guidelines and other applicable laws.	By screening the project EAP, they can evaluate the EIA application and eventually award a clearance certificate.	Disapprove the EIA application, and as a result, refuse to issue a clearance certificate	Application for the clearance certificate and draft the EAP
Local Authority – Town council (External)	Medium	High – Medium	Medium – Low	Project has to be safe, sustainable and able to be maintained easily.	To be able to approve project plans	Disapprove the project plans	Draft project plans and submit
Community/residents of Lüderitz (External)	High	Medium – Low	Medium – Low	Recognizing the necessity of the project and how safe it is for the kids	To express their thoughts and opinions on the project	If the community opposes the project, does not understand its need, and questions whether it will keep the children safe.	Public consultation meetings, conduct a survey and advertise.
Primary stakeholders							
Kelp Blue – Interns (Internal)	High	Medium – Low	High	To provide a safe, sustainable project design for the community on behalf of Kelp Blue, gain some industrial experience and to help meet the requirements for their graduation.	To ensure that the project design is both safe and sustainable.	Obstacles such as inadequate data and/or equipment available, the need to return to school, or the end of a contract of internship.	Collect data and procure/request for required necessary equipment early on.

Stakeholders Name	Estimated Interest	Estimated Project Impact	Estimated Project Influence	What is important to the stakeholder?	How does the stakeholder contribute to the project?	How could the stakeholder block/delay the project?	Strategies to engage with the stakeholder
Primary stakeholders (continued)							
Primary schools (External)	Medium	Medium – Low	Medium – Low	To understand that the project is suited for swimming/training and is safe	To express their thoughts and opinions on the project	-	Schedule consultation with the schools
Secondary schools (External)	Medium	Medium – Low	Medium – Low	To understand that the project is suited for swimming/training and is safe	To express their thoughts and opinions on the project	-	Schedule consultation with the schools
Secondary stakeholders							
Contractor/construction company (External)	High – Medium	Medium – Low	Medium – Low	To determine whether the design project can make use of locally available materials.	To provide materials that are safe, durable, cost-effective, and environmentally friendly for the project.	The materials required are not available locally or easily (in stock) and are not cost-effective	Prepare ahead of time by requesting quotes from several parties and comparing them.
Local business owners (External)	Medium – Low	Medium – Low	Low	To understand how they can turn this project into a business opportunity.	To express their thoughts and opinions on the project.	-	-
Local clubs (swimming clubs) (External)	High – Medium	Medium – Low	Low	To determine how the project can assist them in expanding and exposing their clubs.	To express their thoughts and opinions on the project	-	Advertisement
Potential sponsors (External)	High – Medium	High – Medium	Medium – Low	To determine how they will benefit from the project such as for exposure or good name making.	Provide free swimming lessons, donate or fund project materials, or even participate at cleanup events.	-	Reach out, send emails, advertise

B | Reference Projects

B.1 | Tidal Pools

Definition : A small area of sea water contained by the rocks or sand around it when the tide goes out.
- Cambridge Dictionary

This chapter gives an overview of different tidal pools. They can appear naturally or artificially either on sandy coastlines behind sand banks, or rocky coastlines. In figure B.1, examples of natural tidal pools are given.


Figure B.1: Natural tidal pools

B.2 | Local examples of tidal pools

The coast of Lüderitz alternates between sand beaches and rocky shores. An attempt has been made to provide smaller tidal pools along the beaches, as seen in figure B.2. Unfortunately, they filled up with sand and are currently unusable for swimming.


Figure B.2: Reference project: tidal pool Lüderitz

The nearest functional tidal pool in the area is located at Point Dias, on the peninsula west of the town. The pool is artificial but has a highly natural rocky appearance, and is relatively shallow. The pool is not well-accessible from the city and thus doesn't function well as a safe swimming location for the local community.

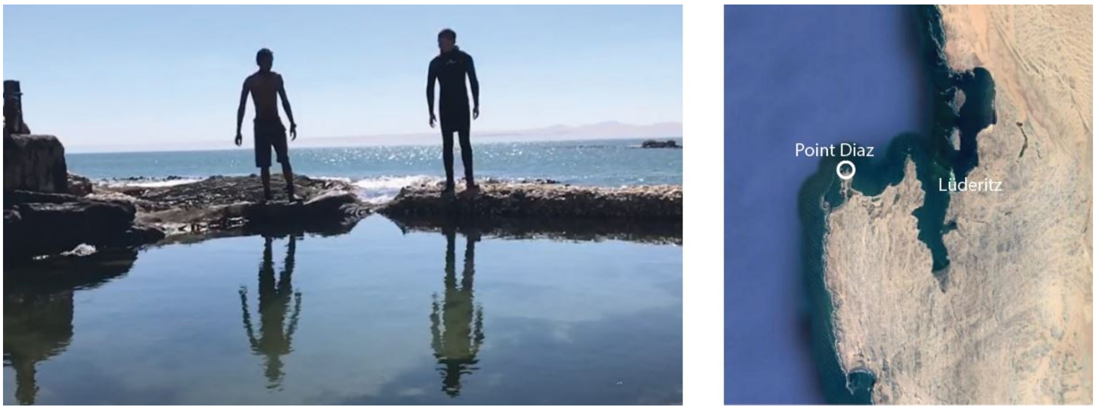


Figure B.3: Reference project: Nearest functioning tidal pool. Location: Point Dias

A functional tidal pool in Namibia is located in Langstrand (Long Beach), located between Walvis Bay and Swakopmund. The pool, as shown in figure B.4, stretches over 70 meters, has a sandy floor, and is of relatively shallow depth.

Figure B.4: Reference project: largest tidal pool in Namibia: Long Beach

B.3 | Shape and Nature Integration

Since one of the criteria for the tidal pool design was a natural appearance, this section categorizes different tidal pools based on their shape and detailing. Figure B.5 shows conventional rectangular tidal pools. figure B.6 shows more natural rocky entrances with a curved and a straight artificial ocean barrier, and figure B.76.5 shows tidal pools with sandy entrances. Finally, the reference tidal pool in figure B.8 shows the integration of natural elements throughout the artificial pool.

Figure B.5: Reference Projects: Rectangular tidal pools. location: Tenerife (left) and Cape Town (right)


Figure B.6: Reference projects: rocky entrances with round and straight artificial ocean barriers. Location: Cape Town (right).

