A predictive view on Durbans flood safety

The Golden Mile protected in style

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by

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An electronic version of this multidisciplinairy project report is available at
http://repository.tudelft.nl/.

TU Delft

Preface

This report is written in order to fulfill the requirements of a Master of Science degree in Civil Engineering at the Delft University of Technology, at the faculty of Civil Engineering and Geosciences (CEG). As a part of our Masters, we are able to do a multidisciplinary project, worth 10 ECTS per person. The multidisciplinary part is covered in the fact that we represent different tracks within the Masters in Civil Engineering. The Master tracks Structural Engineering (SE), Building Engineering (BE) and Hydraulic Engineering (HE) are covered.

In cooperation with the eThekwini municipality and the University of KwaZulu-Natal in Durban, South Africa, we found an assignment in the form of a coastal engineering issue. We would like to thank both the municipality and the university in providing us with working spaces and available knowledge. We would like to make a special thanks to JP Calitz, Natasha Ramdass and Nishal Misthry from the municipality and to Derek Stretch and Justin Pringle from the UKZN for their help and supervision in Durban. Furthermore, we would like to thank IV-Groep, who helped us realising this project and trip in South-Africa in a financial way.

Finally, we would like to thank G.P. van Vledder for being our main supervisor and guiding us in the right directions. This also holds for the other supervisors, C.R. Braam, J.A. Hopkins, S. Pasterkamp. In addition we want to thank A.J.H.M. Reniers for answering our questions.

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Abstract

Durban is the third largest city of South-Africa, located in the province of KwaZulu-Natal. The city suffers from severe floods from time to time, finding its cause in both the Indian ocean as well as the Umgeni river. The eThekwini municipality wishes a better insight in the occurrence of these floods and searches for a structural solution to protect the coastline. The eThekwini municipality has models in operation to predict the hydraulic characteristics in the ocean and the river. However, the existing models don't represent the reality sufficiently, since the interaction between the Indian ocean and the Umgeni river is not modelled properly yet.

An analysis on the area of interest has been executed. The conclusion was drawn that the Umgeni river delta was (partly) tide-dominant, meaning that the Indian ocean imposes the downstream water level. Furthermore, the wave climate was observed, as well as a look into present coastal protections.

The link between the Indian ocean and the Umgeni river has been modelled using Delft3D. Since the Indian ocean imposes a downstream boundary condition (in terms of a water level) for the Umgeni river, a backwater curve might occur. First, the link is made by extending the Delft3D-model which was present for the Indian ocean only. The model has been extended all the way up to the Inanda dam. The part of the river included in the new model is approximately 32 km long. When comparing the models output at the river mouth, at the same location as a measurement point, similar behaviour can be observed. The same phase (lag) is observed, contrary to the tidal range. The tidal range in the model differs from reality, but this is due to a lack of calibration in the amplitudes of the different tidal constituents taken into account. Hence, the renewed model seems to work, but more validation still has to be done. This was not possible yet, as there is a lack of measurement stations along the river.

Next to an extension of the Delft3D model, a script has been written in Python. This script is based on the empirical fit of Bresse and shows an elegant function. The results from the function in Python and the model in Delft3D are similar in a qualitative and a quantitative way. Both the models show an influence of the Indian Ocean, reaching easily to about 12 km upstream of the river mouth. This can be explained by the mild bed slope in this part.

A structural solution for the flooding on the promenade at the height of North Beach was found in the form of a seawall. The most important design demand is to protect against a high water level of a 200 year return period combined with a 50 year return period wave height. These storm conditions are input for the ocean-river model, which delivers wave characteristics at the beach front, linking the structural design to the ocean-river model. After a pre-selection on design options, a Multi Criteria Analysis is carried out for the remaining eight design options. Grading is done based on criteria, representing the viewpoints of the many stakeholders involved and leading to a highest grading of a seawall in combination with an emergency barrier.

Following, the water-retaining height for a vertical wall is determined. Given the the ground level height of the promenade to be $MSL + 2.2\ m$ and a total water-retaining height of $MSL + 2.944\ m$ this leads a practical construction height of $0.80\ m$. Due to the limited height a reinforced concrete seawall is designed with emergency barriers for the beach entrances. The emergency barriers are designed of pinewood. Additionally, in order the fit properly in the surroundings, an integrated design is added with features like benches, thatch umbrellas and plants to disguise the construction and protect the Golden Mile in style.

Contents

1
Introduction

Durban is a coastal city in the eThekwini municipality, in the KwaZulu-Natal province, on the east coast of South Africa. The harbour of Durban is very important to the Southern African region and is the largest in South Africa. Next to that, the King Shaka International airport is a central hub in the Dube Trade Port, a long-term planning initiative. A lot of employment opportunities are generated by the airport and the harbour.

One of the events which accelerated the growth of Durban is the FIFA World Cup in 2010. The FIFA World Cup was organized in South Africa and some of the matches were played in the Moses Mabhida Stadium, in Durban. To prepare the city for the World Cup, the promenade of the Golden Mile, as the Durban beachfront is called, has been rebuilt. This in combination with the subtropic climate, makes Durban in summertime very popular among tourists. To attract even more tourists, both national and international, Durban is improving its infrastructure. [38]

However, the future of the Golden Mile is in danger since parts of the promenade flood from time to time. These floods are caused by a storm surge from the Indian Ocean or from the Umgeni river overflowing its embankments. This results in damage to buildings and infrastructure and in worse cases it has already lead to losses of life. The municipality is maintaining the beaches by intensive nourishment, collecting data on the hydraulic conditions of both river and ocean, and has already partly modelled the coastal area with the adjacent Umgeni river. However, eThekwini can't protect the boulevard from flooding yet. With the modeling being complete, including coastal influences on the river, a warning system as short term solution can be used to prevent more losses of life. The promenade is a vital part of the city beaches, it allows tourists to access the beaches, go to shops and restaurants, and with the many surf shops along the coast, it offers surfers many possibilities. Therefore the municipality is looking for solutions which do not influence the wave climate and protects the promenade from flooding, in the form of a structural element well integrated in the beachfront.

To assess the mentioned topics, the report is divided in different parts. In the next section an overview of floods which occurred in Durban is stated, followed by the formulation of the objective in section 1.2. In chapter 2 the existing situation is mapped. In chapter 3 the model, which is currently under construction, is explained, improved and validated. Chapter 4 used the output of the model, with historical data as input, to try to find a long-term solution for the most vulnerable area. In chapter 5, a conclusion is given followed by some recommendations, model and structural related, in chapter 6.

1.1. Historical floods

This section gives insight in the different big floods which Durban and surrounding areas had to deal with in the past. These are certainly not all the floods which the province of KwaZulu-Natal had to deal with, but a few selected big floods are highlighted. The usual floods happen on a regular basis, often influenced by tropical storms in spring. These floods are a small notice in this section but are of big importance for the research. Therefore, it is assessed later on in the report. The big floods in this section are in chronological order, starting with the one which happened the longest time ago.

One of the first properly recorded floods happened in 1856. It is said that 686 mm of rain has fallen during this flood, causing the river Umgeni to burst out of its banks. The reason behind this flood is obvious, too much rain to handle for the river.

In January 1984, due to the tropical cyclones Domain and Imboa, Durban and parts of the province KwaZulu-Natal flooded. On 27 January of 1984 the tropical cyclone Domain struck the African continent, causing a widespread flooding with 242 deaths and thousands of people homeless. There were places which had 906 mm of rain in five days. The next tropical cyclone, Imboa, hit the coast of Africa only three weeks later and caused heavy rainfall, over 350 mm, and strong winds [37].

Also the floods of October 1987 were caused by heavy rainfall, 800 mm during a period of 5 days. All kind of debris, even large trees, were thrown on the beach by large waves [5]. In appendix A the Umgeni river is shown in the normal form and in the flooding form. The river in flooding form is brown coloured through the amount of sediment which is picked up. This shows another aspect of the problems the floods bring with them, the river is seriously affected.

A more recent flood is the flood from March 2007, in which storm swells coincided with a Saros spring tide. This storm caused serious coastal erosion, as shown in the figure 1.1.

Figure 1.1: Coastal erosion during the flood of 2007

The years ahead of this storm were very prosperous with a lot of economic growth, resulting in a lot of building activity. This in combination with bad urban planning resulted in buildings close to and on the beach. During the flood of March 2007 these buildings were heavily damaged. The estimated return period of this flood is 45 years, which means that is likely to happen again soon. A return period describes the likelihood of an event, which is in this case an event estimated to happen once in 200 years. Thus there is need for more protection of the shores and/or better urban planning. [51]

In October 2017 the beaches of Durban, among others North beach, were flooded through either spring tide or the aftermath of a cyclone which passed by Madagascar the week before. In figure 1.2 the flooding is shown. [48]

Figure 1.2: Flood of North beach 2017

This storm stopped all activities in the city, roads were flooded, the hospitals were affected etc. The flooding was caused by heavy rain and wind. This was again a combination of flooding of the rivers and high waves which pushed the water onto the shore.

At last in January 2018 a seven-year-old girl died because she was washed away from one of the Durban piers by a huge wave. It is not sure where these big waves came from, but they were probably connected to the cyclone Berguitta, which passed by a few days before [6].

All of the above illustrates the need of a good warning system for predicting the flooding of the river and the flooding of the beaches. Through warning, people can take appropriate actions in time to make sure people and their properties are save for the floods. Next to that, if the locations which flood can be detected beforehand, these area's can be fortified to withstand the power of the sea.

1.2. Objective

The objective of this project consists of two parts, related to the modeling of the river influenced by the sea and a flood defence system to protect the coastal region from these high sea levels

- Model related: "Create a better insight in possible flood locations making use of computational models, based on weather forecasts. The potential consequences of a flood must be mitigated by providing the municipality with enough information for a flood warning system. The model should incorporate both the ocean and the Umgeni river, since both are responsible for the floods."
- Flood defense related: "Design an engineered flood defense, which integrates in the surroundings and protects the promenade of Durban against severe floods."

2
Analysis

This chapter contains an analysis on available knowledge about the Durban coastline and the Umgeni river system. The analysis forms a starting point of the project. From here, it's determined what has been done already and also what can best be done in the upcoming years, to make Durban future-proof with respect to its flood safety. The chapter starts with a description of the existing situation. This is followed by defining the system borders of the project. Next, some assumptions are made to simplify big problems. A stakeholder analysis is done and finally a look is taken at some reference projects.

2.1. Existing situation

The existing situation describes the properties of the Durban environment as they currently are. A distinction is made between physical and digital parts. The physical parts are the river system, ocean system and coastal zone. The existing model is also shortly discussed and forms the digital part, together with the available data. In figure 2.1 the map of Durban and its global position is shown. In this way, one can immediately discover the main elements which are discussed later on in the analysis.

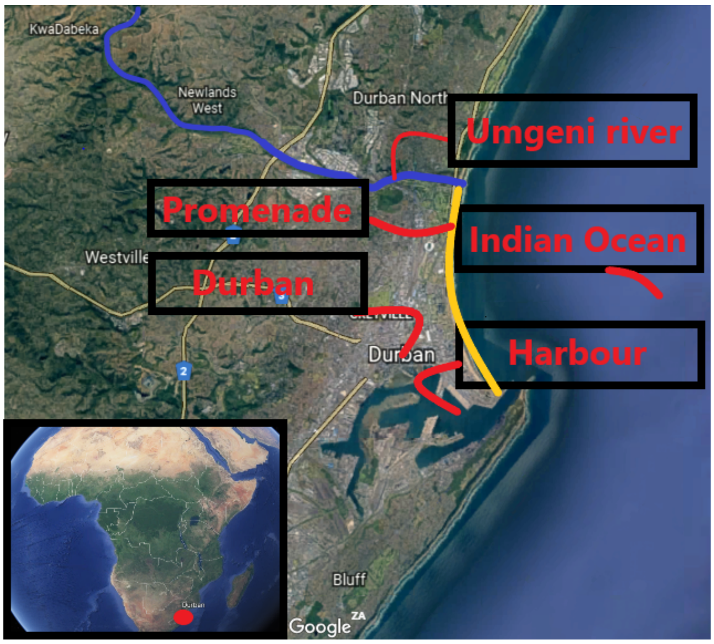


Figure 2.1: The map of Durban with its main elements

2.1.1. River System

The rivers plays an important role in the floods. Due to high peak discharges in the rivers, the water levels rise significantly, leading to floods with severe damage to the surroundings. In the scope of this project, the Umgeni river is the most important river regarding these floods. The Umgeni river is located north of Durban. Besides the Umgeni river, the Mlazi river also contributes to some severe floods. This one is located south of Durban. The two rivers together with the ocean surround Durban with water.

In figure 2.2 below, in orange, the Umgeni catchment area is indicated. This area equals 4439 km^2, which is a large drainage basin. The Umgeni river itself is 232 km long from its source, the Drakensbergen, all the way to its mouth. Some other statistics are listed in table 2.1. [2]

Table 2.1: Statistics about rivers

River (catchment)	Area [km2]	Evaporation/yr [mm]	Rainfall/yr [mm]	Discharge/yr [mln m3/s]
Mooi river	2,868	1,342	800	401.5
Umgeni river	4,439	1,214	921	674

The natural runoff can be calculated per second, since this is a more common value to calculate with later on. Mind that this value is an average throughout the year, but that there are big differences between winter- and summertime.

$$Q = \frac{Q_{year}}{T} = \frac{674 * 10^6}{365 * 24 * 3600} = 21.37\ m^3/s \tag{2.1}$$

The Umgeni river is also very important regarding the morphodynamics of the east coast of South-Africa. Besides, the Umgeni river is used catch water for irrigation and to do so, it has four significant reservoirs with dams. In figure 2.2, the four dams in the Umgeni river are circled in green. From upstream to downstream, the dams are called:

- Midmar dam
- Albert Falls dam
- Nagle dam
- Inanda dam

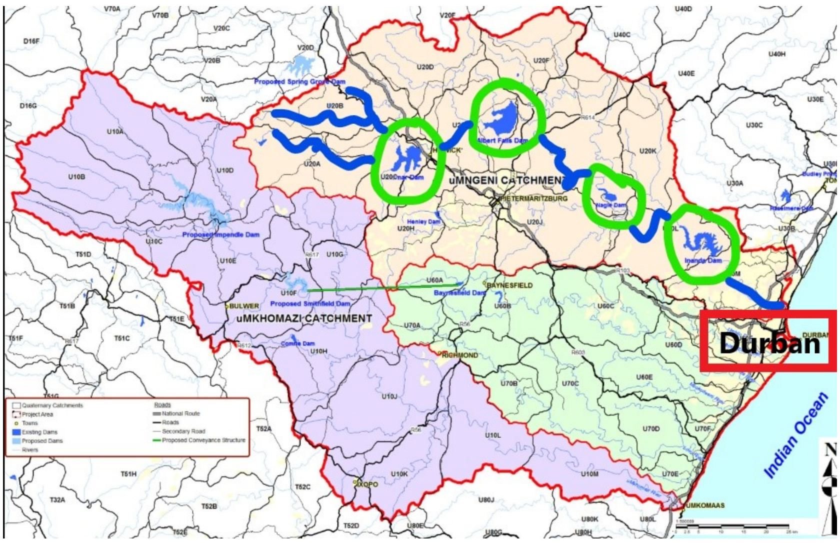


Figure 2.2: Reservoirs and dams in the Umgeni river

These dams trap a lot of sediment, resulting in a lower sediment concentration at the river mouth. This means that the coast of Durban gets less sediment input in the ocean with respect to the situation without the reservoirs. On the long term, erosion occurs north of the Umgeni river mouth because there is less sediment input compared to the situation before the dams. Hence, the shoreline retreats. This shoreline retreat is also the result of other processes, which are covered in other parts of the analysis.

The Mlazi river is responsible of a part of the 'light-green' catchment area in figure 2.2. This river is of less importance, but is now taken into account. The catchment area of the Mlazi river equals 970 km^2 and the length is 172 km. An overview can be seen in figure 2.3.

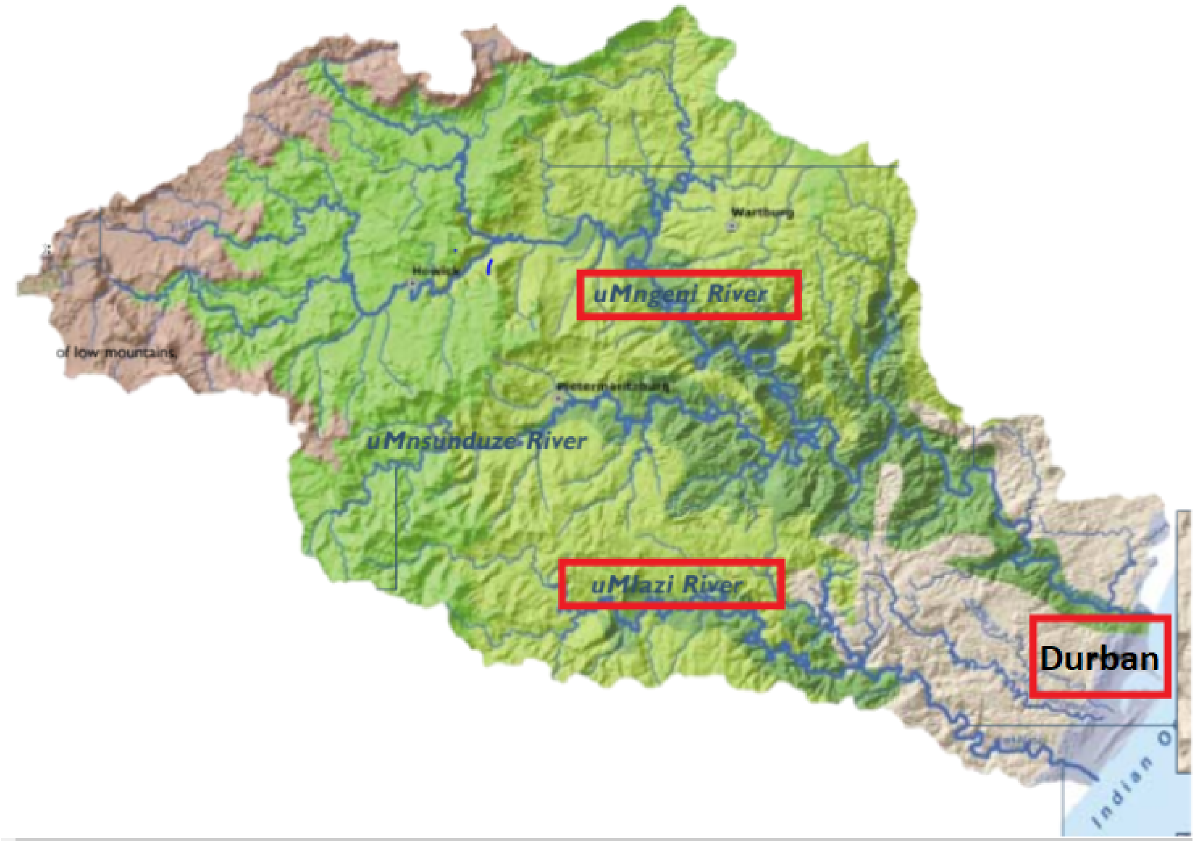


Figure 2.3: Overview of Durban being surrounded by water

The Umgeni and Mlazi river catchments have in total 129 registered dams, of which five significant ones. All of these dams were purely built and designed to create a local water reservoir. Because the dams were not designed to let a reasonable runoff pass, the rivers downstream shrinked heavily. Therefore, these rivers got a lower capacity, so floods downstream were more likely to occur when a sudden flood wave has to pass.

Another significant effect of the dams on the river system is the ecological impact downstream of the dams. Due to the shrinking of the river, the water levels lowered and the river barely flows anymore during the dryer season. As a consequence, the amount of fish decreased significantly and the water quality dropped downstream of the river system, compared to the situation without the dams.[11]

Morphodynamic delta dominance
It is important to understand how the river mouth interacts with the ocean. Later on, when the modelling of the river-ocean interactions is required, it is necessary to know what mechanisms are dominant in making assumptions regarding boundary conditions for example.

William Galloway made a classification in which he distinguished what processes are dominant in shaping the river delta [25]. These classifications are presented in a triangular diagram, shown in figure 2.4.

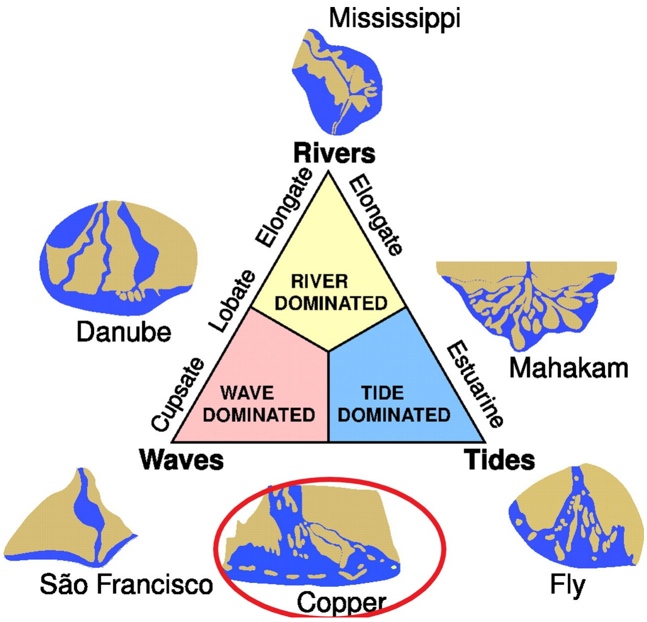


Figure 2.4: Galloway's delta classification

In the left bottom corner of the triangle, wave dominated deltas are shown. These type of river deltas have a smooth shoreline. Besides this, the river mouth is characterized by a rather symmetrically shaped cuspate and a strong alongshore current. On the right bottom corner, tide dominated deltas are shown. These deltas are characterized by high tidal ranges, leading to strong tidal currents, wider river mouths and the formation of estuaries and tidal flats. [9]

Figure 2.5: A satellite view on Umgeni's river mouth

In figure 2.5 a satellite view of the Umgeni river mouth is shown. The red line indicates the embankments of the Umgeni river, which are widening towards the river mouth. This refers to a tide-dominated delta. Moreover, some tidal flats or sand bars are visible at the river mouth. However, they are not as significant as in a 'real' tide-dominated delta. It is more shifted towards a wave-dominated delta, since the shoreline is also quite smooth and symmetric around the river mouth. Therefore, the conclusion can be drawn that the Umgeni river delta is classified between wave- and tide-dominated. The location of the Umgeni river within Galloway's diagram is shown with a red circle in figure 2.4.

For the Mlazi river, the same method can be applied in determining what dominates the river delta. When looking at a satellite view of the Mlazi river mouth, it is visible that this river delta is wave-dominant. This is because the river mouth is small and very symmetric. A very important characteristic of this delta type is that the waves can be strong enough to carry the river sediment away. This is also visible on the satellite view, because the delta is small (shrinking) and might eventually disappear. The beaches at the river mouth are very 'thin'. See figure 2.6

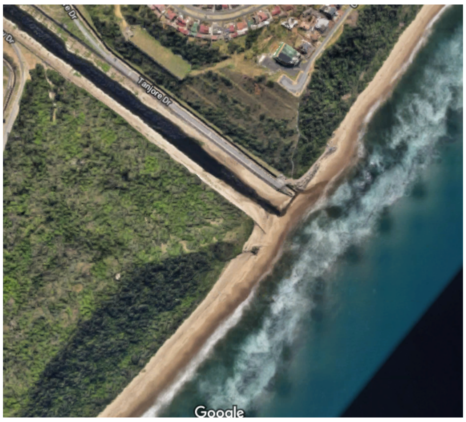


Figure 2.6: A satellite view on Mlazi's river mouth

2.1.2. Ocean system

Durban is located at the eastern coast of South-Africa, where the land meets the Indian Ocean. This ocean provides the Durban beaches with great surf waves, but also reoccurring flood waves. These different kind of waves have different origins and also different wave characteristics, like wave height, wave period and direction. Next to waves, the ocean is also subject to tide and current. These determine the main flow of water along the Durban coast. Multiple studies have already been done on these topic to create a solid overview of the ocean characteristics in front of the coast of Durban. Therefore no new analysis is done for this. The results are briefly discussed here.

Wave climate

The wave climate describes the wave characteristics of waves that occur in a certain area during a certain period. As this is heavily dependent on the time of the year and other meteorological interactions like storms, a wave climate is assumed. This is a generalized view of the waves hitting the coast of Durban. Stretch and Corbella did an attempt to create this by doing a formal analysis of a combination of available wave data from Waverider buoys near Richards Bay and Durban. In this analysis, they visualised the wave climate in front of Durban and the rest of KwaZulu-Natal. With this, a time period of 18 years is covered. [14]

The Durban wave climate is dominated by three main storm sources. First there are cold fronts and coastal lows causing low, short waves from the south. Second are cut-off low pressure systems generating swell with relatively large, long waves from the south-east. Third are tropical cyclones generating large, long waves from the north-east. This last source causes the largest waves, but happens very rarely. The second source therefore dominates the Durban wave climate under normal conditions. Concluding, the governing wave direction causes a wave-driven alongshore current from south to north in terms of sediment transport. The analysis resulted in a significant wave height of $H_s = 1.65\ m$, a peak period of $T_p = 10.0\ s$ and a wave direction of $\theta = 130°$ with respect to the northern direction. These data were measured by the Durban WaveRider at a depth of $d = 30\ m$ [14]. This also corresponds with the earlier mentioned south-eastern direction of the dominant waves. The coastal floods in March 2007 in Durban gave the largest wave heights with a maximum of 12.4 m and a significant wave height of 8.5 m. In the paper, return periods of wave heights were also obtained for design purposes. This 2007 event has a return period between 32 and 61 years. This period decreases for lower waves.

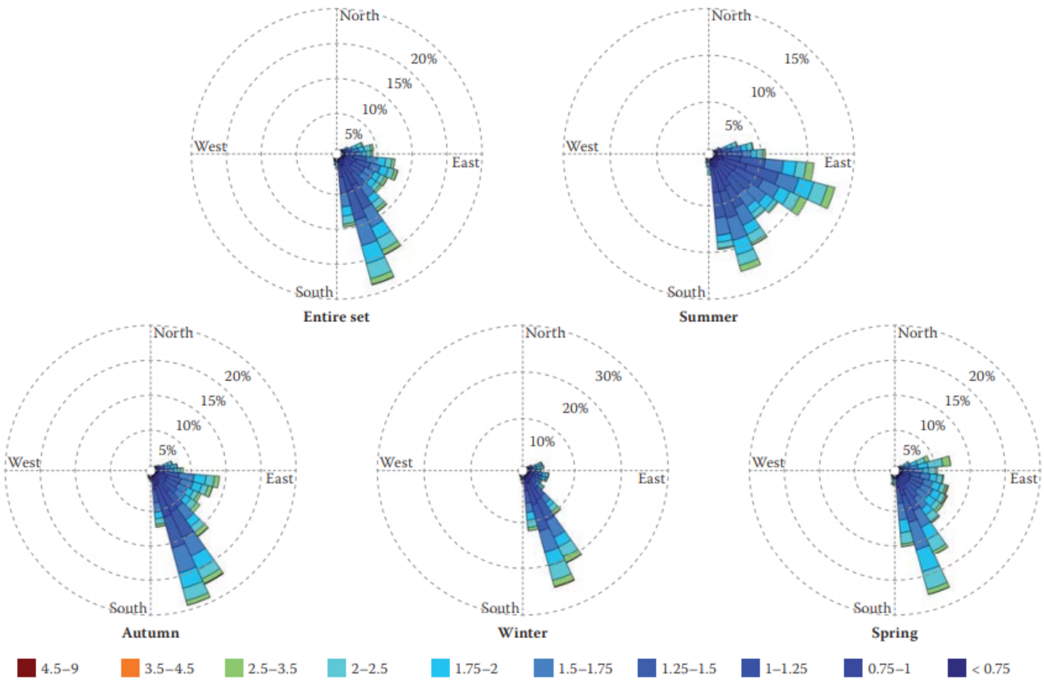


Figure 2.7: Wave roses for all seasons combined, and separately for summer, autumn, winter and spring. The significant wave height associated with the various directions are illustrated by the different colours shown in the legend [14]

Of course, the local wave climate is not constant through a single year. Seasonal changes in weather have significant influence on the waves. In their analysis, Stretch and Corbella looked at this by only taking into account wave events with a significant height larger than 3.5 m and scaling this to the total wave events. This is done per season. The results of this are clear. The percentage of events larger than 3.5 m is largest for autumn (30%), closely followed by spring and winter (28.3%). For summer, this clearly is lower (13.2%). Even though the percentages for autumn and spring and winter are close, the significant wave heights aren't. When only taking into account waves with $H_{wave} > 3.5\,m$, the significant wave height for autumn becomes 4.64 m. Winter is closest to this at 4.12 m. From this, it can be concluded that autumn is the season with the largest waves and also with the highest frequency of large wave events. Summer has the lowest frequency of large wave events. All of this is visualised in figure 2.7. Here, wave roses are shown for a whole year as well as every season. The differences per season can be clearly seen in this.

The paper, referred to above, dates from 2012. It is not expected for a wave climate to change drastically in only 6 years. Therefore, the wave climate governed by the characteristics above can be assumed to still be valid now. For further reference, one other report is looked at. A similar study was done by Derrick Olij in 2015, to find the wave climate for modelling purposes. This time, only the data from the Waverider buoy near Durban was used for a period of 7.3 years. As the conditions at Richards Bay and Durban are not exactly similar, small differences can be expected. The results of this analysis were a significant wave height of 1.68 m, a peak period of 10 s and a angle of incidence 132° with respect to the northern direction. The peak values for the significant wave height and peak period are 3.17 m and 16.6 s respectively. Looking at the seasonal differences, it is observed that in summer and spring the waves are smaller and in winter and autumn the waves are bigger. In summer and autumn the wave period is shorter while in spring and winter the wave period is longer.[42] From this it can be concluded that it's safe to assume the described wave climates above as still resembling the wave climate in the present.

The tropical storms are seen as exceptional events that disrupt uniformity of the statistics mentioned above. These storms are expected to occur mainly during the autumn and winter seasons. Waves caused by these storms also have a high impact in the coast in terms of sediment transport, especially when happening close after each other. Due to their rare occurrence, they barely influence the wave climate.

Bathymetry

To get a good understanding of how waves behave when approaching the coast, the profile of the seabed close to the coast has to be known. This bathymetry basically is an elevation map of the seabed. Especially around the existing piers, the bathymetry can be irregular due to sediment transport. Currently no detailed bathymetry is available, but the data can easily be obtained through measurements.

Tides

The moon and the sun have a gravitational pull force on the Earths ocean water. Because of the rotation of the Earth around its own axis and the orbital motions of the Earth and the Moon, the tide is generated resulting in waves approaching the shoreline. Some big water level differences can be experienced due to the tide, especially when resonance (in a harbour) occurs. These tidal effects are generally semi-diurnal, if it weren't for the daily inequalities. Because of the fact that the Earth-Sun and Earth-Moon orbital motions are not in the equatorial plane, the gravitational pulls can be higher or lower at the different high-tides in a day. This effect increases significantly with higher altitudes on the earth. This can be so dominant, that the tide becomes diurnal instead of semi-diurnal [9].

	Tidal Symbol (description)	Period (hrs)	Coefficient
	O_1 (principal lunar)	25,82	0,415
Diurnal	P_1 (principal solar)	24,07	0,194
	K_1 (lunisolar declinational)	23,93	0,585
	N_2 (elliptical to M_2)	12,66	0,192
Semi-	M_2 (principal lunar)	12,42	1,000
diurnal	S_2 (principal solar)	12	0,466
	K_2 (lunisolar declinational)	11,97	0,127

Figure 2.8: The tidal constituents present at South Africa

In figure 2.8 the coefficients show the relative potential of a component compared to the dominant M2-tide. Also, the periods are shown for each of the constituents. The behaviour of the tide at South-Africa is mainly semi-diurnal according to the following formula:

$$F = \frac{K1 + 01}{M2 + S2} = \frac{0.585 + 0.415}{1 + 0.466} = 0.68 \tag{2.2}$$

If the value for F is between 0.25 and 1.5, the tidal character is mixed but mainly semi-diurnal [9]. The complete elaboration of the generation of the tide goes beyond the extent of this report. However, it is important to understand which tidal constituents/effects are present at Durban. According to Schumann and Perrins, the M2 tidal constituent dominates the coast of South-Africa [49].

Currents
Currents determine the main flow direction of water. They can be split into several groups, based on the scale they act on. First there are the global ocean currents that mainly determine the water circulation around the world. In front of the Durban coast, the Agulhas Current is acting in southern direction. The current is relatively strong compared to other ocean currents and has a maximum flow velocity of 2 m/s at the surface. This is however at the core of the current somewhere far from the coastline. The effects of the Agulhas current near the coast are limited. [45]

Next in line are the tidal currents, which are generated by the change in tide. How the tide works is already described above. As these currents follow the tide, they act perpendicular to the shore and switch direction following high and low tide. This also makes it a reliable current. Especially in bays, high speeds can be reached as the water gets 'trapped'. As the area of interest is at the shore, the tidal currents are likely to have a larger influence than global currents.

Finally, there are wave currents, generated by waves. These mainly occur when waves approach the coast under an angle. In this case they generate a wave-driven alongshore current. The wave climate in front of Durban is already described. From this, it follows that waves indeed arrive under an angle at the Durban coast, meaning these currents are present. Sediment is transported with this current as well, in this case in northern direction.

2.1.3. Model and data
The existing models are already quite extensive. A model for both the river system and the ocean is developed and in control of the municipality of eThekwini. Later on, when 'the river model' is mentioned, it is referred to the model in SWMM for the Umgeni river. When a reference is made to 'the ocean model', the model in Delft3D is meant for the coastal area. Further explanations on the updated models follow in chapter 3. These models require input on a frequent basis, because they have to forecast the flood danger for the upcoming three days. In this paragraph, the data collection and the existing models are discussed.

Existing models
The existing models of the river and the ocean are made with the programs SWMM and Delft3D, respectively. The interface however, is shown with the program Delft-FEWS. FEWS is a platform for real time forecasting and water resources management. It integrates different models to predict among others the hydrological circumstances such as water levels and wave heights. Based on these outcomes, a warning signal can be given on a very local level, so that only the people in question can be alerted.

As already mentioned, a river model and an ocean model are present. Obviously, there is an interaction between the river and the ocean in reality. In the existing situation, this link between river and ocean is not modelled properly. Some of the output of the river model is used as input for the ocean model (the river discharge) as the boundary condition. On the contrary, the output of the ocean model is not used as input for the river model. Hence, it can be said that the link between the two models is one-directional, whereas it has to be two-directional. As mentioned in paragraph 2.1.1, it is concluded that the river delta of the Umgeni river is between wave- and tide-dominated, which means that the ocean influences the river as well. Therefore, a coupled model should be made where ocean and river interact and the water level at the connection is continuous.

Data collection
The input for the river model, the precipitation in the catchment area, comes from radar information. This info is retrieved from the national meteorologic institute of South Africa, called the South Africa Weather Service (SAWS). The amount of rainfall in the drainage basin of the Umgeni river determines the real time discharge for any location along the river. Together with the discharge, the water levels can be predicted to give a proper warning sign.

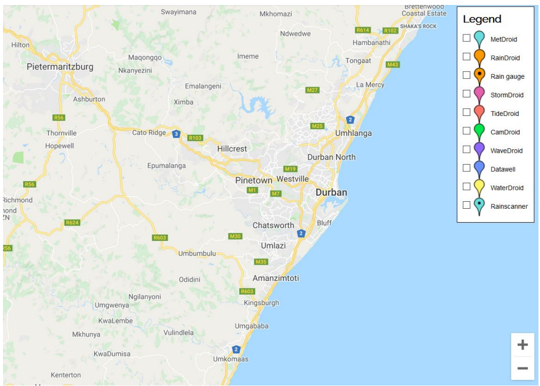
The warning system for the river is based on a three level principle, which are:

- Watch: Create awareness among the inhabitants, signal is given if $h > 0.7\ h_{crit}$
- Alert: Make sure inhabitants are prepared for evacuation, signal is given if $h > 0.8\ h_{crit}$
- Warning: Inhabitants do actually have to evacuate, signal is given if $h > 0.9\ h_{crit}$

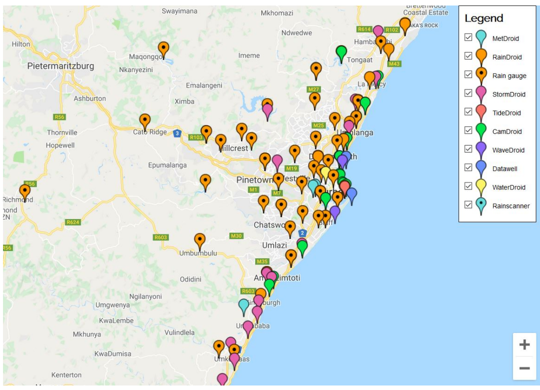
NB: h_{crit} is determined per river section.

For the ocean model, the wind speed is retrieved to calculate and determine the wave climate in front of the coast of Durban. This is modelled using SWAN, which also runs at the background in Delft3D.

The eThekwini municipality is not only using data from other parties, but is also collecting data itself. In a circle, with a radius of 50 km, around Durban the municipality has installed multiple different instruments to measure lightning strikes, rainfall intensity, wind speed, wind direction, gust wind speed, air temperature, relative humidity, solar radiation, water level of rivers, water level on sea, wave heights, wave periods and wave directions. Next to that there are multiple webcams installed to observe the area in real time. All of this data is gathered on an internal site, *DBI0.com*, and can be accessed by municipality workers. The eThekwini municipality is still installing more instruments to expand their real-time knowledge in the area. In figure 2.9a and 2.9b the location of the instruments are shown as on the internal site.



(a) The area in which the instruments are installed



(b) The area with all instruments

When the models, described in paragraph 2.1.3, give a high water level and possible threat of flooding, a warning message is send to workers of the municipality, who pass the message on. This message is send to, among others, the disaster management team. When the risk of a flood is serious the public is informed by the disaster management team. All in all the intention of the municipality is to make this an automatic process, with the use of an app or a public accessible website.

2.1.4. Coastal protection

Since the construction of the harbor, the natural sediment flow has been disturbed by the harbor arm and made the Durban beachfront rely on sand supply by other sources. Nourishment of beaches is needed to secure the beach from erosion and has always taken place. In the case of Durban beaches, no natural supply of sand is present since the long term drift, including sediment transport, next to the shore, going from south to north, is blocked by the entry of the harbor [40]. The entry of the harbor causes a so-called "shadow zone", see figure 2.10, where no natural supply of sand takes place. The beaches within the shadow zone rely on nourishment via a sand bypass system. The sand is captured at the sand trap at the harbour entry and transported via a pipe system and various boosters to the beaches, the purple scheme in figure 2.10. Transnet, owner of the Durban harbor, is responsible for sufficient nourishment of the beaches.[40]

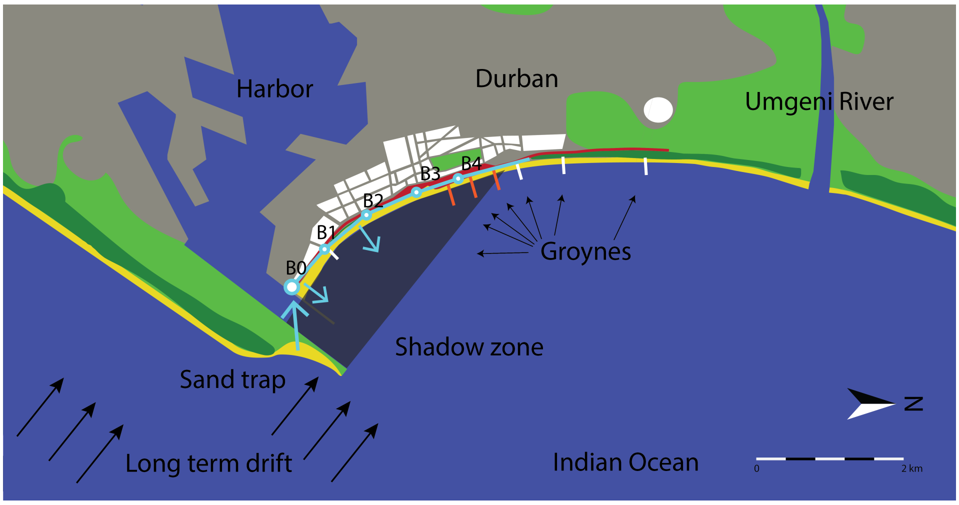


Figure 2.10: An overview of the considered area around the coast of Durban including sand pump scheme. Sand is trapped south of the harbour and pumped via various boosters (B0-B4) up north. Nourishment takes only place at the southern beaches while the pipe is constructed up north. Boosters 3 and 4 are not operational and nourishment of these beaches is done via offshore dredgers. The pumping scheme is shown more detailed in figure 2.11.

Over time, measures have been taken to prevent the beaches of erosion. In the 80's the groynes, see figure 2.11 have been built to prevent erosion of the North Beach. The groynes have withstand big events over time and are believed to be in a good condition, even though some maintenance was needed over the years. The March 2007 event caused most of the sand of the South Beach to be washed away. Intensive soft engineering nourishment was needed to restore the South Beach: in total 500.000 m^3 sand has been nourished in 2009 and 2010 to restore the beach and prepare the beaches for the 2010 World Cup. In the same period, the entire promenade from Addington to Durban North, also known as the Golden Mile, has been rebuilt. An analysis of the different sections of the Golden Mile is assessed in section 4.1.1. [14]

The nourishment in 2009 and 2010 has mainly been done at the southern beaches and came from an offshore borrow site, which also was used for deepening of the berths in the Durban harbor. However, since the long drift is blocked by the harbor groyne, none of this nourished sand reaches the northern beaches [40]. Figure 2.11 shows the wider beaches at Addington and South Beach, compared to the North Beach near the groynes.

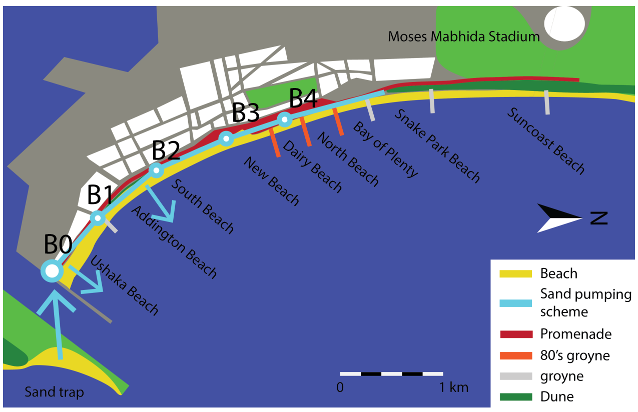


Figure 2.11: A map with the beaches and pumping scheme of Durban . Three of the groynes are constructed in the '80s and are indicated in orange. The groyne at Addington Beach is a pier, not a groyne.

Due to the widening of the harbor entrance in 2009, the sand-pump operated by Transnet, located on the north harbor groyne, had to be demolished and located on a new spot. Therefore a temporary pumping system has been used which was similar to the old facility [56]. According to Transnet in an interview with Times Live, Transnet had nourished 500.000 m^3 sand per year on the southern beaches and said that it was ''the responsibility of the city to redistribute the sand to beaches further north'' [12]. However, due to a lack of proper infrastructure, this has not been done, causing the northern beaches to erode [40]. Sand for nourishment is trapped at the south harbor groyne and transported via 900 mm pipes to the beaches. The existing pipe reaches only the Addington beach, 1.4 km from the sand trap. Corbella and Stretch stated in 2012 already that the goal is to get that pipe to the northern beaches to nourish sand. The sand can be transported via a 3.5 km pipe with the help of four booster stations [14]. At the moment of writing, the pipe is constructed and a few times a year in use, since this is not a continuous process. However, booster station 3 and 4, as indicated in figure 2.10, are not operational yet.

For the 2010 World Cup, the beaches have been restored including the construction of dunes along the Snake Beach in the north and at Addington and South Beach. The dunes form a natural coastal protection. However, the dunes were designed in 2008 but have been lowered in 2012 to create a better view over the promenade, which rises many questions regarding the flood safety of these parts [10]. Besides, for the renewal of the promenade for the World Cup, future climate change aspects have been neglected according to Lisa Guastella in an interview with The Daily Vox [40].

Other hard engineering coastal protection systems like seawalls or revetments are not part of the coastal protection system in this area. In emergency situations Geotextile Sand-filled Containers (GSCs) have been used to protect the beach and promenade from erosion and floods, but they offer no future solutions in the way they have been used now.

2.2. System border

A system border is defined for this report. This limits the contents to only the necessary information and prevents getting dragged off-topic to less important parts. The border is defined by a geographical border and a project scope. The geographical border limits the area which is researched. The project scope mentions the actual subjects that are being researched.

2.2.1. Geographical border

The main source of the problem lies within the flood potentials of the Indian Ocean and the rivers running through Durban. Besides their own specific mechanics, they also influence each other. This mainly is due to the fact that water levels have to be continuous between the rivers and ocean. The ocean can however only influence the river to a certain extent, that is a distance upstream from the river mouth. This distance has to be determined later on. The rivers around Durban however have many dams in them. As they prevent free water flow, they also prevent the ocean influencing the water level in the river upstream of the dam. For the Umgeni River, the first main dam looking from the river mouth is the Inanda Dam. This is then also were the system border is, as looking beyond this point has no use.

Just looking at the river itself, its water level is determined by the rainfall in its catchment, which is already included in the models. In this report is mainly focused on the influence of the ocean on the river. As this is only expected to cause changes in the water level of the main channel, not the entire catchment area has to be taken into account again. Just looking at the main channels is enough.
The other end of the geographical border lies in the ocean. The coastline must be included, as this is where the interfaces between ocean and rivers are. The water level in the ocean is mainly dependent on the tide.

For the design of the structural element, only the Durban beachfront is considered, which stretches from the harbor entry to the mouth of the Umgeni river.

Figure 2.12 shows the outline of the geographical border. The city itself also is inside this reference frame, but is not of interest for the modelling. The sections of the rivers inside the frame as well as the coast are of importance.

Figure 2.12: Geographical border of project scope

2.2.2. Scope

The two main goals of the report are to find a way of linking the two models for the river and ocean system and to find a structural solution as coastal protection.

Currently, the model is linked one-way only, from river to ocean. The link the other way around still has to be made. This means output from the ocean model should be used as input for the river model. This is at the interface of the models, the mouth of the river. For now, there is only to be looked at the water flow, as sediment transport is neglected. The rivers are considered tidal, meaning that the tide of the ocean mainly determines the water level of the river in the lower reaches. This is therefore also the main linking parameter.

For the extent of this report, the Mlazi river is not taken into account in linking the two models. Besides, there are no morphodynamics included in the models at all right now. So the influences of the reservoirs trapping sediment is also left out.

For the design of a flood defence a distinction can be made between engineered defences, or hard methods, and natural defences, or soft methods. In this report, natural defences like salt marches, mangrove forests, wetlands, gravel bars, bypass and nourishment are not taken into consideration. However, one exception is made for a dune, as due to the surroundings this natural defence is already part of the coastal protection of Durban. An engineered flood defence, or dune, is designed up until a preliminary design, it should fit in the existing surroundings and protects the coastal region of Durban against severe floods.
Differences in elevation make specific areas more prone to flooding. It is important and therefore within the scope to use elevation data to assess those area. Differences in elevation come from natural and man-made causes. However, differences in sea water level are natural causes and should be investigated in relation to the local elevation data. Since the structural element is a solution for the next decades, it is within the scope to assess climate change within design life. Hereby it should be noted that climate change is considered to a certain extend, since an entire research can be done to the local effects of climate change.

A different problem is the erosion of the beaches. The structural solution is not a solution to prevent the beaches from eroding, only for the flooding of the promenade, which is assessed further on. However, it is important to be aware of the problem and good to mention it throughout the report.

2.3. Assumptions

In this section, the assumptions are summed up which had to be made in order to continue the research. Afterwards, when reconsidering the design and opted solutions, the assumptions made have to be checked to have a valid conclusion.