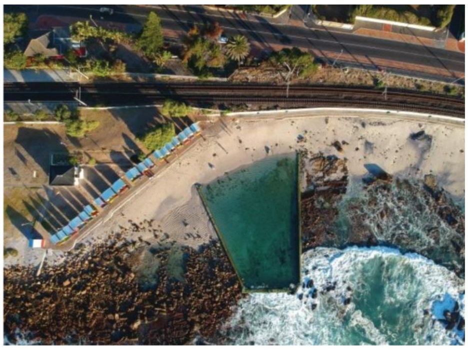
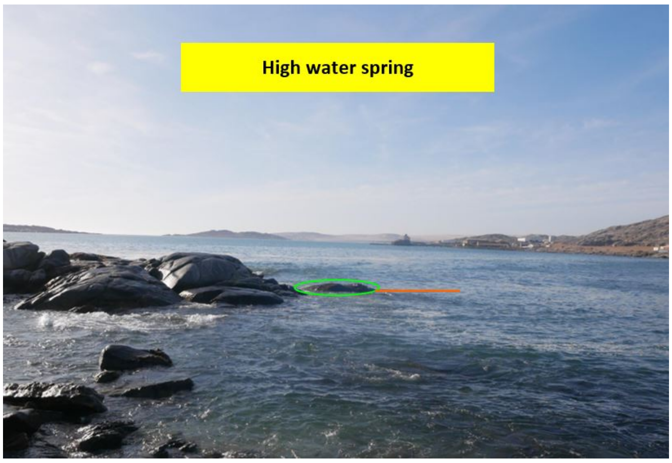



Figure B.7: Reference Projects: Sandy Entrances with Artificial Ocean Barriers. Location: Cape Town (right)


Figure B.8: Reference Tidal Pool: Integration of Natural Elements. Location: Porto Moniz (left and rigth).

C | Appendix: Physical observations

C.1 | Photo documentation



(a) High water spring

(b) Low water spring

(c) Mean sea level

Figure C.1: Different tidal levels captured (orange line illustrates high water spring level)

(a) View looking at east

(b) Detail on end of breakwater

Figure C.2: Pictures taken from left side of pool area

(a) View looking at north
(b) View looking at northwest
(c) View looking at northwest
(d) View looking at west

Figure C.3: Pictures taken from right side of pool area

C.2 | Water level measurements

As described in paragraph 6.2.3, the water level was measured twice in the same day during spring tide. The measurements and predicted water level values are presented in table C.1. The tidal range that result from these values are 1.18 m for the predicted water levels and 1.01 m for the measured tidal levels. This results in a reduction factor from the predicted value to the observed value of 0.85 m.

Table C.1: Water level observations compared to predictions

Date and time	Tide	Predicted water level [m referenced to tide datum]	Measured water level [m referenced to rock]
29-07-22 16:30	High water spring	1.48	-1.36
29-07-22 10:00	Low water spring	0.30	-2.37

C.3 | Bathymetry measurements

Figure C.4 shows the transects that were intended to be measured. Due to dangerous conditions, transect B-B' has not been measured. This transect is very rocky and most rocks reach until just below the water surface. Observations for transect A-A' are presented in table C.2. The errors in these measurements may be ± 10 cm, due to waves and difficult conditions to read the measuring stick.

Measurements were taken during (predicted) mean sea level.

Figure C.4: Bathymetry transects to measure

Table C.2: Water depth measurement transect A-A'

Distance from A	Water depth [m]
2	1.5
4	-
6	0.8
8	1.7
10	1.8
12	1.8
14	1.8
16	1.8
18	2.0
20	1.8
22	1.5
24	1.6
26	1.6

C.4 | GPS anomaly

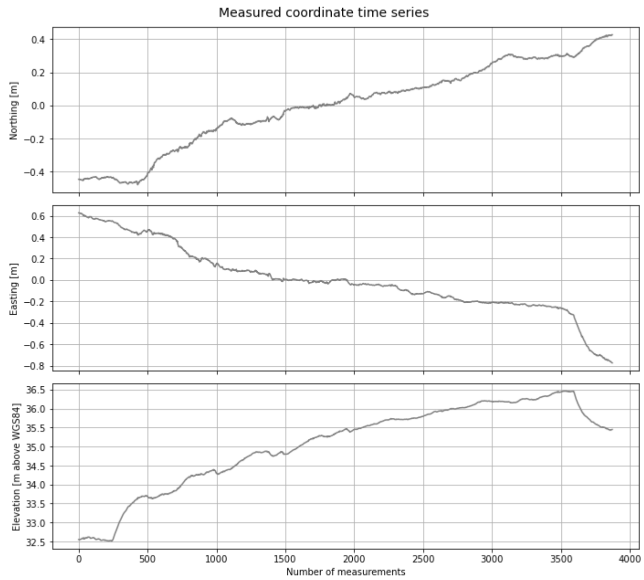


Figure C.5: Drift found in GPS in x, y and z.

In the figure above, we see the x, y and z coordinates as measured over time by the GPS. During this time, the receiver was not used. This seems to imply that something is wrong when the receiver is placed on "tracking" mode. A wider analysis of the suspicious data was made and given to the relevant technician from Kelp Blue (who own the receiver).

D | Appendix: WSP wave and wind roses

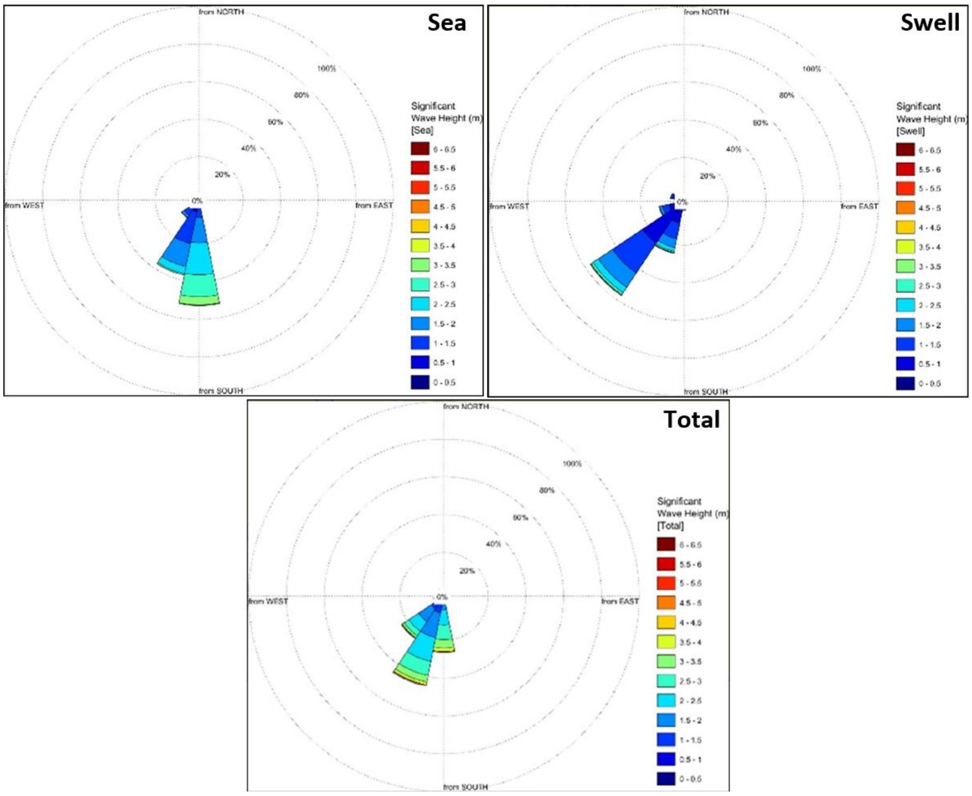


Figure D.1: Sea, Swell and Total wave roses for Site A (Smith, 2021)

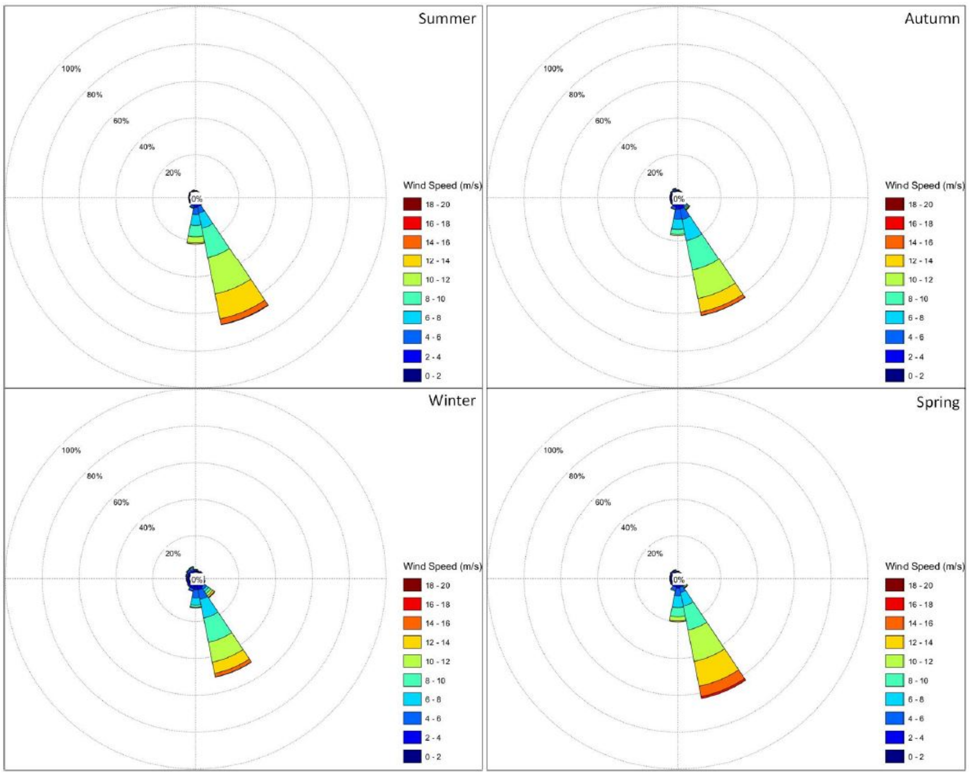


Figure D.2: Seasonal wind roses – offshore Lüderitz (OceanWeather Inc grid point) (Smith, 2021)

E | Appendix: Wave transformation

In this appendix, the methodology for the derivation of the local wave height is explained.

E.1 | Wave data

Since no data sets (offshore or nearshore) were available, wave information was retrieved from (Smith, 2021), a wave analysis report by WSP for several offshore locations for Kelp Blue's kelp farms. Figure E.1 shows the locations of these sites.

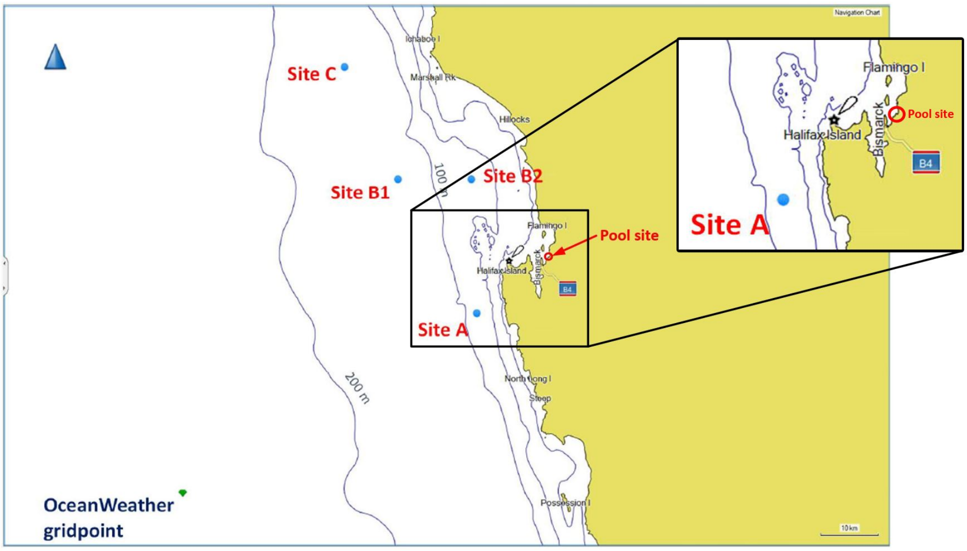


Figure E.1: Overview offshore locations wave analysis report

E.2 | Refraction and shoaling

According to the wave analysis, the dominant wave direction is south to south-west. Swell is predominantly from the south-west, while sea waves are more from the south. Since swell causes more significant waves at the pool location, the south-west wave direction is used for the wave transformation.
Waves that reach the pool site can be traced back to the rough location of Site A. Satellite imagery of Google Earth can help indicate the path of the waves reaching the shore at the pool site. Figure E.2 shows the wave path of a south westerly (swell) wave. The wave path can be split into two parts. The green track start at Site A and ends between Penguin Island (north) and Shark Island (south). The wave transformation for the green track is calculated by hand with the formula for wave refraction and shoaling (see E.1.

$$\left(\frac{H}{H_0}\right)^2 = \frac{\cos\alpha_0}{\cos\alpha}\frac{c_{g0}}{c_g} \tag{E.1}$$

Where H is the wave height, α is the wave angle and c_g is the wave celerity. The subscript 0 denotes the deep-water wave (in this case the wave at Site A). This equation is applicable for situations with parallel depth contours, which is not the case here. However, creating a 2D wave transformation model was not possible in the project time, therefore the inaccuracy is accepted. The depth at Site A is provided by the WSP report and is 73 m. The water depth at the end of the green track is retrieved from (Navionics, 2022) and is 10 m. Based on the water depth and the wave period (as provided by the WSP report, the wave group velocity can be determined, using the linear wave theory.

$$L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi h}{L}\right) \tag{E.2}$$

$$k = \frac{2\pi}{L} \tag{E.3}$$

$$c = \frac{L}{T} \tag{E.4}$$

$$n = \frac{c_g}{c} = 0.5\left(1 + \frac{2kh}{\sinh 2kh}\right) \tag{E.5}$$

To quantify the wave height at point B, the wave angle should be know as well. The wave angle is determined from the Google Earth image (see figure E.2) and the depth contour lines of the bathymetry (see figure E.3). Since the depth contour lines aren't parallel over the entire area, the contour line at point B is used to relate the wave angle to. For the transformed wave the wave angle is approximately $\alpha = 0°$. At Site A, the wave angle is approximately $\alpha = 66°$.

The resulting wave characteristics are given in table E.1.