- The water level at the river mouth is regulated by the water level of the ocean, assuming that the volume of the ocean is big enough to absorb influences of the river discharge.
- Even though climate change is assumed to be of influence on the design, only the sea level rise is assumed in the design and increase in intensity of storms caused by climate change is neglected.
- The friction coefficient calculated in paragraph 3.3 is assumed to be valid for the whole reach from the river mouth until the Inanda dam. This assumptions should be checked afterwards with measurements on discharge and corresponding water levels.
- The booster station located in the vulnerable area is assumed to withstand severe storms, at the same level of the groynes, and do not have to be protected by the flood defence.

2.4. Stakeholder analysis

In order to successfully warn and protect the Durban coastline against flooding and waves, the role of stakeholders has to be closely elaborated, as an opinion of a single stakeholder can already block a possible solution for the project. Therefore, a detailed stakeholder analysis is necessary to save the project valuable time, money and minimize the risk of other negative effects during the whole process of the project. The main aim of the stakeholder analysis is to identify and characterize the stakeholders, with the use of a Power Interest Grid and a Active Thread and Opportunity Model (ATOM).

In this report is dealt with two objectives; modelling an ocean-river system related to a 3-day ahead warning system and a flood defence related preliminary design. For each objective a separate stakeholder analysis is performed, in terms of identification and characterization. The 3-day ahead warning systems is characterized by humanitarian damage, as it only warns and doesn't protect, and the preliminary flood defence design is characterized by material damage, as it is designed to protect properties. Due to the extensive research which has already been performed by an other research team in 2016, the identification of the stakeholders and the characterization regarding flood defense are still relevant for this report [1]. For this report the stakeholder analysis of the previous research project is used and refined, to be applicable for this project.

2.4.1. Stakeholder identification

The identification of the stakeholder depicts which stakeholders are involved in the project of protection the Durban shoreline. The stakeholders involved are listed in table 2.2 and categorized as follows; 1) Authority related, 2) Non Governmental Organizations (NGOs), 3) Organized business, 4) Residential and 5) Recreational. This list of stakeholders is applicable for both objectives, only their viewpoint may differ, which is dealt with in paragraph 2.4.2.

2.4.2. Stakeholder characterization

A correct stakeholder management is obtained by visualizing the impact on each objective of every individual stakeholder and dealing accurately with their opinion. As a part of the identification of the stakeholders a Power Interest Grid and an Active Thread and Opportunity Model (ATOM) are computed for the stakeholders mentioned in table 2.2. This is performed separately for both the 3-day ahead warning system, relating to humanitarian damage, and the flood defence design, relating to material damage.

Table 2.2: Stakeholder identification

Category	Stakeholder
Authority related	South African government
	National Treasury
	South Africa Maritime Safety Authority (SAMSA)
	Government for KwaZulu-Natal Department of Cooperative Government and tradition Affairs
	eThekwini municipality
	Provincial Department of Health
	South African Police Services
	KZN Deparment of Health and private service providers
NGO	Coastal Watch
Organised Business	Insurance companies
	Large businesses (South African Property Owner Association)
	Local businesses
Residentials	Lower class residents
	Higher class residents
Recreational	Tourism organizations
	Daily visitors focus area

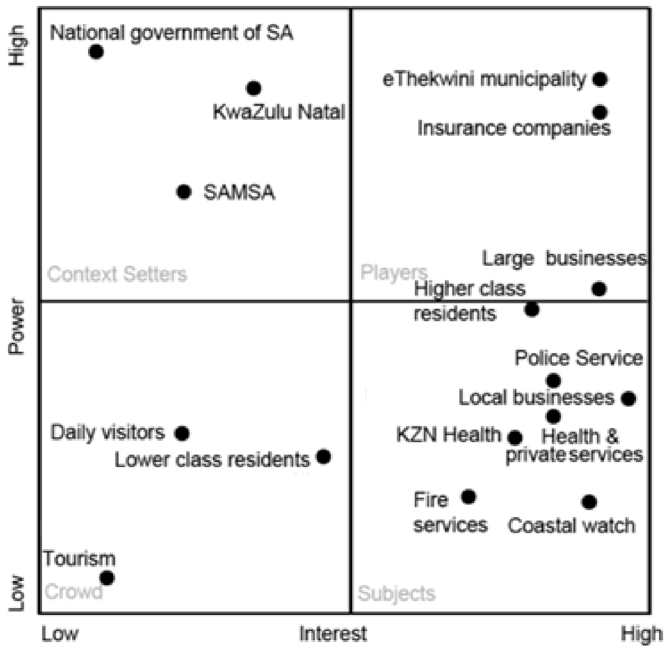


Figure 2.13: Power Interest Grid toward a coastal warning system

Flood warning system

In figure 2.13 the relation between the stakeholders is shown in terms of their power and interest, regarding the coastal warning system of Durban. The stakeholders are divided into four groups: Context setters, Players, Crowd and Subjects. In table 2.3 these stakeholders are defined in more detail, including their general point of view towards the objective. The background information for the characterization of the stakeholders can be found in the report: *Flood Safety Durban (2016)* and the refined characteristics can be found in Appendix B [1].

The national government of South Africa is the most powerful stakeholder involved, followed by the eThekwini municipality and the KwaZulu-Natal government. As the problem is located in the eThekwini municipality, they have the most interest in finding a solution. The insurance companies form another stakeholder with high power, as they have a lot of money. The higher class residents and the large businesses have similar power and interest, only due to the consequences to their businesses, large businesses have slightly more power and interest. The other stakeholders have little power, but they have to be informed properly, especially the stakeholders with high interest. The authority related stakeholders have higher power and interest, due to their power as the whole department. Local businesses also have high interest, as their businesses can be completely washed away by a flood. Due to the high number of local businesses, their power is considerable. Daily visitors of the beach and tourism have little interest and little power as they only come to visit on sunny beach days and therefore aren't affected by the consequences for flood warning.

Table 2.3: Power, interest and attitude of stakeholders towards a coastal warning system

Stakeholder	Power	Interest	Attitude	ATOM
National government and treasury	High	Passive	Backer	Sleeping Giant
SAMSA	High	Passive	Backer	Sleeping Giant
Government of KwaZulu Natal Department of Cooperative Government and Traditional Affairs	High	Active	Backer	Saviour
eThekwini municipality	High	Active	Backer	Saviour
Provincial department of Health	Low	Active	Backer	Acquaintance
South African Police Services	Low	Active	Backer	Friend
eThekwini Fire	Low	Active	Backer	Friend
KZN Department of Health and private service providers	Low	Active	Backer	Friend
Coastal watch	Low	Active	Backer	Friend
Insurance companies	High	Active	Backer	Saviour
Large businesses	Medium	Active	Backer	Friend / Saviour
Local businesses	Low	Active	Backer	Friend
Lower class residents	Low	Passive	Backer	Acquaintance
Higher class residents	Medium	Active	Backer	Friend / Saviour
Tourism	Low	Passive	Backer	Acquaintance
Daily visitors of focus area	Low	Passive	Backer	Acquaintance

Table 2.3 shows the ATOM of the flood warning system, which characterizes the position of the stakeholders based on the Power Interest Grid and their attitude towards the problem. As table 2.3 illustrates, there are no real blockers towards a warning system for the coastal area. This generates an overall positive attitude, but still awareness is required, as a minor change in the solution or a decision during the process can lead to a complete blocking of the project. A detailed explanation of the labels is given in the article, *Making Sense of Stakeholder Mapping by Ruth Murray-Webster & Peter Simon* and are briefly described in appendix B.3 [41].

Flood defence design

For the design of a flood defence the same Power Interest Grid is composed. For Power Interest Grid the stakeholder characterization is generally used from the report: *Flood Durban Safety (2016)* and slightly refined to be applicable for this specific objective. [1] The refinement makes the power medium and the interest active of the daily visitors of the beach, as any structural intervention can damage their view and/or wave climate. Table 2.4 shows the ATOM similar as before, only now relating to material damage.

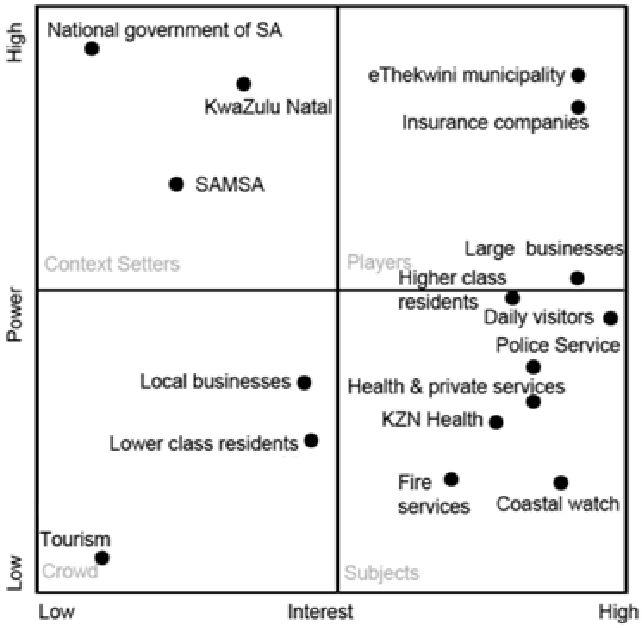


Figure 2.14: Power Interest Grid toward a design of a flood defence

Table 2.4: Power, interest and attitude of stakeholders towards a structural barrier design

Stakeholder	Power	Interest	Attitude	ATOM
National government and treasury	High	Passive	Backer	Sleeping Giant
SAMSA	High	Passive	Backer	Sleeping Giant
Government of KwaZulu Natal Department of Cooperative Government and Traditional Affairs	High	Active	Backer	Saviour
eThekwini municipality	High	Active	Backer	Saviour
Provincial department of Health	Low	Active	Backer	Acquaintance
South African Police Services	Low	Active	Backer	Friend
eThekwini Fire	Low	Active	Backer	Friend
KZN Department of Health and private service providers	Low	Active	Backer	Friend
Coastal watch	Low	Active	Backer	Friend
Insurance companies	High	Active	Blocker	Saboteur
Large businesses	Medium	Active	Backer	Friend / Saviour
Local businesses	Low	Passive	Backer	Acquaintance
Lower class residents	Low	Passive	Blocker	Trip Wire
Higher class residents	Medium	Active	Backer	Friend / Saviour
Tourism	Low	Passive	Backer	Acquaintance
Daily visitors of focus area	Medium	Active	Blocker	Saboteur / Irritant

2.4.3. Conclusion

From both stakeholder analyses, even when exactly the same stakeholders are involved, completely different conclusions can be drawn. For the design of a flood warning system, the overall attitude is positive towards the solution. High influences, in terms of power and interest, come from the local municipality and province authorities, who are responsible towards their inhabitants and the national government. Large and local businesses and higher class residents are also interested in such a system, because due to the warning the cause save the properties and belongings. All the other stakeholders aren't involved and only have to be informed correctly.

The general attitude towards a design of a flood defence are divided. Again, the role of the local municipality and province authorities are similar compared to the warning system, even so for the businesses. It is the public opinion of the daily visitors that has turned drastically against structural intervention, as their priority is not safety of the city, but the pleasure of their day to the beach. For a city as Durban this group is of considerable importance, as tourism and daily visitors generate a great economic impulse for the city. Not keeping this group satisfied can lead to a great opposition of a certain decision and eventually a block of it. Therefore their opinion has to be taken into account with great care, during the whole process of the project.

2.5. Reference projects

Now that the problem is fully described and understood, a solution has to be found. What might help with this, is taking a look at some reference projects. These are projects where there had to be dealt with similar problems under similar circumstances. As these are projects done in the past, a right solution might have already been found that might also be of use in this case. For each reference project, first the project itself is described, followed by how it's relevant for this project.

2.5.1. Early-Warning System in the Gulf of Thailand

Heavy tropical storms are a recurring event in the Gulf of Thailand. These storms cause high water levels and waves at the coastal zones of Thailand. These regions already have a high population density and on top of that keep attracting more tourists. A flooding can therefore cause serious harm. To be able to evacuate people in time, an early warning system is created for the Gulf of Thailand.

The project was a cooperation between Deltares and HAII (Hydro and Agro Informatics Institute of Thailand). The system has three models in it: a regional wave model covering the Gulf of Thailand, a local wave model covering the southern part of Thailand and a hydrodynamic model covering the Gulf of Thailand. The wave models are based on SWAN. The hydrodynamic model uses D-Flow from Delft3D. The mesh was generated using the Flexible Mesh Suite. The models are combined in Delft-FEWS to visualize the total output. The input in FEWS is a 7-day meteorological forecast. The output are a 1-day hindcast and 3-day forecast of water levels and wave heights in the Gulf of Thailand. From this flooding forecasts and wave setup forecasts are generated. The results of the model are considered sufficiently accurate to be included in the early-warning system (EWS) at HAII. [17]

This project is useful for the case of Durban as an early-warning system has been created regarding flooding by oceanic waters. In Durban, the majority of the modelling is done. One important difference is the presence of rivers in the Durban area, which also have to be modelled. Thailand also has rivers around mouthing in the Gulf of Thailand, but these are not included in the model. Another difference is the scale of model. In the case of this report, the model is only focused around Durban, with a length of around 30 km. The model might later be used for the whole eastern coast of South Africa or maybe even larger parts of Africa, but that is still a long time away. The Thailand model however already is 1800 km by 1200 km.

This mainly has influence on the grid size, as such a large model with small grids would take very long to run. The Durban model is significantly smaller and thus smaller grids can be used. The EWS in Thailand is in operation now. As the Durban model still needs some parts added to it, the Thailand model can be used as an example.

2.5.2. Umhlanga rock coastal defence

The flood of March 2007, described in 1.1, was one of the reasons to look further into the protection of the coastline of Durban, in particularly the coastline of Umhlanga Rocks. In 2014, a research team of the TU Delft investigated the improvement of the coastal defence of Umhlanga rocks. Their problem definition is as follows:

"Due to erosion and extreme weather conditions the coastline of Umhlanga Rocks is shifting on shore, causing narrow beaches, decrease of tourism and increased risk of failures of the coastal structures. The existing situation requires a new long term safety strategy, taking into account the social, economic and environmental vitality of the Umhlanga Rocks area as well."

In this report four solutions are investigated: Doing nothing, a bar retaining sill, nourishment and a breakwater. One of the valuable conclusions of the report is that doing nothing will result in loss of the beach resulting in a big loss in tourism, which evidently impacts the economy of the city. Concluding, doing nothing is not an option and doesn't have to be investigated for the coast of Durban. The best option, for mitigating the loss of the beach, is a submerged breakwater. [19]

2.5.3. Scheveningen promenade

In the beginning of the century, Scheveningen (The Netherlands) was marked as weak spot in the protection, of the Netherlands, against the sea. Thus the coastal protection had to be improved. In 2009 the definitive design for the new coastal protection for Scheveningen was finished. In 2013 the designs were realized. The design included a promenade, which is heightened, with an integrated seawall. In figure 2.15 the promenade is shown.

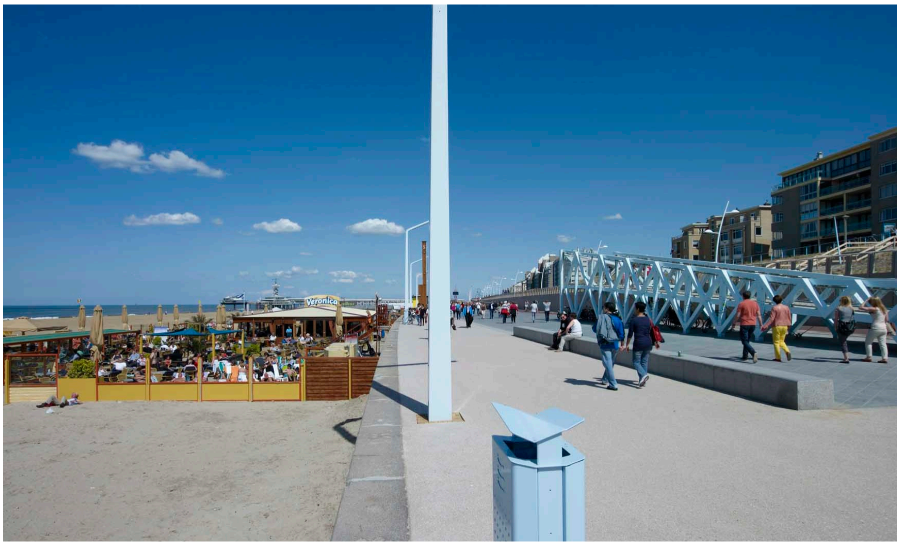


Figure 2.15: Boulevard of Scheveningen with integrated seawall [16]

In figure 2.15 it is shown that the beach, with a restaurant on it, is lower than the promenade. On the right hand side of the figure the buildings are build on a higher level than the promenade. The seawall is integrated in the environment on the right hand side of the promenade. [16]

In this case the promenade could flood during a serious storm, but the hinterland of the promenade is saved for the flood. This example shows that the promenade still has the view of the sea and good access to the beach. One big disadvantage is that the promenade isn't completely safe for flooding. The designers went for safety for the more important buildings in the hinterland of the promenade instead of saving the promenade. Next to that, it is a great example of how to integrate a coastal protection in the surroundings.

2.5.4. Dordrecht flood safety measures

The Dutch are used to fight the water, due to the fact that big areas of the Netherlands are below sea level. Dordrecht, a city in the Netherlands, also fights the water with several defence mechanisms.

The defence mechanisms (or flood measures) consist, among others, of dikes, distribution of sand sacks and an emergency flood barrier. The sand sacks, including instructions are distributed to residences of the lowest areas, the old inner city. The instructions, how to place the sand sacks, can be found on their site. The emergency flood barrier is place at several locations in the city, and shown in figure 2.16. In the same figure the yearly test of the emergency flood barrier is shown. Every year the barrier is placed, such that the municipality workers know how to place them and to see if the material is still in good condition. [26]

Figure 2.16: Yearly test of the flood barrier in Dordrecht [8]

Concluding, an emergency flood safety system is a good possibility to protect big areas as an old inner city. Although this solution only works when two conditions are satisfied. Firstly, there should be a warning system, just like the high water level mail service of the municipality of Dordrecht. Secondly, the equipment for the emergency flood barrier should be checked periodically and maintained.

3

Ocean-river system

The existing model is operating using the interface of the program Delft-FEWS. This is an abbreviation for 'Flood Early Warning System'. Delft-FEWS makes it possible to integrate data from different models into one single interface, which can be used to give warning signs to the local inhabitants. The ocean model is made in Delft3D, which is adapted in this project. As already mentioned, currently there are two models present. One for the river system and one for the ocean. The objective of this project is to interlink the two models. Therefore the focus is rather on making this link, than on explaining the whole model.

In the first paragraph, the problem is elaborated more. Second, a solution in the form of an extended model in Delft3D is presented. Here, the input of the existing model is further evaluated. Consequently, the steps are mentioned in extending the existing model. Besides, the output of the renewed model is showed as well. In the third paragraph, a solution is found in the form of an empirical fit. This is done by an analytical model using the fit of Bresse for the backwater curve, modelled in Python. Finally, a validation of the new model is carried out, comparing it to the analytical solution as well.

3.1. The problem

The two models being present do not interact with each other on a sufficient level. It is only one-directional linked, whereas it has to be two-directional. The water level at the ocean partly determines the water level at the river, because the delta is (partly) tide-dominant as stated in paragraph 2.1.1. Since there is no input from the ocean model into the river model, the river model gives a wrong water level. If the water level at sea exceeds the equilibrium depth of the Umgeni river, a backwater curve occurs resulting in higher water levels in the river for a certain distance. This increases the danger for the inhabitants as the water levels may turn out to be higher than predicted with the river model only. Figure 3.1 shows that a (M1) backwater curve occurs in this situation.
NB: The addition of the 'M1' implies a certain shape of the backwater curve, where M stands for 'mild'. It is a M1 because in this particular example the downstream water level exceeds the equilibrium depth of the river.

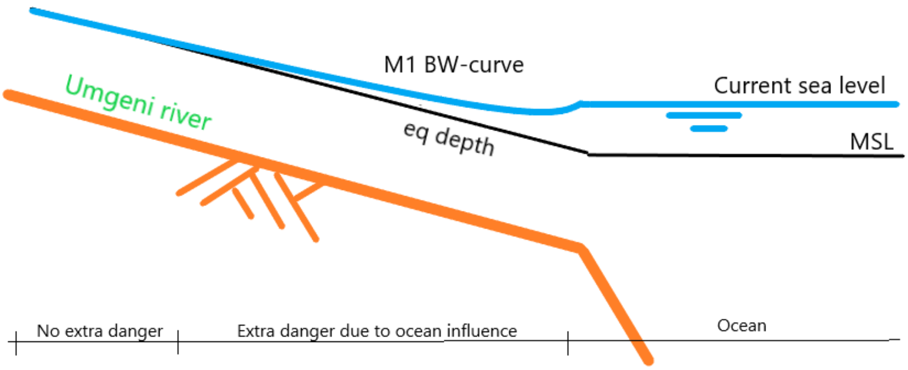


Figure 3.1: The backwater curve that might occur

3.2. Extending the ocean model in Delft3D

The idea here is to extend the ocean grid into the river. The model then continues computing flow and wave output in the river. The result is a water level related to the tidal effects as desired. Although this sounds quite simple, there are some complexities. The mechanics in the river are different than in the ocean. Then there is the problem of the grid itself. First it must be extended up into the river. This can be done by editing the existing grid or adding a separate river grid. In both cases, a correct river bathymetry should be added to the grid. This can be obtained from GIS data, but preferably from measurements.

Extending the ocean grid causes an overlap between the new extended ocean model in Delft3D and the existing river model in SWMM. A solution for this is shortening the SWMM model until there is no overlap anymore. This is most likely at the Inanda dam, as the backwater curve can not reach any further by definition. The output from the SWMM model was a discharge into the ocean. This type of interface can be kept, but now it is a discharge at the Inanda dam. Of course, the discharge here is regulated. There must also be accounted for the precipitation that falls in the catchment of the lower part of the river up to the Inanda dam. This has to be included in the extended ocean model. This could be done through several discharge boundaries.

The validity of the new model can be checked with measurements. Currently, river gauges are only present at the river mouth and at the Inanda dam. The end of the ocean influence must be somewhere in between these points, as it obviously can not pass the dam. Placing more measurement equipment could help in validating the new model.

3.2.1. Model input

The model has many different aspects as input. Meteorological data is used as input, particularly the rainfall, wind speeds and air pressure fields. Radar data from the national weather service is used to predict the rainfall in a certain catchment area. This data is only used in the river model. On the other hand, the wind speed and air pressure are used in the ocean model, since these are important in determining the resulting wave heights. Below, the four main parts of the total existing model system are listed. These are elaborated more as well, including the way they interact with each other.

- The defined grid and bathymetry
- The FLOW model
- The WAVE model
- The river model

As the river model is now (partly) incorporated in the other models in Delft3D it disappears from this list in the new model. The remaining three components have to be adapted to create the new model. The made changes are discussed in paragraphs 3.2.2, 3.2.3 and 3.2.4. First a general description of each part is given.

The defined grid

The defined grid is very important in defining the accuracy of the predicted hydrological conditions and the computational time required. The more refined the grid results in more accurate results. Around critical or vulnerable areas one would like to have a more refined grid, because it gives the most reliable and detailed solutions. On the contrary, a less refined grid is preferable, since it decreases the computation time drastically. In the existing models, the grids are just equally refined. The existing grid including the link from the river model to the ocean model looks like shown in figure 3.2. The bathymetry is the elevation profile of the seabed, attached to the grid. This can be obtained from elevation data, called samples in Delft3D.

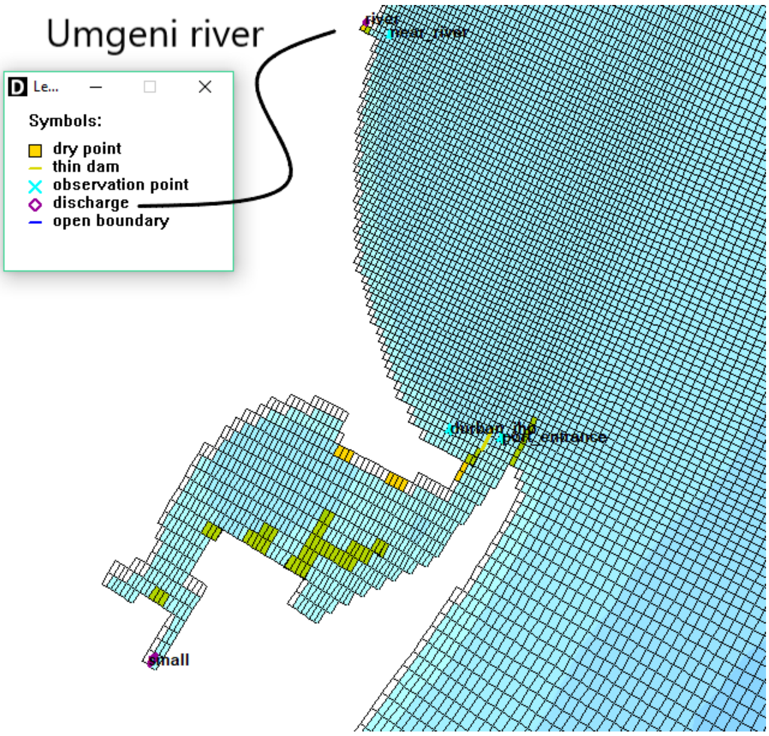


Figure 3.2: The grid of the ocean model in Delft3D, including the boundary condition from the river

The FLOW model

The FLOW model accounts for the flows in the ocean-river system. The grid mentioned above is used together with the known bathymetry. Boundaries are defined at the edges of the model. It is also possible to select which physical processes are included in the computation of the hydrological model: Wind set-ups, secondary flows, sediment transport, pollutants etc. The flow model calculates the water level and currents and gives these as output. A good example of a natural current that needs to be taken into account, is the Algulhas current. This current is shown in figure 3.3.

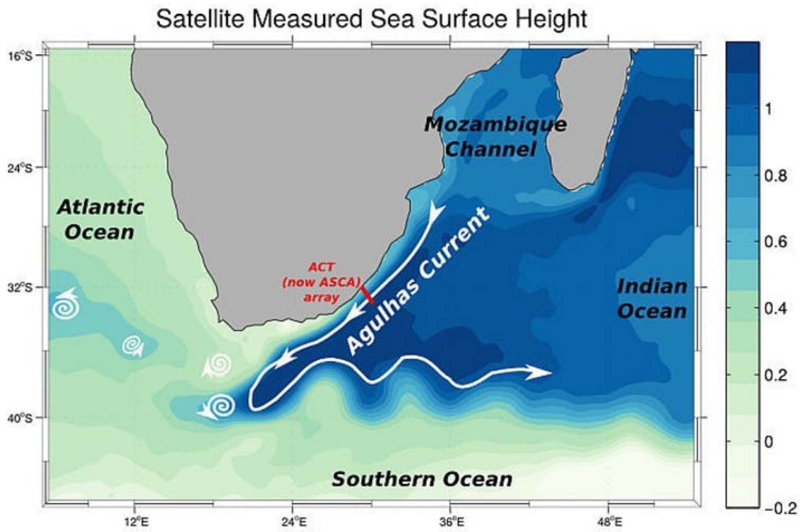


Figure 3.3: The Agulhas current along the Durban coastline

For now, no sediment transport is taken into account. This has consequences for the long term use of the model, if the bathymetry is not in an equilibrium state. Due to the sediment transport, the bathymetry might change significantly over time. Since the sediment transport is not part of the model, it is not 'aware' of this process and it does not change the bathymetry accordingly over time. A solution for this problem would be, to continuously update the bathymetry in the model, based on new measurements in the field. However, this is an expensive solution. Including the sediment transport in the model is planned for the future.

The WAVE model/SWAN Delft

The WAVE model is the second module of Delft3D used for the ocean and accounts for the waves generated in the ocean. The model uses for a large part the same input as the flow model, like the grid and bathymetry. Again, boundaries along the computational area are specified. As these boundaries are (partly) in the ocean, incoming waves must also be defined for storms occurring outside the model grid. For this, the WaveWatch III is used. SWAN is the actual program running in the background in Delft3D. It determines the wave characteristics of the waves closing in on the Durban coastline. In this, multiple physical processes are included, like:

- Depth-induced breaking
- Non-linear triad interactions
- Bottom friction
- Diffraction and Refraction
- Quadruplet interactions
- White-capping
- Shoaling

Next to these physical processes, it is also possible to model physical obstacles or boundaries. Durban has several permeable piers reaching out into the ocean and also the harbour plays an important role in the development of waves as it causes diffraction for example.

The river model
Contrary to the ocean model, the river model is made in SWMM (Storm Water Management Model). It covers all of the river catchment area. Here, it takes the predicted rainfall as input and converts this into a discharge for the river. From this, also a water level in the river can be determined if the river cross-sections are known. The discharge at the river mouth is used as a input for the ocean models. This is the first one-way link between the models. Similar to the ocean model, the river model does not contain sediment transport yet. This is to be included later, together with water quality.

Overview of the models present
Figure 3.4 shows an overview of the different models running. Note: this is a simplified overview with the main input for the two models. In reality, more detailed information goes into the model, which is discussed in paragraphs 3.2.3 and 3.2.4.

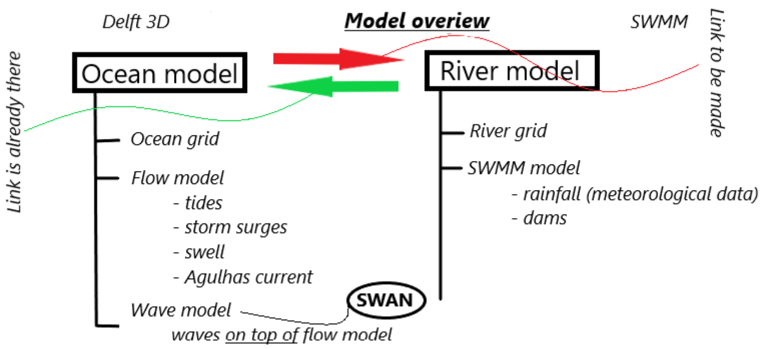


Figure 3.4: The Delft3D model overview

3.2.2. Grid and bathymetry

The downstream river section, which is added to the ocean model, needs a grid itself. Together with the bathymetry, the model includes the river in its calculations. The bathymetry for the river is a combination of two sources. From the ocean to where the highway '2' crosses the Umgeni River, measurements done by EMS [20] are used. These give a very detailed bathymetry of this river section. The rest of the river bathymetry comes from GIS data. This is less accurate, as the water level is included in this measurements. Also, the GIS data is much less dense. A simplified overview of the steps taken are shown in the list below:

1. The GIS data: The GIS data is basically information about the elevation of different coordinates. The input for GIS is a raster format, received from the municipality. This has to be converted to a point data format. With the help of the program QGIS, a certain area of interest can be selected. This is then clipped from the main map. The elevation data of this area is exported as a '.xyz' file, as this is what Delft3D needs as input. The file contains information about the height 'z' at point (x,y)

2. The xyz data: The '.xyz' file is imported as a sample in the RGFGRID tool of Delft3D. The '.xyz' of the lower river reach is added to this. RGFGRID is used in order to create a grid, which can be imported in Delft3D itself at a later moment. An elevation map is shown in the RGFGRID tool, consisting of many individual coloured points. These points visualise the '.xyz' data and thus the depth contours of the river.

3. The grid for the river: Consequently, the grid for the river can be drawn. This is done by drawing splines along the river. If the splines are drawn correctly, RGFGRID can create a grid out of these splines. Once this is done, the grid can be refined and orthogonalised. The grid needs to be orthogonalised, to make sure that the grid cells intersect with an angle of 90 deg. This improves the quality of the grid, because deviations in the $cos(90)$, i.e. values other than 0, result in errors in the pressure gradient in Delft3D-FLOW.

4. Importing the ocean grid: Once the grid for the river is created, the ocean grid can be imported to the same file in RGFGRID. Obviously, the ocean grid was already present because the ocean model already existed. The tool recognizes the different grids and it allows the user to work on them separately.

5. Pasting the two grids: Since Delft3D wants to have one grid file as input, the river grid and ocean grid need to be pasted together. RGFGRID can paste them together decently as long as two conditions are met. Firstly, the different grids are close enough to each other, i.e. they are closer than a quarter grid cell from each other (1). Secondly, the grid cells need to have approximately the same dimensions, since it connects 'grid cell A from grid 1', to 'grid cell B from grid 2' (2). The result is one single grid, containing both the ocean and the river section. However, the river grid is preferred to be more refined than the existing ocean grid. The scale of the river system is much smaller than the ocean system, so more detailed information about the river is wished for and required. This can be achieved by refining the grid of the river system. However this gives complexities, regarding the second condition (2) just mentioned. Since the tool 'Flexible Mesh Suite' of Delft3D can not be applied here, another solution has to be found.

6. Locally refine tool: RGFGRID has a 'local refinement' tool. It can refine the grid along a certain grid line. Hence, a more refined river grid is achieved by refining only the local M- and N-coordinates. See figure 3.5 as an indicative figure to understand what is meant here. An additional complexity of this solution, is that the ocean grid is also more refined along certain grid lines. This is because RGFGRID can only refine a grid locally along the entire grid line. Therefore, this solution costs a bit more computational time, because the ocean grid is (unnecessarily) finer, but it also gives more detailed and reliable solutions in the Umgeni river. Weighing the pros and cons of this solution, it is considered to be positive for the model.

7. The bathymetry file: The bathymetry file is created by combining the new grid and the elevation samples from the .xyz files. As said, the elevation of the river is obtained from GIS and measurement data. For the ocean, this is taken from the existing model and copied into the new model. The elevation data has to be implemented in the grid points. For the river, this is done by averaging per grid point and triangular interpolation. Which method is used depends on the density of the available data at a specific grid point.

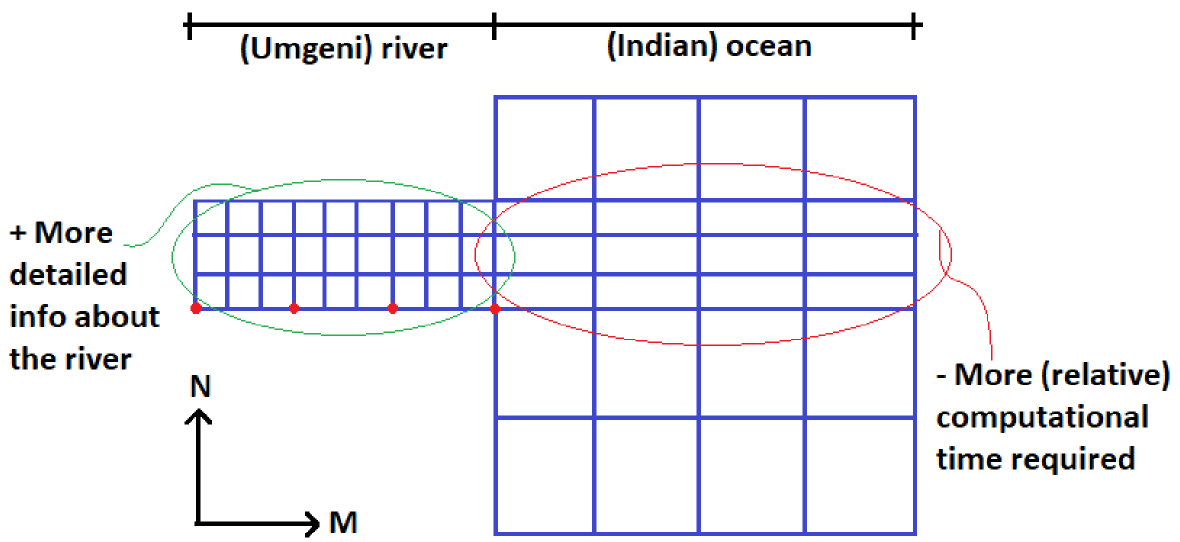


Figure 3.5: Indicative figure about the two grid refinements

The total grid now looks like in figure 3.6. The result is used for both the FLOW and WAVE model. Compare this to figure 3.2.

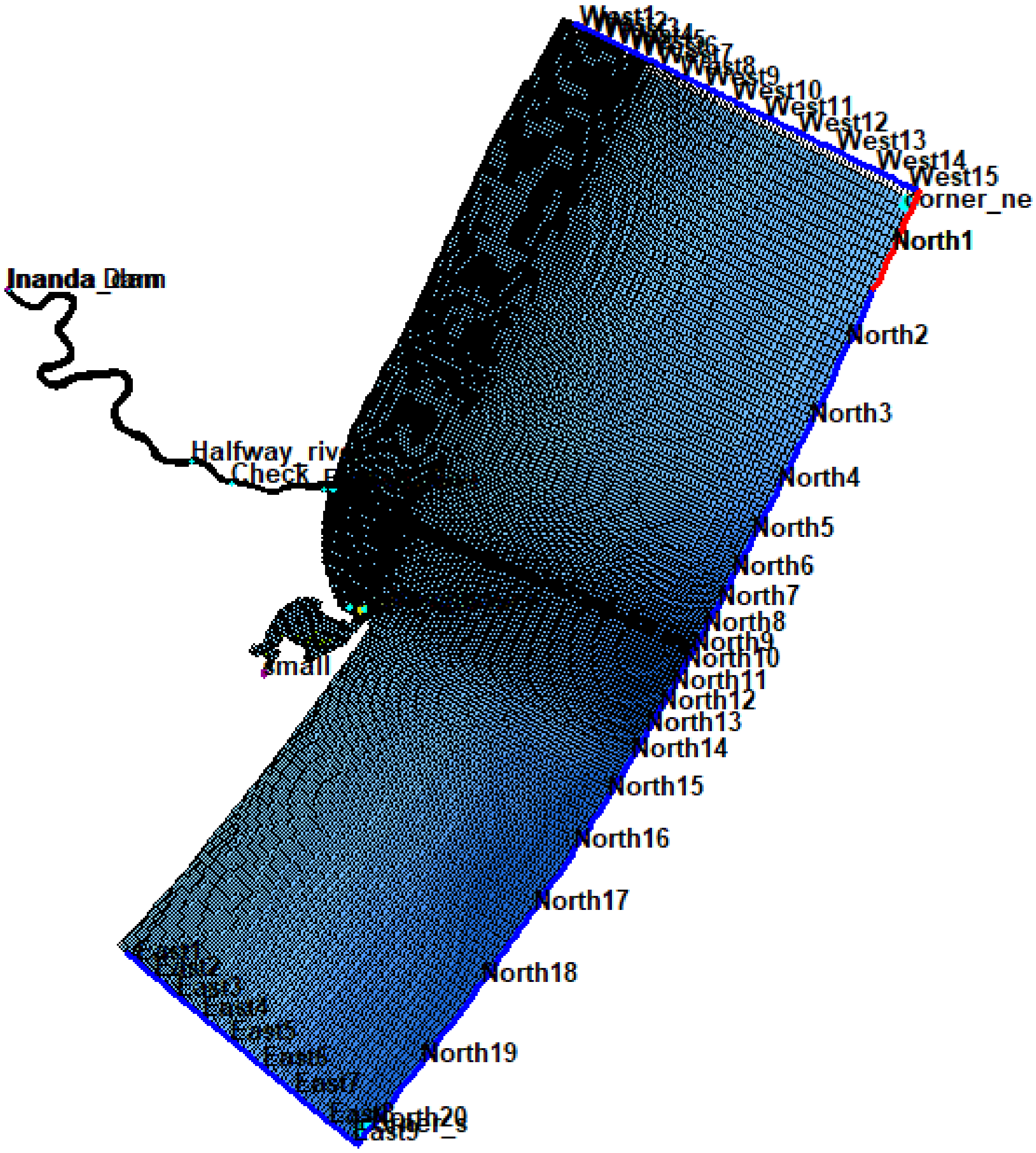


Figure 3.6: The total grid for river and ocean section

3.2.3. Adjusting the FLOW-model

In this part, a more elaborated view is shown on how the model is adjusted. This involves all kind of input data for Delft3D. Explanations are given on why and how certain items are modelled as they are. The grid and bathymetry are skipped as they have been discussed above. For the complete model set up, see appendix C.

Time frame

The time frame simply tells the model from when to when it has to run. A certain reference date has to be filled in, next to the start date/time and end date/time. Next to the start and end date, the time interval also has to be given. The smaller the interval time, the more detailed the output, but the longer the computational time.

Concluding, a three day forecast is chosen as time frame for the model. The time interval between two computational steps is set to 1 minute. The WAVE-model, which is elaborated later on, runs together with the FLOW-model. Hence, it uses the same time frame.

Initial conditions

The initial conditions tell the model which values to start with in its computation. The parameters that need to be defined, depends on the processes taken into account. In the model that is built here, the following parameters need to be defined:

- Water level
- Salinity
- Water temperature

For simplicity, uniform initial conditions are used. The most important one is the water level. This is kept at 0 m, which resembles the mean sea level. This means that at all places where the bed level is below mean sea level, water is present at the start of the simulation.

Objects and boundaries

Special objects can be defined in Delft3D, for example dry points and thin dams can be implemented from the existing ocean model. These points had to be moved though. The internal coordinate system of Delft3D, working with (M,N) coordinates to indicate the different grid cells, has changed due to the extension of the grid.

The boundaries in the model are regulated at the east-side, north-side and west-side. These boundaries consist mostly of incoming tidal waves. Since the tidal constituents are not constant along the whole length of a boundary, they are subdivided into different sections. These sections are indicated with a number, so the north boundary is divided into 20 sections. Each section along the boundary has a different dependency on the tidal constituents.

In this case, boundary 'North1' is defined from North1A until North 1B. In the figure, it is shown how the point North1A is influenced by a whole list of tidal constituents, all defined with a different amplitude and phase. This is done for all the boundary sections, to make sure the model is reliable.

Observation points

Besides, the observation points of the model are moved to the right location as well. Some more observation points are added as the model only keeps track of the hydrodynamics of a certain grid cell at these points. Delft3D also shows some statistics on the whole grid, like the water level or water depth. Within the river there is a lot of interest about the hydrodynamics. Some additional observation points are added. Again, see appendix C for a more elaborated overview of the set up of the model.

Roughness

The roughness of the bottom must be defined as well in Delft3D. The roughness parameter allocated to the Indian Ocean in front of Durban is equal to a Mannings coefficient of 0.024 $s/m^{1/3}$. It is not sure whether this parameter is valid and applicable for the river section as well. Experiments have to be performed to determine the friction coefficient in the Umgeni river. For more information, see Chapter 6. For now, it is simply assumed that the roughness parameter in the river is equal to a Mannings coefficient of 0.012 $s/m^{1/3}$. Note that this value is used just to stress the fact that there is (most likely) a difference in roughness between the river and the ocean. This value is not a substantiated value for the friction in the Umgeni. A roughness file has been generated and used as input in the Delft3D-FLOW module.

Discharges

The discharge, which was currently inserted at the Umgeni river location has to be replaced to the location of the Inanda dam. The Inanda dam produces a discharge, which can be inserted in the renewed ocean model. Moreover, some tributaries flow into the Umgeni river as well. These need to be modelled as a discharge at the location where the tributary flows into the river. In figure 3.7 it is shown where the locations are of the significant tributaries, i.e. the tributaries which are modeled as a discharge in Delft3D.

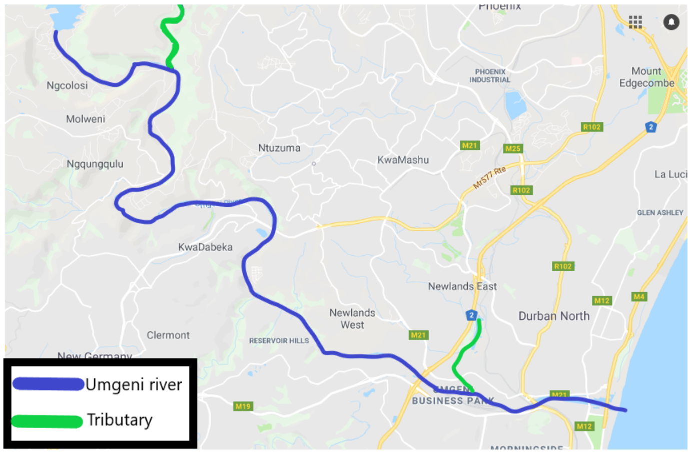


Figure 3.7: The locations of the tributaries

These discharges are modelled in so called dump points. This is a specified point on the grid from where water can be added into the system. The outflow is specified as a time series with a certain volume of water. This is used for both the Inanda dam as well as the tributaries.

3.2.4. Adjusting the WAVE-model

The Delft3D-WAVE model is present from the existing ocean model. This module uses SWAN to compute the wave propagation through the model. The same mechanisms regarding the waves are applicable for the renewed model as for the existing model. Explanations for the input are given below. Obstacles are not used in this model.

Hydrodynamics

The first main option for WAVE is how it should treat hydrodynamic conditions, as these are not generated by the module itself. There are three options for this, specified below:

1. Run FLOW and WAVE together: This is also called an online simulation, as both FLOW and WAVE run and the same time and share input and output. First a time step is done in FLOW. The hydrodynamic output is taken into WAVE and the waves are computed for the same time step. Then the output is taken into FLOW and the next time step is done, and so on.

2. Run WAVE after FLOW: This is called an offline simulation. First a full FLOW simulation is done. The right file of the FLOW simulation is selected in the WAVE module. It then takes the output of the FLOW simulation as input in WAVE. This means WAVE is based on FLOW, but not the other way around.

3. Run standalone WAVE: In this case, the hydrodynamic conditions must be manually specified in the FLOW module. This is done in the fields of the FLOW GUI.

The case of Durban is quite complex compared to other locations around the world where similar models have been built. There are many factors that significantly influence each other as well as the results. Unlike in other cases, no influences (e.g. currents, waves, etc) can be neglected here. Therefore the online simulation is carried out here, so with FLOW and WAVE running together. These calculations are performed in a non-stationary way. In doing so, a two-way coupling can be made allowing an iterative process.

Grid and Bathymetry
The WAVE module of Delft3D also needs a specified grid and bathymetry to work with. In this case, the same files are used as for the FLOW module. This automatically means the river is included in the wave computations. This is initially done to investigate the influence of waves on the river. In addition to this, nested grids can be used to create a locally more refined grid. A more detailed grid automatically means more detailed computations and results. This therefore is good method for places of high interest. For now, no nested grids are used however, as the main grid already is refined at the river.

Also the spectral resolution for the grids has to be defined. Here, the wave directions and wave frequencies that should be included must be specified. These are taken from the existing model.

Time frame
As the FLOW and WAVE simulations run together in Delft3D, the time data from FLOW is automatically adopted in WAVE. Only a water level correction must be specified, which is by default 0 m.

Boundaries
Wave conditions are specified at the boundaries of the computational grid. First a location is set for the boundary by selecting an orientation (north, east and west). Next, the following (uniform) wave conditions are specified per boundary:

- Significant wave height [m]
- Peak period [s]
- Direction w.r.t. the geographical north [deg]
- Directional spreading [-]

These values are usually filled in based on the data retrieved from the WaveWatch. Here some arbitrary values are used in order to estimate the influence of the waves on the Umgeni river. Next to this, the spectral boundary conditions are required. Here, the default input is used again.

Physical and numerical parameters
These parts give additional input in forms of numbers and computation methods. The input contains water properties, wind, wave propagation processes and wave energy dissipation processes. Mostly default values are used, only the wind has to be specified as this is not taken from the FLOW module. Uniform wind conditions are used for now.

Output curves and parameters
Here, the way the output is generated and displayed can be chosen. No output curves are used. The computational mode is set to stationary and the output is generated for the FLOW grid as well as the computational grid (the same). Also additional locations of interest can be specified at which the results are wished to be seen.

Delft3D now knows the incoming wave conditions. Together with this, it also takes wind speed and wind direction into account during its calculations. Consequently, the program can calculate the wave propagation throughout the model, once the grid and the bathymetry of the model are inserted as well. The grid and bathymetry of the model are similar to the ones used for the FLOW-calculations.

The following output for the significant wave height is generated by Delft3D at a certain time step.

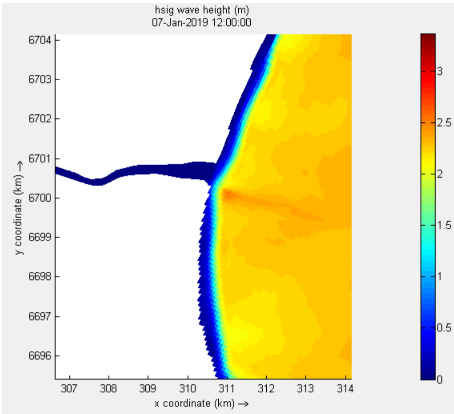


Figure 3.8: The H_s values in the model at the certain time step

Figure 3.8 shows the significant wave height in the model, zoomed in at the river mouth. When looking carefully, it can be observed that the waves propagate into the river for a very short distance, and that they fade out very soon. Because of the scarce influence of the waves on the river, the assumption that the waves do not propagate far into the river seems to be valid.