Table E.1: Wave characteristics at Site A and Point B

Return period		**Site A (h = 73 m)**			**Point B (h = 10 m)**		
	T_p [s]	H_0 [m]	L_0 [m]	c_{g0} [m/s]	H [m]	L [m]	c_g [m/s]
50% exceedance	10.0	2.2	155.3	8.0	1.40	92.4	8.1
10% exceedance	10.0	3.2	155.3	8.0	2.04	92.4	8.1
1 year	12.6	4.7	237.6	11.0	3.37	119.5	8.7
100 year	16.0	6.0	346.7	14.9	4.89	154.3	9.2

Note that according to rules of thumb for waves in shallow and deep water (see equations E.6) waves at Site A and Point B are classified to be in transitional depths. The majority of the bathymetry leading to point B is deeper than 10 m. Therefore bed friction leads to insignificant wave height loss.

$$\begin{aligned} \text{Shallow water:} h/L < 1/20 \\ \text{Deep water:} h/L > 0.5 \end{aligned} \tag{E.6}$$

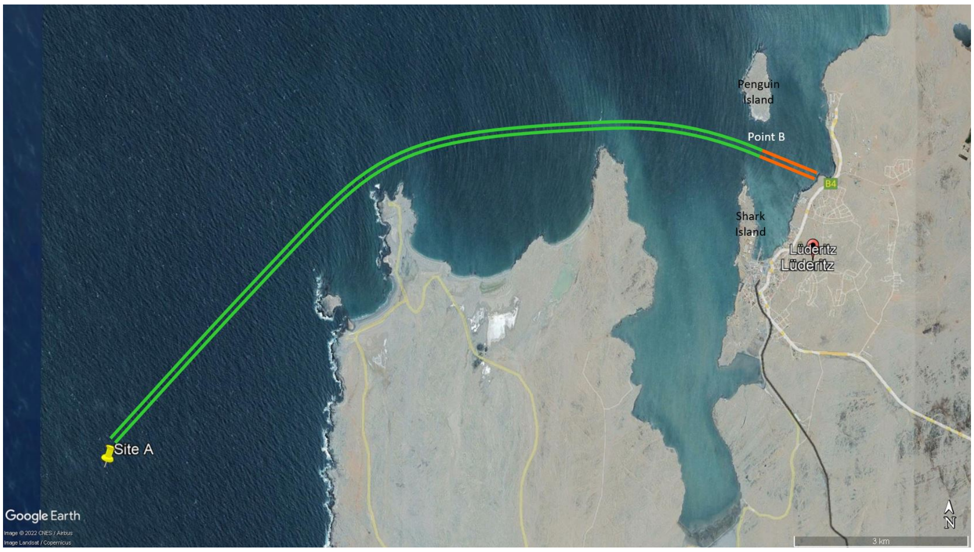


Figure E.2: Wave path from Site A to pool site

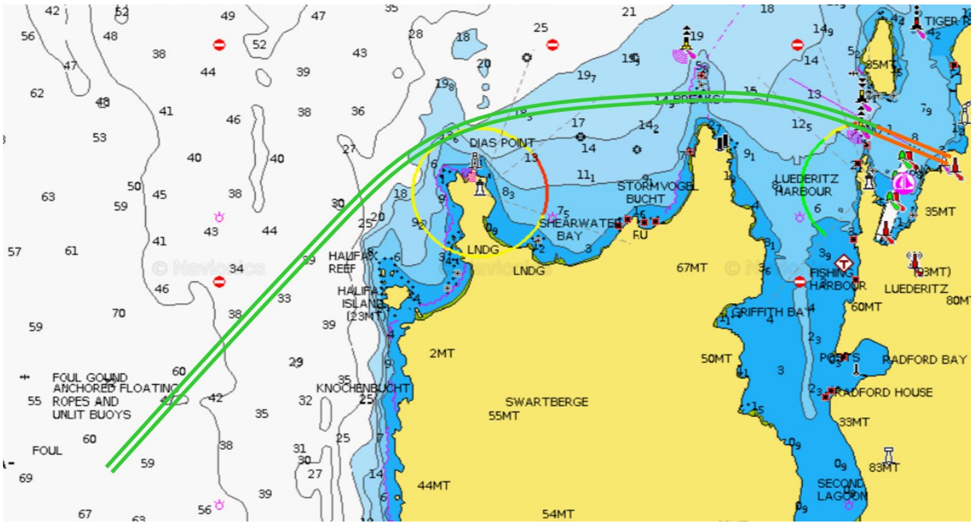


Figure E.3: Wave path and bathymetry

E.3 | Diffraction

Shark Island and Penguin Island form barriers that cause wave diffraction, the gap between the islands is approximately 940 m. This formation is considered a semi-infinite barrier since the ratio between the width of the gap and the wave length (b/L) exceeds 5. From the diagram in figure E.4 and the wave direction observed from Google Earth, a diffraction coefficient of 0.7 is found. It is advised by (Goda, 2000) that the diffraction diagram is not used for design, since directional wave spectra aren't considered. However, since the swell wave climate has a small range of directions and the bathymetry causes significant refraction, a unidirectional wave climate at the pool site is assumed.

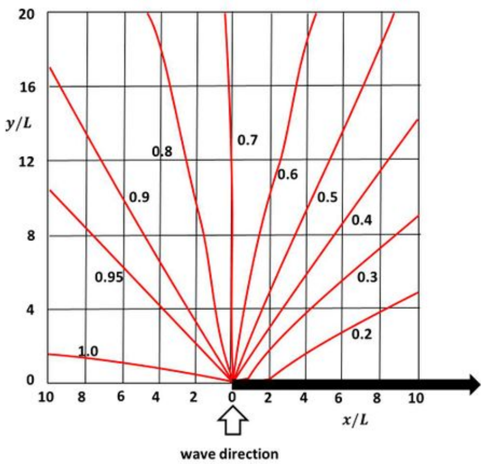


Figure E.4: Diffraction diagram for a semi-infinite barrier, adapted from (Goda, 2000)

E.4 | SwanOne model

Next, the wave transformation for the final section (the orange track in figure E.2) a SwanOne model is used. The bathymetry is derived from Navionics and local observations, this results in the bottom profile as given in figure E.5.

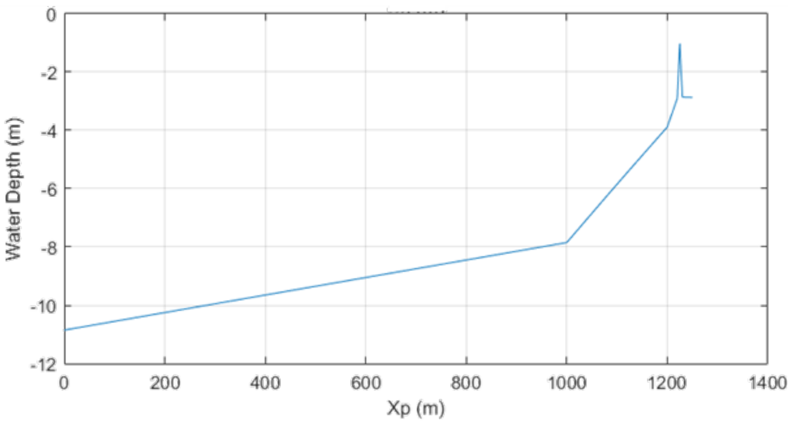
Other orientation of the bottom profile, wind and waves are the following:

- Direction of normal to the coast $\alpha = 120°$
- Wind direction = 160°(wind velocity: 10 m/s)
- Wave direction $\phi = 283°$

For the wave boundary conditions in SwanOne the deep water significant wave height. However, since the wave propagating past point B is much smaller than the deep water wave because of refraction and diffraction, the equivalent deep water height (EDWH) wave is used. The equivalent deep water wave has the same period as the deep water wave, but is adjusted by refraction and diffraction coëfficients, see equation **??**. (Goda, 2000)

$$EDWH = H_{m0}K_rK_d$$
$$K_r = \sqrt{\frac{\cos\alpha_0}{\cos\alpha}} = 0.71 \qquad (E.7)$$
$$K_d = 0.7$$

The water depth is relative to MSL. Waves are computed for water levels MHWS, MHWN and MHWS + SLR.