Therefore, it is plausible to let the Delft3D-WAVE module only run on the ocean grid. This saves a lot of computational time, since the refined river grid demands a lot from Delft3D-WAVE, whereas there is barely any wave propagation in the Umgeni river. In the final model set up, the original wave grid is used to run in cooperation with the FLOW-module, which does run with the extended grid.

In Delft3D-WAVE, it must be specified what hydrodynamic results from the FLOW simulation should be used. The water level, current and wind is used from the FLOW-model, since this gives a more complete estimate as it takes the rivers influence into account. It does not use the bathymetry from the FLOW-model, since it runs only on the wave grid and the morphodynamics are not taken into account. Hence, no influence of the river on the oceans bathymetry is present.

Concluding, for the renewed model:

- The boundary conditions at the ocean have not changed and can therefore be maintained.
- The updated version of the grid and bathymetry does not need to be inserted in the WAVE-model. The ocean grid and bathymetry suffice in the wave calculations.

- The same mechanisms are taken into account as in the existing ocean model, because it is assumed that the wave propagation into the river is negligible. This assumption is made because the river is very narrow compared to the ocean. NB: Figure 3.8 shows that this assumption is rather valid.

3.2.5. Output of Delft3D

In this paragraph, the output of the renewed model is shown. In the next paragraph, some conclusions are drawn based on this output, including the physical explanations and meanings of the plots.

Delft3D uses the tool 'QuickPlot' to show and animate the results of the model. The program creates certain output files, which can be opened with this tool. Within this tool, the (hydrodynamic) statistic of interest can be chosen and printed to the screen at a certain time step. It is also possible to show all time steps. QuickPlot then shows an animation. A screenshot of such an animation looks like in figure 3.9.

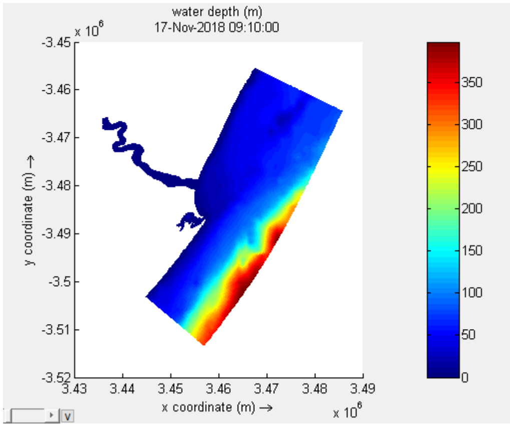


Figure 3.9: A screenshot of the water depth at the certain time step over the whole grid

Tidal influence at the river mouth

Next to the information about the total grid, it is also possible to look at the hydraulic statistics at the observation points. These observation points were defined earlier, as mentioned in paragraph 3.2.3. The information at these observation points can also give a better insight in the influence of the tidal effects on the river. In figure 3.10 the water level elevation near the Umgeni river mouth is shown. The model computes data for three days, as can be seen on the horizontal axis. The vertical axis shows the water depth.

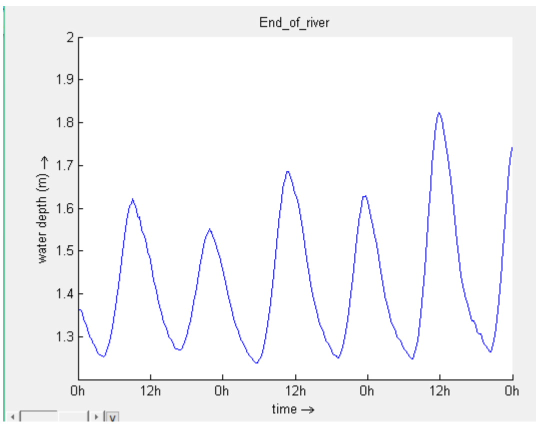


Figure 3.10: The water level elevation near the river mouth

Delay of the tidal wave
The plot below shows the water level elevation at the whole grid. The plot is zoomed in on the harbour area and the Umgeni river entrance. In the next paragraph, some conclusions can be drawn regarding a delay of the tidal wave.

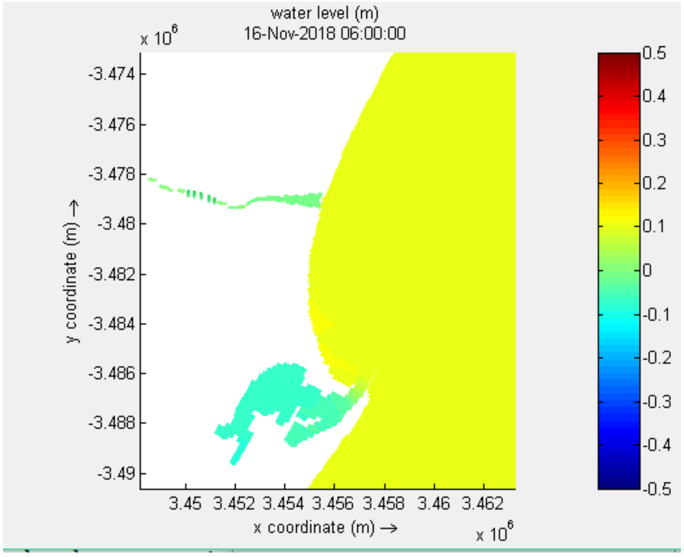


Figure 3.11: The delay of the tidal waves

Reach of influence of the ocean
An important use of the model is that it can show the influence of the ocean on the Umgeni river. Under normal conditions, this is due to the tidal behaviour of the ocean. This is also what is modelled here. The tidal waves flow in through the river mouth and then further upstream, causing water level changes along its way. Figure 3.12 shows the amplitude of the tidal wave as a function of travelled distance. The graph is based on several observation points, from which the output is used. As these measurements are quite rough and only few in number, a trend line was added to give a more smooth graph for the progression of the tidal wave.

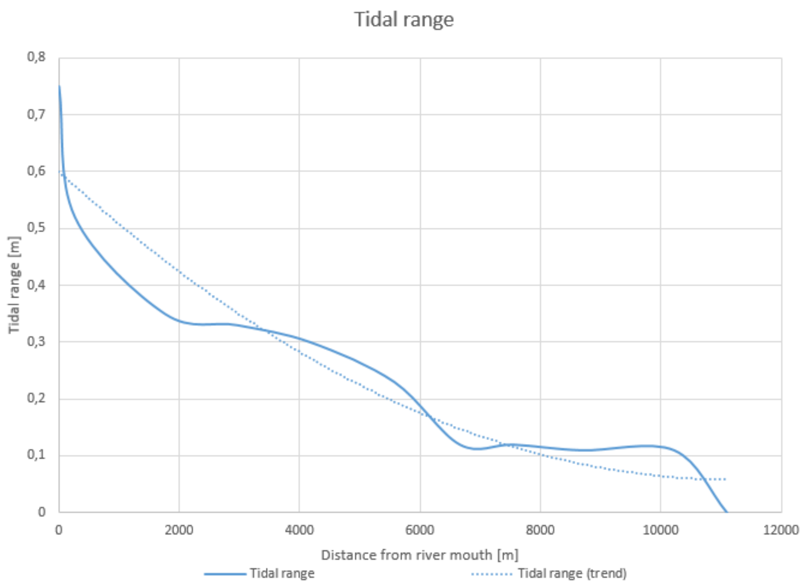


Figure 3.12: The tidal wave propagating up the river

Discharge and ocean influences

The moment of interest is when both the tidal waves and a discharge from the Inanda dam are present. This is because floods are most likely to happen during these circumstances, especially concerning the flooding of the river banks. The discharge accessory to the output presented below was here set to $Q = 20\ m^3/s$. The plot shows the propagating discharge at a certain time step in figure 3.13 below.

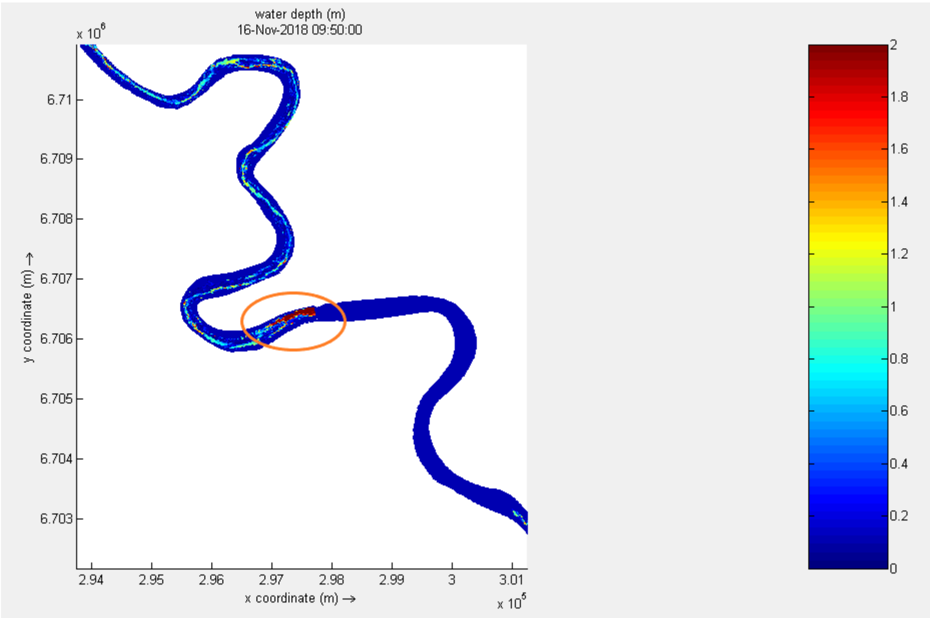


Figure 3.13: The propagation of the flood wave, circled with orange

Next to information about the propagating wave from the Inanda dam, there is also information on the observation point halfway the river. Below, the graph for the water depth halfway the river is shown.

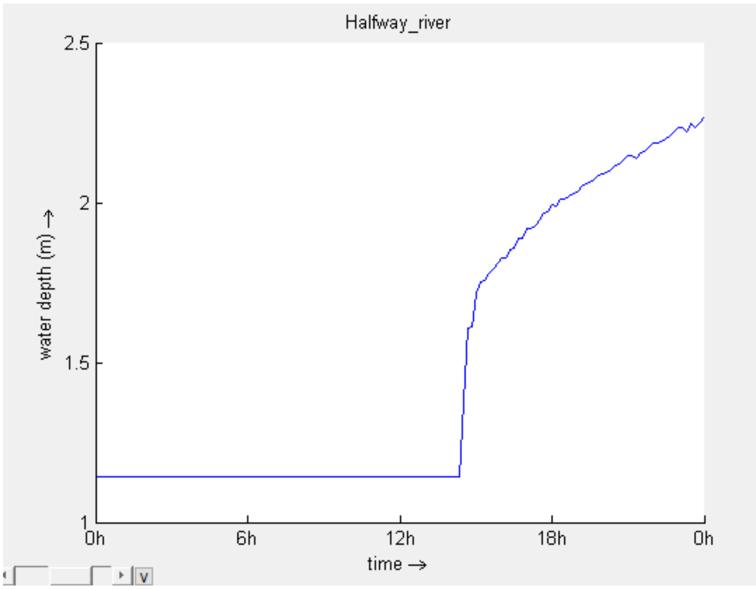


Figure 3.14: The moment where the discharge arrives halfway of the river.

Tributaries
The tributaries from the Umgeni river have an additional discharge flowing into the model. These two main tributaries and its locations were already discussed before in paragraph 3.2.3 under **Discharges**. Next to these two additional 'discharge operations', even more (smaller) tributaries flow in as well. For the extent of this project, only these two main tributaries are taken into account and the result is visible below. The first figure shows a plot/map of the moment just before the two discharges meet and the second figure shows the total time-series of the water depth at the observation point nearby.The tributary shown was set to start flowing after 12 hours of simulation with a discharge equal to Q = 10 m^3/s.

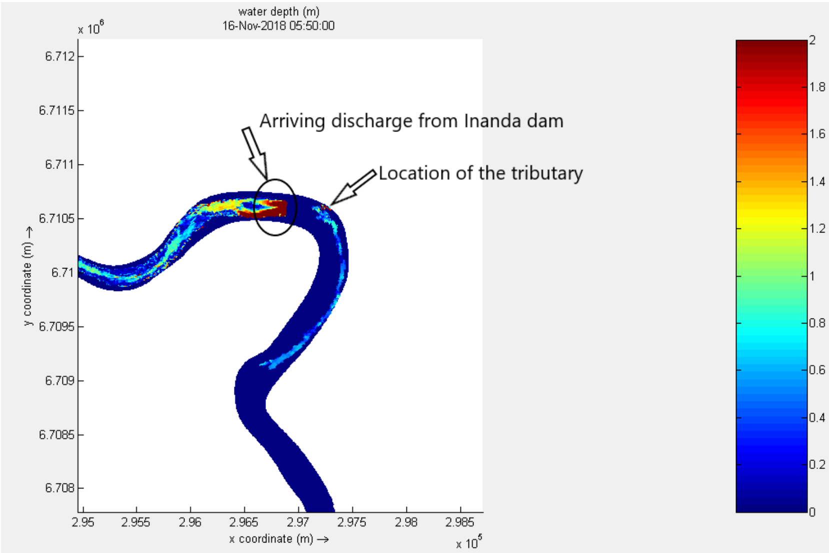


Figure 3.15: The moment before the Inanda discharge and the tributary flow meet.

3.2.6. Conclusions regarding the extended Delft3D-model
In this paragraph, the conclusions that can be drawn from the Delft3D output are discussed. Certain aspects are elaborated and explanations for the phenomena visible on the output graphs are given. The main emphasis in this paragraph is on evaluating the qualitative results, i.e. does the shape of the model output make sense? In section, 3.4, the validation is performed, where the quantitative results are evaluated.

- **The tidal influence at the river mouth:**
  If a harmonic signal is visible, the tide still influences the river at that observation point. This is also clearly visible at the observation point at the end of the river. See figure 3.10. This means that the statement that the river delta is (partly) tidal dominant is supported by the output of the renewed Delft3D model. The semi-diurnal tidal wave can be seen in the figure, with two high-waters and two low-waters a day. Again, this plot emphasizes the influence of the ocean on the downstream end of the Umgeni river section.

- **The delay of the tidal wave:**
  When considering the animation in QuickPlot in figure 3.11, which plots the water levels of the whole grid, a delay in the tidal wave can be observed. The tidal wave needs time to reach certain areas, like the upstream reach of the river mouth. This delay was already visible at the port of Durban. According to the model, the tidal wave also propagates with a certain delay into the Umgeni river. This effect is visible in the figure. The oceans water level (yellow) is rising from ebb to flood at this certain time step. When looking at the port of Durban, a more blue color can be seen. This means that the water level is

still low, i.e. in its ebb tidal phase. See the colour bar to allocate certain colours to water levels. The Umgeni river is more or less green, a mix between the ocean and the port. Hence, the conclusion can be drawn that the delay for the tidal waves are smaller in the river than at the port. This is physically understandable, because the port is more sheltered from the ocean than the river. The port has a narrow inflow area compared to the basin size.

- **The reach of influence of the ocean:**
  As mentioned in the previous paragraph, the plot considered here is the result of a run with rather 'basic' conditions, i.e. not a storm for example. Besides, the river discharge was set to zero. Even during these 'basic' conditions, the backwater curve in the river reaches relatively far. The reach where a water level higher than zero can be observed in figure 3.12. Looking at the distance along the axis, it can be concluded that the tidal wave reaches to just before the transition zone, which was to be expected. See figure 3.16. This is because river sections with a very mild bed slope have a very long adaptation length. More information on this is given in paragraph 3.3.
  It can be seen that the amplitude at first decreases rapidly. This immediate decrease can be explained by the narrow flow channel at the river mouth. A large amount of the wave energy dissipates here. Further upstream, the decrease in amplitude is more gradual. The decrease here is mainly due to friction.

- **Combined discharge and ocean influences:**
  The discharge can be regarded as a 'flood wave', as a large volume of water is released instantaneously. This behaviour is clearly visible in figure 3.13. The red coloured area is the area with the largest water depth. This area keeps clustered together, like a flood wave would do. At the tail of the flood wave, an elevated water depth remains.
  Now consider figure 3.14. After a certain while, the flood wave arrives at the observation point halfway the river. The water depth halfway the river should get an abrupt increase at the moment the discharge arrives. Then, it is expected that the water level increases more slowly to its equilibrium depth, since the imposed discharge remains present during this whole simulation. Again, this behaviour is visible in figure 3.14.

- **Tributaries:**
  The results of the tributaries flowing into the Umgeni are shown in figure 3.15. It can be concluded that Delft3D models the two different discharges correctly and that both discharges travel downstream.

Concluding, the most important result from this section is the fact that the backwater curve can have a significant reach of influence due to the downstream boundary condition imposed by the oceans water level. The renewed model can take this influence into account.

In the next section, an analytical method is evaluated in order to estimate if the results are of the same order of magnitude.

3.3. Analytical check backwater curve

The backwater curve can be calculated using the empirical fit of Bresse. This fit isn't as precise as the Delft3D model, but it can give a good and very fast indication of what the water levels approximately are. Python Jupyter is used for programming the backwater curve.

A backwater curve occurs in the river. The influence on the adaptation length depends on the bed slope, the equilibrium depth of the river and the water level at the river mouth, opposed by the ocean. In a formula:

$$L_{1/2} = 0.24\frac{d_e}{i_b} \cdot \frac{d_0}{d_e}^{4/3} \tag{3.1}$$

When the longitudinal profile of the Umgeni river is known (until the point of the Inanda dam, because the BW-curve can never go beyond this point) the backwater curve can be calculated real time. The profile of the water level, according to the empirical fit of Bresse, looks like:

$$d(x) = d_e + (d_0 - d_e) \cdot 2^{\frac{x-x_0}{L_{1/2}}} \tag{3.2}$$

$$d_e = \left(\frac{q^2 c_f}{g i_b}\right)^{1/3} \tag{3.3}$$

In this way, the only time-variables are the specific discharge and the water level at the ocean. The specific discharge is predicted by means of the river model, while the water level at the river mouth is predicted by means of the ocean model. The only difficulty in using this approach is the determination of the friction coefficient c_f, the bed slope i_b, and the river width, because these three values are not constant for the whole Umgeni river.

3.3.1. Bed slopes and river widths

When looking at the longitudinal profile of the river, different segments can be distinguished where it is plausible to assume a constant bed slope i_b. Each segment then gives the boundary condition d_0 for the segment upstream. In this way, a continuous water level can be calculated which is the most accurate considering the parameter supply. The longitudinal profile of the Umgeni river is shown in figure 3.16. The longitudinal profile of the river is retrieved with the program QGIS, using provided GIS data of the eThekwini municipality. Considering the longitudinal profile, it is valid to assume two sections in the river with two different bed slopes.

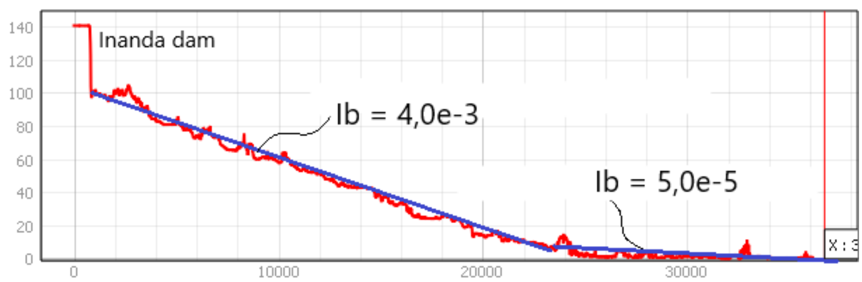


Figure 3.16: The longitudinal profile of the Umgeni river

The model in Python can be divided into two sections according to the bed slope. However, the width of the river also plays an important role, since it determines the specific discharge and thus the equilibrium depth. Each section defined according to the bed slope, can be divided into another two sections with different river widths. The total number of sections then becomes four. These widths are also determined based on the data in QGIS. A total overview of the sections distinguished in the analytical model:

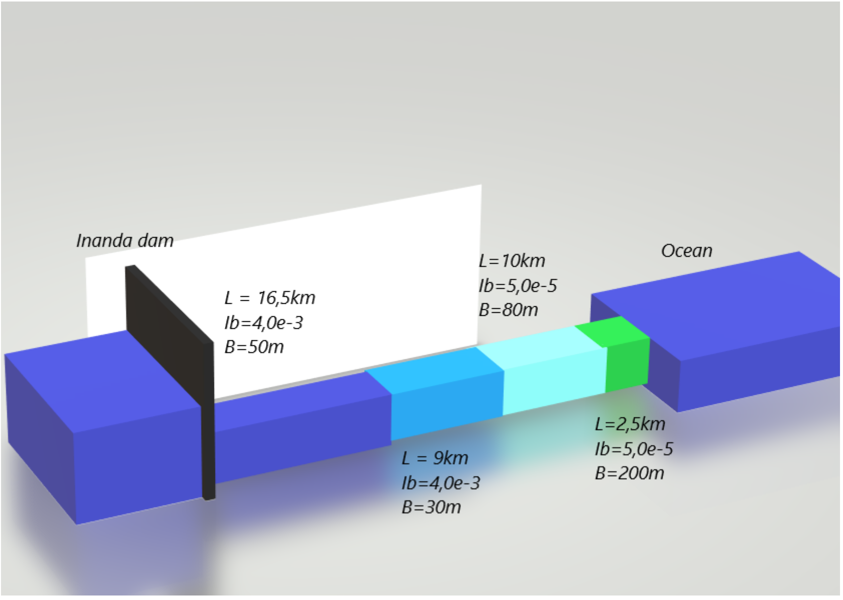


Figure 3.17: The characteristics of the sections

3.3.2. The friction coefficient

For all sections a friction coefficient c_f needs to be determined. This can be achieved, once the specific discharge and the corresponding water level is known. When these data is available, an iterative method can be applied using the White-Colebrook relation, and the friction coefficient is known. However, these data are not available, so a different method needs to be applied. The Manning formula can be applied, since the Manning coefficient is known at 0.024 from the existing models. Consequently, the Manning coefficient n needs to be coupled to the friction coefficient c_f. This is done in the following way:

$$\text{Chézy}: \; U = C\sqrt{Ri_b} \tag{3.4}$$

$$\text{Manning}: \; U = \frac{1}{n}R^{2/3}i_b^{1/2} \tag{3.5}$$

When these two equations are set equal:

$$CR^{1.2}i_b^{1/2} = \frac{1}{n}R^{2/3}i_b^{1/2} \tag{3.6}$$

$$CR^{1/2} = \frac{1}{n}R^{2/3} \tag{3.7}$$

$$C = \frac{1}{n}R^{1/6} \tag{3.8}$$

Now the relation between the Chézy coefficient C and the friction coefficient c_f is inserted:

$$C = \sqrt{\frac{g}{c_f}} \tag{3.9}$$

Resulting in the final link between c_f and n:

$$c_f = \frac{g}{\left(\frac{R^{1/6}}{n}\right)^2} \tag{3.10}$$

Given the Manning constant from the existing model, the only unknown yet is the hydraulic radius R. For very wide rivers, i.e. where $B >> h$, the hydraulic radius can be approximated with $R = h$. The water depth at the river mouth of the Umgeni river is known using measurements. See figure 3.18 for the water level variation in a range of a few days. Note that the tidal effect is clearly visible in the data.

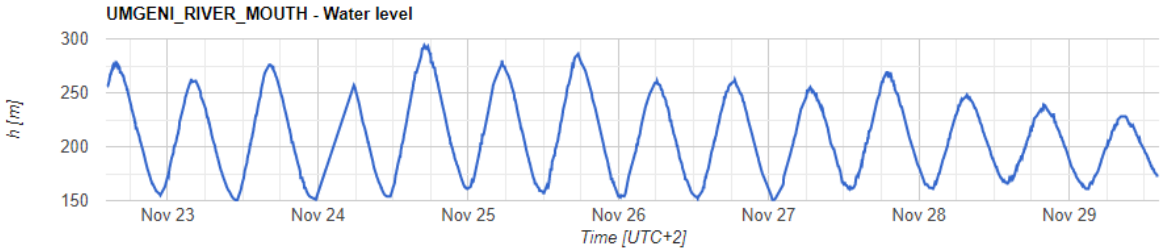


Figure 3.18: The water level variation of the Umgeni river mouth

In this analysis, it is assumed that a water depth of 3 meters is representative. When considering a longer range of data, this turns out to be acceptable. Applying this value for R and the Manning constant $n = 0.024\ s/m^{1/3}$, the friction coefficient becomes:

$$c_f = 0.0039$$

It is assumed that this value is valid for the whole downstream river reach.

3.3.3. Discharge

Next to the water level at the river mouth, i.e. the oceans water level, the discharge is another input for the model in Python. Similar to the Delft3D model, the discharge may vary over the river length due to the tributaries flowing into Umgeni. This causes a different discharge over the total river length, which has to be accounted for. Since this rather simple model is only divided into four river sections with constant characteristics, it is not possible to add the discharges at the exact location. The four river sections defined at the model can each have a different discharge. In other words: the input 'discharge' is an array consisting of four values. Therefore, the inflow of the tributaries is only added from on certain river sections. Right now, only the two tributaries that were evaluated in the Delft3D model are taken into account here as well.

- Tributary 1: This concerns the top tributary in figure 3.7. An additional discharge is flowing into the Umgeni river at 4.5 km downstream of the Inanda dam. The first river section (darkest blue in figure 3.7) has a total length of 16.5 km. Hence, the additional discharge is present over most of this river section and is therefore added to the first river section already.

- Tributary 2: This concerns the lower tributary in figure 3.7. This tributary is located approximately 4.5 km upstream of the river mouth. This means that the additional discharge is present over the full length of the fourth/lowest river section, and present for only the last 2 km of the third river section. The third river section has a total length of 10 km and is thus present for only the last 20 %. Therefore, the additional discharge due to this tributary is only present in the fourth river section in this Python model.

In this way, the tributaries and differing discharges are modelled in Python.

3.3.4. The model in Python

In appendix D, the code for the model in Python can be found. The model includes a warning level, here set to 6 m for indicational purposes. Moreover, the existing warning system which the eThekwini municipality handles is implemented in the Python model. This means that a 'Watch, Alert and Warning' signal is given when 70%, 80% 90% of the warning level is exceeded, respectively. The outcome of the model looks like:

```
In [35]: BWcurve(Q=800,d0=4)
```

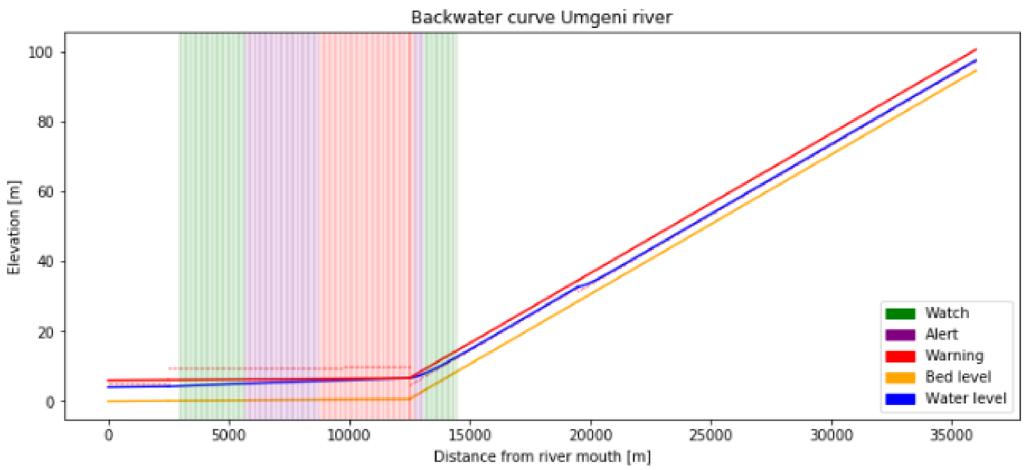


Figure 3.19: The outcome of the models BW-curve function

As already mentioned, the model is divided into four sections with each its own characteristics. The model starts at the river mouth (x=0) and calculates the backwater curve based on the oceans water level. Then, the second segment uses the last water depth from segment one as a d_0-value, etc. The vertical axis represents the absolute elevation. The horizontal axis shows the distance from the river mouth to the Inanda dam. Note that the horizontal axis is reversed compared to figure 3.16.

As can be seen, the only input is the discharge and the depth at the river mouth. This data is available from the Delft3D models, so an analytical check can be done using the Python model.

3.3.5. Conclusion

The conclusion based on this analysis, is that the influence of the downstream boundary can have a very big impact on the water levels in the Umgeni river. This can be explained by the very mild slope at the lower region of the Umgeni river. This causes a very high adaptation length $L_{1/2}$. Therefore it takes a long distance for the river to get the imposed water level at the river mouth towards the equilibrium depth. Even if the river transports barely any discharge, it can still be in a dangerous situation at the lower region of the river. See figure 3.20 for this situation. The very low discharge $Q = 10\ m^3/s$ causes very low water levels upstream, but the boundary condition $d_0 = 5\,m$ has a serious impact at the lower region.

```
BWcurve(Q=10,d0=5)
```

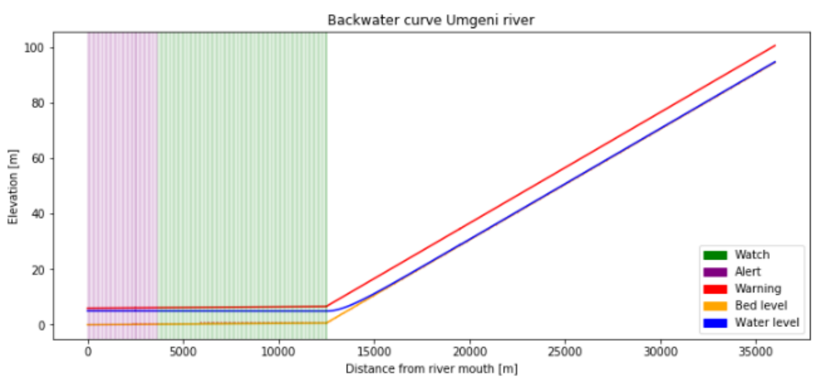


Figure 3.20: The water levels at very low discharge

From the moment where the bed slope starts steepening, the river manages to bring the water level quickly back to its equilibrium depth. This is again understandable given the formula for the adaptation length. Concluding: the link between the ocean and the river is very important for 12.5 km upstream of the Umgeni river mouth and this can be modelled using the Python script.

3.4. Validation

Now the models are there, it has to be checked whether the output makes sense compared to the reality or not. In doing so, the models output has to be compared to the measurements made in reality. For the Umgeni river, the eThekwini municipality has a measurement point at the river mouth. The model has observation points at this location. A direct comparison between model and reality can now be made. Ideally, there are more of these comparison points present throughout the river. However, the scarce availability of measurement points within the Umgeni river makes it hard to validate the models results. Still, this point is used in validating the model. It is now already recommended to validate the model on a more detailed level, i.e. with more measurement points, before it could be used in practice. A more elaborated recommendation is given in chapter 6.

Note that the validation mostly involves the model in Delft3D. This model is the most accurate one in terms of river dimensions and bathymetry data. The model based on the empirical fit of Bresse, written in Python, includes some rough estimates in terms of bed slope and river width. Hence, these results are less reliable by definition and an intensive validation of this model would be redundant. Moreover, there are no measurement points present to validate the model on. The only measurement point present covers the input of the Python model, i.e. the water depth at the downstream boundary condition. However, at the end of the validation, it is checked in what extent the Python model does coincide with the Delft3D model.

3.4.1. Validating the Delft3D model

The approach of the validation is as follows. The input of the model are the measured values for the discharge at the Inanda dam, the wind speed at sea and the direction of this wind speed. These values are assumed correct, since these values are measured instead of predicted like when the model is operating for real. Therefore, the deviations in model results and reality can be attributed to the input in a lesser extent. The deviations which are present can be attributed to certain parameters/attributes of the model. The main parameters/attributes which can cause the deviations between model and reality are listed below. Besides, a small hypothesis per parameter is given in order to estimate certain effects on the output.

- **The bathymetry**
  The bathymetry as it is used right now consists of two parts pasted together. A very dense survey has been done near the river mouth. This survey reaches up to about 6 km into the Umgeni river. From this point on, GIS data is used for the bathymetry, which is less dense. Also the water surface is measured instead of the bed level. (Note that a correction has been applied to this bathymetry file). It might be that the bathymetry points allocated to the grid cells are not precise enough.

- **The grid**
  The grid in the river is already quite dense compared to the grid in the ocean. However, it might be that the grid is still not refined enough for Delft3D to calculate the output precise enough. Though, there is no use in refining the grid more for now. This is because the 'less refined' bathymetry, mentioned above, is not refined enough for a more refined grid. Remark: this holds only for the part more than the 6 km upstream of the river mouth. The 'denser survey' is refined enough for a more refined grid. Besides, Delft3D is struggling to run simulations on a very refined grid.

Especially the river inlet is a critical part. The connection of the Umgeni river with the Indian Ocean is quite narrow, during low tide even only around 10 m. Ideally, a grid cell located here would be smaller than this width, which was found to be impossible however. Therefore, the inflow of the ocean into the river isn't modelled fully accurate. This can lead to wrong volumes of water flowing into the river and thus water levels. This also influences the reach of the ocean into the river.

- **The friction coefficient**
  The friction coefficient for the river is now simply assumed to be identical to the one for the ocean. Whether this assumption is valid or not, is yet to be determined through experiments. It goes beyond the extent of this report to check the friction coefficient of the river. The friction coefficient does influence the water level, flow velocity and hence river discharge. This parameter is very important in the output and likely to be the cause of any deviations in model output and reality.

Model output vs reality
Now the model runs a simulation in order to compare the model output with the reality. The simulation on which the validation is done takes two days, in this case the 6th and 7th of January 2019. In the figures shown later in this paragraph, it is important that one only looks at the data from these dates.

The input for the model is gathered from measured data. The wind speed and direction are retrieved from a weather station, which is in possession of the eThekwini municipality. The closest weather station in the region is located at Durban North, only 1 km away from the shoreline and 3.5 km North of the Umgeni river. It is assumed that this weather station gives acceptable input regarding the wind speed and direction for the ocean-river model.

The contribution of the wind in the wind induced wave set-up can be modelled by a constant wind over certain time frames. In general, the energy input from the wind forcing can be schematized as in figure 3.21.

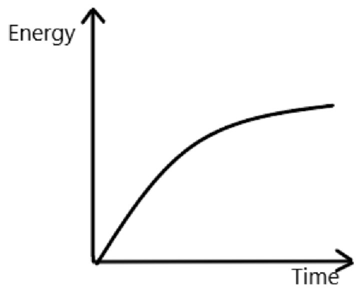


Figure 3.21: The energy input due to wind over time

In a formula:

$$\frac{dE}{dt} = S_{wind} \tag{3.11}$$

Where S_{wind} is the source term for the wind, i.e. the energy input due to the wind. When integrating this formula, for the time related frame it can be concluded that the energy transfer depends linearly on the time which has passed. In formula:

$$\Delta E = S_{wind} * \Delta t \tag{3.12}$$

Hence, the wind needs time to transfer energy to the water surface to build up waves consequently [30]. Therefore, wind gusts do not play a significant role in the formation of wind waves. However, the wind speed measured at the weather station has a lot of small 'time varieties'. These can be easily spread out to gain a constant wind speed over time. For the same reason, the same can be done for the wind direction. In this way, certain time frames are defined in which a constant or linear wind speed/direction can be defined. Both constant and linear functions are allowed, because these type of functions are the ones which Delft3D can cope with.

Measurements and input
When considering the measurements of the weather station, some time intervals can be defined with equal characteristics regarding the wind speed and the wind direction. See figure 3.22

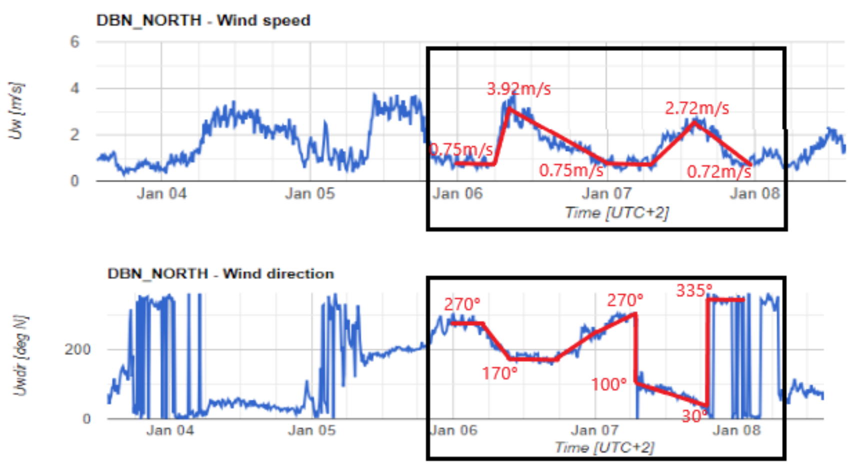


Figure 3.22: The measured wind data

As can be seen in the figure, some values are addressed for the wind speed and direction. The function of such a time interval is either constant or linear. The values for the wind speed and direction over time have both been inserted in Delft3D-FLOW. In the Delft3D-WAVE module, it is defined that the program should use the wind data from the FLOW-module in order to compute the wave development. For now, the WAVE-module is turned off since it barely influences the hydraulics in the river, as stated before.

The discharge flowing through the Inanda dam is another input item which can be gained from measurements. However, these measurements are not present from the municipality. Umgeni Water Amanzi does keep track of the discharge flowing through the Inanda dam [3]. This is a state-owned institute responsible for the water management in this area. Opposed to the data on the wind speed from the municipality, the data on the discharge is less extensive. Amanzi just gives a certain outflow per day, which must assumed to be constant throughout that whole day. The discharge flowing through the Inanda dam for the days of interest is equal to $Q = 0.78\ m^3/s$. Unfortunately, there is no data available on the discharges coming from the tributaries. So again, a lack of measurements makes the validation more complicated.

The model is started already on the 6th of January, so one day before the day of interest, because the model needs some time to 'warm-up'. This means that the model needs some time to fill all the grid cells with water and to reach towards its equilibrium state. This can also be done using a so-called hot-start file, but for the validation it is now chosen to just start the simulation earlier. The measurements at the river mouth look as shown in figure 3.23.

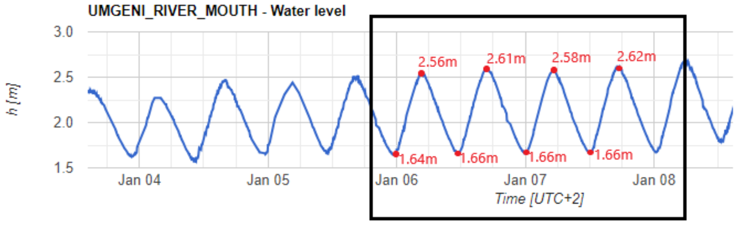


Figure 3.23: The measured water levels at the Umgeni river mouth

The graph inside the red box in figure 3.23 shows the water level variation of the Umgeni river mouth during the 6th and 7th of January 2019. The tidal behaviour is clearly visible in the graph. The maximum and minimum values are explicitly shown in red in order to make a good comparison between the model output and the measurements. It is explicitly stated that this graph only gives insight in the water level variation over time and not in the water level/depth itself. The measurement station is fixed at the bottom of a bridge and measures the distance from the bottom of the bridge to the water surface. Hence, it only gives insight in the water level variations. This means that the tidal range predicted by the model can be compared to the real tidal range.

Model output

The model generates the following output for the 6th and 7th of January 2019. This has to be compared to figure 3.23.

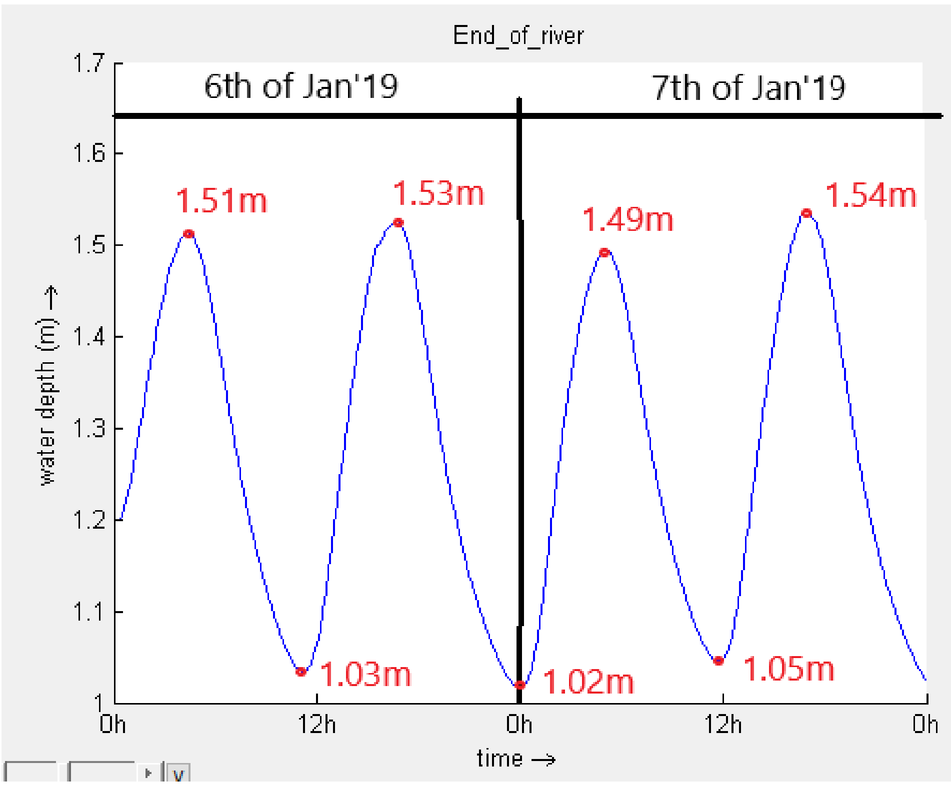


Figure 3.24: The Delft3D model output at the Umgeni river mouth

Comparison model and measurements
Now, the model and the measurements are compared on different aspects with importance for the model. Again, the emphasis of the comparison is on the 7th of January 2019, since the model needs some time to 'warm up', causing the start of the model to be slightly unreliable.

- **Phase of the tidal wave**:
  The phase of the tidal wave shows when during the day it is ebb or flood. When the high and low tide differ from reality, the phase of (one of) the tidal constituents is wrong. When comparing figure 3.23 to figure 3.24, it can be seen that the day starts with low tide. Consequently, the tidal flood wave comes in and a high tide can be observed in both the model and the measurements. This process goes on twice a day, which can obviously be explained by the semi-diurnal tide. The results from the model match reality regarding the phase of the tidal wave. Hence, it can be concluded that the phase of the tidal wave is modelled properly.

- **Daily inequality in the semi-diurnal tide**:
  The daily inequality between two low tides and high tides was already visible from the measurements in figure 3.23. The second flood wave is higher than the first flood wave, for both the 6th and the 7th of January. Considering the models output, the same phenomenon can be observed. The second high tide of the day has a higher water level. This basically is another confirmation of the correct modelling of the phase of the different tidal constituents.

- **Tidal range**:
  On the contrary to the previous two items, the tidal range is not modelled properly. The tidal ranges observed at the measurement station are equal to 0.92 m and 0.96 m for the both tidal waves respectively on the 7th of January. The tidal range calculated by Delft3D are equal to 0.47 m and 0.49 m. These values don't coincide and refer to a wrong definition of the amplitudes of (some of) the tidal constituents.However, the renewed model is not necessarily the cause of this problem. Similar issues have been observed at the existing model running. The calibration of the model, which still needs to be executed, can be held responsible for the differences in tidal range. Therefore, this calibration first needs to be done for the ocean model itself before any conclusions can be drawn on the correctness of the renewed model.

3.4.2. Validating the Python model

The model in Python can be compared to the model in Delft3D. Unless the fact that the tidal range has not been calibrated yet in the Delft3D model, it is still possible to compare the two models. At a certain time step, a certain water depth at the Umgeni river mouth according to Delft3D can be used as input for the Python model. Then, the same discharge is inserted in the river in both models. Doing so, gives similar conditions in the river for both models. Python then calculates the water level elevation along the river. This elevation can be compared to the calculations done by Delft3D. By exporting the water level data from Delft3D manually, a longitudinal profile can be created.

Note that the backwater curve needs time to establish an equilibrium situation. Therefore, the water level at the Umgeni river mouth is set to 3 m throughout the whole simulation in Delft3D. In this way, Delft3D has got the time to reach towards the desired equilibrium situation, which is equal to the situation that the Python script shows. Since the bed level at the Umgeni river mouth is 1.5 m below MSL, the water depth at the downstream boundary condition is equal to 4.5 m. This is quite a high value. The discharge calculated with in both models is relatively low, and equal to 5 m^3/s.

Results
Since the backwater curve fades out rapidly once the 'steeper' bed level is found, the graph in Python is zoomed in from the river mouth to just after this transition point in bed level. The graph is shown in figure 3.25.

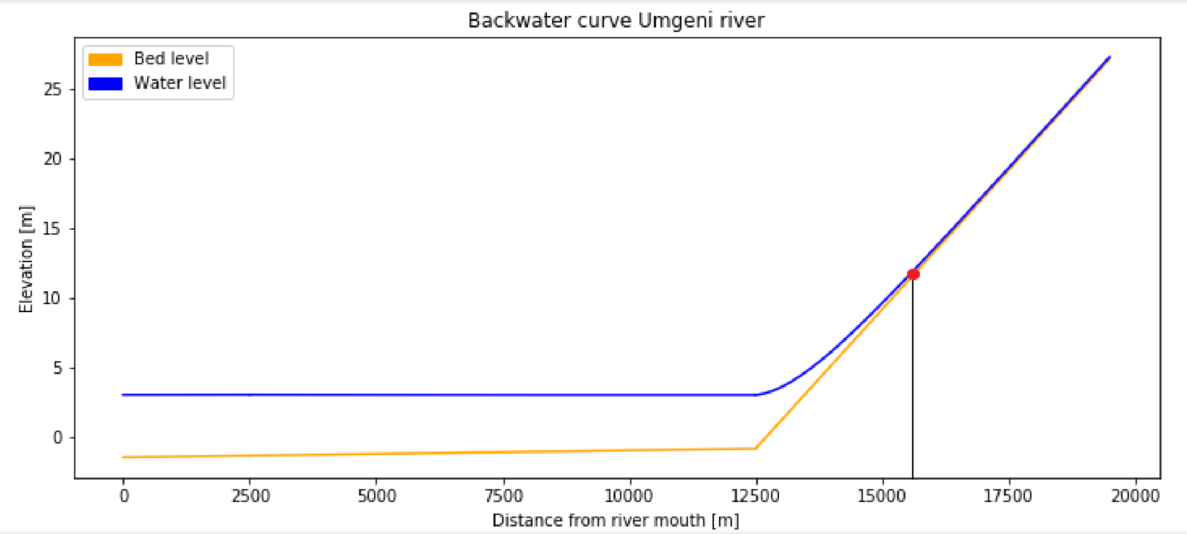


Figure 3.25: The longitudinal profile according to Python

It is visible that the water level remains quite constant at $MSL + 3m$ until the transition point at about 12.5 km upstream of the Umgeni river mouth. After this, it takes the water level about 3 km to 3.5 km to drop back to approximately zero. A very low water level remains because of the fact that the small discharge of 5 m^3/s is present.

Now, the data yielded from Delft3D is exported. It took the model a couple of hours to reach towards its equilibrium situation, so the output has been plotted at a certain time step where the equilibrium had already been met. For the result, see figure 3.26.