Distance from Point B [m]	h [m]
0	-10
1000	-7
1200	-3
1220	-2
1225	0
1230	-1.8
1250	-1.8

Figure E.5: Bottom profile input for SwanOne

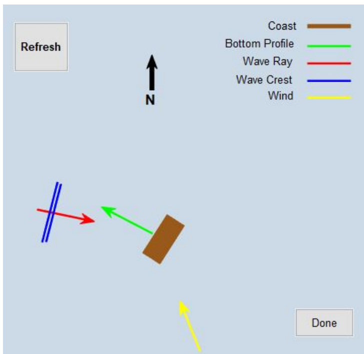


Figure E.6: Orientation of coast, wind and waves for SwanOne

Table E.2: Input and output of SwanOne model

Description	Input wave characteristics			SwanOne output		
	d [m]	T_p [s]	$EDWH$ [m]	H_{m0} [m]	T_p [s]	T_{m-1} [s]
MHWS + 50% exceedance wave	0.6	10.0	1.09	1.03	10.0	5.2
MHWS + 10% exceedance wave	0.6	10.0	1.59	1.19		5.2
MHWS + 1 year wave	0.6	12.6	2.34	1.17		6.1
MHWS + 100 year wave	0.6	16.0	2.98	1.17	15.5	7.3
MHWN + 50% exceedance wave	0.24	10.0	1.09	0.76	10.0	4.5
MHWN + 10% exceedance wave	0.24	10.0	1.59	0.85		4.3
MHWN + 1 year wave	0.24	12.6	2.34	0.88		5.7
MHWN + 100 year wave	0.24	16.0	2.98	0.84	8.0	6.3
SLR + MHWS + 100 year wave	0.85	16.0	2.98	1.44	15.5	

E.5 | Wave breaking

Just in front of the tidal pool (wall) there is a natural breakwater that that can cause significant wave height reduction. The height of the breakwater is assumed to be at MSL in the SwanOne model, although it is higher than that according to visual observations (see appendix C).

If the water level is lower than MHWN, the waves will have lost any significant wave height according to SwanOne. However since the bathymetry is not very accurate, the waves are assumed to have the possible wave height for the depth just in front of the pool wall, which is determined by the relation $H/h = 0.78$. For example, during MLWS the water depth is 1.8 - 0.6 m = 1.2 m, which leads to a maximum wave height of 0.94 m.

F | Appendix: Design calculations pool wall

In this appendix, the calculations for the design of the pool wall are elaborated. The design consists of the determination of the height, the cross-section and any details of the design.

F.1 | Overtopping

In front of a vertical wall, two types of wave conditions are distinguished: impulsive and non-impulsive. In the first case, the waves are, in comparison to the local water depth, relatively small, or remain unaffected by the toe of the structure. These wave conditions will result in a gradually varying overtopping load on the structure. In the latter case, the waves are relatively large to the local water depth causing the waves to break violently against the wall. Since the water retaining wall has a submerged toe $2.4 = h_s > 0$, the wave breaking or "impulsiveness" parameter, h_*, is dependent on the water depth at the toe of the wall, h_s, and incident wave conditions inshore. The calculation for the overtopping discharge consists of three steps.

First, the wave conditions need to be determined by using the impulsiveness parameter:

$$h_* = 1.35 \frac{2\pi h_s^2}{g H_{m0} T_{m-1,0}^2} \tag{F.1}$$

For which: $h_* > 0.3$ indicates non-impulsive wave conditions, and $h_* < 0.2$ indicates impulsive wave conditions in front of the wall. The overtopping regime within the transition zone $0.2 < h_* < 0.3$ is characterized by suddenness and a high-speed, near vertical up-rushing jet in which the wave has only captured a small amount of air, in contrast to impulsive wave-conditions, as it has not yet broken (Van der Meer et al., 2018). Due to the great uncertainty in the prediction of the dominant overtopping behaviour, the "worst-case" (impulsive breaking conditions) is taken (Bruce et al., 2009).

Second, the deterministic design crest freeboard for a plain vertical wall is determined for either impulsive or non-impulsive wave conditions. The equations for a deterministic design, provided by the Overtopping Manual 'Die Küste' (Pullen et al., 2007), incorporate a factor of safety of one standard deviation above the mean prediction.

For impulsive wave conditions:

$$\frac{q}{h_*^2 \sqrt{g h_s^3}} = 3.8 * 10^{-4} (h_* \frac{R_c}{H_{m0}})^{-2.7} \tag{F.2}$$

For non-impulsive wave conditions:

$$\frac{q}{\sqrt{g H_{m0}^3}} = 0.04 \exp{(-1.8 \frac{R_c}{H_{m0}})} \tag{F.3}$$

For near breaking wave conditions:

$$\frac{q}{h_*^2 \sqrt{g h_s^3}} = 3.8 * 10^{-4} (h_* \frac{R_c}{H_{m0}})^{-2.7} \quad \text{valid for} \quad \frac{h_* R_c}{H_{m0}} < 0.02 \tag{F.4}$$

$$\frac{q}{h_*^2 \sqrt{g h_s^3}} = 2.8 * 10^{-4} (h_* \frac{R_c}{H_{m0}})^{-3.1} \quad \text{valid for} \quad \frac{h_* R_c}{H_{m0}} > 0.02 \tag{F.5}$$

For the input parameters 3 different wave conditions have been considered, namely: 50% exceedance, 10% exceedance, and 1 year return period. These conditions have been retrieved from the offshore report for the Kelp Blue project (Smith, 2021) for Site A, SW from Halifax Island, after applying wave transformations as described in appendix E.

The design-overtopping discharges are based on findings of the reference project 'South Curl Curl' in Australia. According to the report on the Ballina Ocean Pool in Australia (Bosman & Scholtz, 1982), it is found that the ocean pool South Curl Curl has a favorable overtopping discharge as the pool remained clean (flushed sufficiently) and safe in the long-term. Since South Curl Curl experiences a mixed semidiurnal tide, the allowable overtopping discharge during high water spring (HWS) at Aeroplane Bay, which experiences a semidiurnal tide, is approximated as half the overtopping discharge at South Curl Curl. From the implementation of the allowable overtopping discharges at Aeroplane Bay into equations F.2 - F.5 it follows that the crest freeboard of the water retaining wall equals 0.06 [m].

Wave conditions	South Curl Curl [l/m/s]	Aeroplane Bay [l/m/s]
50 % exceedance	10-50	30
10 % exceedance	40-120	80
1 year return period	100-250	175

Table F.1: Allowable overtopping discharges at South Curl Curl (Australia), and Aeroplane Bay (Namibia)

Third, a multiplication factor is applied on the calculated discharge for a plain vertical wall to convert it to a valid discharge for a battered wall. For a 10:1 battered wall the overtopping discharge is determined by multiplying the overtopping discharge of a vertical wall with a multiplication factor of 1.3 (Van der Meer et al., 2018).

$$q_{battered} = 1.3 * q_{vertical} \tag{F.6}$$

Table F.2 provides an overview of the overtopping discharges at Aeroplane for $R_c = 0.06$ [m]. The results show that for a crest freeboard of 0.06 m, the water inflow during high water spring (HWS) is sufficient for both the 50% and the 10% exceedance wave condition. Also, it should be noted that with this value for the crest freeboard, there will even be refreshment during high water neap (HWN).

Wave conditions	WL [m]	H_{m0}[m]	T_{m-1}[s]	Q_{design}[l/m/s]
HWS 50 % exceedance	2.4	1.03	5.2	41.38
HWS 10 % exceedance	2.4	1.19	5.2	67.62
HWS 1 year return period	2.4	1.17	6.10	79.81
HWS 100 year return period	2.4	1.17	7.3	67.69
HWN 50 % exceedance	2.04	0.76	4.5	26.69
HWN 10 % exceedance	2.04	0.85	4.3	10.71
HWN 1 year return period	2.04	0.88	5.7	17.88

Table F.2: Overtopping discharges at Aeroplane Bay (Namibia), for different wave conditions

Thus with a crest freeboard of 0.06 [m], the tidal pool will have an overtopping discharge that ensures both sufficient water refreshment during HWS and HWN.

F.2 | Pool wall stability

F.2.1 | Goda method for wave pressure on vertical breakwaters

The wave pressures on the wall are calculated using method of Goda for design of vertical breakwaters (Goda, 2000). This method is slightly adapted to the situation of this design. Goda's method is originally meant for caisson breakwaters, whereas the pool wall is a gravity wall. For the determination of the design wave height, a safety factor of 1.5 is applied, instead of the equations Goda provides. The reason for this is that bathymetry of the site is unusual, since there is a naturally breakwater just in front of the pool wall. Calculations for the design wave height are related to the slope of the bed in front of the structure, which is not constant (and based on very rough estimates). On top of that, wave heights determined as explained in appendix E also have large uncertainty.

Following equations are used to calculated the wave pressure excerted on the wall.

$$\begin{cases} p_1 = 0.5(1+\cos\beta)(\alpha_1+\alpha+\cos^2\beta)\rho_w g H_{max} \\ p_3 = \alpha_3 p_1 \\ p_4 = \begin{cases} p_1(1-h_c/\eta^*) & \text{if } \eta^* > h_c \\ 0 & \text{if } \eta^* \leq h_c \end{cases} \\ p_u = 0.5(1+\cos\beta)\alpha_1\alpha_3\rho_w g H_{max} \end{cases} \tag{F.7}$$

With:

$$\begin{cases} \alpha_1 = 0.6 + 0.5\left(\frac{2kh}{\sinh 2kh}\right)^2 \\ \alpha_2 = \min\left\{\frac{h_b - d}{3h_b}\left(\frac{H_{max}}{d}\right)^2, \frac{2d}{H_{max}}\right\} \\ \alpha_3 = 1 - \frac{h'}{h}(1 - 1/\cosh kh) \end{cases} \tag{F.8}$$

And:

$$\eta^* = 0.75(1 + \cos\beta)H_{max} \tag{F.9}$$

In which:

- h: design water depth [m].
- d: water depth on top of berm [m] $= h$.
- h': submerged height of caisson [m].
- h_c: free board height [m] $=$ 0.
- H_{max}: design wave height [m].
- β: angle of incidence in degrees. It is assumed that $\beta = 0°$, to design for the largest impact on the wall.
- h_b: water depth 5 times the wave height seaward from the structure, here it is assumed that that depth is equal to h. $h_b = h = d$.

With these calculations only the wave pressure is defined, since for normal breakwaters the water level is equal on both sides, cancelling out the hydro-static pressure. In this case however, that is not the case. Several design conditions are considered, in which the pool side is either completely filled or completely empty. The hydrostatic pressure is calculated as follows in equation F.10, in which $\rho_w =$ 1030 kg/m^3.

$$p_{static} = \rho_w g h \tag{F.10}$$

Next the forces can be calculated

$$F_{H,sea} = \frac{1}{2}(p_1 + p_3)h' + \frac{1}{2}(p_1 + p_4)h' + \frac{1}{2}p_{static}h' \tag{F.11}$$

$$F_{H,pool} = \frac{1}{2}\rho_w g h_{pool}^2 \tag{F.12}$$

$$F_U = \frac{1}{2}p_u B \tag{F.13}$$

With h_{pool} is the water depth in the pool, $h_c^* = min(\eta^*, h_c)$ and B is the width of the bottom of the structure in m.

$$M_{H,sea} = \frac{1}{6}(2p_1 + p_3)h'^2 + \frac{1}{2}(p_1 + p_4)h'h_c^* + \frac{1}{6}(p_1 + 2p_4)h_c^{*2} + \frac{1}{6}p_{static}h'^2 \tag{F.14}$$

$$M_{H,pool} = \frac{1}{3}F_{H,pool}h_{pool} \tag{F.15}$$

$$M_U = \frac{2}{3}F_u B \tag{F.16}$$

Next, to determine the stability against sliding and overturning, the mass m and distance from the heel to the center of gravity t are needed. The mass is calculated by multiplying cross-section area with the density of the concrete (2300 kg/m^3 is assumed). t is calculated with equation F.17. In figure F.1 the dimensions of the wall are presented. With these dimensions m and t are calculated: $m =$ 7680 kg, $t =$ 1.29 m.

$$t = \frac{A_i z_i}{A_{tot}} \tag{F.17}$$

On the toe of the wall, there is a significant weight of water that provides a counter-moment for overturning and extra weight for the sliding failure mechanism. The mass and arm of the water are calculated in similar fashion as the mass and arm of the wall. Next, the safety factors for sliding and overturning can be determined with equations F.18.

$$\begin{aligned} SF_s &= \frac{\mu(mg - F_u + m_{water}g)}{F_H} \\ SF_o &= \frac{(mgt - M_u + m_{water}gt_{water})}{M_H} \end{aligned} \tag{F.18}$$