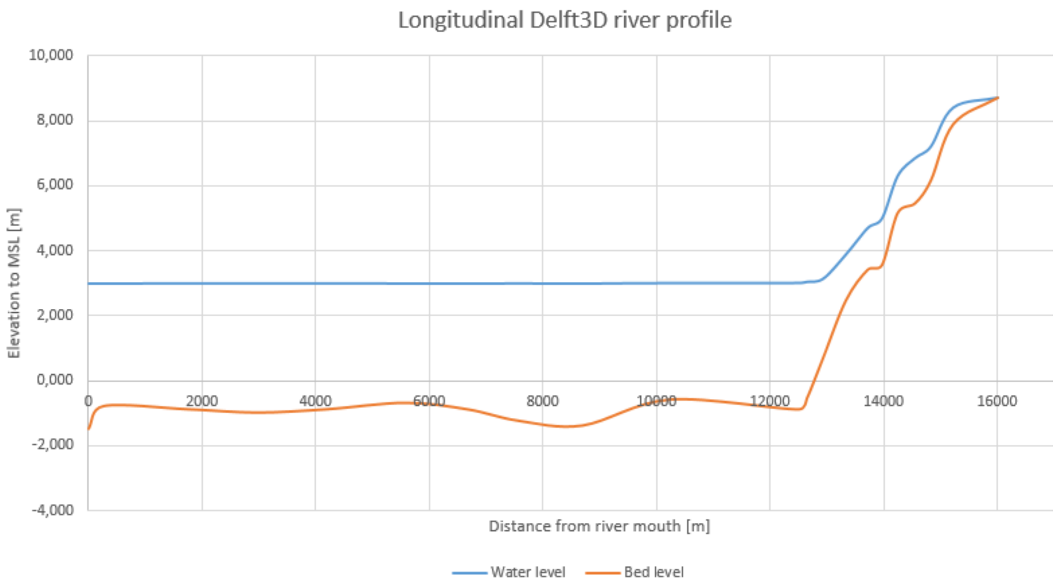


Figure 3.26: The longitudinal profile according to Delft3D

Again, the same water level of $MSL + 3$ m is visible according to the Delft3D model. Some more deviations are visible in the bed level, which is understandable since Delft3D uses a detailed bathymetry file instead of an interpolated linear function. Moreover, the transition point in bed level is clearly visible as well. Again, it takes the model about 3 km to 3.5 km drop the water level back to zero.

Concluding, the results between Python and Delft3D are quite similar, implying that the division into the four subsections seems to be rather valid. The shape of the backwater curve upstream of the transition point is also quite similar, except for the most upstream part. This can be explained by the bumps in the bed level causing an abrupt change in the flow as well. In general, it can be said that the two models have similar results. Again, for a complete validation some more measurement stations are required.

4

Structural design of a flood defence

In this chapter a preliminary design of a flood defence for the protection of the Durban coast is worked out. At first, an analysis on the promenade is executed in order to locate the vulnerable area for the design. The first section is concluded with a complete program of requirements and notes to the used reference level. In section 4.2, different solutions towards the problem are presented. Many solutions are possible, in particular many combinations are possible which are discussed and finally all options are scaled down to a small list. These solutions are assessed in a Multi Criteria Analysis (MCA) in paragraph 4.3. Here, several types of flood defences are compared and graded towards criteria. Based on the grading from the MCA the most suitable option is chosen for the selected vulnerable area. Next in 4.4, the corresponding water-retaining height of the flood defence is calculated by taking into account various aspects, such as design water level, wind set-up and overtopping, using data output from the Delft3D model from chapter 3. At last, a preliminary design of the flood defence is presented in section 4.5.

4.1. Analysis

Additional to the analysis done in chapter 2, this section is more detailed. First, the vulnerable areas are determined based on elevation and historical data. Then, a complete program of requirements is presented, containing demands, constraints and wishes based on the needs of the stakeholders. At last, the reference level is elaborated, since different sources use different reference levels.

4.1.1. Vulnerable area

To assess which areas are susceptible to floods, it is interesting to know the elevation of the area. Elevation data is obtained from the eThekwini Municipality, which has data of the entire area, including the Umgeni river area and promenade. The data has a grid of 10x10 m and is relative to the mean sea level of 2016. To analyse the data, *QGIS 3.4 Madeira* is used and to improve visualization *OpenStreetMap* is used.

Historical floods
One of the most recent flooding happened in October 2017. Photographs of this event are presented in figure 4.1 and show the flooded area, next to that, the photographs already indicate vulnerable areas.

Figure 4.1: Floods in October 2017 caused a large part of the promenade to be flooded. From top to bottom: result of flooding facing North Beach to southern parts of the promenade [47], result from flooding at Dairy Beach [54], during the event at New Beach [32], during the event at New Beach facing south [32]

Elevation

The elevation of the Durban beachfront is presented in figure 4.2. The beaches can easily be recognized due to the greenish colour, indicating relatively low altitude. The beaches are east of a 'hill', which is the reddish/black part in the figure. On this hill, plenty of buildings are located. Important to notice is the area west of the hill. This is a relatively low-level area which the sea can reach via the harbour and is located just south of this area. In this research this low-level area is not considered, but the area is susceptible to flooding.

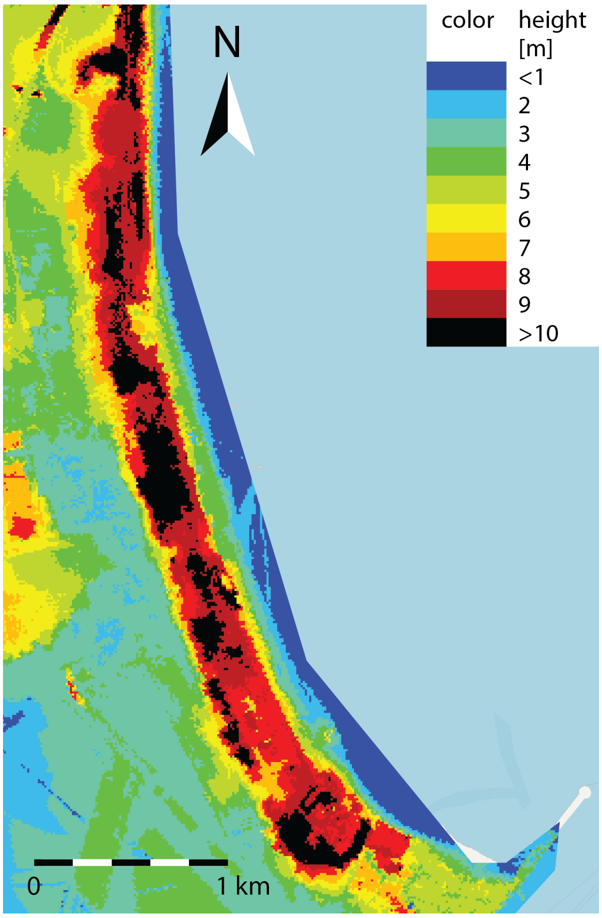


Figure 4.2: Elevation of the Durban beachfront relative to the 2016 mean sea level.

More detailed elevation profiles are presented in appendix E. Figure E.1 shows the beaches and the promenade is presented in figure 4.3. Here, one could easily recognize the lower elevation of the promenade between New Beach and North Beach, where in figure 4.1 a pool can be seen as a result of the flooding.

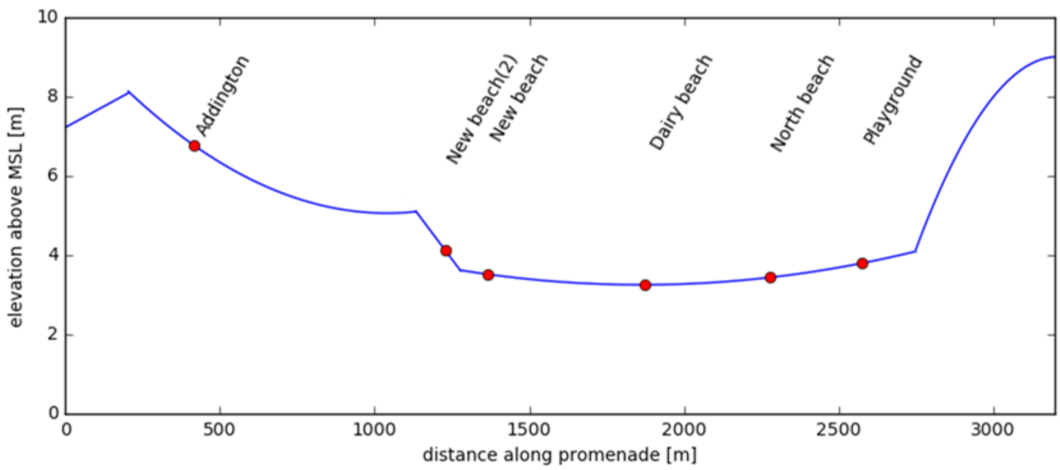


Figure 4.3: Elevation profile of the promenade with indication of the beaches, with direction from south to north. The profile is fitted for readability. However, the promenade has a smooth surface which is not indicated in the figure. Since two beach sections are made at New Beach, one is called New Beach (2).

Additionally, figure E.2 in appendix E also indicates the vulnerable promenade from New Beach up north. The promenade south of New Beach has a higher elevation than other parts, also indicated by previous figures. The road next to the promenade can mainly be found at +8.0 m and the promenade south of New Beach varies between +3.6 m and +5.8 m. Here, and north of the playground, dunes are present as well. Due to the $10x10$ m grid, precise data is difficult to extract because details are left out, especially near spots where the facilities and promenade do not fit in the grid.

Conclusion
Hereby it can be concluded, based on elevation and historical data, that the area north of New Beach up to the playground is the most vulnerable area and is used for this report, since the promenade otherwise is too large to find an integral solution for and this area is the most prone to flooding. The border is chosen at the points where the beaches start to have dunes, where in the obtained area no dunes are present making a single solution likely for that part. At this location of the promenade many shops, various swimming pools and restaurants can be found. One of the booster stations to nourish the beaches can be found at the low level area at Diary Beach, also indicated in figure 4.4. The height of the promenade along this specific part of the beach is 2.2 m above MSL.

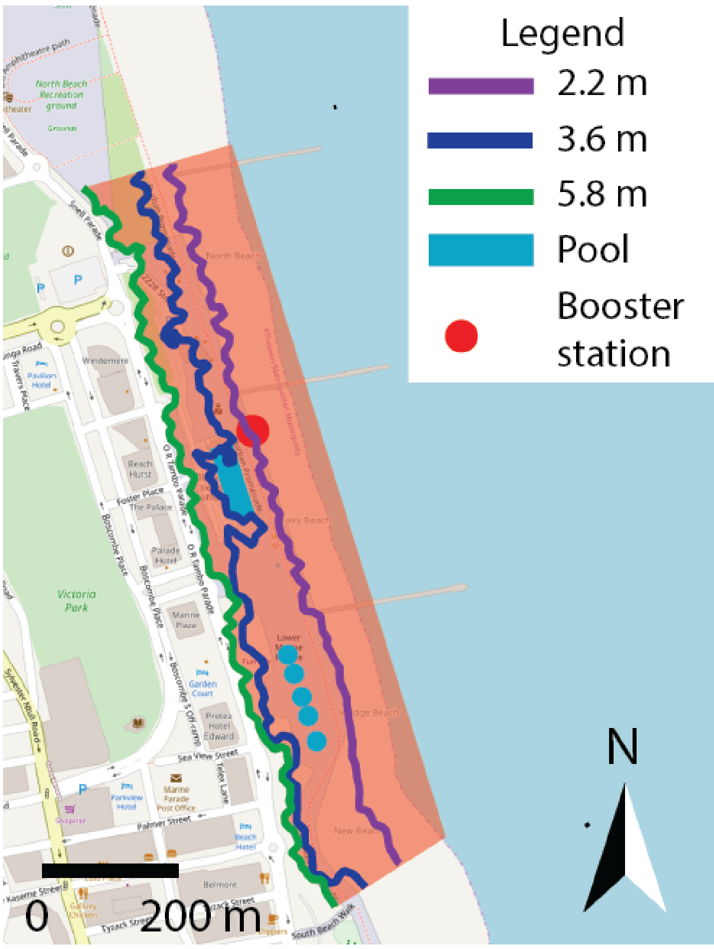


Figure 4.4: Obtained vulnerable area in red including elevation indication and booster location. Along this part of the promenade pools can be found as well.

4.1.2. Program of requirements

In this paragraph the requirements are selected in addition to the analysis done in chapter 2. Also wishes and constraints are mentioned.

Demands

Demands are strict design parameters to which should be designed, for which the South African standards are strictly followed. However, a hydraulic design standard doesn't exist, for these cases standards given by the eThekwini municipality are used. Some of these demands are related to the code and are enumerated and explained below. Others are demands by the client, also the eThekwini municipality. The demands are:

- A 50-year design life, which corresponds with the South African design code.
- Design for a 200 year return period, which has been set by the eThekwini Municipality. A return period describes the likelihood of an event, which is in this case an event estimated to happen once in 200 years.
- For the local wind set-up and shower gusts, the obtained height has to be added with another 0.5 %, which is also set by the eThekwini Municipality.
- The exceedance probability for overtopping is set at 2 % for a 50 year return period wave height. Just like mentioned previously, this standard is also set by the eThekwini municipality.
- The solution may not influence the wave-climate, given the importance for surfers.
- The booster station is important for nourishment of the beaches and should not be affected by the solution.
- The solution should fit in the surroundings.

Constraints

Constraints are already present before the project starts and are important to be aware of since it can influence the design. Here, the constraints are:

- The solution should be build from low-value materials. Given the crime-rate in South Africa, a low-value material is less prone to theft. Especially for details, for example railings, this constraint is of interest.

Wishes

To make a project more likely to succeed and more valuable to the stakeholders, wishes of different stakeholders are enumerated:

- The view along the promenade and beach shouldn't be affected.
- The pools at the promenade are preferably kept in the current conditions, changing the pools has a high impact on the promenade during construction, besides, it is a costly operation. Also changing the restaurants along the promenade can become a costly operation, preferably the restaurants are kept in current conditions.
- Preferably the solution fits other parts of the promenade as well, being more of an integral solution for other parts of the Golden Mile.
- The beach is preferred to be protected as well, although it isn't the main aim of the report.
- There should be minimal nuisance to the promenade, in terms of construction time, construction site area and noise.

4.1.3. Storm input model

One of the demands for the design of the flood defence is: the exceedance probability for overtopping is set at 2 % for a 50 year return period wave height. Figure 4.5 shows the significant wave height measured 1.7 km from the Durban coast caused under storm conditions. In the same figure the March 2007 event is highlighted, which corresponds approximately with the 50 year return period wave height of the demand.

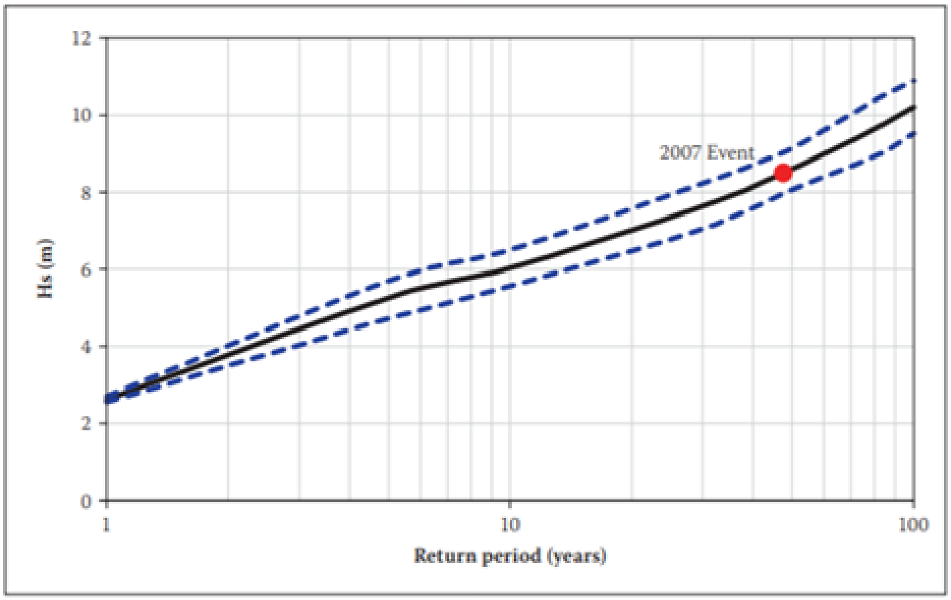


Figure 4.5: Significant wave height, H_s , return periods with a 95 % confidence interval and the March 2007 event for the annual maxima method [14]

Since the March 2007 event is representative for the demand, this storm condition can be used to determine the overtopping height in paragraph 4.4.4. Figure 4.5 gives the significant wave height at open sea, while the significant wave height at the toe of the structure is needed in the calculations. The model described in chapter 3 is used to calculate this required significant wave height, by using storm characteristics as input such as, significant wave height, wind direction and peak period at the boundaries of the model and wind speed on the grid.

To find these input parameters, besides the March 2007 event, the January 1984 and October 2017 event are investigated. Choices for these events are made, because the 1984 event was comparably rare as the 2007 event and of the 2017 event the most documentation is available.

Table 4.1: Storm characteristics of 1984, 2007, 2017 and climate average

	1984 event	**2007 event**	**2017 event**	**Climate average** [14]
Significant wave height [m]	8-9 [28]	8.5 [28]	2.9-4.2 [44]	1.65
Peak period [s]	-	-	-	10
Wind direction [deg]	N 0-20	SE 135	SE 135	130
Wind speed [km/h]	-	64-74 [31]	70 [44]	-

Based on all the available data table 4.1 is composed. As input for the Delft3D model a full column of data is required, which is not there for the March 2007 event. Therefore it is assumed that a 10 s peak period is also applicable for a 50 year return period wave height. As earlier mentioned, these are input conditions on the boundaries of the model.

4.1.4. Reference level

For the design of the flood defence for the coast of Durban, in this report two data sets are used; for water levelling and for land levelling. In order to be able to combine both findings, the data has to be measured relative to the same reference level, which is in South-Africa relative to Mean Sea Level (MSL).

The first data set, for the water levelling, is used in the determination of the water-retaining height due to the astronomical tide. This data set is retrieved from the database of the University of Hawaii Sea Level Center (UHSLC) [55]. From this database the raw data set (research quality) of the measurement station in Durban is used over a period of 36 years, form 1970 till 2016. The water level is given above a specific reference level, named Chart Datum, which is the Lowest Astronomical Tide (LAT) determined for the port of Durban on 01 January 2003. The second data set, for the land levelling, is used to draw height maps of the Durban coastline, in order to visualise the vulnerable areas in paragraph 4.1.1 [21]. This data is given with respect to MSL 2016.

For combining both data sets, the relationship between the LAT and MSL is given in the meta file of the UHSLC as follows: LAT is 0.904 m below MSL by 01 January 2003. All recorded data from the UHSLC is scaled to 01 January 2003 reference level. Since then no changes have been made anymore, which gives the Land Levelling Data of 2016 the same reference level. In the remainder of the chapter all heights from calculations are given relative to MSL. [21]

4.2. Design options

As it comes to solutions for the promenade many different methods can be considered. This section introduces a large number of alternatives and its combinations which concludes in variants. Not all alternatives form a solution on itself, but in a combination with another solution, the alternative or variant becomes more likely. Considering the many options resulting from the combinations, a selection is done to select only the best and realistic options for the Durban promenade, which all are elaborated. In the next section a Multi Criteria Analysis (MCA) is performed to select the best out of these options.

Solutions which have a clear dependence on the nature are left out in this research as given in the scope, section 2.2.2, pointing at for example mangrove solutions. The problem of flooding of the promenade has two main causes: the sea level at an extreme event and the waves additional to this. Both effects need to be solved, first solutions to the sea level are enumerated. These solutions are existing and known (structural) concepts to prevent the promenade of flooding and are assumed to be, as principle, known to the reader.

1. Relocation of the promenade. By relocating of the promenade to a safer height and a location more inland, the promenade is not longer prone to flooding.
2. Heightening of the promenade. By heightening the promenade to a certain elevation, the promenade is no longer prone to flooding.
3. Sea wall at the promenade/beach border. By building a sea wall at the border, the wall holds back the water and thus prevents floods of the existing promenade.
4. Dune. As done at other places along the Durban coastline, placing dunes in front of the promenade protects the promenade from flooding.
5. Emergency barrier at the promenade/beach border. In the case of extreme events, the emergency barrier is placed in advance to protect the promenade from flooding and removed later when the swells have lowered.
6. Dike. As done in other places in the world, locating a dike in front of the promenade protects the promenade from flooding.

Additional to the previous solutions, one can think of combining the solution with one of the enumerated additions, all limiting the wave impact. The solutions are called 'barrier' in this part.

1. Revetment. The combination makes it possible to dissolve energy of the waves before it hits the barrier. Revetments come in different variants.
2. Revetment integrated in the barrier. By integrating an energy dissolving system in the barrier, one could effectively dissolve wave energy. Think of a sea wall built from rocks where water can flow through for example
3. Groynes or piers. Additional groynes dissolve energy of the waves before they hit the beach and therefore barrier, when they are obliquely incident waves.
4. Piles. By placing piles at the beach (or offshore), energy is dissolved from the waves.
5. Offshore breakwater. Probably most effective to dissolve wave energy is an offshore breakwater.

In theory, all alternatives can be combined, which results in a large amount of options. To limit the amount of options in further sections, each combination presented in table 4.2 is (verbally) discussed and concluded to take into consideration or is excluded in further research. Here, distinction can be made between exclusion, not matching the demands, considered depending on certain conditions and fully considered.

Reasons for exclusion are: being an unrealistic solution in South Africa, the combination is impossible in South Africa or the combination is superfluous since a part of the solution already covers the other function of the combination. If the combination does not match the demands at this point, then there is no reason to look into this options since the solution should fulfill the demands as minimum.

Table 4.2: All combinations of solutions and the result after discussion. X = not possible, D = not possible due to not fulfilling demands, C = considered depending on certain conditions, OK = possible combination. The colour grey indicates that the combination already has been considered via another option.

	Relocation	Heightening	Sea wall	Dune	Emergency barrier	Dike
Relocation	X	X	X	X	X	X
Heightening		X	OK	X	X	D
Sea wall			X	X	C	X
Dune				X	X	X
Emergency barrier					X	D
Dike						X
-no combination-	X	X	C	C	X	D
Revetment	X	OK	C	X	X	D
Integration in barrier	X	OK	C	X	X	X
Groynes or piers	X	D	D	D	D	D
Piles	X	D	D	D	D	D
Offshore breakwater	X	D	D	D	D	D

From table 4.2 can be concluded that only three options seem possible and five are possible depending on certain conditions which are all being elaborated. Concluding, the possible combinations are:

- Heightening of the promenade combined with a revetment.

- Heightening of the promenade with an integration of a wave energy dissolving system in the promenade.

- Heightening of the promenade combined with a sea wall.

The possible combinations depending on certain conditions are:

- A sea wall alone. The result depends on the height, since a very high wall does not seem viable in this area.

- A sea wall combined with a revetment, depending on the same condition

- A sea wall with an integration of an wave energy dissolving system, depending on the same condition

- A sea wall combined with an emergency barrier, depending on the same condition

- Dune, also depending on the resulting height.

For an elaboration on the design options, one is referred to Appendix F. The remaining eight design options are further assessed in the MCA in section 4.3.

4.3. Multi Criteria Analysis

In this section the Multi Criteria Analysis (MCA) is performed. First the objective of this MCA is discussed. Secondly, the criteria on which these options are evaluated are talked over. Thirdly, the options are analysed and at last choices are being made and the best option for the objective is chosen. [18]

4.3.1. Objective

The objective of this MCA is correlated to one of the objectives of the report. The objectives of the report are discussed in section 1.2. The flood defence related objective is as follows: ”An engineered flood defence, or dune, is designed up until a preliminary design. It should fit in the existing surroundings and protect the promenade of Durban against severe floods.” The objective of the MCA is to find a (structural) flood defence. The options are the different structural elements which are applicable for this situation. In the previous paragraph these options are listed.

4.3.2. Criteria

To analyze the options, criteria are needed. In this paragraph each criterion, which comes from the stakeholders, and its weight is discussed. The weight is determined with the use of method called *relative weight of the criteria*. The method works as follows; each criteria is graded in importance relative to the other criteria. If the row criteria is more important than the column criteria, than a 1 value is given, if not than a 0 value is given. There is one special case, if both criteria are equally graded in importance than a 1 value is given to both criteria. All values are summed up, from which a weight factor is generated by dividing the sum of a single criteria by the total sum of all criteria. The weight factors are used in the evaluation of the MCA in the next paragraph. All criteria and their weights are shown in table 4.3. [29]

Table 4.3: Criteria and their weights

		a)	b)	c)	d)	e)	f)	g)	Sum	Factor	Weight
Blockage of view	**a)**		1	1	1	1	1	1	6	0.22	22.22
Pools/restaurants are not affected	**b)**	1		1	1	1	1	1	6	0.22	22.22
Fits in the surroundings	**c)**	1	1		1	1	1	1	6	0.22	22.22
Protection of the beach	**d)**	0	0	0		1	1	1	3	0.11	11.11
Nuisance	**e)**	0	0	0	1		1	1	3	0.11	11.11
Booster station is not affected	**f)**	0	0	0	0	1		1	2	0.07	7.41
Extendability along the beach	**g)**	0	0	0	0	0	1		1	0.04	3.70
									27	1	100

From table 4.3 follows that the criteria; 'blockage of the view', 'pools/restaurants are not affected' and 'fits in the surroundings', are weighted the most. These criteria most affect the promenade and therefore a large number of stakeholders. Protection of the beach and nuisance affect the promenade less and therefore are also less important. For the protection of the beach there already is a project in operation (nourishment) and the nuisance is a temporary effect. At last, the booster station is not affected and extendibility along the beach are least important. This is because design options can easily be adapted to the location of the booster station and extendibility along the beach can be valuable, but this is not the main aim. For the fully elaborated criteria and their motivations see appendix G.1.

4.3.3. Evaluation

All the options are evaluated on the basis of the criteria. Each option can achieve a score between 1 and 5. If an option scores a 1 than it performs bad at the criteria. The following scale is used:

1. Bad
2. Insufficient
3. Average
4. Good
5. Excellent

Multiple options can have the same score on the same criteria. In Appendix G the scores are explained. In table 4.4 the total scores are shown.

Table 4.4: Total scores of each option in MCA

Option	Total score
Dune	314.81
Heighten promanade + wave energy dissolving	229.63
Heighten promanade + seawall	237.04
Heighten promanade + revetment	255.56
Seawall	259.26
Seawall + revetment	288.89
Seawall + wave energy dissolving	274.07
Seawall + emergency barrier	348.15

4.3.4. Conclusion

Clearly shown in table 4.4 is that the highest score is obtained by the seawall in combination with an emergency barrier. The second best is the dune and the third best is the seawall in combination with the revetment. All three have the condition that they can't be build too high. After the design this condition should be evaluated. The seawall with emergency barrier is the best option according to the MCA, thus in the next section (section 4.5), this is designed.

Note that there should be a party which is responsible for the storage and maintenance of the emergency barrier, if this is neglected or not possible the option should be left out. Besides, the warning system should be operational so that the barrier can be placed in time to protect the promenade.

4.4. Water-retaining height

In order to design a proper flood defence, firstly the water-retaining height has to be determined, as it is an essential design parameter of the flood defence. For the determination of the water-retaining height the exceedance probability approach is performed, for a given return period of 200 years, see paragraph 4.1.2. The governing water-retaining height of the flood defence is determined based on mainly three components; the design water level, the maximum allowable discharge of wave overtopping and a surplus height (see figure 4.6). The design water level is generally determined by taking into account the (highest) astronomical tide, wind set-up, funnel-effect, shower gusts, seiches, tidal resonance and sea level rise.[39]

For the remainder of the report, the following aspects are considered as ineffective on the water-retaining height for the coastline of Durban; funnel-effect, seiches and tidal resonance. As the water body considered is the Indian Ocean on a shoreline of hundreds of kilometres wide, it can be said that for the specific location of Durban the coastline isn't enclosed on three sides. Therefore the influences of funnel-effect, seiches and tidal resonance can be considered negligible. Each of the other influences are fully elaborated in the following paragraphs.

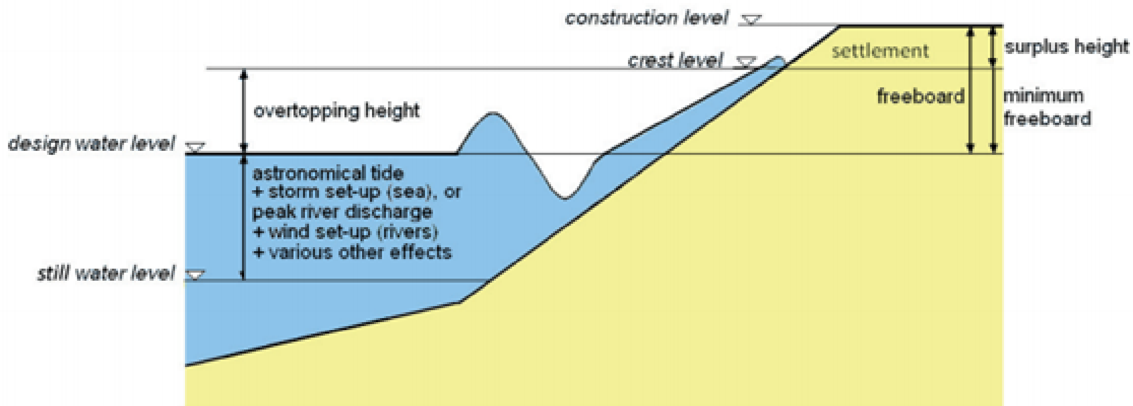
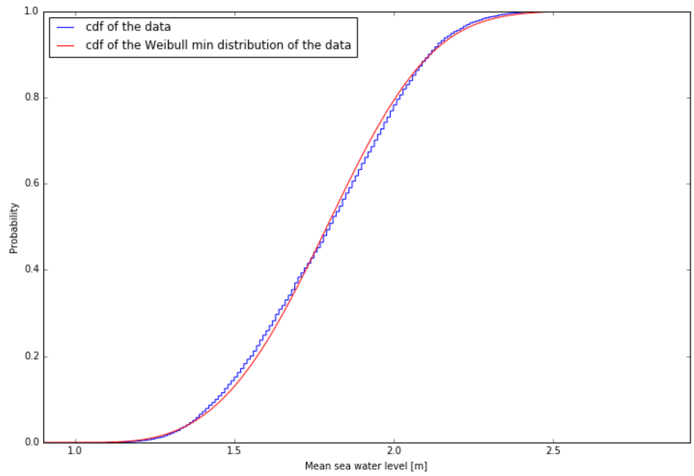


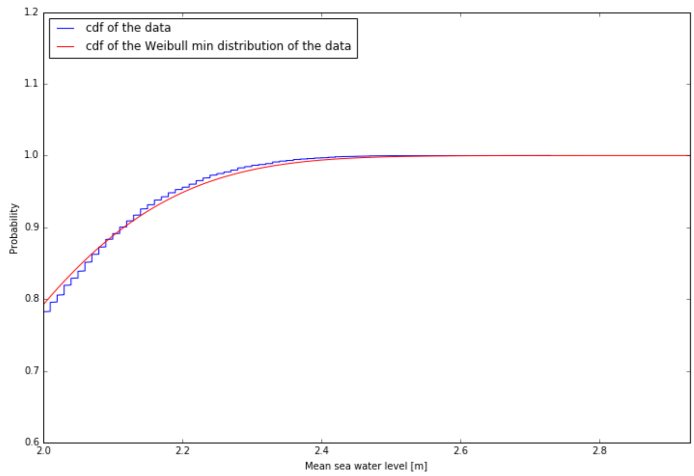
Figure 4.6: Determination of the water-retaining height of the flood defence

4.4.1. Astronomical tide

For determining the design water level, at first the effects of the astronomical tides have to be taken into account. This is done by using the exceedance probability approach on the highest astronomical tides of an hourly water level data set of Durban over a period of 36 years, 1970 till 2016 [55]. For the exceedance probability approach the case of a 200 year return period is considered. As the data set consists of purely raw data, which includes numerous measurement errors, it has to be filtered first. The complete filtering process is explained in detail in appendix H. Filtering is applied in a way such that only days with a complete hourly date set are considered to be valid and thus relevant. As the period of a single cycle of the dominant tidal constituent (M2) takes 12 hours and 25 minutes, in combination with the discontinuous data set, the determination of the water-retaining height due to astronomical tide is performed by taking the daily maximum water level relative to MSL. In figure 4.7a the daily maximum water level is plotted cumulatively as the blue line.



(a) Overall cumulative distribution



(b) Zoomed upper limit cumulative distribution

Figure 4.7: Cumulative distribution of the data set vs. Weibull min distribution

In order to obtain the exceedance probability, various probability functions are plotted and compared with the data set. As the exceedance probability deals with the upper limit, the curve fitting of this part is considered to be most important. For this data set follows that the Weibull min distribution, equation 4.1, fits the best, especially for the upper limit, see figure 4.7b, for which the exceedance probability is determined. For the full determination and comparison of various probability functions see appendix H.

$$Weibull\ min\ distribution : f(x, c) = cx^{c-1}e^{-x^c} \tag{4.1}$$

Following the Weibull min distribution given in figure 4.7b and the exceedance probability for a return period of 200 years, this gives a water level of 2.760 m relative to LAT. For calculations the same reference height as the height map is used, namely MSL. The height for a return period of 200 years then becomes 1.856 m above MSL, see appendix H.

4.4.2. Wind set-up

Whether the influence of the wind set-up has to be taken into account is determined by looking at the wave steepness. The wave steepness is computed by the ratio between the wave height and the wave length ($s_0 = H_{mo}/L_{m1,0}$). For an indication, generally a wave steepness $s_0 = 0.01$ is found for a typical swell sea and a wave steepness $s_0 = 0.04\ to\ 0.06$ is found for a typical wind sea. [57] The wave steepness under storm conditions is extracted from the model described in chapter 3, see figure 4.8.

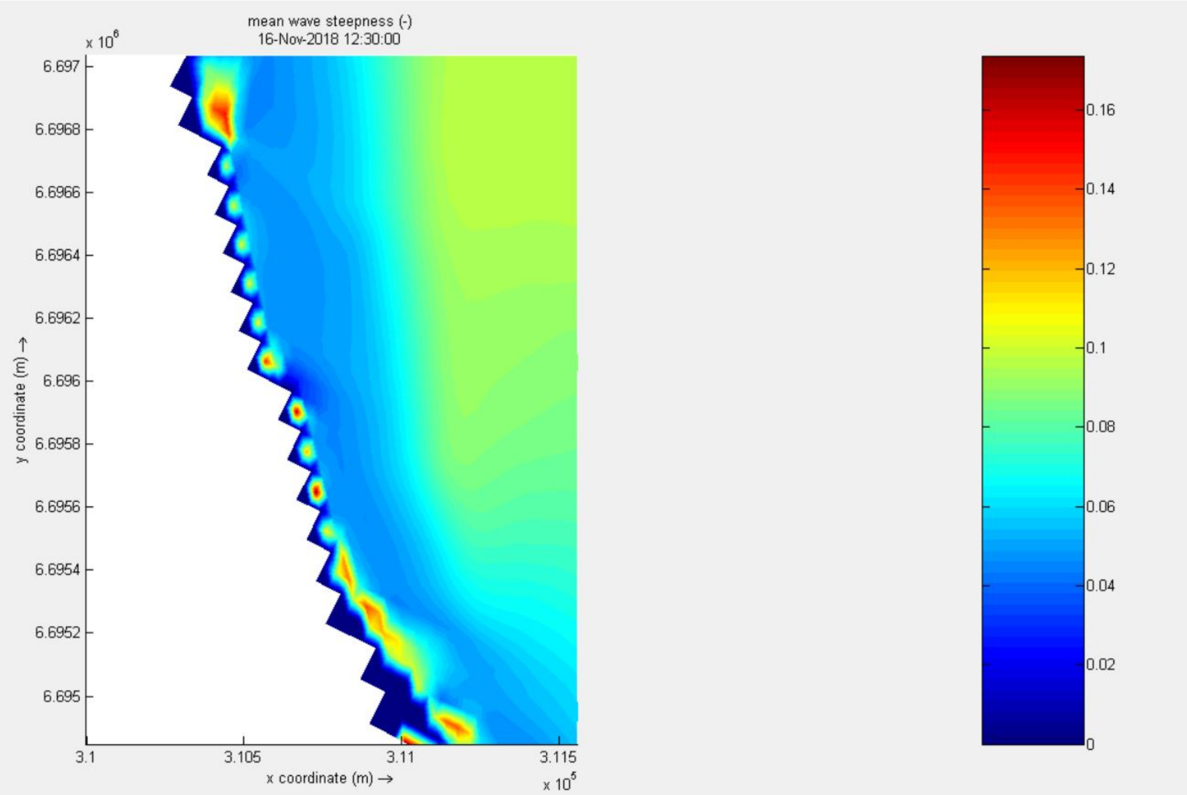


Figure 4.8: Wave steepness for the Durban coastline extracted from the model of chapter 3

From figure 4.8 can be concluded that the wave steepness close to the beach boundary is between the 0.04-0.06, which makes it a wind sea according to previously mentioned standards. Therefore the influence of wind set-up has to be taken into account. In consultation with the eThekwini Municipality, for local design the influence of wind set-up and shower gusts is taken into account by adding 0.5 % to the water-retaining height found due to the astronomical tide in the previous paragraph. Given the water-retaining height due to astronomical tide to be 1.856 m relative to MSL, 0.5 % gives an addition of 0.009 m due to wind set-up and shower gusts.

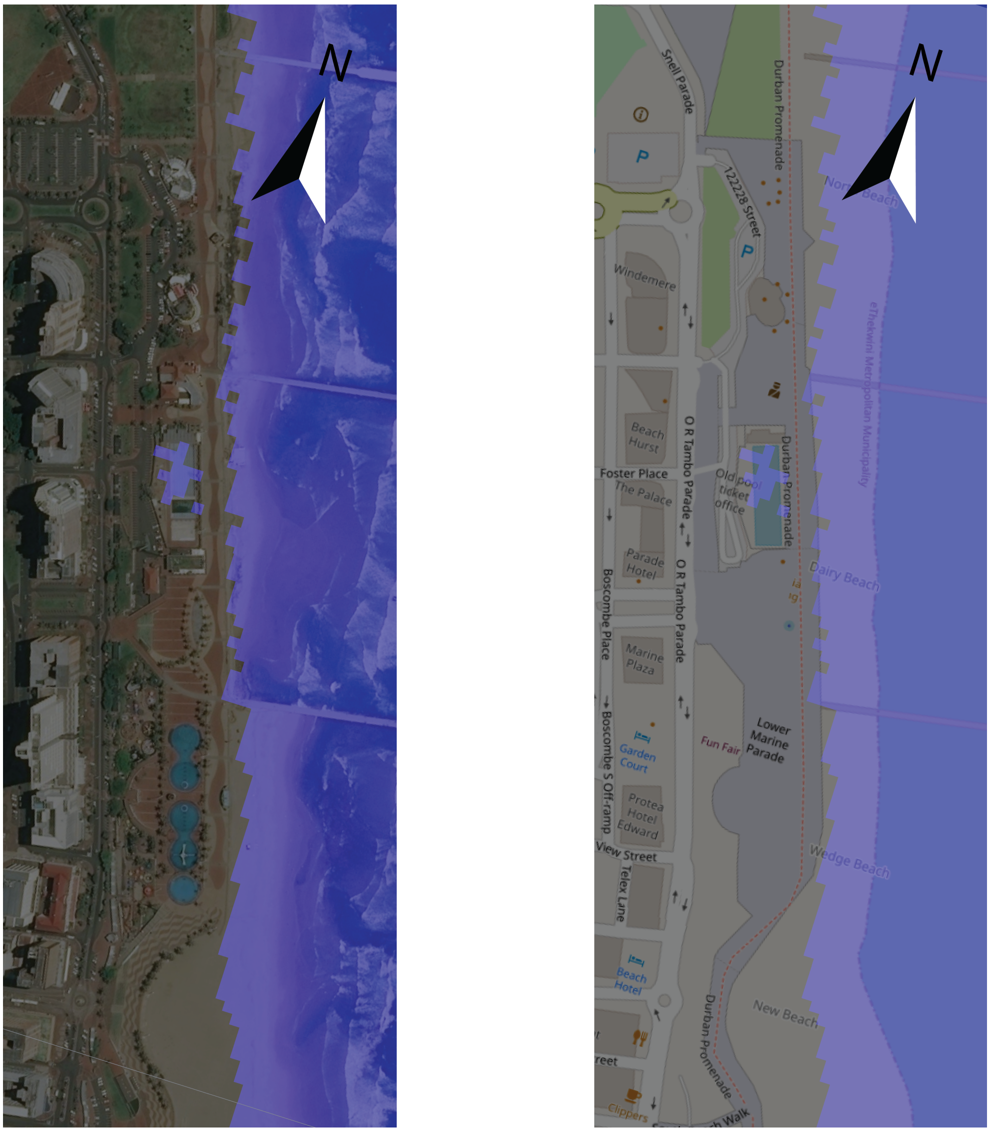
4.4.3. Sea level rise (SLR)

Additional to the previously mentioned effects, the sea level rise also contributes to the water-retaining height. The impact of climate change on the sea level rise is evaluated, taking into account both global and local effects. For an elaborated review on the SLR, one is referred to Appendix I. By adding the SLR, the full story of the design water level is complete. The additional water-retaining height, so-called allowance due to climate change is 0.46 m.

4.4.4. Overtopping

Given the water-retaining heights due to the astronomical tide, wind set-up and sea level rise, this leads to a design water level of $1.856 + 0.009 + 0.460 = 2.325\ m$ above MSL. For the depicted vulnerable area in paragraph 4.1.1 the height along the promenade is minimal 2.2 m

above MSL, as shown in figure 4.9. For these values can be concluded that the overtopping is determinant for the height of the flood defence, taking into account the remaining 0.13 m from the design water level.



(a) With an Microsoft Bing aerial as background (b) With an OpenStreetMap map as background

Figure 4.9: The area which lies lower than $2.33m$ above MSL coincides with the border of the promenade and is indicated in blue

To determine the water-retaining height, due to overtopping, input parameter for the formula are retrieved from the model described in Chapter 3. However, Delft3D does not have a bathymetry reaching until the promenade and therefore is unable to calculate wave characteristics at this point. Following, two alternatives are explained to still determine these wave characteristics. For a full elaborated derivation see appendix J.

The first method, appendix J.1, takes the output of the Delft3D model at 310 m from the coast. Via a handful of calculations, an extra wave set-up, due to accumulation of broken waves, of 0.32 m is found. Combining this with the design water level, this leads to a water level of 0.45 m relative to ground level at the toe of the structure.

The second method, appendix J.2, uses SWAN Delft to make a 1D-model for waves approaching the coastline until the promenade. Again, Delft3D output is used as input of the SWAN model, only this time at 2 km from the coast. Following from the 1D-model, a water depth of 0.15 m and an extra wave set-up of 0.41 m is found, leading to a water level of 0.56 m relative to ground level at the toe of the structure.

The difference of 0.11 m between the two methods can be explained by the fact that the manual calculation method starts at 310 m from the coastline. Apparently, this distance does not cover the full range of wave development. Namely, according to the 1D-SWAN model, at 310 m from the coast there is already a wave set-up of approximately 0.10 m. This explains the difference and by considering this, both methods are justified, as the design water level found by SWAN, 0.15 m, is similar to the statistically derived design water level of 0.13 m.

To be conservative the highest design water level is used in further calculations. Thus there is $0.15 + 0.41 = 0.56$ m of water against the wall relative to ground level. Next to that, roller height has to be added. Rollers are remainders of broken waves, and result in a wave of 0.07 m. With this information the freeboard, thus the height between the top of the structure and the water level, can be calculated as described in appendix J.3.

For the calculation of the height of a vertical wall which can resist the water from the sea and protect the hinterland, the EurOtop manual on wave overtopping of sea defences and related structures (second edition 2018) is used. [57] This calculation gives a freeboard height of 0.004 m. thus the total construction height (under and above water level) is $0.56 + 0.004 = 0.564$ m relative to ground level.

The main objective of the seawall is retaining the water. The freeboard height could be lowered, through a bullnose (for example). In this specific case this is not an option, because the freeboard height is low. Thus this is not further investigated in the report.

4.4.5. Surplus height

To compensate for settlements a surplus height is applied on top of the crest level of the flood defence. This surplus height is composed out of two components; mathematical surplus height and geotechnical surplus height.

First, the mathematical surplus height is determined based on the length effect and robustness surcharge. The length effect doesn't apply for this design, as the flood defence is divided in various sections, due to openings to the beach. The robustness uncertainties do apply, which is determined by technical uncertainties, like magnitude of discharge, modelling uncertainties and statistical uncertainties in load parameters. The climate change scenario uncertainties are already taken into account within the water-retaining height, due to sea level rise in paragraph I. The mathematical surplus height for the other uncertainties is assumed to be 2 % of the total water-retaining height of $2.2 + 0.564 = 2.764$ m, which gives a mathematical surplus height of 0.06 m.

For the geotechnical surplus height only the settlement is applicable, which is 5 % for a not well compacted sand layer. As further explained in paragraph 4.5.5, the top soil layer consists of fairly loose sand, for which the settlement has to be taken into account. Followed from the previous paragraph the water-retaining height above ground level is 0.564 m. Combining this value with a 2.06 m deep foundation beneath ground level, see paragraph 4.5.5, the total height to be taken into account for settlement calculations is $0.564 + 2.06 = 2.624$ m. Given the 5 % settlement for a not well compacted sand layer, the geotechnical surplus height becomes 0.13 m.

To conclude, the total surplus height of the structure is 0.19 m, composed by 0.06 mathematical surplus height and 0.13 m geotechnical surplus height. [39]

4.4.6. Conclusion

Considering the astronomical tide, wind set-up and sea level rise, the design water level is determined to be 2.33 m above MSL. As is indicated in figure 4.9 this water level reaches till the promenade. The ground level of the edge of the promenade is 2.2 m above MSL, as is indicated in section 4.1.1 and in appendix E, which leads to a 0.13 m water level above ground level. Adding the overtopping for a vertical wall and the surplus height, the total water-retaining height of the flood defence needs to be at least $2.2 + 0.564 + 0.19 = 2.954$ m above MSL. This leads to a construction height of 0.754 m above ground level of the promenade, which is for design purposes set to 0.80 m above ground level.

According to the MCA performed in section 4.3 the flood defence has to be made as seawall combined with an emergency barrier. Due to the limited height of the flood defence a combination of the seawall with an emergency barrier on top of that, is not needed according to the criteria of blockage of the view. Two options are left, which are an emergency barrier over the full height or a permanent seawall of 0.80 m. The second option is preferred, as it leads to a more simplified design, which is favourable towards working conditions and the building process in South Africa. Therefore it can be concluded that a permanent seawall is chosen to be the design of the flood defence. Conditions are that it is well integrated in the surroundings and emergency boards must be used to close the beach entrances during high water circumstances.

4.5. Preliminary design

In this section a preliminary design for the flood defence in the vulnerable area determined in paragraph 4.1.1 is elaborated. A concrete seawall is designed along the promenade with at every 50 m a 3 m wide beach entrance, accessible for vehicles from the beach watch or other services. These entrances are closed during high water and/or storm circumstances by emergency barriers.

First, the loads are calculated and explained, after which the same is done for the load factors. From these loads the seawall design is discussed in paragraph 4.5.3 followed by the emergency barrier design in paragraph 4.5.4 and the foundation in 4.5.5. At last, an integrated design is explained in paragraph 4.5.6.

4.5.1. Loads

To result in a design of the sea wall, the different loads working on the structure should be assessed. The loads have been assessed with reference to the South African National Standards (SANS).

The working loads on the structure are divided in permanent, variable and accidental loads. Permanent loads involve only self-weight. Variable loads involve wind, water (both forces from hydrostatic pressure and wave driven forces), other horizontal induced forces and accidental loads.

Permanent loads

The permanent load which is relevant for the sea wall is self weight. Self weight is only of relevance in vertical direction.

Variable loads

Wind

Wind load is highly depending on local conditions, which are elaborated in appendix K.1 and are based on SANS10160-3:2018. Here, the results of the local conditions are presented, considering the wind zone, orography factors, depended on landscape geography, and pressure coefficients.

The wind pressure $q_{w,e} = 2.48\ N/mm2$ is illustrated in figure 4.10 and is divided in two parts, given the load combinations in a later stage, which is described in the next section.