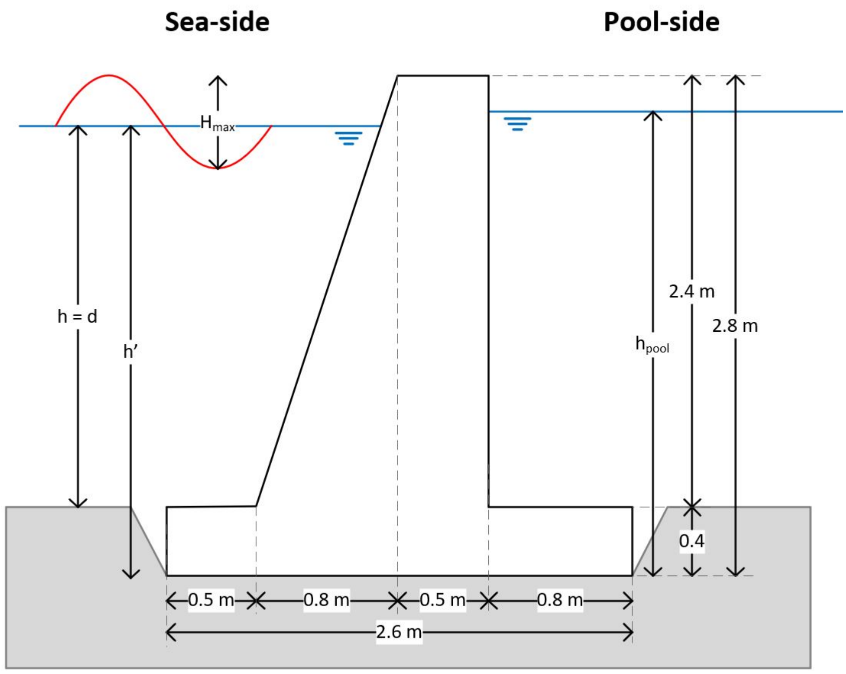


Figure F.1: Dimensions of vertical wall

F.2.2 | Pressure under a wave trough

A wave can also exert a sea-ward pressure on a vertical wall, which can cause instability and lead to sliding and overturning seaward. Goda, 2000 provides a method to calculate the pressure under a wave trough. This method is not used in the Coastal Engineering Manual, which only mentions the Sainflou Formula. However, when comparing the two methods, the Goda method resulted in larger forces.
From the ratios h/L and H/L, the non-dimensional values for the total wave pressure and lever arm can be determined using the diagrams in figure F.2. The values can be translated into a force and arm according to equation F.19.
Next the safety factors for seaward sliding en overturning van be determined. The weight of water is not taken into account, and note that the moment is calculated around the toe instead of the heel.

$$F_{wave} = P_{min,non-dimensional} w_0 H h$$
$$s_{wave} = s_{min,non-dimensional} h \tag{F.19}$$

With the specific weight of water $w_0 = 10.1\ kN/m^3$.

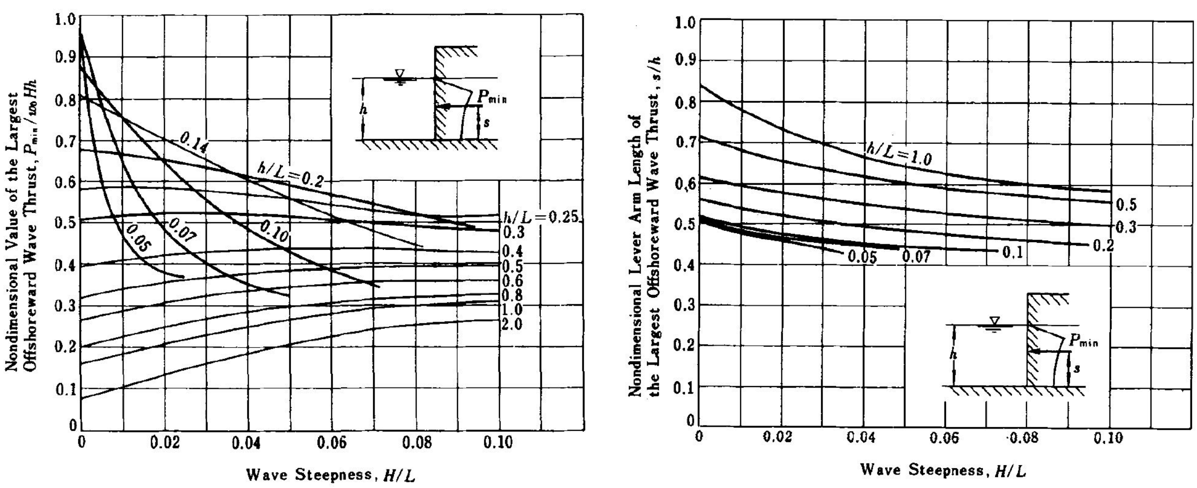


Figure F.2: Calculation diagrams for standing waves under a wave trough

F.2.3 | Design conditions

Several design conditions are considered, see table F.3. For design condition 1, the 100 year return period wave is combined with a filled pool, since this will be advised in the pool management guidelines. However, the wall should be able to resist "regular" waves when the pool is empty (design condition 2). Design conditions 1 and 2 are considered for both spring and neap tide, since with a lower water level, the wave will load a larger surface area of the wall. From table F.3 it is concluded that the spring tide situations will lead to the largest loads.
Design condition 3 describes the negative wave load (pulling it to the sea-side). Design condition 4 includes sea level rise (SLR) in combination with the highest water level and biggest wave. SLR is estimated to be 0.25 m in 25 years. This means the water level will exceed the wall height, so the water level will be the same on both sides (net no hydrostatic pressure load). Note that for the design wave, the safety factor is not taken into account.
Design condition 4 combines all offshore pressures, meaning the lowest water level, the pressure under a wave trough and a filled pool.
The input parameters are taken from table E.1 (with exception of the wave of design condition 4).

Table F.3: Design conditions

Design conditions		Input	Resulting forces	Overturning moments (around heel)	Safety factors
1	MHWS + 100 y wave filled pool	H = 1.17 m T = 15.5 s h = 2.4 m h' = 2.8 m	FH,pool = -34.2 kN FH,sea = 92.3 kN FU = 24.2 kN	MH,pool = -29.6 kNm MH,sea = 111.0 kNm MU = 41.9 kNm	SF_s = 0.7 SF_o = 1.6
	MHWN + 100 y wave filled pool	H = 0.84 m T = 8.0 s h = 2.04 m h' = 2.44 m	FH,pool = -34.2 kN FH,sea = 60.5 kN FU = 15.6 kN	MH,pool = -29.6 kNm MH,sea = 61.6 kNm MU = 27.0 kNm	SF_s = 1.7 SF_o = 4.2
2	MHWS + 50% wave empty pool	H = 1.03 m T = 10.0 s h = 2.4 m h' = 2.8 m	FH,pool = 0 FH,sea = 83.7 kN FU = 19.9 kN	MH,pool = 0 MH,sea = 99.3 kNm MU = 34.5 kNm	SF_s = 0.5 SF_o = 1.4
	MHWN + 50% wave empty pool	H = 0.76 m T = 10.0 s h = 2.04 m h' = 2.44 m	FH,pool = 0 FH,sea = 58.8 kN FU = 14.9 kN	MH,pool = 0 MH,sea = 59.5 kNm MU = 25.9 kNm	SF_s = 0.8 SF_o = 2.3
3	Negative wave load: MLWS + possible wave* filled pool	H = 0.94 m T = 5.0 s h = 1.2 m h' = 1.6 m	FH,pool = -34.2 kN FH,sea = 6.5 kN	MH,pool = 29.6 kNm MH,sea = -3.4 kNm (overturning around toe)	SF_s = 1.4 SF_o = 3.8
4	SLR + MHWS + 100 y wave	H = 1.44 m** T = 15.5 s h = 2.65 m h' = 2.8 m	FH,wave = 43.1 kN FU = 19.8 kN	MH,wave = 60.6 kNm MU = 34.3 kNm	SF_s = 1.1 SF_o = 2.4

*possible wave is based on breaking relation H/h = 0.78, the safety factor is also applied to this wave
**no safety factor on wave

F.2.4 | Impulsive wave breaking

Goda, 2000 advises that impulsive wave breaking is avoided for vertical breakwaters. To assess this, the method of Tanimoto and Takahashi is used.

A coefficient α_I is calculated, there is no risk for wave breaking if $\alpha_I < \alpha_2$. From the calculation before, it follows that α_2 = 0. From equations F.20 follows that α_I = -0.002. Therefore it can be concluded that there is no risk for impulsive wave breaking.

$$\alpha_I = \alpha_{IH}\alpha_{IB} = 0.22 < 0.30$$
$$\alpha_{IH} = \min(H/d, 2.0) = 1.40$$
$$\alpha_{IB} = \begin{cases} \cos\delta_2/\cosh\delta_1 & \text{if } \delta_2 \leq 0 \\ 1/(\cosh\delta_1 \cosh^{1/2}\delta_2 & \text{if } \delta_2 > 0 \end{cases} = 0.15$$
$$\delta_1 = \begin{cases} 20\delta_{11} & \text{if } \delta_1 1 \leq 0 \\ 15\delta_{11} & \text{if } \delta_1 1 > 0 \end{cases} = -2.35 \quad \text{(F.20)}$$
$$\delta_2 = \begin{cases} 4.9\delta_{22} & \text{if } \delta_2 2 \leq 0 \\ 3.0\delta_{22} & \text{if } \delta_2 2 > 0 \end{cases} = -0.62$$
$$\delta_{11} = 0.93(B_M/L - 0.12) + 0.36(0.4 - d/h) = -0.12$$
$$\delta_{22} = -0.36(B_M/L - 0.12) + 0.93(0.4 - d/h) = -0.13$$

With B_M the berm length, which is 0 m in this case.

F.3 | Inlets

F.3.1 | Fully contracted weir limitations

A fully contracted weir may be described by the limitations as shown in table F.4.

Design limitations
$B_1 >= 4h_1$
$h_1/p_1 <= 0.5$
$h_1/b_c <= 0.5$
$0.07[m] <= h_1 < 0.60[m]$
$b_c >= 0.30[m]$
$p_1 >= 0.30[m]$

Table F.4: limitations of a rectangular sharp-crested fully contracted weir

F.3.2 | Determination of the filling time

The filling time of the tidal pool can be determined by using two basic equations: head discharge over a sharp crested weir (F.21) and the volume-discharge relation (F.22).

$$Q = C_e \frac{2}{3}\sqrt{2g} b_c h_1^{1.5} \quad \text{(F.21)}$$

$$t = V_{pool}/Q \quad \text{(F.22)}$$

Equation F.21 is dependant on the effective discharge coefficient C_e. As shown in F.5, C_e is a function of the ratios b_c/B_1 and h_1/p_1 with b_c (inlet width), B_1 [7](length of the water retaining wall / number of inlets, and h_1 (water level - height of the inlet).

b_c/B_1	C_e	b_c/B_1	C_e
1.0	$0.0602 + 0.075h_1/p_1$	0.5	$0.592 + 0.011h_1/p_1$
0.9	$0.599 + 0.064h_1/p_1$	0.4	$0.591 + 0.0058h_1/p_1$
0.8	$0.597 + 0.045h_1/p_1$	0.3	$0.590 + 0.0020h_1/p_1$
0.7	$0.595 + 0.030h_1/p_1$	0.2	$0.589 - 0.0018h_1/p_1$
0.6	$0.593 + 0.018h_1/p_1$	0.1	$0.588 - 0.0021h_1/p_1$
		0	$0.587 - 0.0023h_1/p_1$

Table F.5: Values for C_e as a function of the ratios b_c and h_1/p_1 (from Georgia Institute of Technology)

[7]Rectangular weirs should be located in a rectangular approach channel. However, since the approach channel of the fully contracted weir is sufficiently large ($B_1(h_1 + p_1) >= 10b_c h_1$), the velocity of approach may be considered negligible and the shape of the approach channel unimportant (LMNO Engineering, Research, and Software, 2014). Hence, for the calculation of the filling time, the width of the approach channel is assumed rectangular.

Table F.6 provides an overview of the filling times related to different values of the inlet width (b_c) and the amount of inlets (n). The values are based on a tidal pool volume of 1800 $[m^2]$, an average water level of 2.3 $[m]$, an inlet height of 2.2 $[m]$, and a value of C_e as presented table F.5.

Inlet width	n=1	n=2	n=3	n=4	n=5	n=6	n=7	n=8	n=9	n=10
0.8	684	342	228	171	137	114	98	85	76	68
0.9	608	304	202	152	121	101	87	76	67	61
1.0	547	274	182	137	109	91	78	68	61	55
1.1	498	248	166	124	99	83	71	62	55	49
1.2	456	228	152	114	91	76	65	57	50	45
1.3	421	210	140	105	84	70	60	52	47	42

Table F.6: Inlet design filling times [minutes]

F.3.3 | Available filling-time

To assess which of the combinations of inlet widths and amount of inlets (shown in table F.6) applies to the project, the available filling-time (time at which the water level is higher than 2.2 [m] during spring high water conditions.) has been determined. Figure F.3 shows the available filling time per day, based on the tidal predictions of tide-forecast.com (tide-forecast.com, 2022). An available filling time of 100 minutes has been chosen to guarantee sufficient refilling, on a 2-week frequency, of the tidal swimming pool.

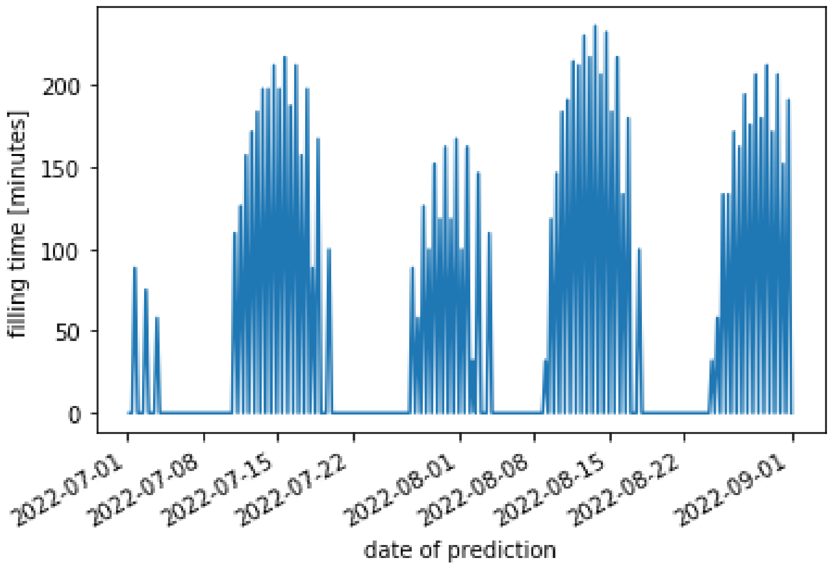


Figure F.3: Available filling time [minutes]