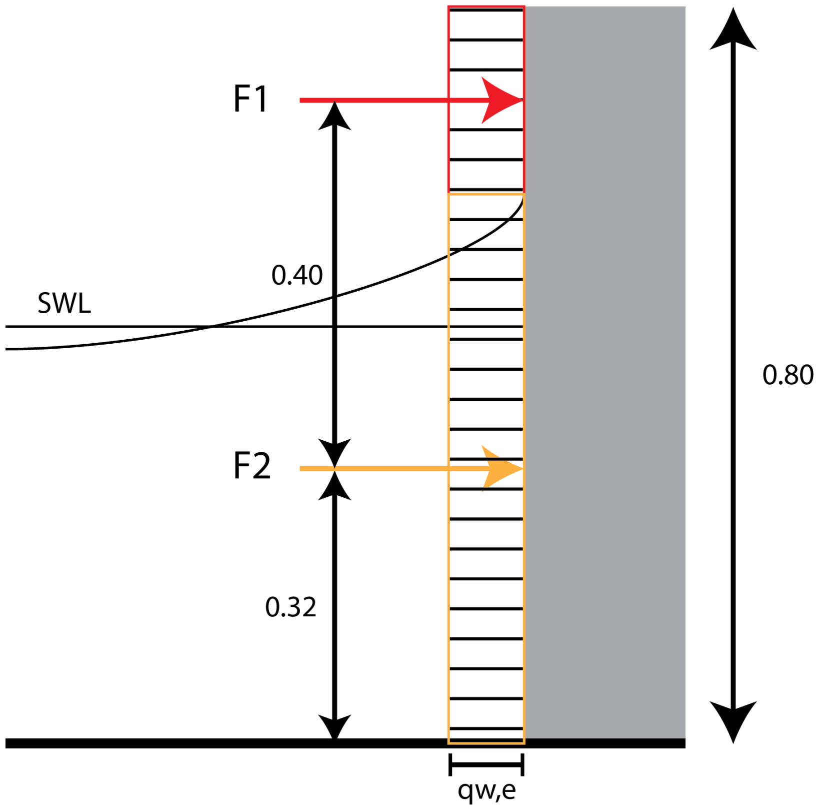


Figure 4.10: Wind pressure on the wall. Distances in meter

On an assumed section of 1 $meter$, the wind force becomes:
$F1 = 0.39\ kN$
$F2 = 1.60\ kN$

Water
Two situations can be assumed for the loads coming from water, wave induced forces and hydrostatic pressure. The wave induced forces are determined with Sainflou's method, which is a simple method but valid for a preliminary design. Sainflou's method is described in appendix K and the results are shown in figure 4.11.

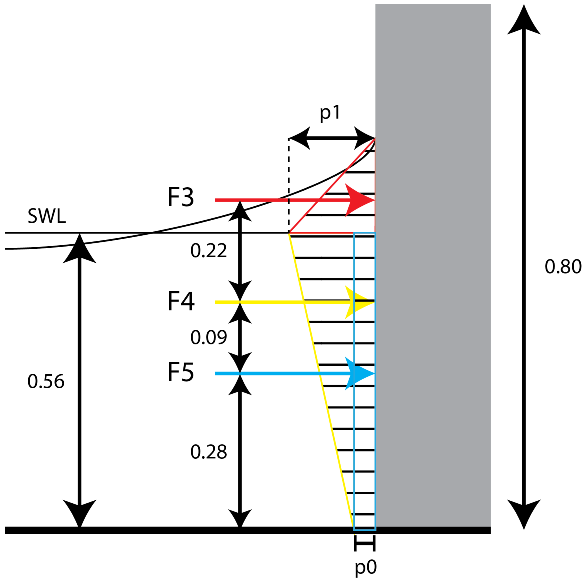


Figure 4.11: Wave induced pressure according to Sainflou's method. Distances in meter.

On an assumed section of $1 meter$, the wave induced forces become:
$F3 = 0.18\ kN$
$F4 = 0.01\ kN$
$F5 = 0.19\ kN$

The hydrostatic pressure causes forces which are simple to determine and can be seen in figure 4.12

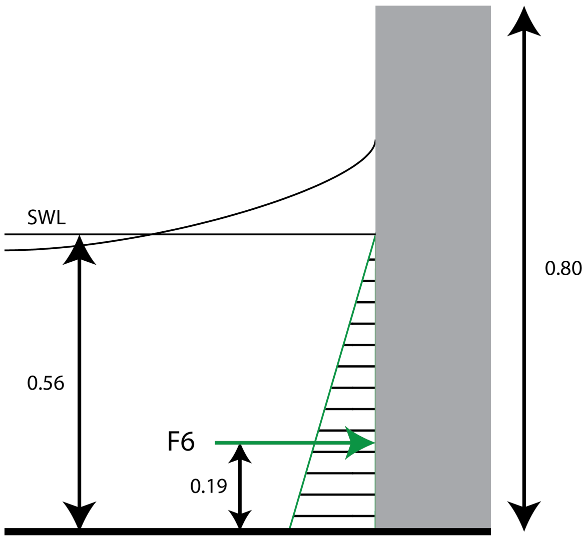


Figure 4.12: Hydrostatic pressure at the structure. Distances in meter.

On an assumed section of $1 meter$, the hydrostatic pressure causes the force to become:
$F3 = 0.28\ kN$

Horizontal load
Another variable load according to on SANS10160-2:2011 is the horizontal load caused by impact from a person falling against or bumping into the wall. The wall is a structure in a low risk group, indicating that the wall belongs in consequence class 1 (CC1). The wall is located in a public area which is susceptible to crowding, however SANS assumes these categories for buildings, not for open area outside. Therefore it is assumed that the wall belongs in category C1, in which usually class rooms, areas in schools, cafés, restaurants are categorised. Lower classes A and B are respectively residential and area not susceptible to crowding and classes C2-C5 and D are more in the range of area with fixed seats, grandstands, museums, concert halls and shops.

The horizontal load according to on SANS10160-2:2011 is either a point load Qk or a spreaded load qk. Both loads cannot be applied simultaneously. The loads are shown in figure 4.13 and are respectively $1.0kN$ and $0.5kN/m$.

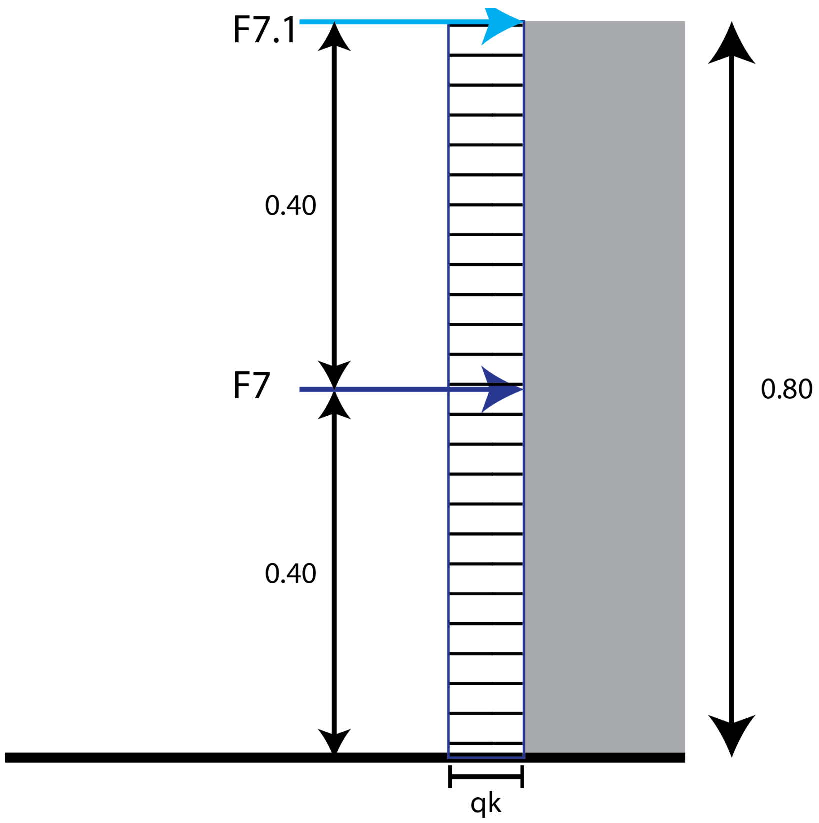


Figure 4.13: Horizontal forces acting on the sea wall due to impact. Distances in meter.

On an assumed section of $1meter$, the impact forces are either:
$F7 = 0.8\ kN$
$F7.1 (= Qk) = 1.0\ kN$

Vertical loads
No vertical variable loads are known or given in the SANS. Assuming no load seems very unlikely, since it is easily possible that people sit on the wall. To use a vertical variable load, it is assumed that a similar impact load of 0.5 kN/m can be applied vertically on the wall, representing 500 kg load condition on a section of a meter.

Accidental loads
Besides permanent and variable loads, accidental loads should be considered. Accidental loads can be in the form of seismic activity or car impact. On seismic level, South Africa is a stable region and therefore it is assumed that seismic activities have no impact on the design [15]. Car impact however, seems plausible given the accessibility to cars on the promenade, although this is limited to emergency vehicles and other traffic related to work. Besides, life guards and others drive also on the beach.

No guidelines are given for accidental load as car impact, but for multi-storey parking garages for vehicles with a gross weight not exceeding 25 kN, horizontal loads are assessed. In SANS10160-2:2011 it is assumed that a horizontal load of 30 kN, distributed over any 1.5 m length of barrier, acting normal to the barrier and at a height of 550 mm above floor level, shall be applied. The load situation is illustrated in figure 4.14.

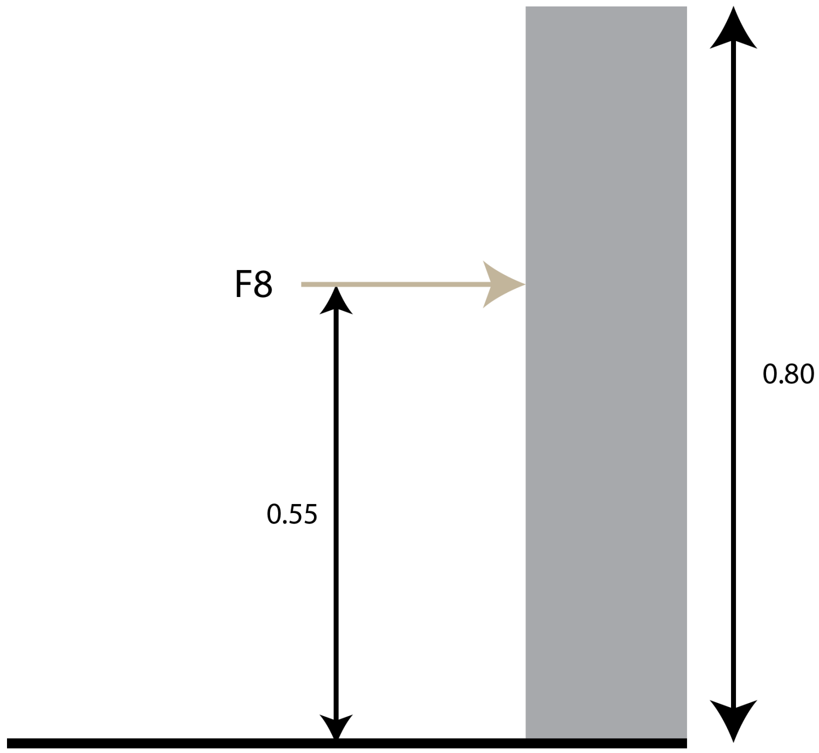


Figure 4.14: Horizontal forces acting on the sea wall due to accidental car impact. Distances in meter.

On an assumed section of $1 meter$, the accidental force is:
$F8 = 20.0kN$

4.5.2. Load factors and combinations
Given the possibility that loads can occur simultaneously, different load combinations should be assessed. To allow for uncertainties, load factors γ are applied. Different combinations can be governing, all related to the previously mentioned loads, and are combined with the combination factor Ψ. Load factors and combinations are described and based on SANS10160-1:2018.
The wall is in Reliability Class 1 (RC1), indicating that a consequence of failure has low consequences for human life, economic, social or has small or negligible environmental consequences. Therefore, the load combinations will be multiplied with a factor $Kf = 0.9$.

Load factors
Within the load factors, one should distinguish the Serviceability Limit State (SLS) and the Ultimate Limite State (ULS). An entire overview of all the load factors in ULS is given in appendix K.3, the ones relevant are presented in table 4.5.
Three different situations can be distinguished from appendix K.3 and are shown in table 4.5 for structural resistance (STR): Permanent load (STR-P), variable load (STR) and accidental load (ACC).

Table 4.5: Load factors γ which are applicable to the sea wall, based on SANS10160-1:2018. F = Favourable, Un-F = Unfavourable.

Type of action	Ultimate Limit State					
	STR		STR-P		ACC	
	Un-F	F	Un-F	F	Un-F	F
Self weight	1.2	1.0	1.35	0.0	1.0	
Wind action	1.6	1.0	1.0	0.0	1.0	
Loads from fluids	1.6	1.0	1.0	0.0	1.0	0.0
Other loads	1.6	1.0	1.0	0.0	1.0	0.0
Accidental loads	Not applicable				1.0	

For SLS, less factors are known, being: Ψ = 1.1 for unfavourable, and = $1,0$ for favourable permanent actions.
Ψ_Q = 0.6 for wind loads;
Ψ_Q = 1.0 for all other imposed loads.

Combination factors
Combination factors are introduced to combine uncorrelated loads on the structure. For category C structures, the load combination factor Ψ = 0.3 and for wind forces Ψ_{wind} = 0.3. The entire table can be found in appendix K.4

Load combinations
In ULS the load combinations are derived according SANS10160-1:2018 and know two different forms, which are:
For STR-P and STR:

$$\gamma_{G,j}G_{k,j} + \gamma_{Q1}Q_{k1} + \Sigma\psi_i\gamma_{Q,i}Q_{k,i} \tag{4.2}$$

For ACC:

$$\gamma_{G,j}G_{k,j} + A + \Sigma\psi_i\gamma_{Q,i}Q_{k,i} \tag{4.3}$$

Where:
$\gamma_{G,j}$ is the load factor for the permanent load;
$\gamma_{Q,i}$ is the load factor for a variable load;
γ_{Q1} is the load factor for the leading variable load;
psi_i is the combination factor;
G is the permanent load;
Q is a variable load;
Q_1 is the leading variable load;
A is the accidental load.

In SLS no accidental load has to be considered and different load factors should be used. The loads should be combined according SANS10160-1:2018 via:

$$\gamma_{G,j}G_{k,j} + \gamma_{Q1}Q_{k1} + \Sigma\gamma_{Q,i}\psi_iQ_{k,i} \tag{4.4}$$

Concrete wall
All load combinations to design the concrete wall are shown in appendix K.5 and are based on table 4.5 and equations 4.2 and 4.3. Combinations or parts that need explanation are further elaborated in 4.5.3. Besides, the governing loads are elaborated. The combinations presented in appendix K.5 do not use the permanent load, since the permanent load is composed of only self weight, which on itself has no lever arm which could have been of interest for the moment, see figure L.1.

The governing situation in ULS for the moment (and shear force) in the section is in case of accidental load 1, as presented in appendix K.5. Here the accidental load is combined with an horizontal impact load and a wind load. Water is left out the combination by the combination factor $\psi = 0$, for the reason that water and wind do not cause loads at the same spot at the same time. The load situation is presented in figure L.1. The combination is further elaborated in section 4.5.3.

$$M, V : 1.0G + 1.0q_{vehicle} + 0.3 \cdot 1.6q_{impact} + 0.3 \cdot 1.6q_{wind} + 0 \cdot 1.6q_{water} \tag{4.5}$$

The governing situation in SLS is in case of an impact load as governing variable load, in combination with wind. Since water has a combination factor $\psi = 0$, it has no influence, besides the permanent load has no impact on the SLS. The combination is presented in 4.6 and is further elaborated in 4.5.3.

$$w = 1.1G_{k,j} + 1.0F_{impact} + 0.6 \cdot 0.3 \cdot q_{wind} \tag{4.6}$$

Emergency barrier
All load combinations to design the emergency barrier are shown in appendix K.5 and are based on table 4.5 and equations 4.2 and 4.3. Combinations or parts that need explanation are further elaborated in 4.5.4, besides, the governing loads are elaborated. The combinations presented in appendix K.5 do not use the permanent load, which has the same reason as for the design of the concrete barrier. The accidental load is assumed not to be valid, since it does not seems likely that in an emergency (when the wall is in use), a car crashes into wall, given that cars are -some exceptions a side- not allowed on the promenade.

The governing situation in ULS for the moment in the board is in case of impact load 3.1, as presented in appendix K.5. Here the impact load is combined with a horizontal wind load. Water is left out the combination by the combination factor $\psi = 0$, for the reason that water and wind do not cause loads at the same spot at the same time. The load situation is presented in section 4.5.3. The governing load combination is presented in equation 4.7 and is further elaborated in section 4.5.4.

$$M : 1.2 \cdot G_{k,j} + 1.6 \cdot F_{impact} + 0.3 \cdot 1.6 \cdot q_{wind} \tag{4.7}$$

Given that the impact load knows two different forms, either as spread load q_{impact} or as point load F_{impact}, a different load combination becomes governing for the shear force. This is easy to explain since the shear force in a single board for the point load becomes $V_{Fimpact} = \frac{F_{impact}}{2}$ and $V_{qimpact} = q_{impact}L/2$. With the knowledge that $F_{impact} = 1.0\ kN$, $q_{impact} = 0.5\ kN/m$ and $L = 3.0\ m$, the governing shear force is $V_{qimpact} = 0.75\ kN$.

The governing load combination for shear force is wind load 1, as presented in appendix K.5. Here the impact load is combined with an horizontal impact load. The combination is presented in equation 4.8 and is further elaborated in section 4.5.4.

$$V : 1.2 \cdot G_{k,j} + 1.6 \cdot q_{wind} + 0.3 \cdot 1.6 \cdot F_{impact} \tag{4.8}$$

Also for SLS, the possible combinations are shown in appendix K.5. The governing situation is in case of an impact load as main variable load, in combination with water. Since wind has a combination factor $\psi = 0$, it has no influence, besides the permanent load has no impact on the SLS. The water load has is maximum at ground level, there where the hydrostatic pressure is at largest. The combination is presented in 4.9 and is further elaborated in 4.5.3.

$$w : 1.1 \cdot G_{k,j} + 1.0 \cdot F_{impact} + 0.3 \cdot 1.0 \cdot q_{water} \tag{4.9}$$

Foundation
The governing load combination for the foundation is a combination of self weight and the variable load. Given that only two variants are possible, both are presented in equation 4.10 and 4.11. The governing load combination is the second one, equation 4.11.

$$w = 1.35 \cdot G_{k,j} + 1.0 \cdot Q_{vertical} \tag{4.10}$$

$$w = 1.2 \cdot G_{k,j} + 1.6 \cdot Q_{vertical} \tag{4.11}$$

4.5.3. Seawall design

From appendix L.1.2 the governing load combinations for the bending moment capacity (ULS), shear capacity (ULS) and calculated deflection (SLS) are determined. The following values are determined:

- Maximum bending moment: $M_{Ed} = 10.59\, kNm/m$
- Maximum shear force: $V_{Ed} = 19.29\, kN/m$

With the resulting thickness of 150 mm, the deflection can be determined, also elaborated in appendix L.1.2. A section of the seawall is shown in figure 4.15.

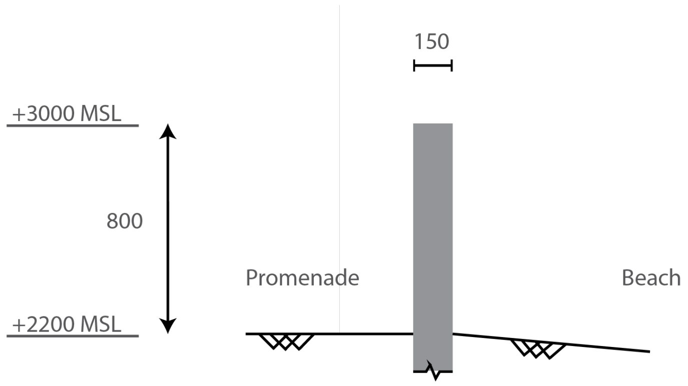


Figure 4.15: A part of the section of the seawall, all measurements are in mm

- Calculated deflection: $w_{calculated} = 0.018\, mm$

Based upon the three unity checks on bending moment capacity, shear capacity and deflection, the dimensions of the concrete wall are determined iteratively. For full calculations of the unity checks see appendix L.1.2.

- Bending moment unity check: $\frac{M_{Ed}}{M_{Rd}} = \frac{10.59}{11.71} = 0.90$
- Shear unity check: $\frac{V_{Ed}}{V_{Rd}} = \frac{19.29}{58.75} = 0.33$
- Deflection unity check: $\frac{w_{design}}{w_{max}} = \frac{0.018}{3.2} = 0.006$

To calculate the thickness and the amount of reinforcement in the seawall, the South-African standard (SABS 0100-1 edition 2.2, from the year 2000) for concrete is used. The calculation shows that the seawall is strong enough to withstand the normative load combination if it is 0.15 m thick, 0.8 m in height. And has reinforcement for tension. The bars are 11 mm in diameter and are placed every 0.125 m in the middle of the beam, shown in figure 4.16. The seawall is strong enough to take the shear force, thus there is no need for reinforcement for shear. The calculation are shown in appendix L.1.2.

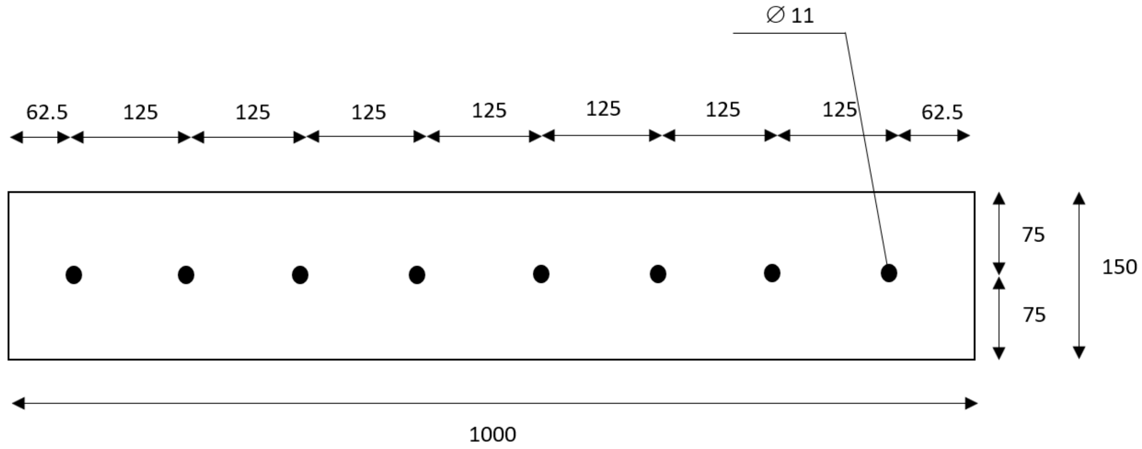


Figure 4.16: top view of the cross-section of the seawall, all measurements are in mm

If you hold to certain spacing standard, according to the South-African standard, the crack width is controlled. The rule is not to exceed 0.215 m spacing between individual bars, when using $f_y = 250\,Mpa$. With the 0.125 m between bars the design suffices, thus the crack width does not have to be checked (see A.3.1.1 from the SABS 0100-1 edition 2.2, from the year 2000).

The governing load condition can be applied from both sides, the car impact can come from the beach and the boulevard. Due to the fact that the reinforcement steel is in the middle, the calculation of the other sides is exactly the same. Thus this will suffice too and does not have to be calculated.

A special note is given to the edge of the concrete wall, where it meets with the emergency barrier. When the barrier is in use, the forces which act on the barrier will be transferred to the edge of the concrete wall via an ‘ slot, which is described in the next section. Therefore, extra forces act on the barrier due to the emergency barrier, making the assumed 1 meter wall in theory invalid. However, when the emergency barrier is in use, it is assumed that the accidental load by vehicle impact is not governing, causing only the SLS requirement to be invalid. Given the low $UC = 0.02$, there is enough room for additional deflection, like those of the emergency barrier.
At last, due to the difference in temperature the concrete seawall will expand and shrink. This difference in size should be accounted for by a dilatation joint in the seawall. For the calculation and placement of the joint additional research is required.

4.5.4. Timber emergency boards

As mentioned in the introduction of section 4.5, at every 50 m along the coastline, of the depicted vulnerable area, a 3 m wide beach entrance is located. These entrance are closed under certain storm and/or high water conditions with emergency barriers, which are slid into a U-shape slot on both sides in the concrete seawall, an impression is shown in figure 4.17. As material for the emergency barrier softwood pine timber is chosen, with the motivations that it is a low value product, concerning corruption and theft, and widely available in South Africa. Given the numbers from the report; *South African Forestry and Forest Products Industry 2016*, KwaZulu-Natal has 115922ha of pine plantation by 2016. [27] Together with the fact that pine timber is used in the SANS, the choice of material is made.

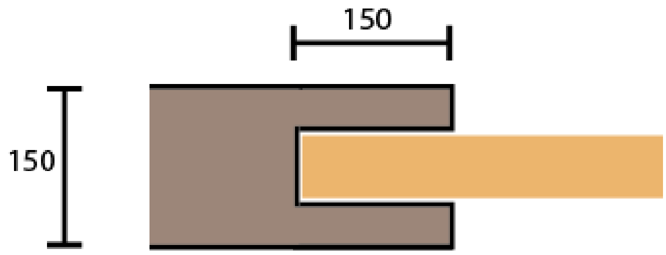


Figure 4.17: An impression of the U-shaped slot, all measurements are in mm

From appendix K.5 the governing load combinations for the bending moment capacity (ULS), shear capacity (ULS) and calculated deflection (SLS) are determined. The following values are determined:

- Maximum bending moment: $M_{Ed} = 1.32\, kNm$
- Maximum shear force: $V_{Ed} = 3.54\, kN$
- Calculated deflection: $w_{calculated} = 6.22\, mm$

A special note is given by the SANS for the deflection of timber boards, using equation 4.12. For the complete derivation of the factor see appendix L.2.

$$w_{design} = w_{calculated} d_1 d_2 = 10.57\, mm \tag{4.12}$$

Based upon the three unity checks on bending moment capacity, shear capacity and deflection the dimensions of the timber boards are determined iteratively. For full calculations of the unity checks see appendix L.2. From this the dimensions of the timber boards are determined as length x width x height of $3300x80x200\ mm$, with a $L = 3\ m$ span and 15 mm fit in the U-slot on both sides. These dimensions correspond with the following unity checks:

- Bending moment unity check: $\frac{M_{Ed}}{M_{Rd}} = \frac{1.32}{1.63} = 0.81$
- Shear unity check: $\frac{V_{Ed}}{V_{Rd}} = \frac{3.54}{10.18} = 0.35$
- Deflection unity check: $\frac{w_{design}}{w_{max}} = \frac{10.57}{12} = 0.88$

To conclude, all unity checks are satisfied and per entrance in the flood defence four timber emergency barriers are required. Each one is slid into the U-slot of the reinforced concrete wall and fit on top of each other with a small rubber edge to prevent water from seeping through the barrier. The total weight of a single timber board then becomes 22.2 kg.

4.5.5. Foundation

Shallow foundation

For the foundation, very limited information was available. Known is that piers and groynes are founded on friction piles, going to 25 meter below ground level. This seems not a reasonable comparison with the sea wall. If CPT data would have been available an actual pile design could have been performed, however this is not the case. It is therefore shown what it would take to design the wall on a shallow foundation. To allow for settlements, additional height has been reserved in the surplus height, as described in section 4.4.5.

At the promenade poor conditions are found: loose sand and an undrained conditions indicate limited bearing resistance. To compete these poor conditions, a foot of 1.4 m is needed at the toe of the structure. The calculations are performed in appendix L.3. A section is shown in figure 4.18.

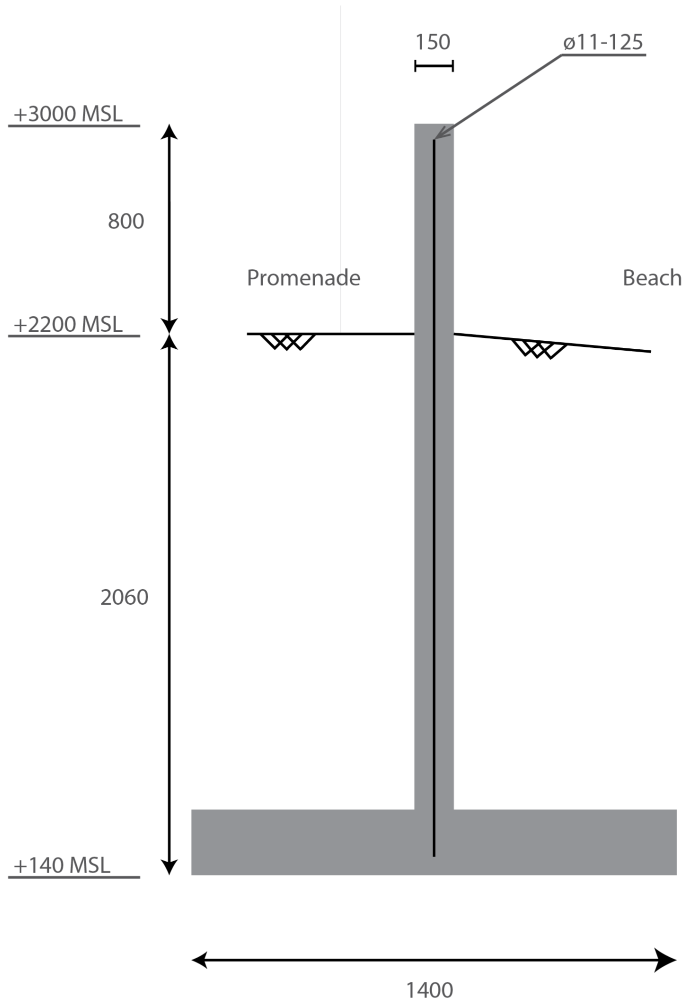


Figure 4.18: A section of the seawall, all measurements are in mm

Stability
The design of the foundation of the wall is done by assessing the method of Blum. With this method, the minimum depth of the wall is derived with as key requirement not to have too much horizontal deflection. Blum's method is presented in appendix L.3. The result of Blum's method shows an embedded depth of the sea wall of $t = 2.06\ m$ allowing limited horizontal deflection and therefore allowing the wall to be stable.

Concerning stability, the depth of the foundation is depending on the design. Blum's method is applied for a I-shaped seawall, but given the, for example, shallow foundation, the effects of the shallow foundation on the horizontal stability have been neglected and could possibly benefit the depth. It should be noted that a very shallow foundation is not preferred, since in large storm events, the beach can be eroded and open-up the foundation.

4.5.6. Integrated design

As earlier mentioned in paragraph 4.3.2, the blockage of the view and fitting in the surroundings are the highest weighted criteria in the MCA. Due to the limited construction height of 0.80 m a reinforced concrete seawall is chosen for a design, which doesn't block the view, but still has to fit into the surroundings of the promenade and beach. An impression of the integrated design is given in figures 4.19 and 4.20.

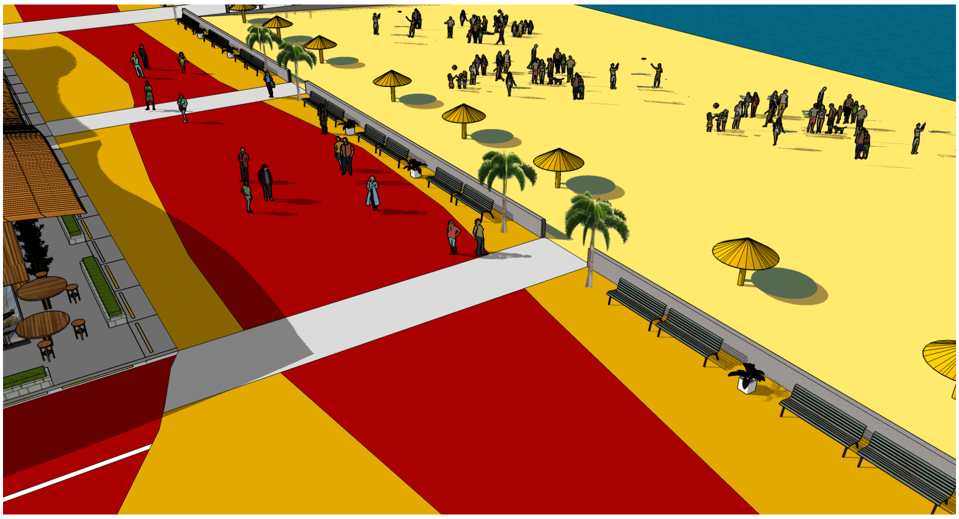


Figure 4.19: Impression integrated design of the seawall

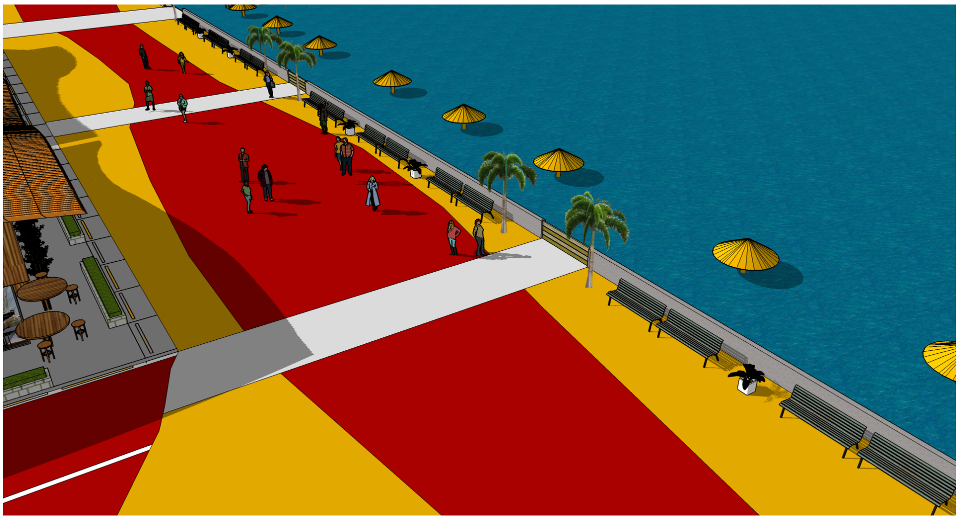


Figure 4.20: Impression integrated design of the seawall and emergency barrier

The seawall is integrated in the surrounding by placing benches on the side of the promenade in combination with plants and palm trees to give an extra exotic dimension to the Golden Mile. Benches should be made out of concrete, which is valuable in two ways; one as it fits perfectly with the concrete seawall and two as it is not able to be stolen. Besides, on the beach side thatch umbrellas are placed, again made from low value material with anchorage in the ground to prevent theft. At last, a slight slope on the promenade towards to seawall has to be realized in order to drain the discharge due to overtopping.

5 Conclusion

5.1. Model-related

The objective for the model-related part of this multidisciplinary project is to create a link between the Indian ocean in front of Durban and the Umgeni river, which flows into this ocean. This missing link is required, because a backwater curve can be opposed in the river by the downstream boundary condition of the oceans water level. Since the river delta of the Umgeni river is (partly) tide-dominant, these influences can reach up far into the Umgeni river.

Two solutions are presented. The first one shows a more time-consuming, but qualitative solution for the missing link. This is the extended Delft3D model. The second one shows a quick and quantitative solution for the link based on the empirical fit of Bresse, written in Python. Both solutions show a big influence of the downstream boundary condition on the Umgeni river. This influence reaches up to about 12 km into the downstream part of the Umgeni. This behaviour can be explained by the mild bed slope of the river at this downstream part. A mild bed slope implies a high adaptation length of the backwater curve, where the river reaches to. From that point on, the rivers bed slope starts inclining drastically. From there, the oceans influence on the river vanishes quickly.

It is recommended to use the extended Delft3D-model in the Delft FEWS system, since this gives the most reliable and accurate outcomes.The influence of the waves into the Umgeni river is negligible. Hence, within the online simulation in Delft3D, the Delft3D-WAVE module can run only on the ocean grid. This saves a lot of computational time, whereas it costs a minimum amount of accuracy in the models output.
The model in Delft3D as it is accompanying this project shows the quantitative behaviour accordingly to the reality. However, in a qualitative way, it still lacks some accuracy in its output. This can be explained by the fact that the model still has to be calibrated to the tidal amplitudes properly.

The model in Python can be run manually in order to get a quick estimate of the resulting water levels in the Umgeni. The two main parameters included in this Python script are the downstream water level from the Indian ocean at the river mouth and the discharge flowing through the Inanda dam. The total Umgeni river has been divided into four sections, with each its own characteristics. These characteristics are mainly based on the different bed slopes and river widths in the Umgeni. Moreover, within this Python script, it is shown what sign has to be given to the local inhabitants based on the current warning signs from the municipality (Watch, Alert and Warning).

The validation of the model is not sufficient yet, due to a lack of measurements stations to make a comparison between the model and the reality.

5.2. Structural-related

The objective for the structural-related part is "Design an engineered flood defence, which integrates in the surroundings and protects the promenade of Durban against severe floods." The flood defence designed is a seawall in combination with a emergency barrier for the beach entrances. The designed seawall is 0.15 m thick, 0.8 m high and made of reinforced concrete. Every 50 m there is a beach entrance where an emergency barrier of pinewood boards can be placed to protect the promenade from flooding. The sea wall is founded on a shallow foundation at 2.06 m deep, with a foot of 1.4 m in width.

Different solutions are considered by designing the barrier at the beachfront. Based on an enhanced Multi Criteria Analysis (MCA) where plenty of alternatives are compared to each other and graded to the criteria, the sea wall is considered to be the best option. For a 50-year design life, the height of the sea wall is determined by considering the effects of astronomical tide, wind set-up, sea level rise, overtopping and an extra surplus height. By determining the effects of wind setup, the Delft3D model is used. Delft3D computes the wave steepness, from which the wind setup is calculated. For the effects of overtopping, the wave characteristics are necessary at the beach front, which are obtained via the 1D-SWAN model with storm conditions as input.

The shallow foundation is designed with very little knowledge of the soil conditions, making an alternative foundation possible, given that more knowledge is obtained. A piled foundation can be considered, or a different configuration of the footing could be possible. However, it is important that the sea wall is not founded too shallow, since in large storm events erosion of the beaches might open-up the footing.

Important for the emergency boards, the timber beams need to be stored, maintained and tested. Our advise is to give the disaster team of the eThekwini municipality control of these actions. Also, for this solution to properly work, there needs to be a warning system which gives a warning when the water is expected to be too high and the emergency barrier need to be placed. A similar warning system is already in use for the Umgeni river, where among others weather input is used to project water levels. Given the Delft3D model, integration of this system is possible.

6
Recommendation

In this chapter recommendations are given upon subjects, which either are outside the scope of the project, of which insufficient information is available, or which are too time consuming to deal within the available time span. As the report consists of the main subjects, the ocean-river system and the structural design of a flood defence, the recommendations are also divided on subject. First, in section 6.1 recommendations on the model are listed and in section 6.2 all recommendations on the flood defence design are listed.

6.1. Model related

In this section all recommendations on the model described, in chapter 3, are listed. The following paragraphs deal with recommendations regarding sediment transport, tributaries in FLOW-model, the friction coefficient, an up-to-date and detailed bathymetry, an improved grid and measurement stations.

6.1.1. Sediment transport

As mentioned in the scope of the project, paragraph 2.2.2, sediment transport is neglected in this report. Commissioned by the eThekwini municipality the main goal of this report is to link the coastal model to the river model and generating a working coastal warning system, similar to the already existing river warning system. As the problems of coastal flooding are common and severe, their number one priority is to warn and to a certain extend protect, by for example structural intervention, their city and its inhabitants. For a future refinement of the models the municipality wants to implement the morphodynamics, by changing the bathymetry from a boundary conditions to a Flexible Mesh Suite of Delft3D, variable over time, as it is dependent on the sediment transport. This implementation also effects the equilibrium situation of the river model and the corresponding parameters like, bed slope and friction coefficients, as stated in section 3.4. The sediment transport is on the other hand influenced by the nourishment. This is regulated by the municipality, so these inconsistencies have to be taken into account while implementing the morphodynamics. By including morphodynamics in the model one can assure an accurate model through time.

6.1.2. Tributaries in FLOW-model

In the renewed model, some discharges are added in the river. The most important discharge is the one at the start of the model, at the Inanda dam. Next to this one, two other discharges are inserted in the river at the location where two tributaries flow into the Umgeni river. Out of all tributaries, these two are considered to be of the most importance in means of discharge supply to the Umgeni river. It is recommended to investigate whether more tributaries have a significant influence on the discharge in the Umgeni river. If the statement above turns out the be true for some tributaries, these can be easily implemented in the model by adding another discharge at the location of that certain tributary. This is also how rainfall can be included, as the majority of the water ends up in these tributaries.

6.1.3. Friction coefficient

The friction coefficient in the river might differ from the already known friction coefficient in the ocean. Different types of sediment and bed slope have a significant influence on the friction coefficient. Since the friction coefficient of the Umgeni river is yet unknown, it is recommended to do some investigation to this value. This can either be done in the same way as for the ocean, resulting in a Manning constant for the friction coefficient. This can also be done by executing flow velocity measurements. The result is the Chezy coefficient, which can be inserted in Delft3D. According to the formula:

$$U = C\sqrt{Ri_b} \tag{6.1}$$

In which:

- U = depth-averaged flow velocity $[m/s]$
- C = Chezy coefficient $[m^{1/2}/s]$
- R = Hydraulic radius $[m]$
- i_b = bed slope [-]

It is recommended to do at least two of these flow velocity measurements, such that one can determine the friction coefficients for both parts in the Umgeni river with a significant different bed slope. See figure 3.16 to identify the two parts of the Umgeni river mentioned. The cross-sections of the rivers are known by the municipality, so the hydraulic radius can be easily determined.

6.1.4. Up-to-date and detailed bathymetry

To have a good start of the model, it is crucial to have a good bathymetry. Especially when the morphodynamics will be in future included in the model. For the results now, only the bathymetry near the Umgeni river mouth is rather recent (2017). The remaining part of the Umgeni is modelled with GIS data, which is less accurate. Using the same survey method as for the lower part of the Umgeni helps in creating a more detailed bathymetry. The ocean bathymetry can be less detailed. Currently, the outer edge of several 100 metres along the coast has bathymetry entries of 1 m only. New measurements should be carried out that include the elevation of the beaches up to the promenade. The current bathymetry does not allow for accurate results, especially in wave calculations.

6.1.5. Improved grid

The new grid for the model is an extension of the old ocean grid, created by drawing a grid along the river contours all the way up to Inanda Dam. Ideally, the grid cell size of the river is much smaller as the flow is more complex, especially around the river mouth. This is attempted to obtain by locally refining the grid. This works to some extent, but it also automatically refines grid cells in the ocean as it can only be done along a full grid line. This results in unnecessarily small grid cells in the ocean. Delft3D has difficulties with these changes from large to small cells, which can cause errors. Also, small grid cells result in more computation time as there simply are more of them. Grid cell sizes smaller than those adopted in this report tend to give many errors, but still it is desired to make them smaller for more detailed results. A possible solution is to implement the Flexible Mesh Suite of Delft3D, which automatically creates a detailed grid if necessary. This is also a good way to optimize computation time. The grid edge along the shoreline should also be treated carefully. Complex wave processes occur here and a too coarse grid can result in inaccurate results. A finer mesh is also recommended here.

Another way to improve the grid is by reducing the overall size. In the conclusion it was described that the ocean has a certain reach into the Umgeni river. For the purpose of this model, which is predicting potential floods, is has no use to include the remaining part of the river up to the Inanda Dam. Leaving this part out helps reducing computation time. One consequence is that new boundary conditions must be defined at the new model edge.

6.1.6. Measurement stations

In order to validate the renewed model properly, more measurement stations need to be placed. As described before, the Umgeni can be divided into a section with a steep slope and a section with a mild slope. The oceans influence is expected to reach easily to this transition and then to fade out rapidly. Therefore, it is recommended to place a measurement station just before this transition and just after this transition. To find the exact location of this transition, see figure 3.16. Besides, it is recommended to apply two or three additional measurement stations in between this transition point and the Umgeni river mouth, in order to generate a global longitudinal profile which can be compared with the models output.

6.2. Structural related

In this section all recommendations on the structural design, described in chapter 4, are listed. The following paragraphs deal with recommendations regarding the ocean warning system, harbour extension, climate change, vulnerable areas, a dune design and the soil data research.

6.2.1. Ocean warning system

For the emergency barrier to work properly, there should be an active ocean warning system. The municipality needs to be warned for high incoming water and should respond by contacting the disaster team, who is responsible for evacuating the coastal regions and installing the emergency barrier. The ocean model runs in Delft3D and loses accuracy near the beach front, thus wave conditions more out of the coast (ca. $200-500\ m$) have to be used. From this point onward waves conditions and water level can be estimated with formulas/calculations of what can be expected on the beach front. A certain warning value can be derived for a specific point out of the coast and if these values, significant wave height and/or water level, is exceeded, the beach front must be evacuated and the emergency barrier must be installed. This only holds under the condition that the warning system acts in advance, as the installation of the emergency barrier requires certain time, due to organization, transportation and installation itself. The Umgeni river projects conditions with a 3-day ahead forecast, which can be considered to be applied similar for the beach front.

Another option is to use SWAN Delft in a 1D-model to predict the wave height and water set-up at the beach front directly. The 1D-model can be linked with the Delft3D ocean-river model, which then can be linked to the weather forecasts as input. When the 1D-model of SWAN is linked with the Delft3D model, warnings for both river and ocean side can be given at the same time. The 1D-model can be modelled for various cross-sections along the coastline and supply reliable wave characteristics at the beach front. If boundaries are formulated and if exceeded the beach needs to be evacuated and the emergency barriers need to be installed. Similar as mentioned above, this only holds under the conditions that the warning system acts ahead.

Both options described above are recommended with a preference to the 1D-model in SWAN Delft, as it is fast and quick to integrate in the already existing Delft3D model. However, the best solution is when Delft3D is self able to give accurate and reliable data at the beach front. In order to be able so, bathymetry has to be extended including the beach profile and the grid has to be refined at the beach front. On the other hand, this slows down the running process and therefore one of the above mentioned options can be integrated.

6.2.2. Harbour extension

The current extension of the harbour entrance has caused a disequilibrium in the sediment mass balance, $S_{in} < S_{out}$. In a perfect situation the lost amount of sediment should be trapped near the breakwater. Nevertheless, in the situation of the Durban harbour entrance, less sediment is trapped than is eroded from the beaches. For this report, sediment transport is neglected, as mentioned in paragraph 2.2.2, and therefore no further research is done in optimizing the sediment mass balance. Due to the long shore sediment transport and the wide opening of the harbour entrance, optimization can be achieved by dredging the sediment efficiently in the harbour channel. Another option is to increase the efficiency of the sand trap near the breakwater by adapting the design or shape. A last option, which can be analyzed is an improved design of the harbour entrance, which stimulates a bypass effect for the long shore sediment transport and results possibly in less erosion on the other side of the harbour entrance.

6.2.3. Climate change

Allowances for sea level rise have been calculated to a certain extend but are not exact given the assumptions made. Assumptions regarding the different RCP scenarios are hard to project, but assumptions related to the available data to actually calculate the allowances show already some possible deviation. For example, the Gumbel parameter λ could already show a maximum benefit (less allowance) of 3.4 cm. The assumed standard deviation is, compared to the real standard deviation, different and could improve the calculation for the allowance and benefit the allowance too. Here is room for improvement and would be recommended to look into. Especially for detailed calculations in a later stage given that, for example, a 5 cm lower concrete seawall over the length of $3-4$ km already reduces the costs considerably.

6.2.4. Vulnerable areas

This report only focuses on the protection of the promenade against flooding. For an integral solution for the complete city of Durban one can also look into flooding of the hinterland via the harbour area, since that area is also at low altitude and many people live and work in that specific area. Besides, flooding caused by the Umgeni river can be looked at, as due to sudden high discharges the river can flood adjacent areas.

6.2.5. Dune design

Based upon the MCA in section 4.3 the seawall combined with an emergency barrier is the most suitable solution regarding the given criteria, followed by a dune. In this report the seawall in combination with an emergency barrier is fully elaborated. Due to a lack of expertise, different formulations for determining the overtopping of a dune and given the scope/time span of the project, this option is left out. However, for the completeness of a qualitative comparison of the best design option one can look at a more detailed elaboration of a dune design, in order to give a complete, worked out advice on the structural design of a flood defence for the selected vulnerable area.

6.2.6. Soil data research

For future research and for the construction process, soil information is required at the exact location of design. Due to poor documentation, insufficient data was available to come up with a detailed design for the foundation. As is determined for this design, there is no pile foundation required but this can be different depending on the soil conditions and in terms of optimizing the design. Besides, no information is available on cables and pipes for the construction site. In order to have a proper building process, further soil data research has to be carried out, by for example cone penetration tests (CPTs).

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A

Flood of Umgeni river in 1987

Figure A.1: Umgeni river during flooding (top) and the normal Umgeni river (bottom)

B Stakeholder analysis

This appendix contains a detailed elaboration of all the refinements of the stakeholder analyses performed, different from the report: *Flood Durban Safety (2016)*. [1] Both the stakeholder identification and the stakeholder characteristics of the humanitarian analysis and the material analysis are treated, followed by a brief description of some specific terms.

B.1. Stakeholder identification

In this section, the stakeholders which have changed with respect to the situation described in the report *Flood Safety Durban* are discussed.

Daily users of the focus area
For this report the ocean users and daily visitors of the beach are combined in one group, as the daily users of the focus area. The only difference is the way the use the focus area, as one uses the ocean and the other the beach. In general both groups are similar affected by the objectives of the stakeholder analyses. Due to flooding both the beach and the ocean are unusable.

B.2. Stakeholder characteristic

In this section first the refined characteristics regarding the humanitarian analysis are treated and next the refined characteristics regarding the material analysis.

B.2.1. Humanitarian analysis

Again, only the specific differences are discussed compared to the report *Flood Safety Durban*.

Insurance companies
Insurance companies pay for the flood damage of the insured properties. The whole insurance business is about money making and therefore the interest is active, as they want to pay as little as possible. Their power is high as they have a lot of money. Their attitude towards a warning system is positive as it doesn't reduce the risk of flooding, which keeps their business alive, but it reduces the potential damage, which saves them money.

Local businesses
Local businesses are small and have low power compared to multinationals. Although they aren't able to have a great influence on the municipality, the are active because a warning system can give them just enough time to save their businesses on the short term.

Lower class residents
Lower class residents have little property and therefore also little to loose. For this reasoning it makes them a low power and passive stakeholder. Nevertheless a warning system can

save their lives, as many of them life on the streets or in weak houses and now the get an opportunity to bring themselves in safety.

Daily users of the focus area
The daily users of the focus area, are the fishermen, surfers and beach visitors. These groups of people don't come to the beach when it is bad weather and therefore don't profit much from a warning system. For this reason their power is low and their interest is passive, but their attitude to the system is positive as it also predicts the weather in advance, which ensures that they won't come to the focus area on bad days.

B.2.2. Material analysis

Here, the stakeholders opinion regarding the type of solution is discussed. Some solutions or materials are not preferred by some of the stakeholders for specific reasons.

Daily users of the focus area
The daily users of the focus area, are the fishermen, surfers and beach visitors. Even when their actual influence in decision taking is low, this group covers a great part of the economic welfare of Durban, which makes their power medium. Besides, they are extremely affected when structural intervention is needed, for example on one hand a breakwater damages the complete wave climate and on the other hand a quay wall has a big impact on the use of the promenade. This reasoning makes this group active in term of interest and a blocker, because their priority is not safety of the city, but the pleasure of the beach day.

B.3. Stakeholder management

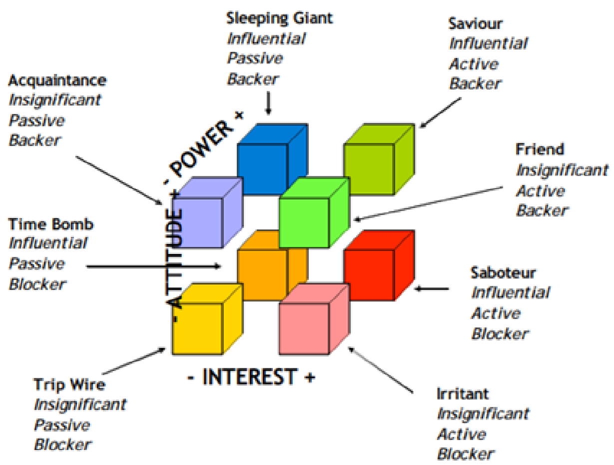


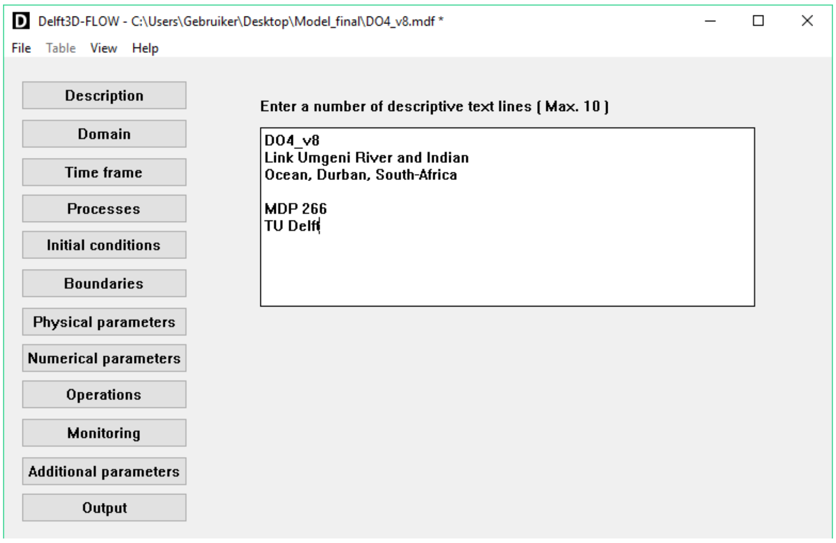
Figure B.1: Stakeholder management in power, interest and attitude

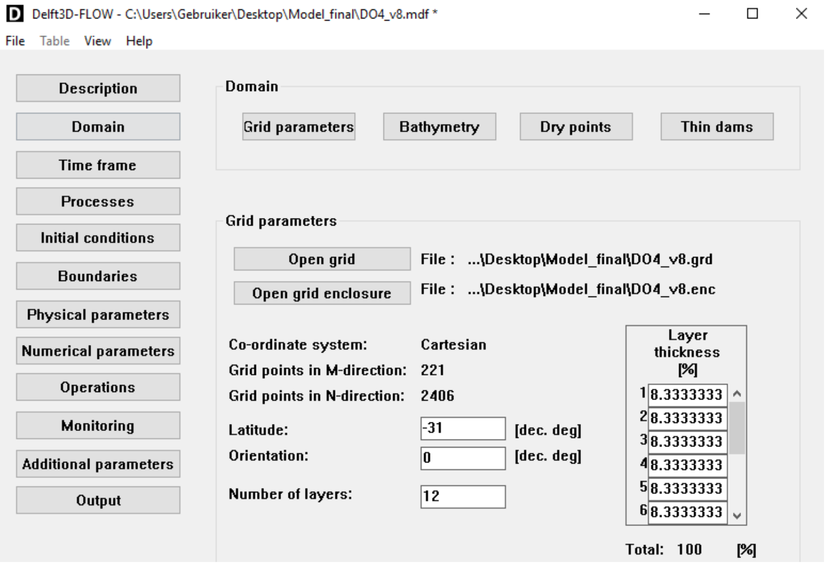
- **Saviour**: powerful, high interest, positive attitude or alternatively influential, active, backer. They need to be paid attention to; you should do whatever necessary to keep them on your side – pander to their needs.
- **Friend**: low power, high interest, positive attitude or alternatively insignificant, active, backer. They should be used as a confidant or sounding board.
- **Saboteur**: powerful, high interest, negative attitude or alternatively influential, active, blocker. They need to be engaged in order to disengage. You should be prepared to 'clean-up after them'.
- **Irritant**: low power, high interest, negative attitude or alternatively insignificant, active, blocker. They need to be engaged so that they stop 'eating away' and then be 'put back in their box'.

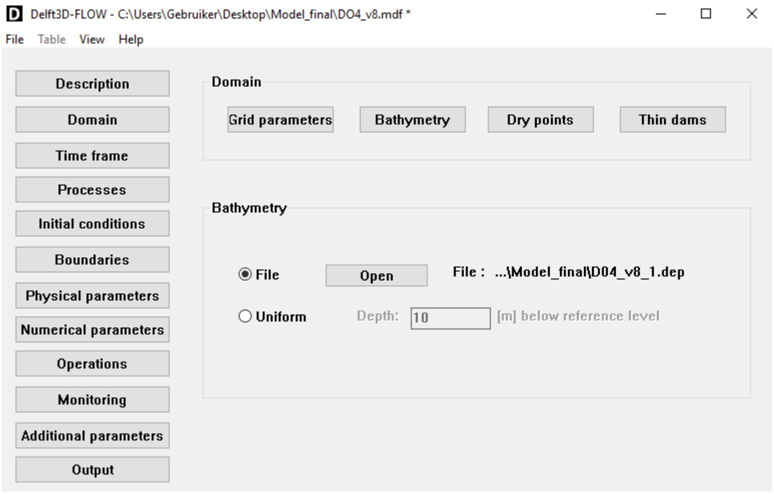
- **Sleeping Giant**: powerful, low interest, positive attitude or alternatively influential, passive, backer. They need to be engaged in order to awaken them.
- **Acquaintance**: low power, low interest, positive attitude or alternatively insignificant, passive, backer. They need to be kept informed and communicated with on a 'transmit only' basis.
- **Time Bomb**: powerful, low interest, negative attitude or alternatively influential, passive, blocker. They need to be understood so they can be 'defused before the bomb goes off'.
- **Trip Wire**: low power, low interest, negative attitude or alternatively insignificant, passive, blocker. They need to be understood so you can 'watch your step' and avoid 'tripping up'.

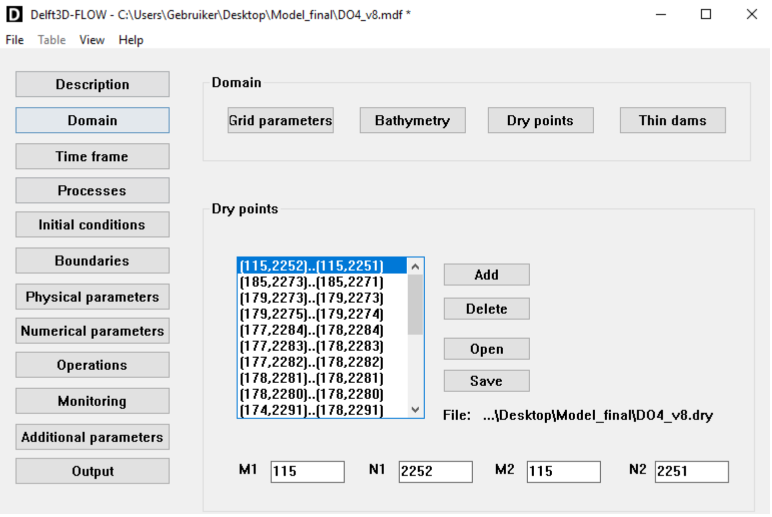
C Set-up of the Delft3D model

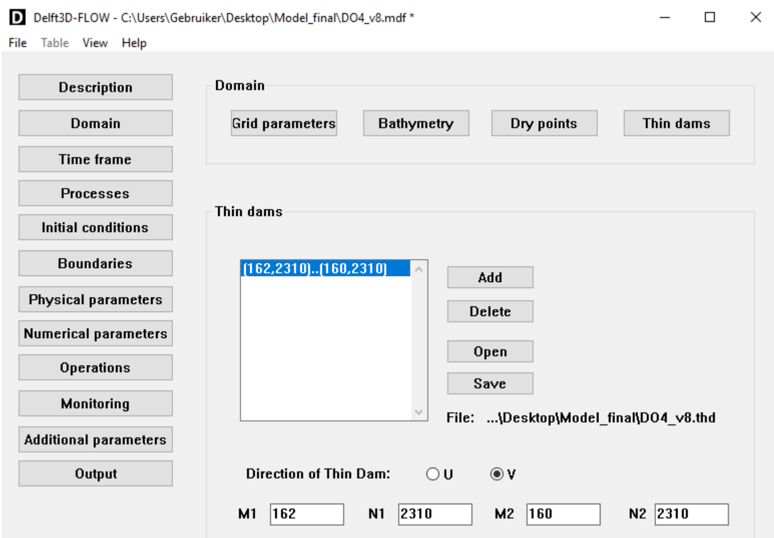
In this appendix, the set-up of the model in Delft3D accompanying Chapter 3 is shown. Both the Delft3D-FLOW module and the Delft3D-WAVE module are presented. The set-up is shown in order to give a better insight in the settings of the Delft3D-model. It is not possible to copy the model, since the required files uploaded in Delft3D are not attached.

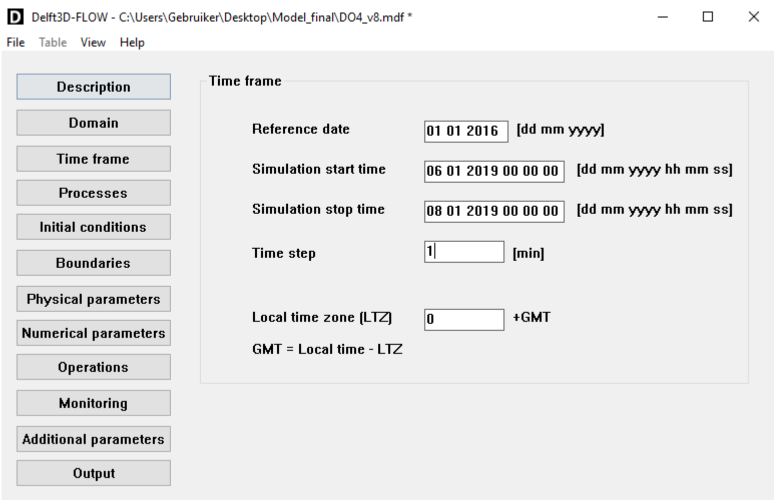


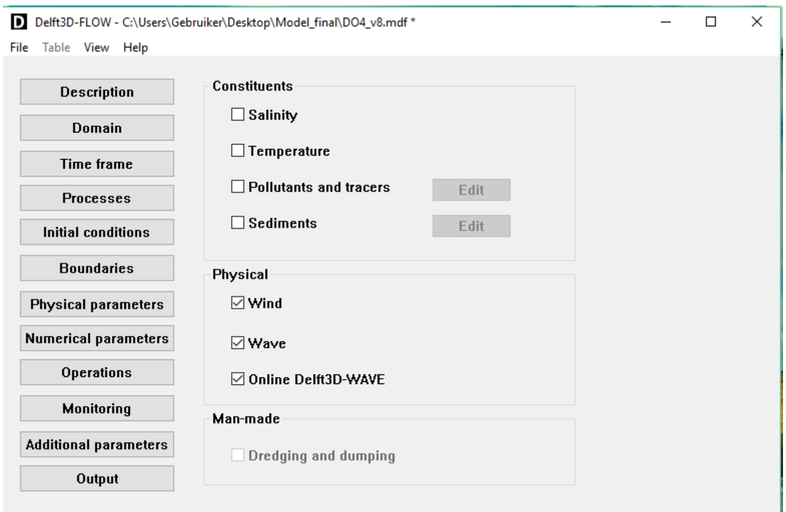

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Domain
Grid parameters
Bathymetry
Dry points
Thin dams
Grid parameters
Open grid
File : ...\Desktop\Model_final\DO4_v8.grd
Open grid enclosure
File : ...\Desktop\Model_final\DO4_v8.enc
Co-ordinate system: Cartesian
Grid points in M-direction: 221
Grid points in N-direction: 2406
Latitude: -31 [dec. deg]
Orientation: 0 [dec. deg]
Number of layers: 12
Layer thickness [%]
1 8.3333333
2 8.3333333
3 8.3333333
4 8.3333333
5 8.3333333
6 8.3333333
Total: 100 [%]

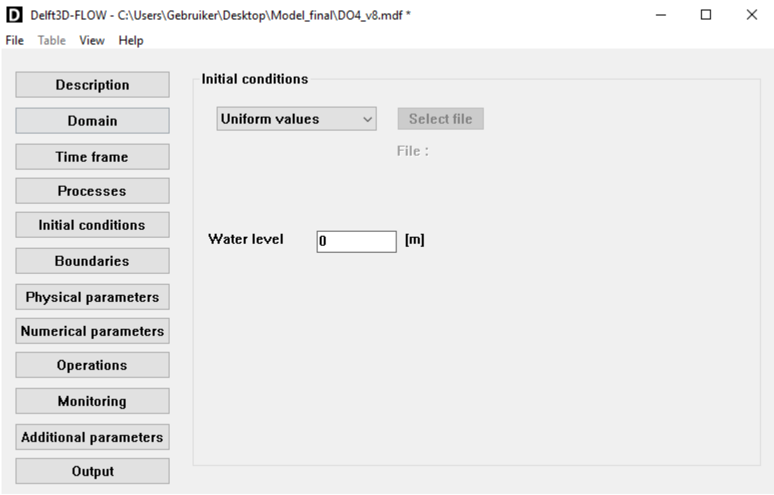

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Domain
Grid parameters
Bathymetry
Dry points
Thin dams
Bathymetry
File
Open
File : ...\Model_final\DO4_v8_1.dep
Uniform
Depth: 10 [m] below reference level

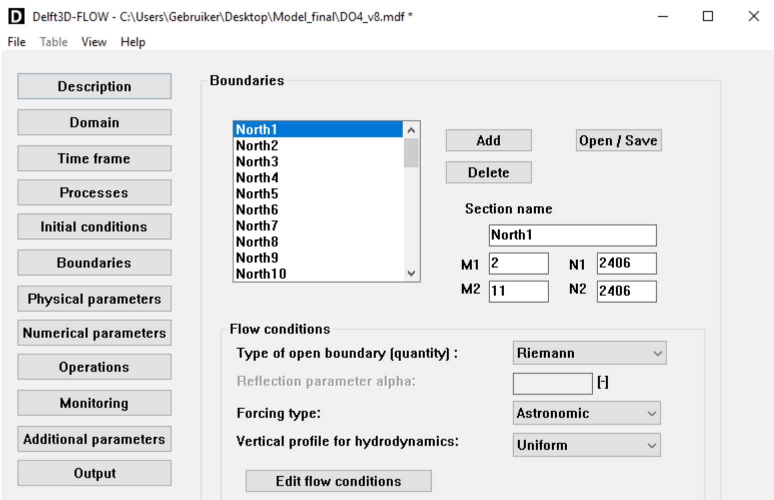

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Domain
Grid parameters
Bathymetry
Dry points
Thin dams
Dry points
(115,2252)..(115,2251)
(185,2273)..(185,2271)
(179,2273)..(179,2273)
(179,2275)..(179,2274)
(177,2284)..(178,2284)
(177,2283)..(178,2283)
(177,2282)..(178,2282)
(178,2281)..(178,2281)
(178,2280)..(178,2280)
(174,2291)..(178,2291)
Add
Delete
Open
Save
File: ...\Desktop\Model_final\DO4_v8.dry
M1 115
N1 2252
M2 115
N2 2251

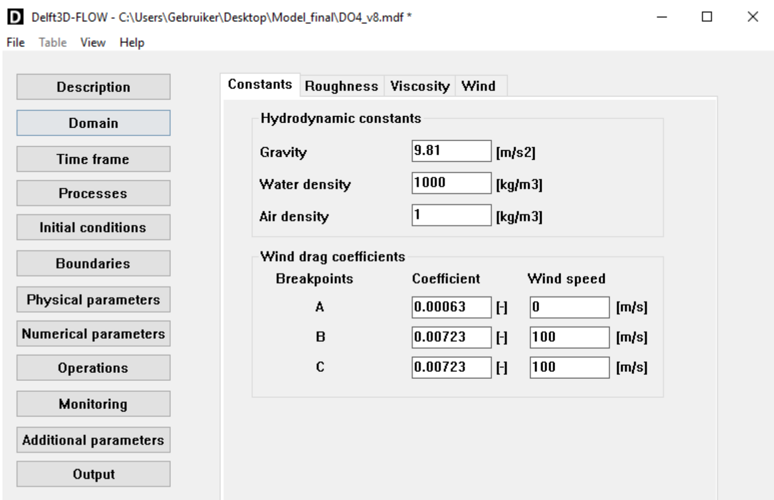

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Domain
Grid parameters
Bathymetry
Dry points
Thin dams
Thin dams
(162,2310)..(160,2310)
Add
Delete
Open
Save
File: ...\Desktop\Model_final\DO4_v8.thd
Direction of Thin Dam: U V
M1 162 N1 2310 M2 160 N2 2310

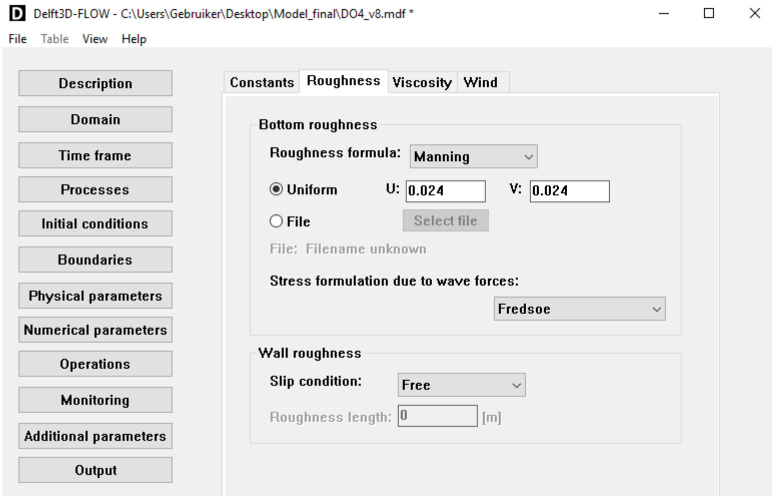

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Time frame
Reference date 01 01 2016 [dd mm yyyy]
Simulation start time 06 01 2019 00 00 00 [dd mm yyyy hh mm ss]
Simulation stop time 08 01 2019 00 00 00 [dd mm yyyy hh mm ss]
Time step 1 [min]
Local time zone (LTZ) 0 +GMT
GMT = Local time - LTZ

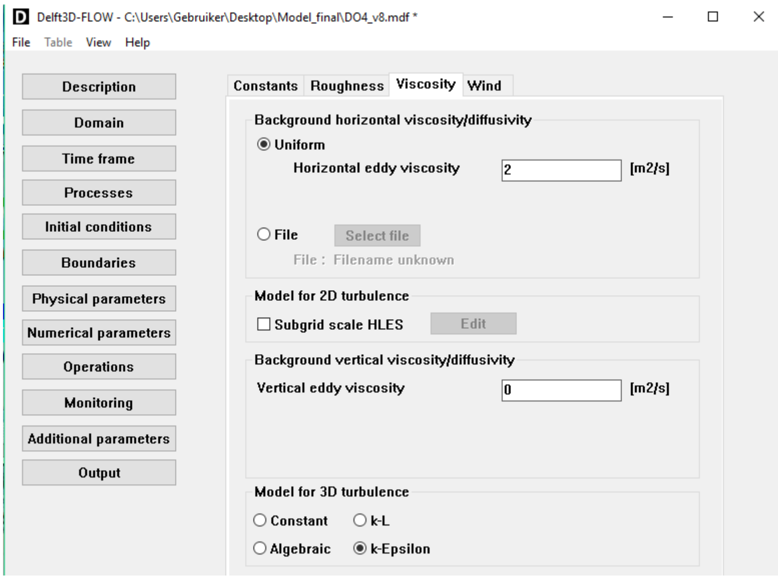

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Constituents
Salinity
Temperature
Pollutants and tracers
Edit
Sediments
Edit
Physical
Wind
Wave
Online Delft3D-WAVE
Man-made
Dredging and dumping

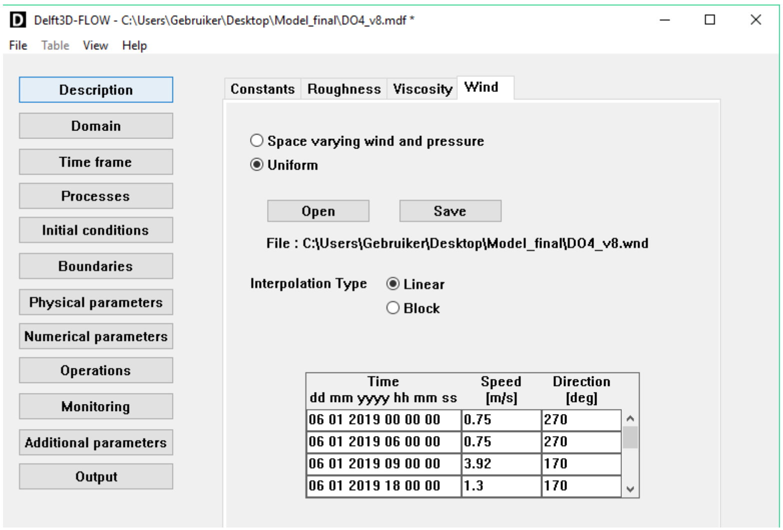

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Initial conditions
Uniform values
Select file
File :
Water level
0
[m]

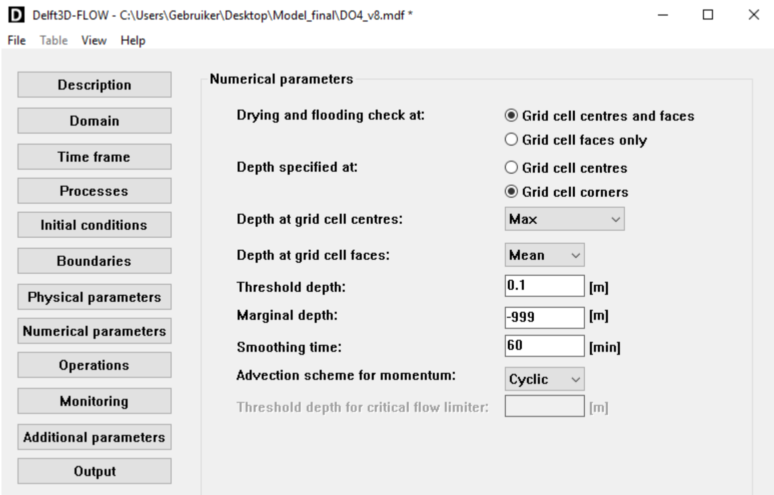

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Boundaries
North1
North2
North3
North4
North5
North6
North7
North8
North9
North10
Add
Open / Save
Delete
Section name
North1
M1 2
N1 2406
M2 11
N2 2406
Flow conditions
Type of open boundary (quantity) :
Riemann
Reflection parameter alpha:
[-]
Forcing type:
Astronomic
Vertical profile for hydrodynamics:
Uniform
Edit flow conditions

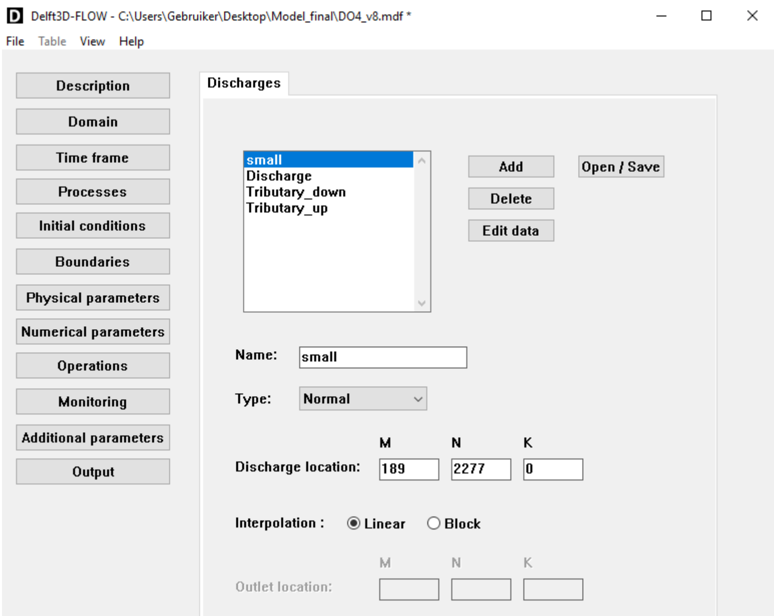

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Constants
Roughness
Viscosity
Wind
Hydrodynamic constants
Gravity 9.81 [m/s2]
Water density 1000 [kg/m3]
Air density 1 [kg/m3]
Wind drag coefficients
Breakpoints
Coefficient
Wind speed
A 0.00063 [-] 0 [m/s]
B 0.00723 [-] 100 [m/s]
C 0.00723 [-] 100 [m/s]

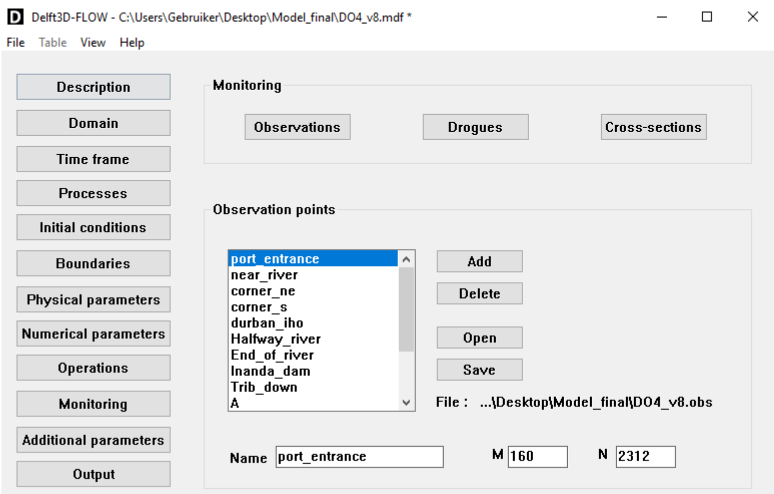

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Constants Roughness Viscosity Wind
Bottom roughness
Roughness formula: Manning
Uniform U: 0.024 V: 0.024
File Select file
File: Filename unknown
Stress formulation due to wave forces:
Fredsoe
Wall roughness
Slip condition: Free
Roughness length: 0 [m]

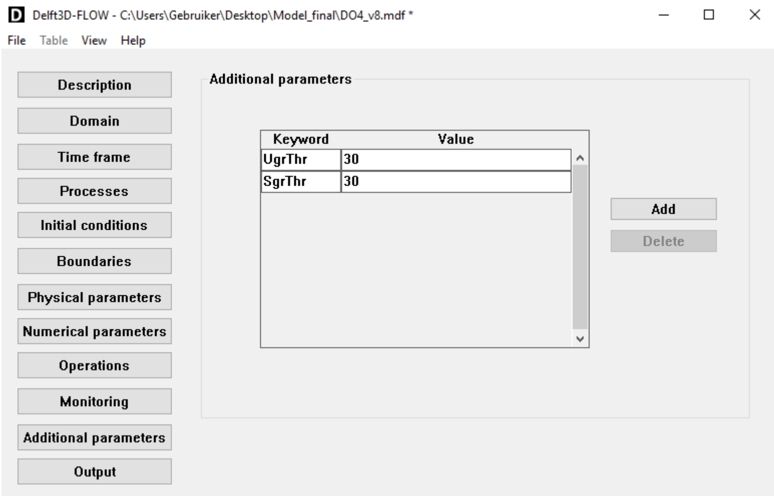

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Constants Roughness Viscosity Wind
Background horizontal viscosity/diffusivity
Uniform
Horizontal eddy viscosity 2 [m2/s]
File Select file
File : Filename unknown
Model for 2D turbulence
Subgrid scale HLES Edit
Background vertical viscosity/diffusivity
Vertical eddy viscosity 0 [m2/s]
Model for 3D turbulence
Constant k-L
Algebraic k-Epsilon

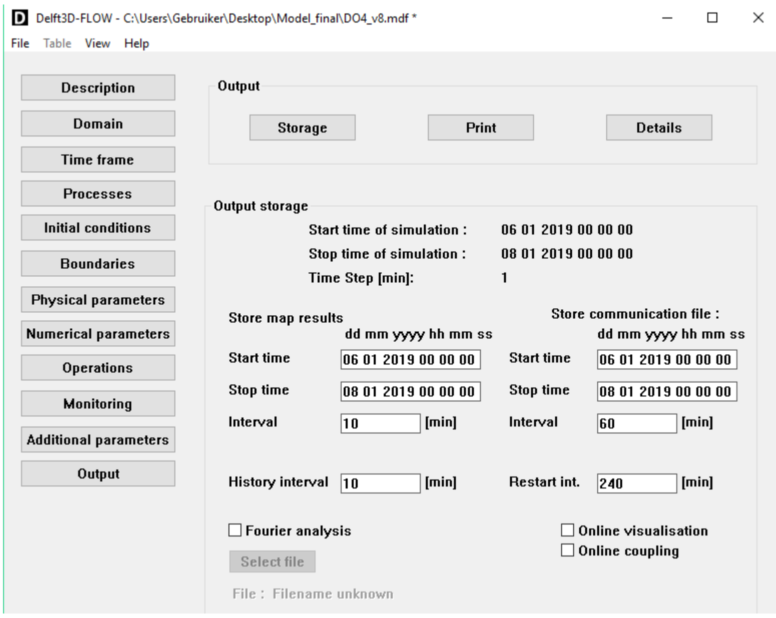

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Constants Roughness Viscosity Wind
Space varying wind and pressure
Uniform
Open Save
File : C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.wnd
Interpolation Type Linear
Block
Time dd mm yyyy hh mm ss
Speed [m/s]
Direction [deg]
06 01 2019 00 00 00 0.75 270
06 01 2019 06 00 00 0.75 270
06 01 2019 09 00 00 3.92 170
06 01 2019 18 00 00 1.3 170

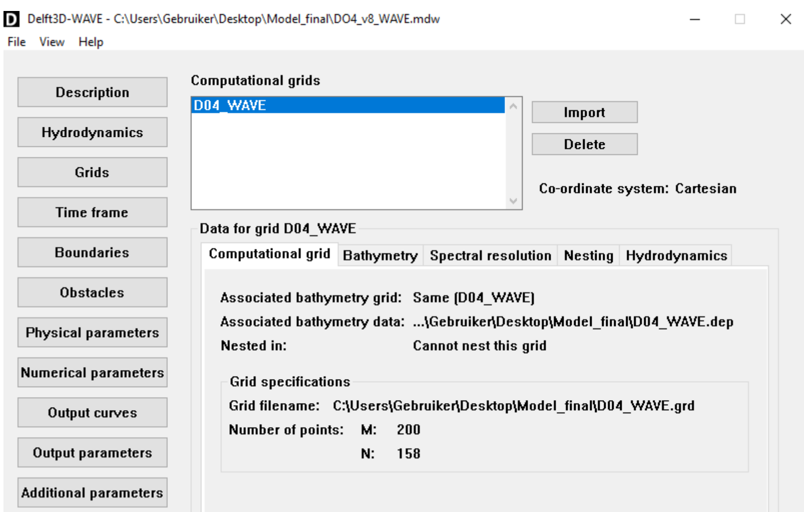

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Numerical parameters
Drying and flooding check at:
Grid cell centres and faces
Grid cell faces only
Depth specified at:
Grid cell centres
Grid cell corners
Depth at grid cell centres:
Max
Depth at grid cell faces:
Mean
Threshold depth:
0.1
[m]
Marginal depth:
-999
[m]
Smoothing time:
60
[min]
Advection scheme for momentum:
Cyclic
Threshold depth for critical flow limiter:
[m]

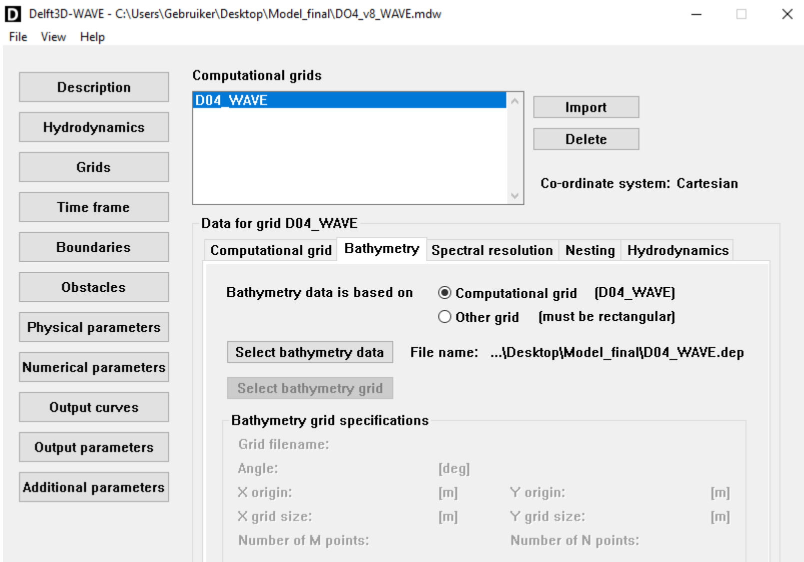

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Discharges
small
Discharge
Tributary_down
Tributary_up
Add
Open / Save
Delete
Edit data
Name:
small
Type:
Normal
M
N
K
Discharge location:
189
2277
0
Interpolation :
Linear
Block
M
N
K
Outlet location:

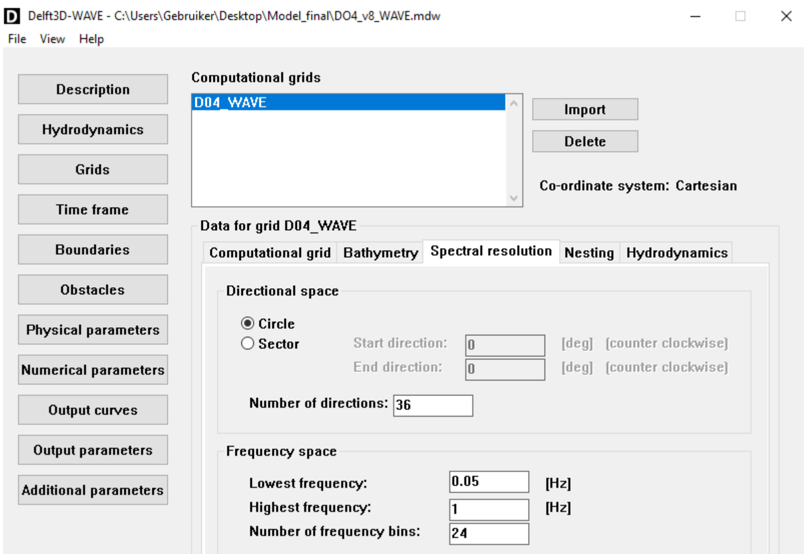

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Monitoring
Observations
Drogues
Cross-sections
Observation points
port_entrance
near_river
corner_ne
corner_s
durban_iho
Halfway_river
End_of_river
Inanda_dam
Trib_down
A
Add
Delete
Open
Save
File : ...\Desktop\Model_final\DO4_v8.obs
Name
port_entrance
M
160
N
2312

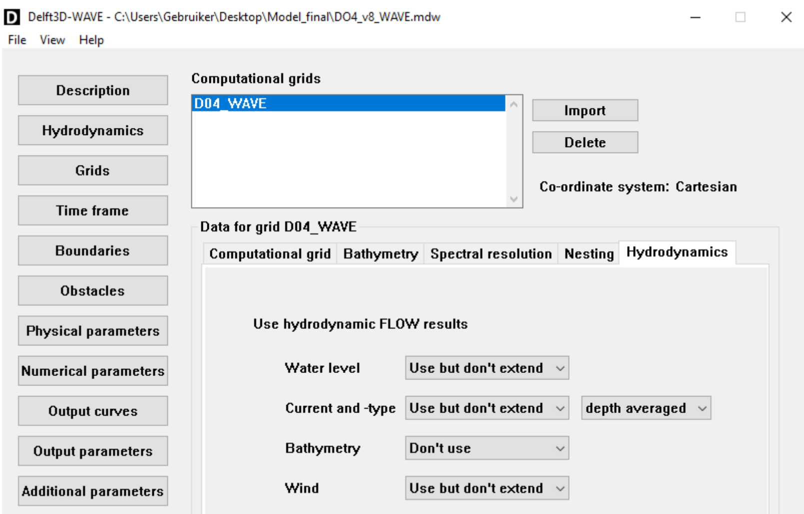

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\D04_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Additional parameters
Keyword Value
UgrThr 30
SgrThr 30
Add
Delete

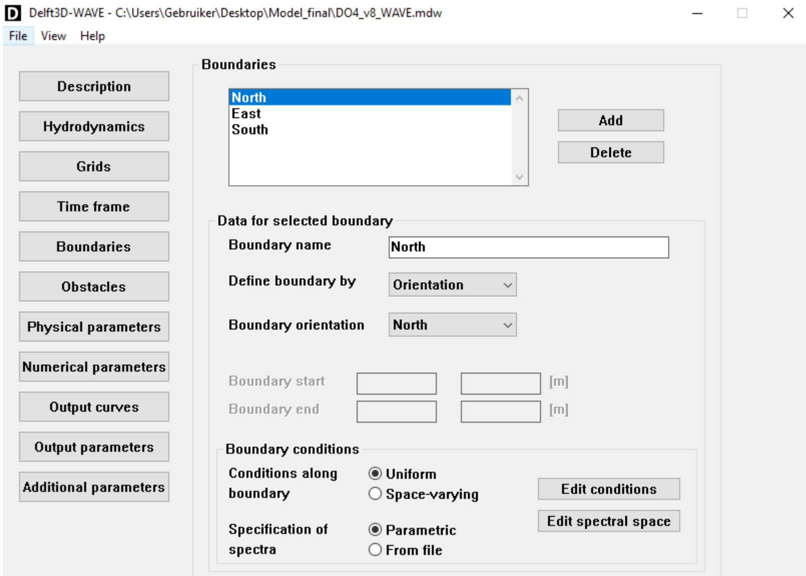

Delft3D-FLOW - C:\Users\Gebruiker\Desktop\Model_final\D04_v8.mdf *
File Table View Help
Description
Domain
Time frame
Processes
Initial conditions
Boundaries
Physical parameters
Numerical parameters
Operations
Monitoring
Additional parameters
Output
Output
Storage
Print
Details
Output storage
Start time of simulation : 06 01 2019 00 00 00
Stop time of simulation : 08 01 2019 00 00 00
Time Step [min]: 1
Store map results
dd mm yyyy hh mm ss
Start time 06 01 2019 00 00 00
Stop time 08 01 2019 00 00 00
Interval 10 [min]
Store communication file :
dd mm yyyy hh mm ss
Start time 06 01 2019 00 00 00
Stop time 08 01 2019 00 00 00
Interval 60 [min]
History interval 10 [min]
Restart int. 240 [min]
Fourier analysis
Select file
File : Filename unknown
Online visualisation
Online coupling

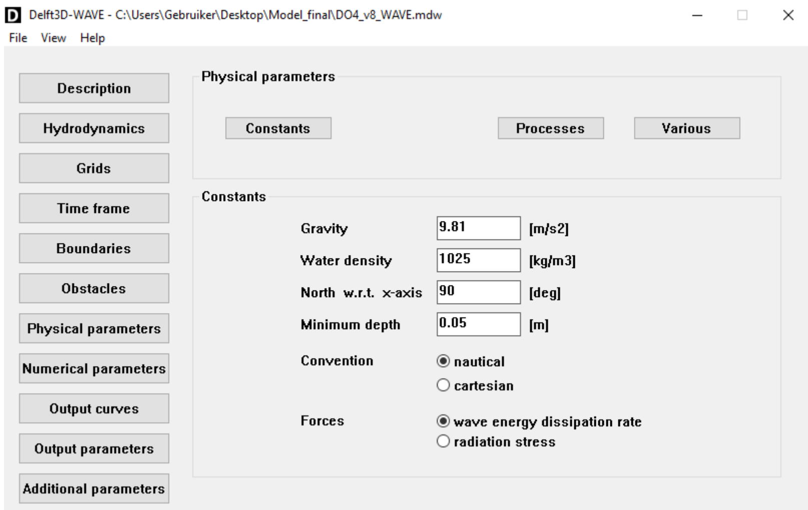

Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\D04_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Computational grids
D04_WAVE
Import
Delete
Co-ordinate system: Cartesian
Data for grid D04_WAVE
Computational grid
Bathymetry
Spectral resolution
Nesting
Hydrodynamics
Associated bathymetry grid: Same (D04_WAVE)
Associated bathymetry data: ...\Gebruiker\Desktop\Model_final\D04_WAVE.dep
Nested in: Cannot nest this grid
Grid specifications
Grid filename: C:\Users\Gebruiker\Desktop\Model_final\D04_WAVE.grd
Number of points: M: 200
N: 158

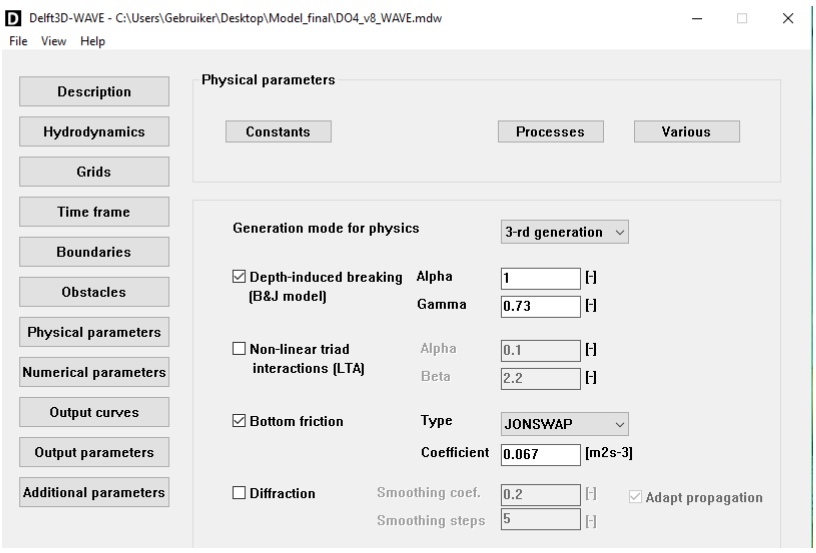

Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\D04_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Computational grids
D04_WAVE
Import
Delete
Co-ordinate system: Cartesian
Data for grid D04_WAVE
Computational grid
Bathymetry
Spectral resolution
Nesting
Hydrodynamics
Bathymetry data is based on
Computational grid [D04_WAVE]
Other grid [must be rectangular]
Select bathymetry data
File name: ...\Desktop\Model_final\D04_WAVE.dep
Select bathymetry grid
Bathymetry grid specifications
Grid filename:
Angle: [deg]
X origin: [m]
Y origin: [m]
X grid size: [m]
Y grid size: [m]
Number of M points:
Number of N points:

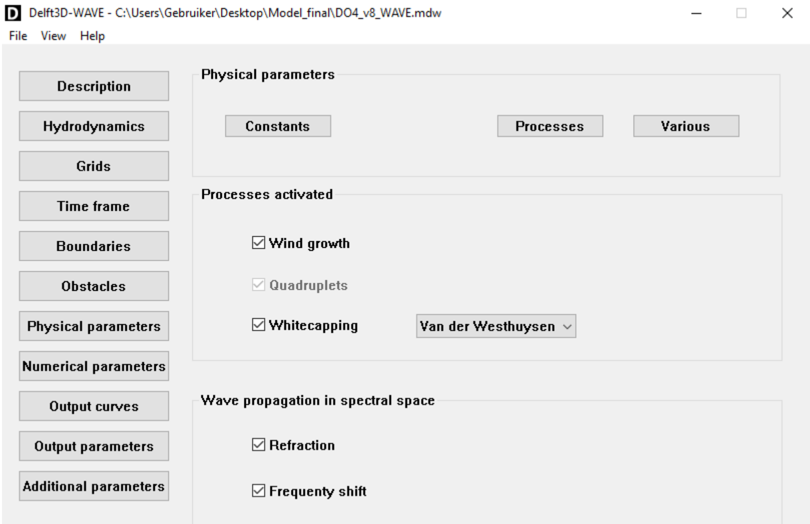

Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\D04_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Computational grids
D04_WAVE
Import
Delete
Co-ordinate system: Cartesian
Data for grid D04_WAVE
Computational grid
Bathymetry
Spectral resolution
Nesting
Hydrodynamics
Directional space
Circle
Sector
Start direction: 0 [deg] [counter clockwise]
End direction: 0 [deg] [counter clockwise]
Number of directions: 36
Frequency space
Lowest frequency: 0.05 [Hz]
Highest frequency: 1 [Hz]
Number of frequency bins: 24

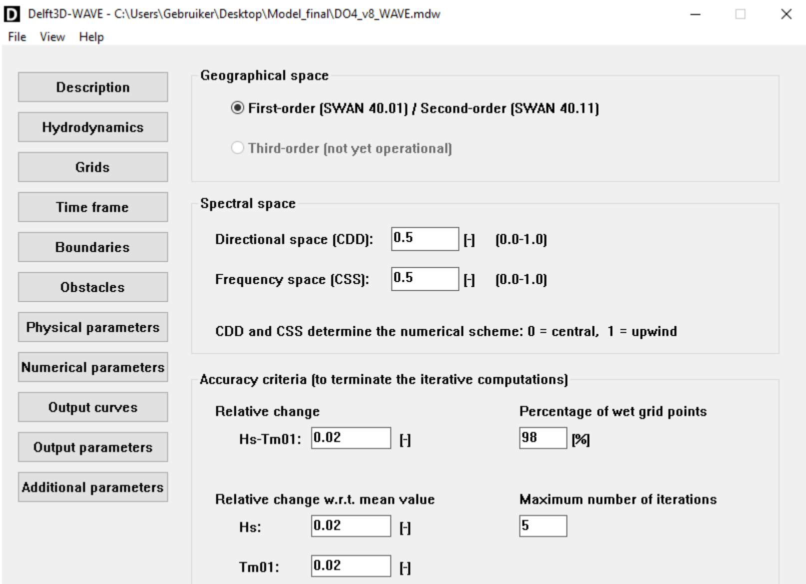

Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\D04_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Computational grids
D04_WAVE
Import
Delete
Co-ordinate system: Cartesian
Data for grid D04_WAVE
Computational grid
Bathymetry
Spectral resolution
Nesting
Hydrodynamics
Use hydrodynamic FLOW results
Water level Use but don't extend
Current and -type Use but don't extend depth averaged
Bathymetry Don't use
Wind Use but don't extend

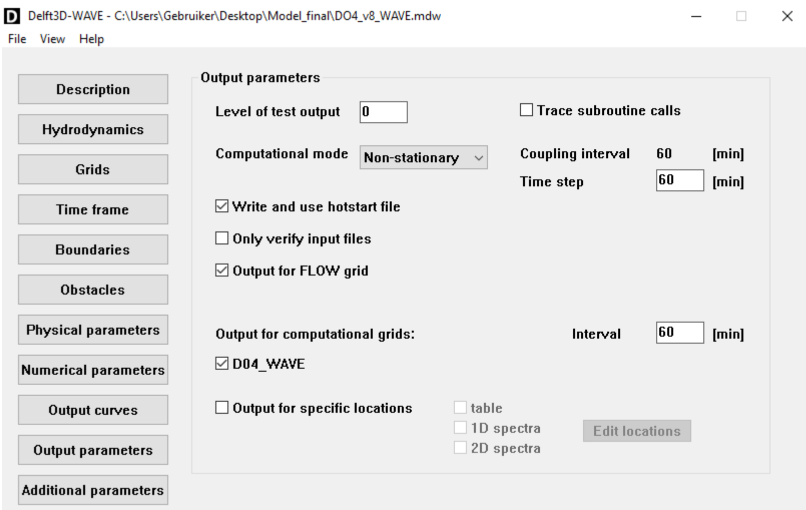

Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Boundaries
North
East
South
Add
Delete
Data for selected boundary
Boundary name
North
Define boundary by
Orientation
Boundary orientation
North
Boundary start
[m]
Boundary end
[m]
Boundary conditions
Conditions along boundary
Uniform
Space-varying
Edit conditions
Specification of spectra
Parametric
From file
Edit spectral space


Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Physical parameters
Constants
Processes
Various
Constants
Gravity
9.81
[m/s2]
Water density
1025
[kg/m3]
North w.r.t. x-axis
90
[deg]
Minimum depth
0.05
[m]
Convention
nautical
cartesian
Forces
wave energy dissipation rate
radiation stress


Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Physical parameters
Constants
Processes
Various
Generation mode for physics
3-rd generation
Depth-induced breaking (B&J model)
Alpha
1
[-]
Gamma
0.73
[-]
Non-linear triad interactions (LTA)
Alpha
0.1
[-]
Beta
2.2
[-]
Bottom friction
Type
JONSWAP
Coefficient
0.067
[m2s-3]
Diffraction
Smoothing coef.
0.2
[-]
Smoothing steps
5
[-]
Adapt propagation


Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Physical parameters
Constants
Processes
Various
Processes activated
Wind growth
Quadruplets
Whitecapping
Van der Westhuysen
Wave propagation in spectral space
Refraction
Frequenty shift


Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Geographical space
First-order (SWAN 40.01) / Second-order (SWAN 40.11)
Third-order (not yet operational)
Spectral space
Directional space (CDD): 0.5 [-] (0.0-1.0)
Frequency space (CSS): 0.5 [-] (0.0-1.0)
CDD and CSS determine the numerical scheme: 0 = central, 1 = upwind
Accuracy criteria (to terminate the iterative computations)
Relative change
Hs-Tm01: 0.02 [-]
Percentage of wet grid points
98 [%]
Relative change w.r.t. mean value
Hs: 0.02 [-]
Tm01: 0.02 [-]
Maximum number of iterations
5


Delft3D-WAVE - C:\Users\Gebruiker\Desktop\Model_final\DO4_v8_WAVE.mdw
File View Help
Description
Hydrodynamics
Grids
Time frame
Boundaries
Obstacles
Physical parameters
Numerical parameters
Output curves
Output parameters
Additional parameters
Output parameters
Level of test output 0
Trace subroutine calls
Computational mode Non-stationary
Coupling interval 60 [min]
Time step 60 [min]
Write and use hotstart file
Only verify input files
Output for FLOW grid
Output for computational grids:
Interval 60 [min]
D04_WAVE
Output for specific locations
table
1D spectra
2D spectra
Edit locations

D
Python script BW-curve

```
import numpy as np                #importing numpy package for scientific computing
import matplotlib.pyplot as plt  #importing matplotlib package for plots
import scipy.signal               #importing scipy.signal package for detrending
from pandas import read_csv
import matplotlib.patches as mpatches

%matplotlib inline
#plots are shown inside notebook
```

```
def BWcurve(Q,d0):
    plt.figure(figsize=(12,5))
    #INPUT = Q and d0
    Q = np.array([Q,Q,Q,Q])                        #Discharge (per section)
    B = np.array([200,80,30,50])                   #Width per section
    cf = [0.0039,0.0039,0.0039,0.0039]                     #Friction coefficients per section
    ib = [5e-5,5e-5,0.004,0.004]                   #Bed slopes per section
    x1 = np.linspace(0,2500,32)                    #Defining sections
    x2 = np.linspace(2500,12500,125)
    x3 = np.linspace(12500,19500,88)
    x4 = np.linspace(19500,36000,206)
    warnlevel = 6 #m                               #Warning water level
```

```
    #CREATING THE BED
    bed1 = np.zeros(len(x1))
    bed2 = np.zeros(len(x2))
    bed3 = np.zeros(len(x3))
    bed4 = np.zeros(len(x4))
    for i in range(len(x1)):
        bed1[i]=ib[0]*x1[i]
    for i in range(len(x2)):
        bed2[i]=bed1[31]+ib[1]*(x2[i]-2500)
    for i in range(len(x3)):
        bed3[i]=bed2[124]+ib[2]*(x3[i]-12500)
    for i in range(len(x4)):
        bed4[i]=bed3[87]+ib[3]*(x4[i]-19500)

    #CREATING THE WATER LEVELS USING THE EMPRICIAL FIT ACCORDING TO BRESSE
    h1 = np.zeros(len(x1))
    h2 = np.zeros(len(x2))
    h3 = np.zeros(len(x3))
    h4 = np.zeros(len(x4))
    for i in range(len(x1)):
        d0 = d0
        q = Q[0]/B[0]
        de = (q**2*cf[0]/9.81/ib[0])**(1/3)
        L = 0.24*de/ib[0]*(d0/de)**(4/3)
        h1[i] = de+(d0-de)*2**((-x1[i]/L))
        plt.plot(x1[i],de+bed1[i],'ro',markersize=0.2)
```

```
    for i in range(len(x2)):
        d0 = h1[31]
        q = Q[1]/B[1]
        de = (q**2*cf[1]/9.81/ib[1])**(1/3)
        L = 0.24*de/ib[1]*(d0/de)**(4/3)
        h2[i] = de+(d0-de)*2**((-(x2[i]-2500)/L))
        plt.plot(x2[i],de+bed2[i],'ro',markersize=0.2)
    for i in range(len(x3)):
        d0 = h2[124]
        q = Q[2]/B[2]
        de = (q**2*cf[2]/9.81/ib[2])**(1/3)
        L = 0.24*de/ib[2]*(d0/de)**(4/3)
        h3[i] = de+(d0-de)*2**((-(x3[i]-12500)/L))
        plt.plot(x3[i],de+bed3[i],'ro',markersize=0.2)
    for i in range(len(x4)):
        d0 = h3[87]
        q = Q[3]/B[3]
        de = (q**2*cf[3]/9.81/ib[3])**(1/3)
        L = 0.24*de/ib[3]*(d0/de)**(4/3)
        h4[i] = de+(d0-de)*2**((-(x4[i]-19500)/L))
        plt.plot(x4[i],de+bed4[i],'ro',markersize=0.2)

    #PLOTTING THE FIGURES
    plt.plot(x1,bed1,color='orange',label='Bed level')
    plt.plot(x2,bed2,color='orange')
    plt.plot(x3,bed3,color='orange')
    plt.plot(x4,bed4,color='orange')
    plt.plot(x1,h1+bed1,'b',label = 'water level')
    plt.plot(x2,h2+bed2,'b')
    plt.plot(x3,h3+bed3,'b')
    plt.plot(x4,h4+bed4,'b')
    plt.title('Backwater curve Umgeni river')
    plt.xlabel('Distance from river mouth [m]')
    plt.ylabel('Elevation [m]')
```

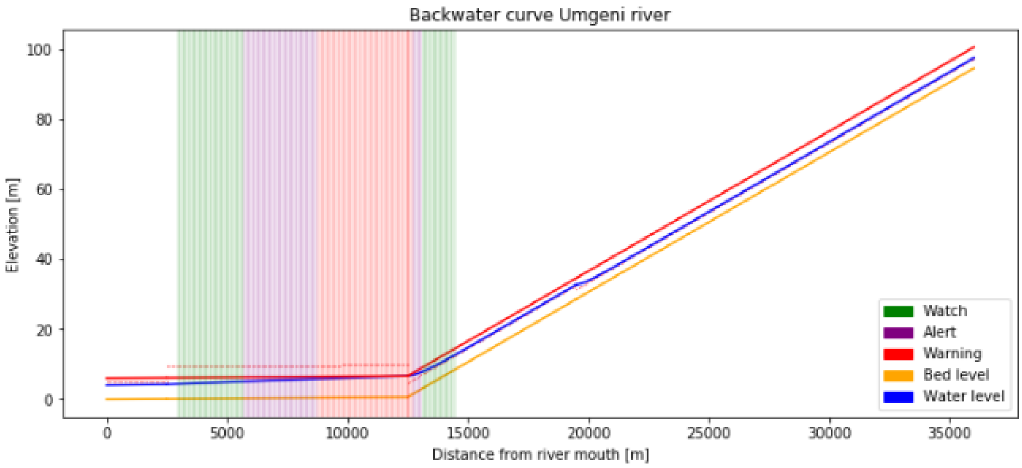
```
    #CREATING THE WARNING LEVELS
    warning1 = bed1+warnlevel
    warning2 = bed2+warnlevel
    warning3 = bed3+warnlevel
    warning4 = bed4+warnlevel
    plt.plot(x1,warning1,'r',label='warning level = 6m')
    plt.plot(x2,warning2,'r')
    plt.plot(x3,warning3,'r')
    plt.plot(x4,warning4,'r')
    for i in range(len(x1)):
        if 0.7*warnlevel<h1[i]<0.8*warnlevel:
            plt.axvline(x1[i],linewidth=0.25, color='g')
        elif 0.8*warnlevel<h1[i]<0.9*warnlevel:
            plt.axvline(x1[i],linewidth=0.25,color='purple')
        elif h1[i]>0.9*warnlevel:
            plt.axvline(x1[i],linewidth=0.25,color='red')
    for i in range(len(x2)):
        if 0.7*warnlevel<h2[i]<0.8*warnlevel:
            plt.axvline(x2[i],linewidth=0.25, color='g')
        elif 0.8*warnlevel<h2[i]<0.9*warnlevel:
            plt.axvline(x2[i],linewidth=0.25,color='purple')
        elif h2[i]>0.9*warnlevel:
            plt.axvline(x2[i],linewidth=0.25,color='red')
    for i in range(len(x3)):
        if 0.7*warnlevel<h3[i]<0.8*warnlevel:
            plt.axvline(x3[i],linewidth=0.25, color='g')
        elif 0.8*warnlevel<h3[i]<0.9*warnlevel:
            plt.axvline(x3[i],linewidth=0.25,color='purple')
        elif h3[i]>0.9*warnlevel:
            plt.axvline(x3[i],linewidth=0.25,color='red')
```

```
    for i in range(len(x4)):
        if 0.7*warnlevel<h4[i]<0.8*warnlevel:
            plt.axvline(x4[i],linewidth=0.25, color='g')
        elif 0.8*warnlevel<h4[i]<0.9*warnlevel:
            plt.axvline(x4[i],linewidth=0.25,color='purple')
        elif h4[i]>0.9*warnlevel:
            plt.axvline(x4[i],linewidth=0.25,color='red')
    green_patch = mpatches.Patch(color='g', label='Watch')
    purple_patch = mpatches.Patch(color='purple', label='Alert')
    red_patch = mpatches.Patch(color='red', label='Warning')
    orange_line = mpatches.Patch(color='orange', label = 'Bed level')
    blue_line = mpatches.Patch(color='b', label = 'Water level')
    plt.legend(handles=[green_patch, purple_patch, red_patch, orange_line, blue_line])

    plt.show()

    return
```

```
In [35]: BWcurve(Q=800,d0=4)
```



E
Elevation maps

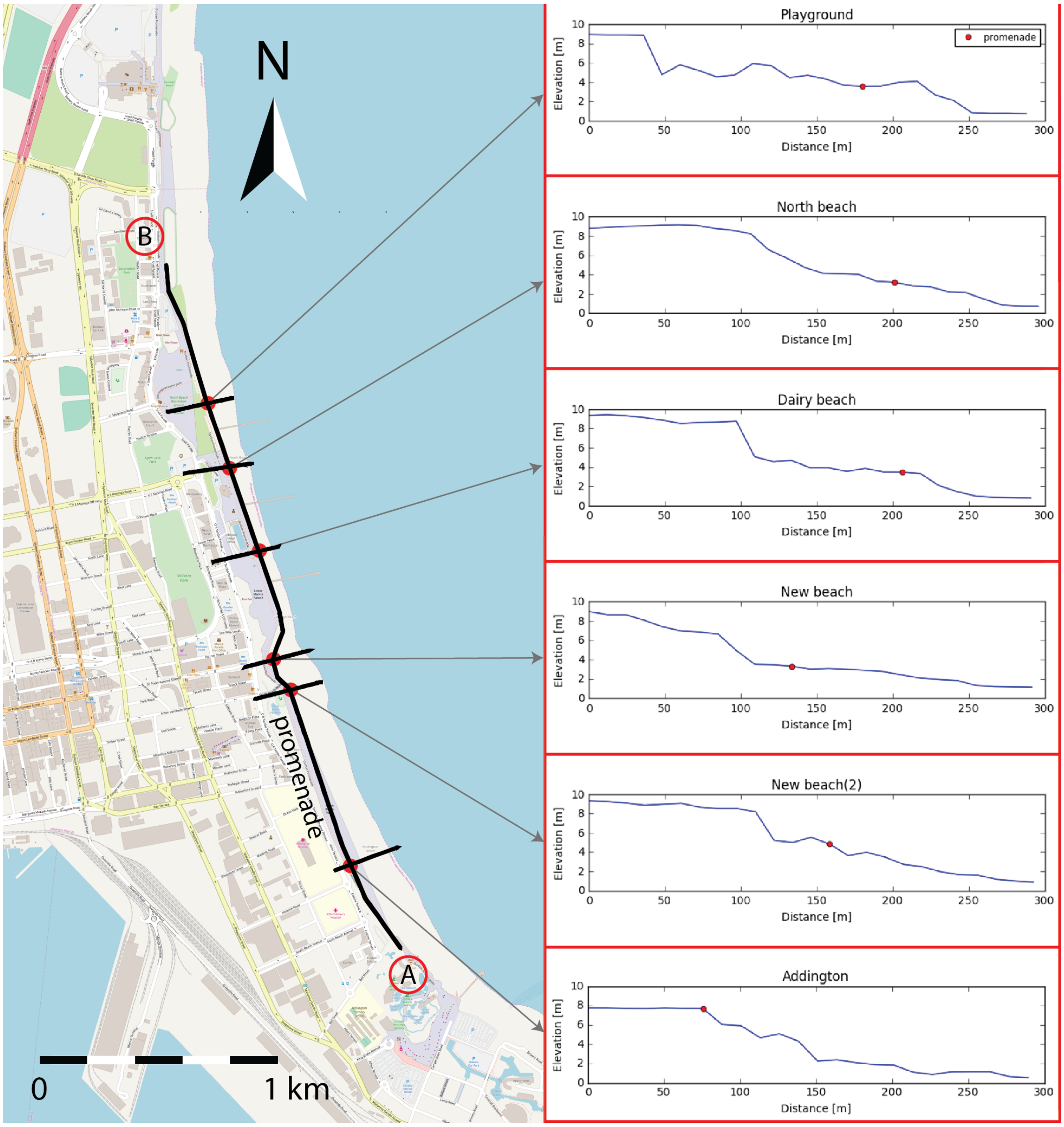


Figure E.1: Elevation profiles of the beaches along the promenade with indication of the promenade in black and red-dotted.

Figure E.2: Elevation of the Durban beachfront relative to the 2016 MSL. The elevation change around New Beach can clearly be seen in this figure.

F
Design options

In this appendix the elaborated design options are shown. Each design options is described in more detail.

Heightening of the promenade combined with a revetment

An option to protect the promenade and its hinterland is to heighten the promenade. This can be combined with a revetment, through which the promenade needs to be less elevated. The revetment mitigates the wave energy, through which the waves are less powerful when they hit the promenade. In figure F.1a and F.1b two different versions of revetments are shown.

(a) Revetment of wood [52]

(b) Revetment of stones [33]

Figure F.1: Various revetments

Heightening of the promenade with an integrated wave energy dissolving system

To dissipate wave energy, gabions can be used along the shore. Gabions are cages filled with rocks, where water can flow through. An example of a gabion is presented in figure F.2. Gabions have various different applications, on the beach, in the water or integrated in other surroundings.

Figure F.2: En example of an application of gabions along the shore. [24]

Heighten the promenade in combination with a sea wall
Heightening the promenade can also be combined with a sea wall. The benefit of this in comparison with a revetment is that there is more beach available. A good example of this is the promenade of Scheveningen, The Netherlands, which is described as reference project in section 2.5.

Sea wall
Another option is to just use a sea wall as protection and leave the promenade on the same level as it lays now. This means that between the beach and the promenade a sea wall is placed. At this moment, there already is a separation between the beach and the promenade. This one may be heightened or renewed. If it is renewed this can be, for example, a concrete or even glass wall as presented in F.3. The solution is viable if the sea wall is not too high, since that may conflict with the surroundings as it blocks the view. Besides, if built higher it is a potential danger to health risk is children climbing it fall off.

(a) Curved concrete seawall beach view [7]

(b) Curved concrete seawall promenade view [7]

(c) Glass seawall [23]

Figure F.3: Various sea walls

Sea wall combined with a revetment
A combination of a sea wall and a revetment could be applied, here many variants are possible given the previous impressions. Combining the revetment with the wall could lower the sea wall. Calculations should prove this. Similar to an individual sea wall the height may not exceed the above mentioned height for the same reasoning.

Sea wall combined with an integrated wave energy dissolving system
Another combination is the seawall combined with a wave energy dissolving system, as for example a gabion. The gabion and sea wall are integrated in the surroundings forming a

solution to the problems. Combining the gabion with the wall could lower the sea wall. Calculations should prove this. Similar to an individual sea wall the height may not exceed the above mentioned height for the same reasoning.

Sea wall combined with an emergency barrier

Combining a sea wall with an emergency barrier allows the sea wall to be designed lower than in stand-alone conditions. Currently, as stated in paragraph 2.1.4, Geotextile Sand-filled Containers (GSCs) are used to protect the beach and promenade from erosion and floods in emergency conditions. A different emergency barrier is for example a barrier which consists of sections placed between columns as shown in figure F.4. Preferably the poles can be taken out when these are not necessary. Important is that there should be a party which is responsible for the possible storage and maintenance of the emergency barrier, if this is neglected or not possible the option should be left out. Besides the warning system should be operational so that the barrier can be placed in time to protect the promenade. Similar to an individual sea wall the height of the permanent construction may not be too high for the same reasoning, as for the emergency barrier it is accepted to have occasionally a higher height.

Figure F.4: Emergency flood defence [4]

A more advanced system is shown in figure F.5. [22]. This option is a self closing barrier and works as follows: the barrier resides below ground in a vertical position. When the flood water rises to a pre-determined height, the barrier will float and arises. [22] Combined with a sea wall, plenty of options arise but should be looked after since the system is advanced and usually designed for small openings, not entire promenades.

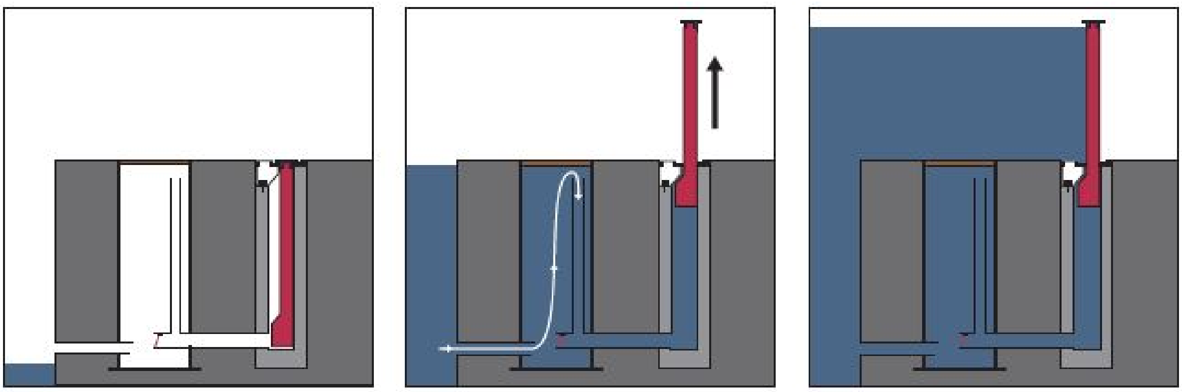


Figure F.5: Illustration of the self-closing barrier

Dune
Dunes are applied along the promenade, both north and south from the assumed vulnerable area. Here the dunes are integrated in the surroundings and plenty of openings have been made to allow people to access the beaches, as can be seen in figure F.6. Given that most of these area are at a higher elevation, the dunes are more acceptable, also taken into account the blocking of the view. Depending on the design height, dunes might be an option.

Figure F.6: Dunes near New Beach form a protection for the promenade.

G

Multi Criteria Analysis

In this appendix the analysis of the MCA is shown, firstly the criteria and their weight factors are discussed. Secondly for each criteria there is explained which score each option obtained and at last the scores are shown in a table. At last, a detailed overview is given for the stakeholders management in terms of power, interest and attitude with added description of the characteristics.

G.1. Criteria

The relative grading of the criteria in table 4.3 is merely depending on the opinions of the various stakeholders involved. Thus there is referred to the stakeholders (section 2.4) to support the choice of weights. In this section each of the criteria is fully elaborated and motivated with the opinions of the various stakeholders involved.

There is only one disclaimer to be made for the criteria regarding the costs, which is not taken in account, although it is of considerable importance. The criteria is not considered, because at this stage of the design no real values can be attached to the costs of the various design options. If a particular design option is for no reason possible in South Africa, due to local conditions regarding costs, this one is by forehand already excluded for the MCA. Therefore for all remaining option can be said that the are realisable for South Africa standards, working conditions and materials. Due to this statement the MCA is slightly simplified and costs can be assumed reasonable for all design options.

Blockage of the view
First criteria discussed is the blockage of the view of the beach and the sea. The important stakeholders regarding this criteria are the eThekwini municipality, tourism and daily visitors. The promenade, beach and sea is quite a big attraction in Durban and attracts lots of tourists/daily visitors, which is beneficial for the local economy. Thus beneficial for the eThekwini municipality and the city of Durban. There is one note, the eThekwini municipality wants to protect their inhabitants and tourists, which means that they need to mitigate their demands if there are no other options. For both tourism and daily visitors the view is important, as it is one of the main motivations to travel to Durban. Thus all three parties are keen to keep the view. The municipality is a party with a lot of power, the daily visitors have a low power but are in great numbers. Considering all mentioned before, this criteria is highly important to the grading of the design options.

Pools/restaurants are not affected
Pools and restaurants are both very important to local businesses, daily visitors and tourism, as they are main attractions along the promenade and attract many people. Affecting these attractions by a new design for the promenade, local businesses are damaged enormously. Beside this, it is not favourable for the eThekwini municipality to redesign the complete recre-

ational area in terms of money and time. A new design for the pools and restaurants upsets many people and costs lots of time and money to be realised. Given all this, this criteria can be considered as important as the blockage of the view.

Fits in the surroundings
As the Durban Golden Mile with its promenade is the main attraction of the city, a new solution for the flood defence should fit in the surroundings to keep attracting the daily visitors, tourism to the area. Besides, for large and local businesses it is very important that their shops/hotels are not affected by the solution. In other words, the solution should fit in the surroundings of their businesses. These businesses profit from the daily visitors and tourism, which is also good to the eThekwini municipality. Many stakeholders are affected if this criteria is not satisfied, which makes it a valuable and for this reason, this criteria is of equal importance to the two criteria mentioned above.

Protection of the beach
Currently the eThekwini municipality is using nourishment to save the beach from washing away and as discussed before, erosion is left out in this report. For a lot of stakeholders namely, residents, businesses and all services, the protection of the beach is of no importance, as their properties and belongings aren't located on the beach. Besides, this report focuses mainly on the protection of the promenade. The protection of the beach isn't the aim, nevertheless the beach is an important criteria for tourists and daily visitors. To satisfy these stakeholders, it is beneficial if the flood defence also protects the beach. Therefore this criteria is taken into account in the MCA, although it is of less importance than the above mentioned criteria.

Nuisance
After a design is chosen, it also needs to be realised. This can cause nuisance in terms of construction time, construction site area and noise. The large and local businesses, daily visitors and tourism are most affected by the nuisance. As the eThekwini municipality is most responsible for the nuisance and prefer to keep the nuisance as low as possible, in order to keep the other stakeholders satisfied and on the same time build a defence to protect the area. Considering all this, this criteria is as important as protection of the beach.

Booster station is not affected
At the end of the beach for the vulnerable area, a booster station, used for the nourishing, is located. For the eThekwini municipality, the most important stakeholder, this station is of great importance to their current project to protect the beaches from eroding. Therefore preferably this station should not be affected by the design options. As no other stakeholders are involved in this criteria and the booster station only covers a small spot in the vulnerable area, it is weighted less important than the previously mentioned criteria.

Extendability along the beach
The preliminary design obtained by the end of this chapter is designed for the specific vulnerable area determined in paragraph 4.1.1. The main objective of this report is to find a structural solution to protect the area which is most prone to flooding. Nevertheless the extendability of this design along the rest of the beach can be valuable for the ethekwini municipality, as it saves time and money. On the other hand, the eThekwini municipality is the only stakeholder is this criteria and note that extendability along the beach isn't the main objective of this report. Thus the criteria is less important than the ones mentioned above.

G.2. Evaluation

In this section the scores are explained and a overview of all the scores are shown in tables, firstly the explanations.

G.2.1. Substantiation of scores

In this section the scores are explained, each criteria is handled separately.

Blockage of the view
The dune affects the view on the sea and beach. Just as the seawall (and a combination with the seawall) the height, thus the amount of blockage of the view, depends on the water height. When the promenade is heightened there is no blockage of the view, thus the highest score. The seawalls in combination with a revetment, integration or emergency barrier don't have to be as high as a sea wall alone, and thus score better than the seawall.

Pools/restaurants are not affected
The seawall doesn't affect the pools or restaurants, concerning their location at all, thus all the combinations with the seawall scores the maximum points. If the promenade is heightened than the restaurants and pools are affected, so it has a bad performance on this criteria. The dune performs good, but not perfect because the access to the beach is harder. Through which people can be hindered to go to the promenade.

Fit in the surroundings
As mentioned before in the section 4.1.1, the boundaries of the vulnerable area are dunes, thus the dunes do fit in the surrounding and scores excellent. A seawall alone doesn't fit in the surrounding, because it makes a barrier between the beach and the promenade. The seawall in combination with the revetment performs better, because in this case the seawall is lower than the seawall alone and the revetment can be made into an extension of the promenade. Also for the seawall in combination with the emergency barrier, the seawall doesn't have to be as high, thus the separation between the promenade and the beach is less. Heightening the promenade doesn't fit properly in the surrounding, due to height difference.

Protection of the beach
In general, all options perform bad on the fourth criteria of protecting the beach. Only the combinations with revetment take some energy out of the waves and protect the beach. All of the others don't protect the beach, and thus score bad.

Nuisance
The heightening of the promenade in general performs bad on the fifth criteria, nuisance during the build, due to the amount of work that needs to be done. The dune has a minimal nuisance for the daily users of the beach/promenade during the build. The sea wall has more nuisance, because the promenade has to be partly rebuild. The seawall in combination with the revetment causes more nuisance due to the placement of the revetment on the beach.

Booster station is not affected
The booster station is located on the beach and should not be affected. Heightening of the promenade does only affect the booster station if there is a slope in front of it. A revetment has a bigger slope than an integrated wave dispersion barrier. The seawall alone and in combination with the emergency barrier have no effect on the booster station thus score excellent. The same as with the heightening of the promenade, if there is a slope in front of it, the booster station is affected, thus a lower score.

Extendability along the beach
On the last criteria, can be extended along the beach, the dune scores well, because at the boundaries of the vulnerable area the dunes of the other beaches start (see paragraph 4.1.1). These eventually have to be levelled to each other. The heightening of the promenade scores

positive in general, but the revetment and the integration need more space on the beach. The beach further op north of the vulnerable area can be quite small, thus more difficult to extend. This also holds for the seawall in combination with a revetment and integration. The seawall alone or in combination with the emergency barrier is easy to extend along the beach.

G.2.2. Scores

In this paragraph all the options and their scores are shown in tables.

Table G.1: Scores of the dune

Criteria	WF	Score	WF x score
Blockage of the view	22.22	2	44.44
Pools/restaurants are not affected	22.22	4	88.89
Fits in the surroundings	22.22	5	111.11
Protection of the beach	11.11	1	11.11
Nuisance	11.11	4	44.44
Booster station is not affected	7.41	2	14.81
Extendibility along the beach	3.70	4	14.81
Total score			314.81

Table G.2: Scores of the heighten of the promenade with an intergrated wave energy dissolving system

Criteria	WF	Score	WF x score
Blockage of the view	22.22	5	111.11
Pools/restaurants are not affected	22.22	1	22.22
Fits in the surroundings	22.22	2	44.44
Protection of the beach	11.11	1	11.11
Nuisance	11.11	1	11.11
Booster station is not affected	7.41	4	29.63
Extendibility along the beach	3.70	4	14.81
Total score			229.63

Table G.3: Scores of the heighten of the promenade combined with a seawall

Criteria	WF	Score	WF x score
Blockage of the view	22.22	5	111.11
Pools/restaurants are not affected	22.22	1	22.22
Fits in the surroundings	22.22	2	44.44
Protection of the beach	11.11	1	11.11
Nuisance	11.11	1	11.11
Booster station is not affected	7.41	5	37.04
Extendibility along the beach	3.70	5	18.52
Total score			237.04

Table G.4: Scores of the heighten of the promenade combined with a revetment

Criteria	WF	Score	WF x score
Blockage of the view	22.22	5	111.11
Pools/restaurants are not affected	22.22	1	22.22
Fits in the surroundings	22.22	3	66.67
Protection of the beach	11.11	2	22.22
Nuisance	11.11	1	11.11
Booster station is not affected	7.41	3	22.22
Extendibility along the beach	3.70	3	11.11
Total score			255.56

Table G.5: Scores of the seawall

Criteria	WF	Score	WF x score
Blockage of the view	22.22	2	44.44
Pools/restaurants are not affected	22.22	5	111.11
Fits in the surroundings	22.22	1	22.22
Protection of the beach	11.11	1	11.11
Nuisance	11.11	3	33.33
Booster station is not affected	7.41	5	37.04
Extendibility along the beach	3.70	5	18.52
Total score			259.26

Table G.6: Scores of the seawall combined with a revetment

Criteria	WF	Score	WF x score
Blockage of the view	22.22	3	66.67
Pools/restaurants are not affected	22.22	5	111.11
Fits in the surroundings	22.22	2	44.44
Protection of the beach	11.11	2	22.22
Nuisance	11.11	2	22.22
Booster station is not affected	7.41	3	22.22
Extendibility along the beach	3.70	3	11.11
Total score			288.89

Table G.7: Scores of the seawall combined with an integrated wave energy dissolving system

Criteria	WF	Score	WF x score
Blockage of the view	22.22	3	66.67
Pools/restaurants are not affected	22.22	5	111.11
Fits in the surroundings	22.22	1	22.22
Protection of the beach	11.11	1	11.11
Nuisance	11.11	3	33.33
Booster station is not affected	7.41	4	29.63
Extendibility along the beach	3.70	4	14.81
Total score			274.07

Table G.8: Scores of the seawall combined with an emergency barrier

Criteria	WF	Score	WF x score
Blockage of the view	22.22	4	88.89
Pools/restaurants are not affected	22.22	5	111.11
Fits in the surroundings	22.22	3	66.67
Protection of the beach	11.11	1	11.11
Nuisance	11.11	3	33.33
Booster station is not affected	7.41	5	37.04
Extendibility along the beach	3.70	5	18.52
Total score			348.15

H Astronomical tide

In this appendix the determination of the water-retaining height due to the astronomical tide is elaborated. Firstly, the filtering process of the data is explained, from the raw data to the daily maximum water level. Secondly, the numerous probability distribution functions are compared and after curve fitting the water-retaining height due to the astronomical tide is determined, based on the best fit.

H.1. Filtering data set

For the determination of the water-retaining height, due to the astronomical tide, an hourly tidal data set is retrieved from the University of Hawaii Sea Level Center (UHSLC) for the measuring station in Durban. The retrieved data is gathered over a period of 36 years, from 1970 till 2016, and consists of raw data, which means it contains many measurement errors. In order to be able to use this data set qualitatively, it has to be filtered first. A csv file with the hourly measured water level is retrieved from the UHSLC database and the measurement errors are deleted from the data set, in this cause. Due to measurement errors the data set is no longer continuous anymore and therefore the determination of the correct maximum water level distribution is worsened, as a single period of an astronomical tide takes 12 hours and 25 minutes. Therefore the derivation is simplified by taken the maximum water level day, which means that the complete data set of a day which previously contained a measurement error has to be deleted. After this the filtered data set is loaded into a Python Jupyter script from figure H.1.

```
#### Load data ####
data = np.loadtxt('tide_complete.txt')/1000    #Meters
Datapoints = len(data)

#### Split data in days ####
Days = Datapoints/24
z = np.split(data[:,4],Days)

#### Find water levels for each days ####
x = np.zeros(Days)
for i in range(Days):
    #x[i] = (max(z[i])+min(z[i]))/2.       #gemiddelde waterstand per dag
    x[i] = max(z[i])                       #maximum waterstand per dag

#### For plotting the data in cdf function ####
sorted_data = np.sort(x)
yvals=np.arange(Days)/float(Days-1)

#### Finding some interesting values ####
minx = min(x)
maxx = max(x)
differencex = maxx - minx
meanx = np.mean(x)
stdx = np.std(x)
```

Figure H.1: Filtering Python script of the raw data

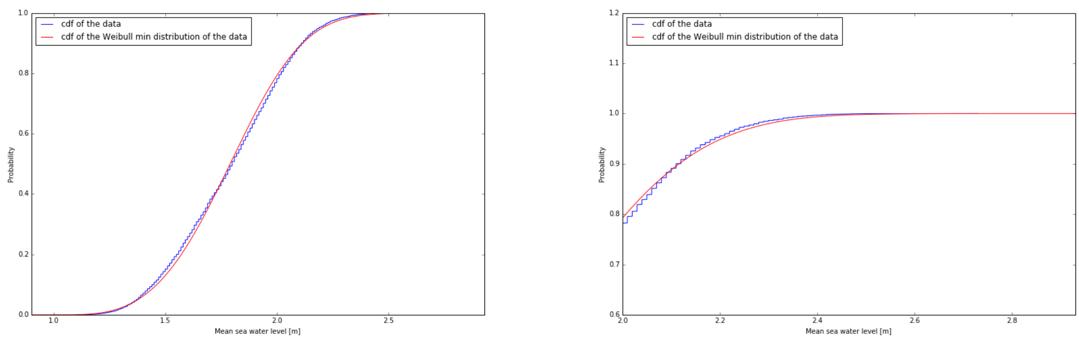
In the Python script of figure H.1 first the data is sorted per day, in order to obtain the daily maximum water level. In total the maximum water level of 10977 days is sorted cumulatively to obtain the exceedance probability for a return period of 200 years.

H.2. Probability distribution

In order to obtain the exceedance probability of the daily maximum water level, various probability distribution functions are computed and plotted versus the data cumulatively. The probability distribution functions used are listed below:

- Weibull min distribution: $f(x,c) = cx^{c-1}e^{-x^c}$.
- Weibull max distribution: $f(x,c) = c(-x)^{c-1}e^{-(-x)^c}$.
- Log normal distribution: $f(x,c) = \frac{1}{sc\sqrt{2\pi}}exp(-\frac{log^2(x)}{2c^2})$.
- Generalised Pareto distribution: $f(x,c) = (1+cx)^{-1-1/c}$.
- Gumbel r distribution: $f(x) = e^{-x-e^{-x}}$.
- Gumbel l distribution:$f(x) = e^{x-e^x}$.
- Normal distribution: $f(x) = \frac{1}{\sigma\sqrt{2\pi}}e^{-(x-\mu)^2/(2\sigma^2)}$.

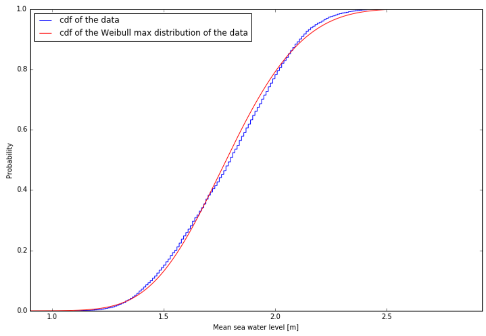
For each of the probability distribution function mentioned above the cumulative plot versus the cumulative distribution of the daily maximum water level, for both complete and upper limit, are shown in figures H.2 - H.8.



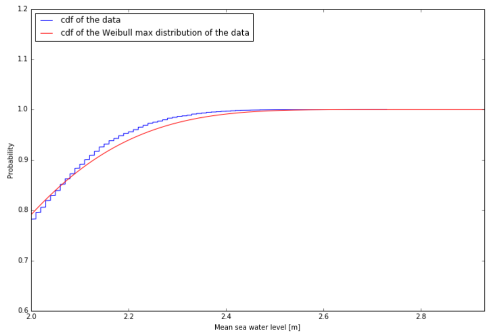
(a) Overall cumulative distribution

(b) Zoomed upper limit cumulative distribution

Figure H.2: Cumulative distribution of the data set vs. Weibull min distribution

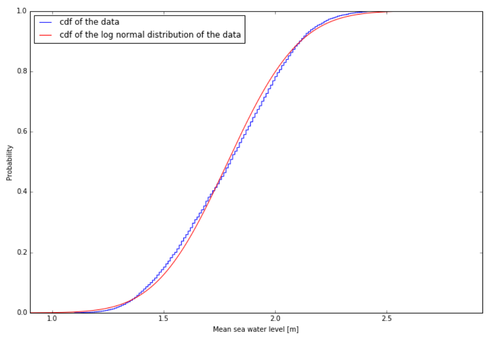


(a) Overall cumulative distribution

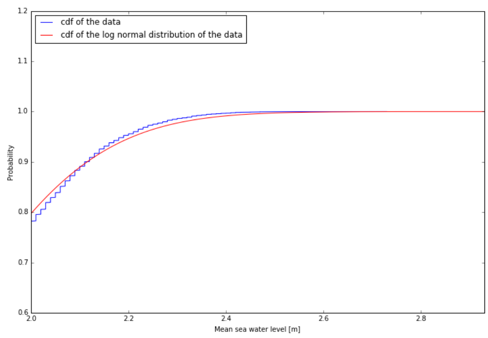


(b) Zoomed upper limit cumulative distribution

Figure H.3: Cumulative distribution of the data set vs. Weibull max distribution

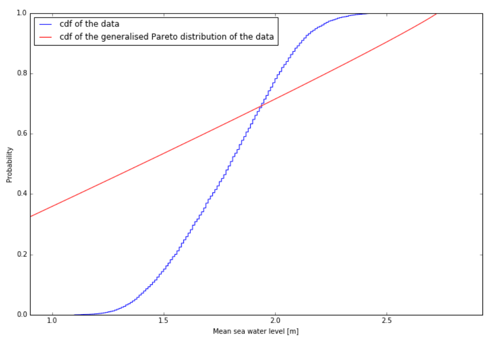


(a) Overall cumulative distribution

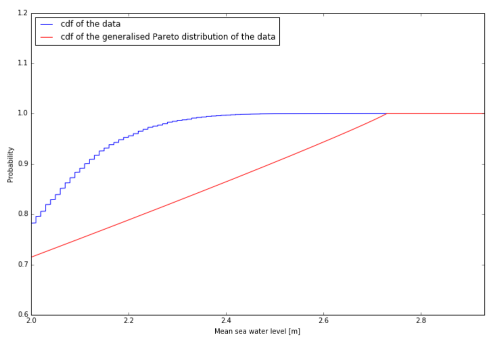


(b) Zoomed upper limit cumulative distribution

Figure H.4: Cumulative distribution of the data set vs. log normal distribution

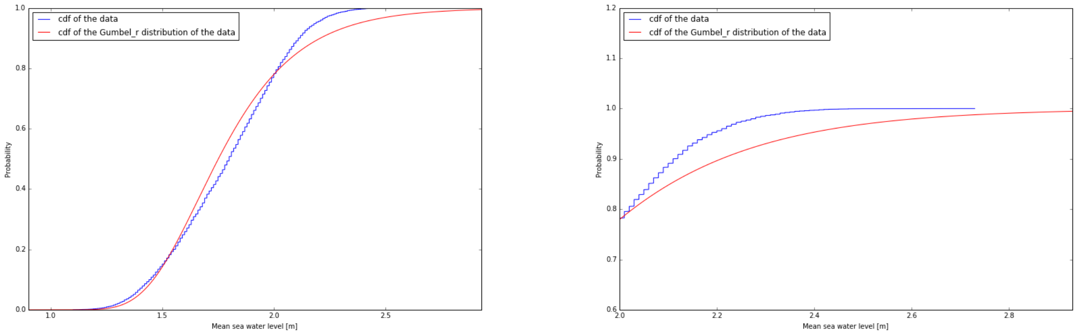


(a) Overall cumulative distribution



(b) Zoomed upper limit cumulative distribution

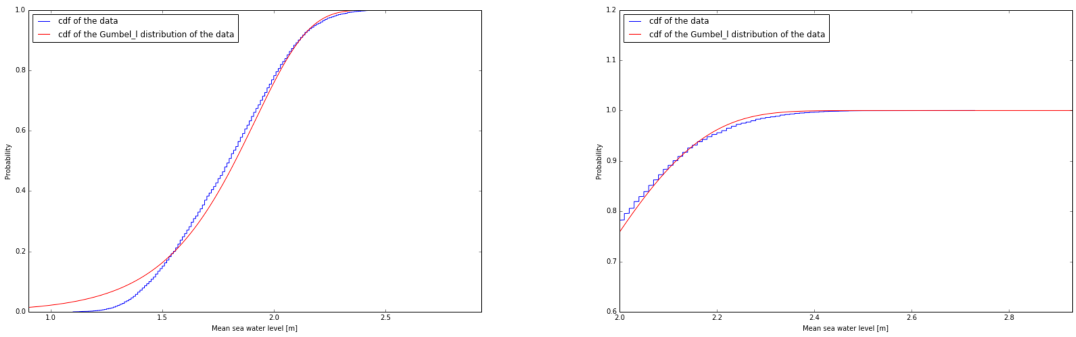
Figure H.5: Cumulative distribution of the data set vs. generalised Pareto distribution



(a) Overall cumulative distribution

(b) Zoomed upper limit cumulative distribution

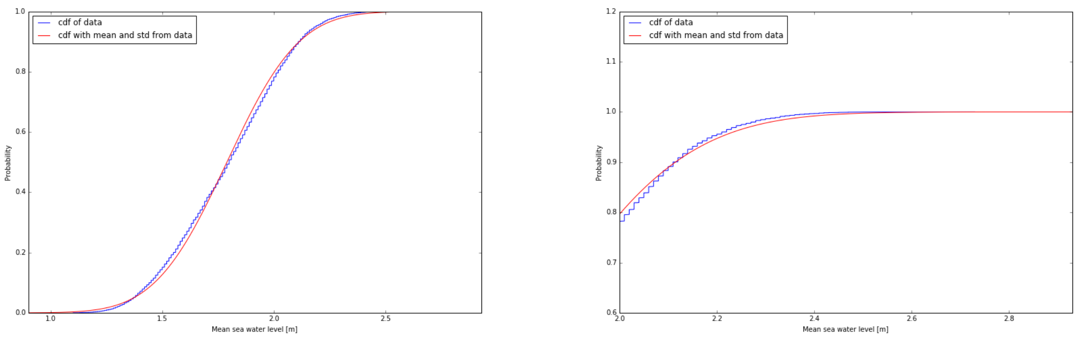
Figure H.6: Cumulative distribution of the data set vs. Gumbel r distribution



(a) Overall cumulative distribution

(b) Zoomed upper limit cumulative distribution

Figure H.7: Cumulative distribution of the data set vs. Gumbel l distribution



(a) Overall cumulative distribution

(b) Zoomed upper limit cumulative distribution

Figure H.8: Cumulative distribution of the data set vs. normal distribution

From figures H.2 - H.8 can be concluded that the Weibull min distribution fits the cumulative distribution of the data the best, especially for the upper limit. As the exceedance probability focuses on the upper limit of the cumulative distribution, to be a bit conservative the Weibull min distribution curve is located just below the actual data, as can been seen in figure H.2b, which makes this distribution extremely applicable.

```
Once in the 200 years, the mean sea level will exceed 2.76 meter (weibull min).
Once in the 200 years, the mean sea level will exceed 2.666 meter (weibull max).
Once in the 200 years, the mean sea level will exceed 2.874 meter (lognormal).
Once in the 200 years, the mean sea level will exceed 2.73 meter (Generalised pareto).
Once in the 200 years, the mean sea level will exceed 4.382 meter (gumbel_r).
Once in the 200 years, the mean sea level will exceed 2.497 meter (gumbel_l).
Once in the 200 years, the mean sea level will exceed 2.854 meter (normal).
```

Figure H.9: Exceedance height of a 200 year return period

From figure H.9 follows that the exceedance height for a return period of 200 years is 2.76 m, relative to LAT, for the Weibull min distribution. Also from figure H.9 can be concluded that the Weibull min distribution isn't the most conservative, but based on the plots from figures H.2 - H.8 is concluded that the (log) normal distribution is considered to be too conservative, which leads to unnecessary high heights. In conclusion, for calculations the height relative to MSL is used and therefore the heights are adapted to this reference level. The exceedance height for a return period of 200 years is then 1.856 m relative to MSL.

Sea Level Rise (SLR)

Much research has already been done to determine the future sea level rise (SLR), which in the end depends on the Representative Concentration Path (RCP). Different RCP scenarios can be distinguished, indicating the amount of pollution in future terms and therefore highly depending on the policy on climate change, RCP2.6 indicating strong policy and RCP8.5 barely any policy. With the global warming, the SLR is mainly caused by thermal expansion of the ocean and melting of land ice (glaciers and ice sheets). [13]

I.1. Global

Depending on the RCP, different global mean sea levels (GMSL) can be expected over time. This is indicated in table I.1 for the years 2050, 2070 and 2100. These three years are chosen since many research refers to 2050 and 2100, and 2070 indicates a 50-year design life, which is usual in South Africa to choose as design life for a structural element, also see 4.1.2. Note that RCP4.5 and RCP6.0 differ very little but differ in rate of rise. The rate of rise has influence on, amongst others, ice-caps and glaciers, resulting in a different outcome. RCP4.5 has a quicker change in SLR but in the end RCP6.0 has a higher result. Therefore, both RCPs will end up in a similar result in this report.

Table I.1: GMSL rise for the years 2050, 2070 and 2100 depending on four different RCP with a likeable range ($5-95\%$) between brackets, in meter relative to 1986-2005[34]

Scenario	RCP2.6	RCP4.5	RCP6.0	RCP8.5
2050	0.22 [0.16-0.28]	0.23 [0.17-0.29]	0.22 [0.16-0.28]	0.25 [0.19-0.32]
2070	0.31 [0.21-0.41]	0.35 [0.25-0.45]	0.33 [0.24-0.43]	0.42 [0.31-0.54]
2100	0.44 [0.28-0.61]	0.53 [0.36-0.71]	0.55 [0.38-0.73]	0.74 [0.53-0.98]

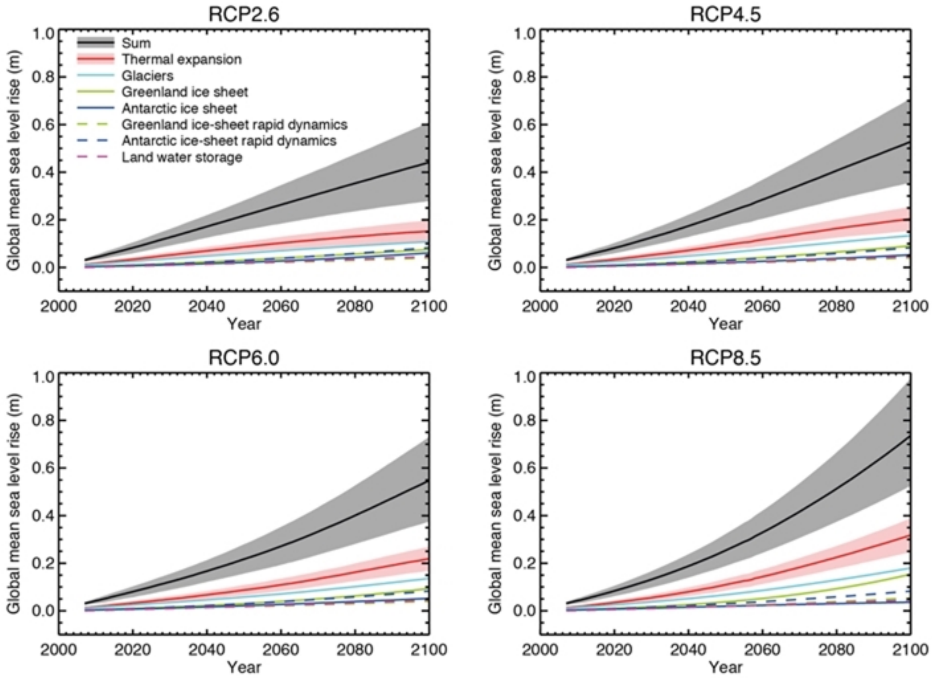


Figure I.1: Different RCP scenarios and its influence on the GMSL rise relative to 1986-2005. The bands shown are the likely ranges of the different contributions.[13]

I.2. Local

Since GMSL rise is different across the world, local influence can become quite significant, as can be seen in figure I.2. Local differences are caused by dynamical processes in the ocean, movements of the sea floor and changes in gravity of the land ice, all having a different 'fingerprint', an area of which a single effect has influence across the world.

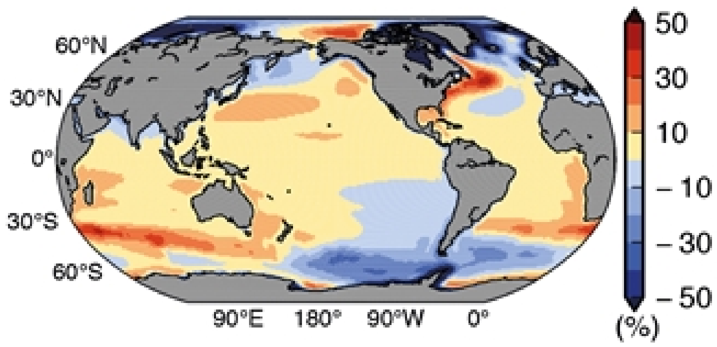


Figure I.2: Differences caused by fingerprints make the GMSL not homogeneously across the globe. In this figure is the deviation (%) of GMSL rise between 1986-2005 and 2081-2100 shown across the globe. [13]

For the case of the Durban beachfront, the data behind figure I.2 has been analysed locally and is presented in figure I.3. The results are computed for RCP4.5, but according to the IPCC the values can be used to first order for all RCPs. For this report, it is assumed that these percentages can be used to cope with local effects. As can be seen, local deviation is generally in the range of 9 – 13%, however the value (3.24%) closest to Durban deviates a lot from other given values in the area. The data of IPCC is considered valid, so the value can't be excluded considering local effects, but it is assumed that the value deviates too much compared to others. Regarding the values most north (12.94%) and south (13.18%), it is not unusual that close to shore lines the sea levels do rise with more than 10%. To assess these differences which occur at relatively short distance, in the order of a couple hundreds of kilometres, it is chosen to use the mean of the three values which are closest to shore as assumed local sea level rise, 9.79%.

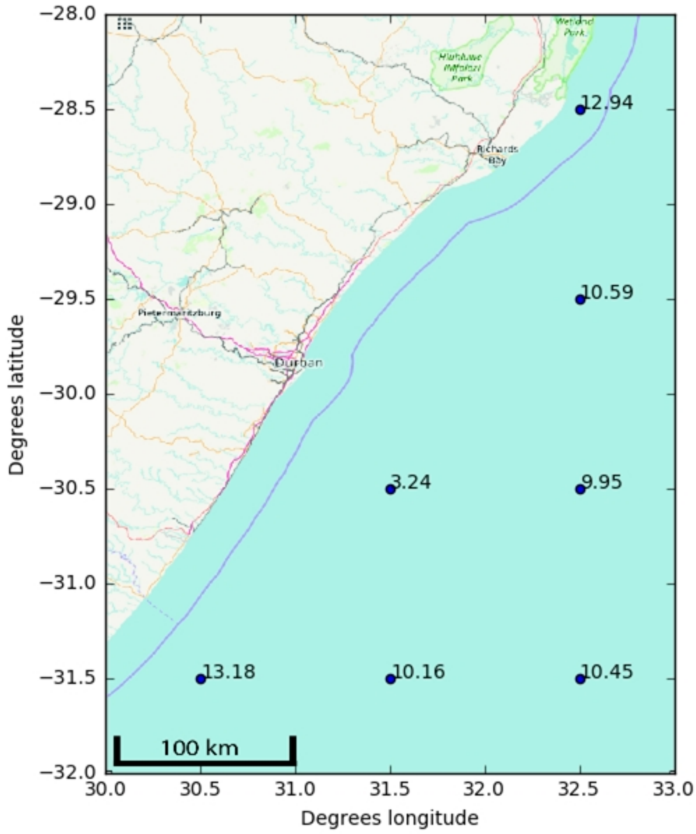


Figure I.3: Local SLR in front of the Durban coast extracted from figure I.2. Other values at specific coordinates do not exist since these are too close to land or on land. [13]

Regarding the different RCP scenarios, the local correction of 9.79% is used to increase the MSL. It is assumed that the likely range also increases with the same factor. This choice is further discussed in the next paragraph

Table I.2: Local SLR in front of the Durban coast. The GMSL rise has been corrected with an increase of 9.79 %, also the range has been increased by this.

Scenario	**RCP2.6**	**RCP4.5**	**RCP6.0**	**RCP8.5**
2050	0.24 [0.18-0.31]	0.25 [0.19-0.32]	0.24 [0.18-0.31]	0.27 [0.21-0.35]
2070	0.34 [0.23-0.45]	0.38 [0.27-0.49]	0.36 [0.26-0.47]	0.46 [0.34-0.59]
2100	0.48 [0.31-0.67]	0.58 [0.40-0.78]	0.60 [0.42-0.80]	0.81 [0.58-1.08]

I.3. Allowances

To design against climate change, the structural element needs to be elevated to cope with the expected raise in MSL. These so-called allowances are discussed and determined via equation (I.1). The allowance is dependent on local distribution of extremes, MSL μ, the standard deviation of the SLR σ and the Gumbel scale parameter λ, which describes the sea level extreme events, and which is assumed to remain unchanged under sea-level rise. A smaller λ means less variability in extremes. [50]

$$a = \mu + \sigma^2/2\lambda \tag{I.1}$$

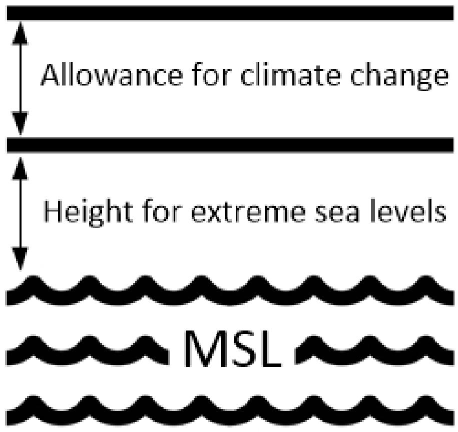


Figure I.4: Allowances relative to the MSL and extreme sea levels

It must be noted that equation (I.1) is only valid for a single normal distribution, where the SLR is not standard deviated but is composed of multiple standard deviations of different effects (land ice, temperature expansion etc.). In theory the equation should be modified including weight effects for the different standard deviations and effects. By doing so, correct standard deviation and weight factor per effect should be obtained for the three observed years and per scenario, which is a time-consuming job and is expected to only result in a more correct result in the order of magnitude of centimetres.

Since the scope of the report is not to fully assess climate change and its consequences, the likely SLR range is assumed to be the standard deviation, where the 5% and 95% value are corrected to the mean so that an averaged standard deviation is used, and the mean is assumed not to change. The order of magnitude of the limited difference can be shown by applying equation (I.1) with both (utmost) standard deviations, $|\mu - 5\%|$ and $|\mu - 95\ \%\ |$. Scenario RCP8.5 results in 2100 in the highest SLR as can be seen in table I.3, and the utmost difference is 9.8 cm, which is for the heaviest scenario relatively small.

Table I.3: The limited effect by choosing a different standard deviation on the allowance for the utmost global RCP8.5 scenario in 2100.

μ **[m]**	λ	5% **or** 95%**value**	σ **(=**$\vert\mu - \%\vert$**)**	**Allowance [m]**
0.81	0.083333	0.58	0.23	1.131
0.81	0.083333	1.07	0.26	1.229

In figure I.5 is shown what the consequences are for the assumptions regarding the (standard) deviation and local corrections. In black is the original global distribution given, which is not normally distributed but accounts for the different effects (land ice, temperature expansion etc.) which all have a different distribution and weight factor [50]. In red is the assumed, standard deviated, original global distribution given and in blue the distribution corrected for local effects. The distributions are shifted, especially near the 95th percentile. However, this shift lowers the 95th percentile, which therefore does not harm the sea level negatively.

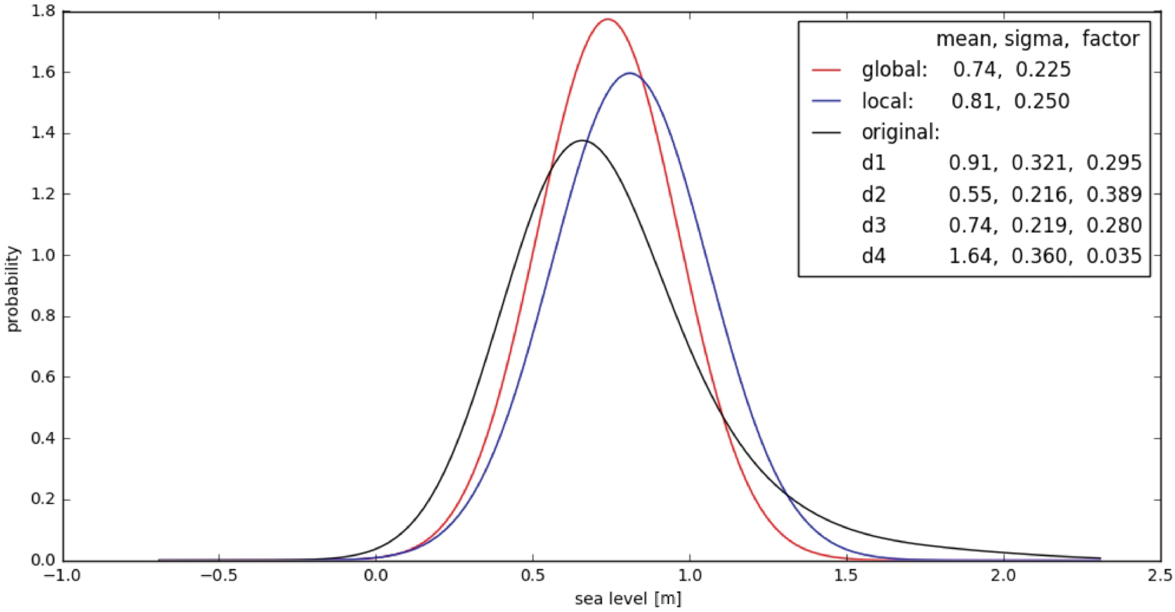


Figure I.5: Shift in distributions to calculate the allowances

λ is as mentioned dependent on the distribution of the extreme sea levels and is assessed by Slangen et al [50] for many tide gauge stations around the world, including the tide gauge station at the Durban harbour. There, from graphs, can be observed that λ has a value between $0.083\ m$ and $0.092\ m$, only a range is available and not an exact value. In theory, it is possible to calculate the exact value, but given the scope and the time-consuming job of critically selecting the correct data, the smaller value is assumed, indicating the highest contribution to the allowances. The consequence for this assumption is minor, the differences are $3.4\ cm$, as can be seen in table I.4.

Table I.4: The limited effect of the λ on the allowance for the utmost global RCP8.5 scenario in 2100.

μ [m]	λ	5% to 95% value	σ =($\|\mu$ – 5%$\|$+ $\|\mu$ – 95%$\|$)/2)	Allowance [m]
0.81	0.083	0.56 to 1.06	0.25	1.179
0.81	0.092	0.56 to 1.06	0.25	1.145

I.4. Result

The result for all the allowance depending on year and RCP scenario including the previously described effects and assumptions is shown in table I.5.

Table I.5: Allowances for the RCP scenarios included the local SLR [m].

GMSL rise scenario	RCP2.6	RCP4.5	RCP6.0	RCP8.5
2050	0.27	0.28	0.27	0.31
2070	0.41	0.46	0.43	0.56
2100	0.68	0.80	0.83	1.18

Given the results in table I.5, many allowances depending on the RCP scenario are possible. To decide which design height to consider, it is important to determine the RCP scenario for a 50-year design life. Kopp et al. (2014) researched the probability of exceeding the GMSL in 2100, presented in table I.6. Note that the RCP6.0 scenario is missing. Based on table I.6 and table I.1 the allowance can be determined. Since the data in table I.6 is for 2100, that year is considered to decide the RCP scenario which is of interest in this research.

Given the 50-year design life, the difference in allowance between the RCP2.6 and RCP8.5 scenario is 0.15 m. It is hard to say which RCP scenario is governing since that is impossible to predict but given the relatively small difference between the RCP4.5 and RCP6.0 scenario, that a RCP8.5 scenario is quite pessimistic (governments already show policy on the climate change topic which makes no policy hard to believe) and that the RCP2.5 scenario is quite optimistic (it is not given that the current policies allow for this scenario), it is chosen that the allowance is 0.46 m for 2070. Governments world-wide, including, South Africa have signed the Paris Agreement in 2015, willing to limit the global warming and therefore willing to make policy on this topic [46].

The probability that the GMSL rise in 2100 rises higher than 0.5 m for the RCP scenarios is quite confident (respectively 49, 73 and 96 %) but given the exponential character of the RCP scenarios in figure I.1 it can be expected that the probability is lower for the year of 2070, which makes the 0.46 m allowance reasonable. Besides, the range given of the SLR for the different RCP in table I.1 is in line with the allowance of 0.46 m, except for RCP8.5.

Table I.6: Probability of exceeding GMSL (median value) scenarios in 2100 based upon Kopp et al. (2014).[36]

Scenario	**RCP2.6 [%]**	**RCP4.5 [%]**	**RCP8.5 [%]**
Low (0.3m)	94	98	100
Intermediate-Low (0.5m)	49	73	96
Intermediate (1.0 m)	2	3	17
Intermediate-High (1.5 m)	0.40	0.50	1.30
High (2.0 m)	0.10	0.10	0.30
Extreme (2.5 m)	0.05	0.05	0.10

J

Overtopping calculations

In the first section of this appendix the wave breaking is determined, as wave set-up and roller height are explained and calculated. Next, a 1D-model is SWAN is described, which similarly derives wave set-up and water level at the beach front relative to ground level. At last, the overtopping height is determined using the retrieved parameters from the 1D-model.

J.1. Wave set-up and rollers

Waves closing in on the shoreline will undergo shoaling. This means the amplitude of a wave changes as a result of changing water depth. The bed profile therefore largely determines the wave height close to the shore. How much the amplitude changes can be expressed with the shoaling coefficient. This increase in wave height only goes on for a certain extent though. As can be observed by anyone, waves break when they get too close to the coastline. This depth-induced breaking depends on the wave steepness and the wave breaking parameter γ. The area where waves break is called the surf zone. The water level here shows an increase compared to the water level offshore. This phenomenon is caused by the waves and is called wave set-up. When waves break, a sudden loss in radiation stresses occur, which pushes the water level up according to the energy balance. The set-up causes an extra water column height that has to be resisted by the wall. How much this is, is calculated in this section. Additionally, a look is taken at wave rollers, which are the remaining part of a broken wave.

Starting point is the output from Delft3D. A simulation is ran with the storm conditions described in paragraph 4.1.3 as input. In an observation point at 310 m from the coast the results are obtained. The significant wave height is 1.7 m, the peak wave period 8.5 s and the wave length 42 m. The water depth here is 3.1 m. Assumed is that this wave travels perpendicular to the shoreline. For the following calculations, the root mean square wave height should be used, and not the significant wave height. The two are related to each other as shown in equation J.1.

$$H_{rms} = 0.5\sqrt{2}H_{sig} \tag{J.1}$$

Finding the ratio between the water depth and wave length results in 0.071. This is just above the limit of shallow water of 0.05. To keep calculations simple, shallow water is assumed. In shallow water, the wave velocity can be found with $c = \sqrt{gd}$, with g being the gravitational constant of 9.81 m/s^2. For shallow water it also holds that the group velocity is equal to the wave velocity, or $c_g = c$. The shoaling coefficient is defined by equation J.2.

$$K_{sh} = \sqrt{\frac{c_{g1}}{c_{g2}}} \tag{J.2}$$

The value of c_{g2} can also be found through the relation expressed before, depending on the local water depth. This is done along a single line, running from the observation point to the start of the promenade. At the promenade the water level is still 0.13 m. In equation J.3, the bed slope gradient is determined. The bed profile is assumed to be linear between these two points.

$$\alpha = \frac{\Delta d}{\Delta x} = \frac{3.1 - 0.13}{310 - 0} = 0.0096 \tag{J.3}$$

Now the water depth at every distance from the coast can be calculated and from that the local group velocity and shoaling coefficient. The new wave amplitude can be calculated with $a_2 = K_{sh}a_1$.

Close to the shore, the water depth is small and thus the velocity is small. This results in a large shoaling coefficient, what would seem to lead to large wave heights. In reality this never seem to occur, as waves break before they come this close. The breaking is caused by the depth-induced breaking, depending on wave steepness and ratio of wave height to water depth. The latter is also described by the breaking parameter described in equation J.4. γ can be found with experiments. In general a value between 0.7 and 0.8 is used. For the case of Durban, none of such experiments have been carried out. Therefore the default value of 0.73 of Delft3D is used, like what was used in the simulations. With the breaker parameter known, the maximum wave height for each depth value can be found. As the bed profile is linear, this maximum wave height also increases linearly with distance from the coast.

$$\gamma = \frac{H}{d} \tag{J.4}$$

In figure J.1 below, the graphs for the wave height due to shoaling and the maximum wave height due to depth-induced breaking are shown. The point where they intersect defines H_{br}, the largest wave height that will occur near the coast. In this case, it has a value of 1.36 m. The resulting set-up at the waterline can be calculated with equation J.5. This shows that the set-up here only is dependent on the breaking parameter and the wave height.

$$\eta_{waterline} = \frac{5}{16}\gamma H_{br} \tag{J.5}$$

This results in a wave set-up of around 31.7 cm. The formula used above is a simplified form of the horizontal momentum balance equation. Equation J.6 relates the gradient in the radiation stress to the gradient of the surface elevation. The set-up can be found as a function of the distance to the coast. The only additional thing required is the radiation stress, see equation J.7. The derivation of the equations is left out here.

$$S_{xx,w} = \left(2n - \frac{1}{2}\right)E_{wave} = \left(2n - \frac{1}{2}\right)\frac{1}{8}\rho g H^2 \tag{J.6}$$

$$\frac{d\eta_{wave}}{dx} = -\frac{1}{\rho g D}\frac{dS_{xx,w}}{dx} \tag{J.7}$$

In the above, n is the ratio between c_g and c, which in this case is 1. D is a summation of the water depth and the surface elevation at a specific location. H is the maximum wave height than occurs at a specific location, limited by the graphs of shoaling and depth induced breaking in figure J.1. Now everything is known to calculate wave the set-up. The radiation stresses and wave set-up can be seen in figure J.2 and J.3.

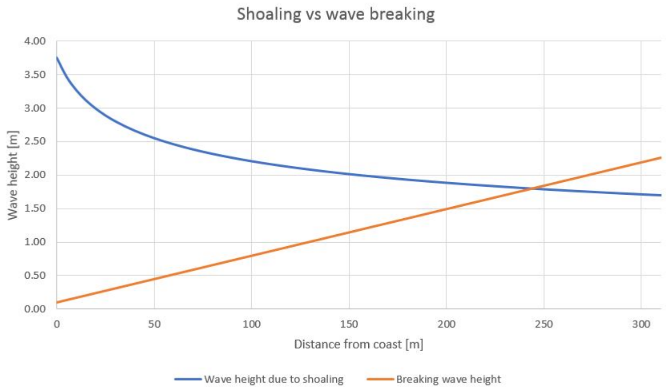


Figure J.1: The shoaling effect and the wave breaking limit visualised

In addition to the wave set-up, another phenomenon takes place, being wave rollers. This is the remaining part of a broken wave, still containing momentum. This roller, like a wave travels on the water surface, thus increasing the water height. Rollers are often left out of the momentum balance, as they are neglected. In this case they are taken into account. The same relation holds to find the surface elevation due to rollers. See equation J.8 and compare it to equation J.7.

$$\frac{d\eta_{roller}}{dx} = -\frac{1}{\rho g D}\frac{dS_{xx,r}}{dx} \tag{J.8}$$

Several formulae have to be used to compute the change in radiation stresses for rollers. These are listed below as equations J.9, J.10 and J.11.

$$S_{xx,r} = 2E_r \tag{J.9}$$

$$E_r = \frac{\rho A c^2}{2L} \tag{J.10}$$

$$A = 0.9H^2 \tag{J.11}$$

The effect of rollers is described by Reniers and Battjes [43]. This is also where equations J.7, J.9 and J.10 come from. A new parameter is the roller surface area A, which is related to the wave height [53]. Everything is known to calculate the radiation stresses and thus the roller height. The results are can be seen in figures J.2 and J.3. The results are a wave set-up of 0.32 m and a roller height of 0.09 m.

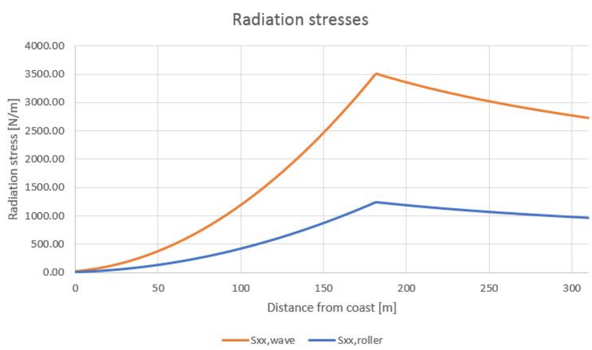


Figure J.2: Radiation stresses of waves and rollers

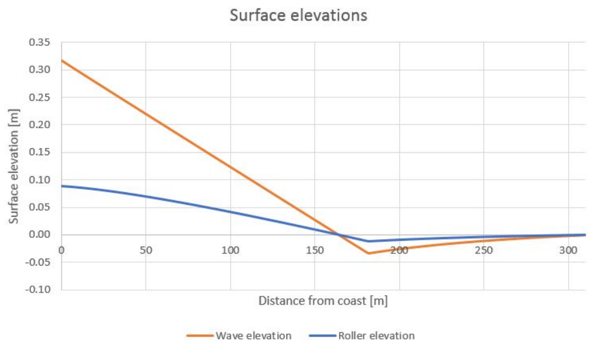


Figure J.3: Surface elevations due to wave set-up and rollers

J.2. 1D SWAN model

Delft3D does not have a bathymetry reaching all the way up to the promenade. Therefore, it is not able to calculate the wave characteristics at this point. SWAN Delft offers a solution in calculating the wave propagation until the edge of the promenade. SWAN has been used in cooperation with Python here. Python was also used to plot the output presented later on in this appendix.

A simple 1D-model was set up to model a governing wave approaching the Durban coastline. The output required from SWAN is the water depth, including the wave-induced water level set-up at the waterline. On top of this, some small waves might have been able to build up again. These parameters are required to design a barrier at the promenade.

The SWAN model consists of a one-dimensional grid starting 2 km from the shoreline, into the ocean. It also needs input for the bathymetry, which could be retrieved for the biggest part from the Delft3D-model. The bathymetry data missing concerns the part closest to the shore/promenade. Some values for this area are retrieved from QGIS and via interpolation the missing bathymetry values have been generated. Next to this, the boundary conditions at the start of the 1D-grid are retrieved from Delft3D. A governing storm has been inserted as input in Delft3D. This model then calculates the wave propagation into the ocean until the point where the SWAN grid starts, i.e. 2 km from the shoreline. The Delft3D-output for the significant wave height, the peak period and the wave direction at this location are used as input for the boundary at the SWAN model. Together with a wind equal to 20 m/s blowing normally incident on the shore, the SWAN input has been completed.

The following output was generated by SWAN. In figure J.4 the development of the significant wave height, the energy dissipation and the wave-induced water level set-up are plotted over the distance of the 1D-model. These plots are generated in a Python script.

Concluding from the 1D-SWAN model, the following parameters at the edge of the promenade are used as input for the preliminary design of the water retaining barrier.

- $H_s = 0.07\ m$
- $d = 0.15\ m$
- $\eta_{waterline} = 0.41\ m$

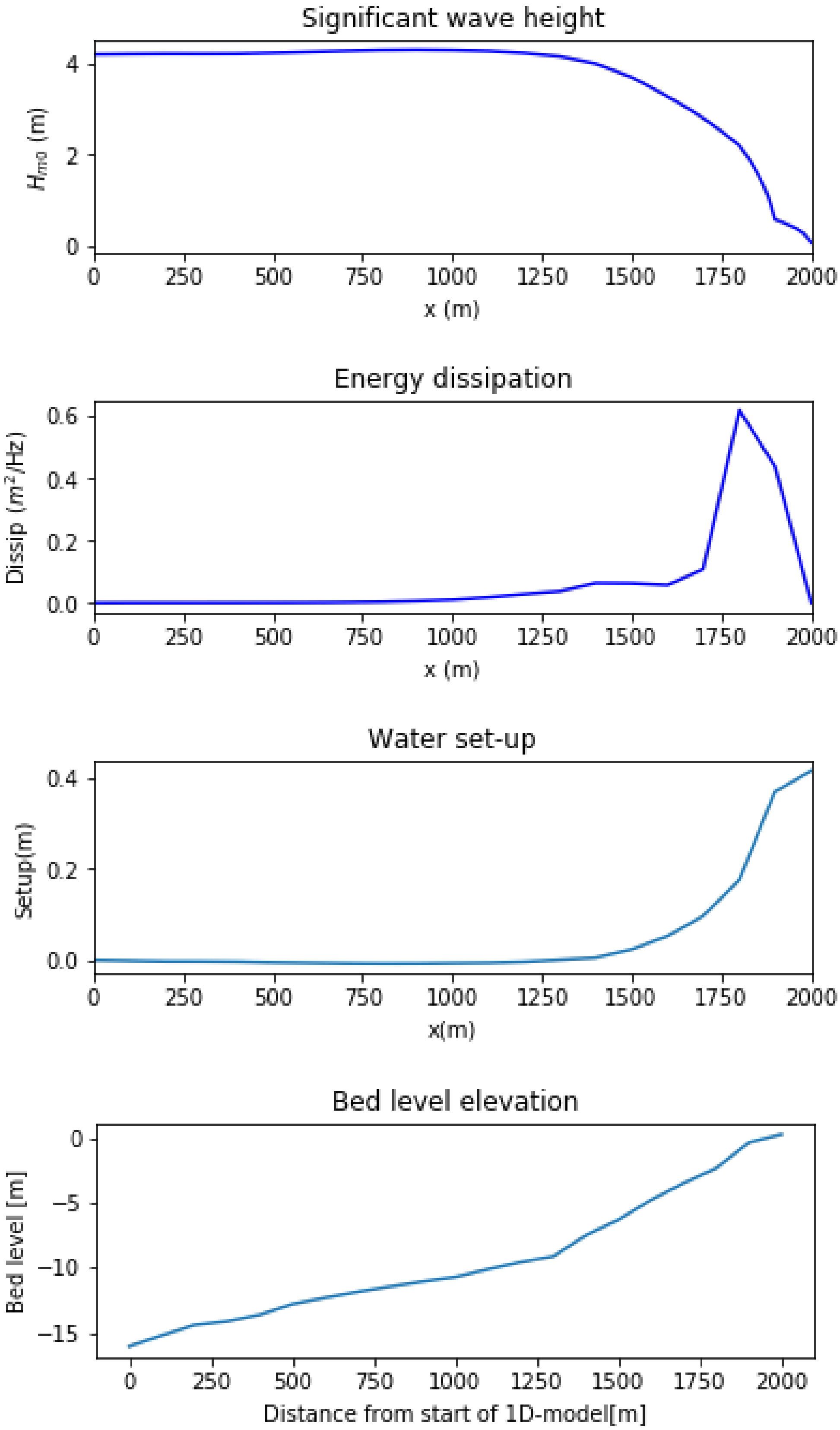


Figure J.4: The output of the 1D-SWAN model

J.3. Overtopping height

First the values of each parameter are stated and explained if needed. Secondly the steps and equations to find the height of the wall are stated. The following values for the parameters are used in the calculation:

- $h = 0.56\ m$;
- $H_{m0} = 0.07\ m$;
- $L_{m-1.0} = 22.27\ m$;
- $q = 0.005\ m^3/s/m$;
- $g = 9.81\ m/s^2$;
- $s_{m-1.0} = 0.0031$;
- $T_{m-1.0} = 9.5\ s$

The H_{m0} is equal to H_s and follows from the calculation in appendix J.2. The depth of the water in front of the construction, h, is 0.56 m as explained in J.3.The $L_{m-1.0}$ is the length of a wave in shallow water, thus at the toe of the structure, which can be calculated according through equation J.12.

$$L_{m-1.0} = \sqrt{gh}T_{m-1.0} \tag{J.12}$$

In paragraph 3.3.5 of the Eur0top manual the value for q is between 1 - 5 liter per second per meter structure width if the waves are smaller than 2 m.[57] According to the Rock manual the overtopping discharge can be between 1 - 10 liter per second per structure meter width before it's starting to get unsafe for people whom are aware of a storm. [35] Locally, a value is used in the range of of 5 - 15 liter per second per structure meter width. The value of the municipality are higher in comparison with the other two ranges. The boulevard is such that the pedestrians have a clear view of the ocean, thus are aware of the high water and possible treat of overtopping. There is only one value which is present in all three ranges, which is 5 liter per second per structure meter width, thus this will be value which is used in the calculations.

The $s_{m-1.0}$ is the steepness of the waves, which is the ratio between the length of the wave and the height of the wave at the toe of the structure. And the period of the wave at the toe of the structure, $T_{m-1.0}$, is taken from the wave model. The model isn't precise at the boundaries, thus the value of $s_{m-1.0}$ will deviate from the the stated values in 4.4.2. The value for $T_{m-1.0}$ is taken from the 1D SWAN model.

The steps which are used to calculate the height of the water retaining structure are described below. Firstly, due to the shallow water at the toe of the structure the foreshore is not influencing. Secondly, according to the bathymetry , there is no mound in front of the structure, thus this doesn't have to be taken in account.

Thridly, there must be a check if the calculations need to be treated with impulsive or non-impulsive conditions. If equation J.13 holds, then the conditions are impulsive.

$$\frac{h^2}{H_{m0}L_{m-1.0}} \leq 0.23 \tag{J.13}$$

Which holds for the values, thus impulsive conditions. The formula in equation J.14 holds with the conditions mentioned above.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0155\left(\frac{H_{m0}}{hs_{m-1.0}}\right)^{0.5}\left(-2.2\frac{R_c}{H_{m0}}\right)\text{valid for}\quad 0.1 < \frac{Rc}{H_{m0}} < 1.35 \tag{J.14}$$

With our values the freeboard height, R_c, is 0.004 m. $0.004/0.07 = 0.0571$, which is smaller than 1.35 and greater than 0.1, thus usable.

K

Loads and load combinations

K.1. Wind load

Wind load is highly dependent on local conditions, which are elaborated here and based on SANS10160-3:2018. Different wind zones can be distinguished in figure K.1 where Durban is located in a wind zone with a governing wind-speed of 36 m/s.

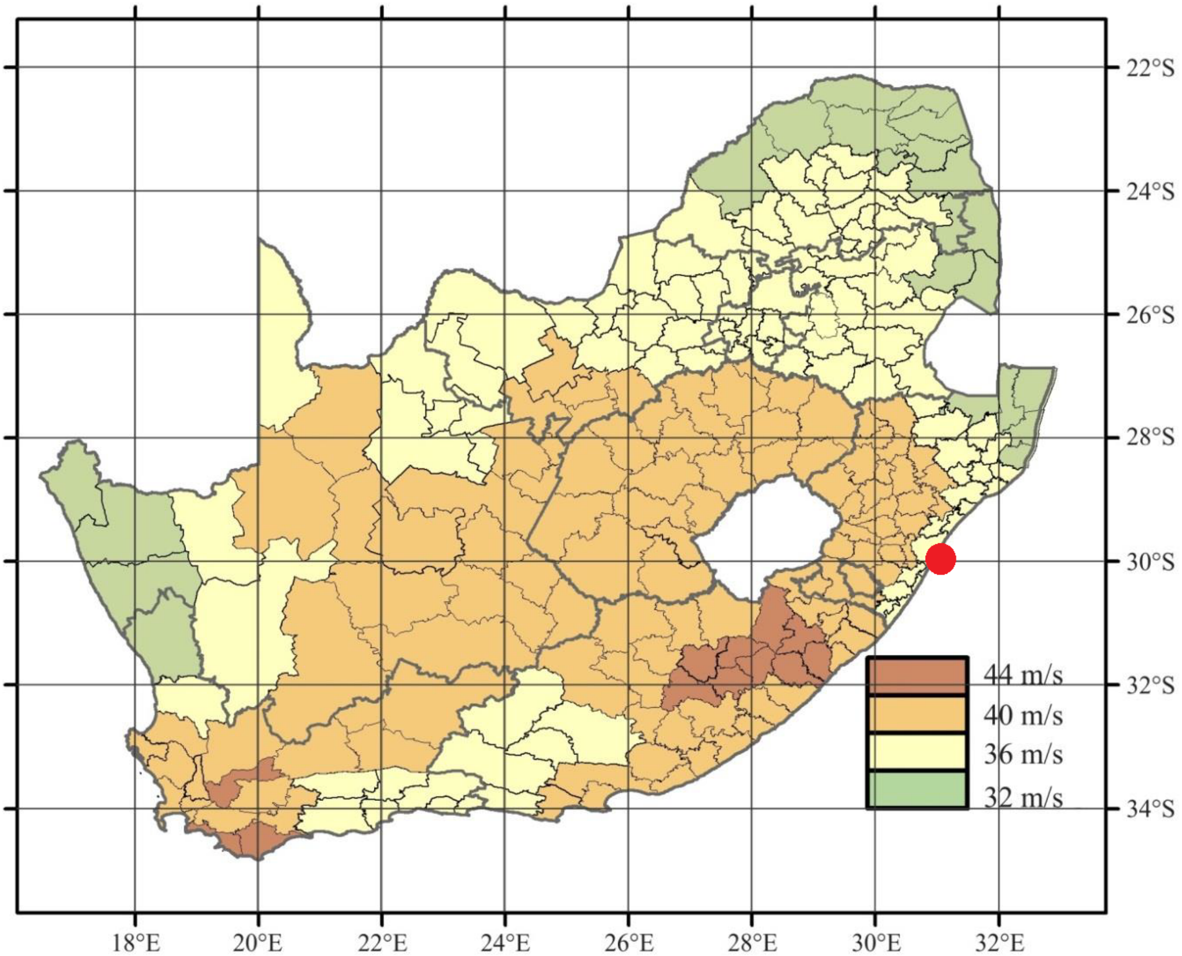


Figure K.1: Wind zones in South Africa in accordance with SANS10160-3:2018.

To determine the wind pressure, equation K.1 is introduced. The orography factor co(z) is depending on the terrain category, which is for the Durban shore category A: 'Flat horizontal terrain with negligible vegetation and without any obstacles'. The orography factor is of influence in area with hills and escarpments, here at the shore line it has no influence, therefore $c_o(z) = 1.0$. From table K.1 can be concluded that for a two meter tall wall $c_r(z) = 0.97$.

Table K.1: Variation of the $c_r(z)$ factor with height above ground level in accordance with SANS10160-3:2018.

1	2	3	4	5
Elevation	Category			
m	A	B	C	D
0	0,92	0,85	0,73	0,71
2	0,97	0,85	0,73	0,71
4	1,02	0,90	0,73	0,71
6	1,05	0,94	0,77	0,71
10	1,09	0,98	0,85	0,71
15	1,12	1,02	0,91	0,78
20	1,14	1,05	0,95	0,83
30	1,17	1,09	1,00	0,90
40	1,20	1,12	1,04	0,95
50	1,22	1,15	1,07	0,98
60	1,23	1,17	1,09	1,01
70	1,24	1,18	1,12	1,04
80	1,26	1,20	1,14	1,06
90	1,27	1,21	1,15	1,08
100	1,28	1,23	1,17	1,10

$$v_p(z) = c_r(z)c_o(z)v_{b,peak} \tag{K.1}$$

Where:
$v_{b,peak}[m/s]$ = 1,0vb;
$c_r(z)$ is the roughness factor;
$c_o(z)$ is the topography (orography) factor;

Concluding, $v_p(z) = 1.0 \cdot 0.97 \cdot 36.0 = 34.92\ m/s$. To determine the peak wind pressure, equation K.2 is used. This concludes in $qp(2) = 731.64\ N/m^2$.

$$q_p(z) = \frac{1}{2}\rho v_p^2(z) \tag{K.2}$$

Where:
$\rho[kg/m^3]$ is the air density, at sea level 1.20 kg/m^3

Finally, the wind pressure in external conditions which is used in further calculations involving wind is introduced in equation K.3. The pressure coefficient c_{pe} can be retrieved from K.2.

$$w_e = q_p(z_e)c_{pe} \tag{K.3}$$

Where:
$q_p(z_e)[N/m^2]$ is the peak wind speed pressure;
$z_e[m]$ is the reference height relevant to the external pressure;
$c_{pe}[-]$ is the pressure coefficient for the external pressure.

Table K.2: Recommended net pressure coefficients cp,net for free-standing walls and parapets in accordance with SANS10160-3:2018.

1	2	3	4	5	6	7
Solidity ratio	**Return corners**	**Aspect ratio**	**Zone**			
			A	**B**	**C**	**D**
$\varphi = 1$	Without	$L/h \leq 3$	2,3	1,4	1,2	1,2
		$L/h = 5$	2,9	1,8	1,4	1,2
		$L/h \geq 10$	3,4	2,1	1,7	1,2
	length $\geq h$		2,1	1,8	1,4	1,2
$\varphi = 0{,}8$			1,2	1,2	1,2	1,2
NOTE Linear interpolation may be used for return corner lengths between 0 and h.						

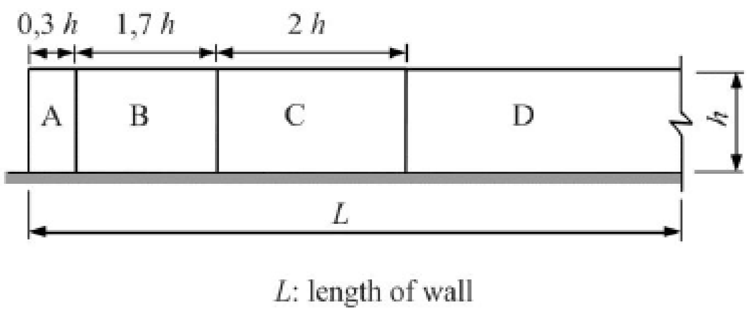


Figure K.2: Zones for free standing walls and parapets with a $L > 4h$ in accordance with SANS10160-3:2018.

Indicated from K.2, the maximum c_{pe} can be found in zone A and is for the preliminary design, conservatively estimated to work all over the wall, $c_{pe} = 3.4$. This concludes that $w_e = 2.48\ kN/m^2$.

Finally, the wind pressure $q_{w,e}$ can be determined with equation K.4 and equals $q_{w,e} = 2.48\ kN/m^2$.

$$q_{w,e} = c_s c_d w_e \tag{K.4}$$

Where:
$c_s c_d$ is the structural factor equal to 1,0;
$w_e[N/m^2]$ is the external pressure on the individual surface at height, z_e, given in equation K.3;

K.2. Water load

The wave induced forces are determined with Sainflou's method, which is a simple method but valid for a preliminary design. Sainflou's method is described in the hydraulic structures manual [39] and in figure K.3. The results are presented in 4.5.1.

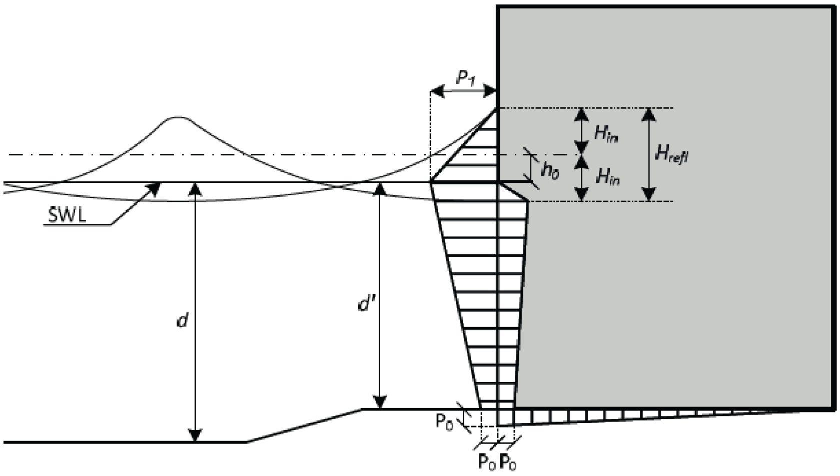


Figure K.3: Sainflou's method to determine forces from waves.[39]

$$h0 = \frac{1}{2} k H_{in}^2 coth(kd) \tag{K.5}$$

Where:
$h_0[m]$ = increase of the mean water level in front of the structure
$H_{in}[m]$ = height of the incoming wave; which is $0.07/2 = 0.035\ m$ as described in [39]
$d[m]$ = water depth in front of the sill, 2 or 3 wave lengths away from the wall, which is $0.4\ m$ as described in [39]
$k[m-1]$ = wave number of the incoming wave, $k = 2\pi/L$
$L[m]$ = wave length, SQRT(9.81 · d') Tm-1,0, as described in [39]
$d'[m]$ = water depth above foundation level of the structure, which is $0.56\ m$ as described in [39]
$Tm{-}1,0[s]$ = spectral wave period at the toe of the structure, which is $9.5\ s$ as described in [39]

Concluding in $h_0 = 0.0015\ m$. The maximum pressures caused by the wave are p_0 and p_1 and are determined in equation K.6 and K.7. Both result respectively $358.4\ N/m^2$ and $339.1\ N/m^2$.

$$p_1 = \rho g (H_{in} + h_0) \tag{K.6}$$

$$p_0 = \frac{\rho g H_{in}}{\cosh(kd')} \tag{K.7}$$

K.3. Load factors γ

Table K.3: Load factors γ from SANS10160-1:2018.

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Type of action			**Part**	**Partial action factor γ_F**									
				Ultimate limit state									
				STR		**STR-P**		**EQU**		**GEO**		**ACC**	
				Un-F[d]	**F**[e]	**Un-F**[d]	**F**[e]	**Un-F**[d]	**F**[e]	**Un-F**[d]	**F**[e]	**Un-F**[d]	**F**[e]
Permanent actions	**Self-weight**		2	1,2	1,0	1,35	-	1,2	0,9	1,0	1,0	1,0	
	Geotechnical actions	Soil parameters un-factored	5	1,2	1.0	1,35	-	Not applicable				1,0 [(a)]	
		Soil parameters factored		Not applicable				1,0		1,0			
	Loads from fluids with a physical control on the maximum fluid level		5	1,2	0	1,35	0	1,2	0	1,0	0	1,0	0
	Imposed deformations due to pre-stressing			1,0	1,0	1,0	1,0	[b]		[b]		1,0	
	Other imposed permanent deformations (for example, settlement)			1,2	-	1,2	-	[b]		[b]		1,0	
Variable actions	Imposed loads: floors and roofs		2	1,6	0	1,0	0	1,6	0	1,3	0	1,0	
	Wind action		3	1,6[c]	0	1,0	0	1,6[c]	0	1,6[c]	0	1,0	
	Imposed variable deformation (for example, temperature)		1 and 7	1,6		1,0	0	[b]		[b]		1,0	
	Overhead travelling cranes and machinery		6	1,6	0	1,0	0	1,6	0	1,3	0	1,0	
	Geotechnical actions	Soil parameters un-factored	5	1,6	0	1,0	0	Not applicable				1,0 [a]	
		Soil parameters factored		Not applicable				1,0		1,0			
	Loads from fluids that vary with time		5	1,6	0	1,0	0	1,6	0	1,3	0	1,0	0
	Other types of variable loads not considered above (for example, material loads) in the absence of more detailed information			1,6	0	1,0	0	1,6	0	1,3	0	1,0	0
	Accidental and seismic actions		1 and 4	Not applicable								1,0	

[a] Soil parameters for the accidental design situation are determined in accordance with SANS 10160-5.
[b] Imposed deformations need not be considered in cases where the achievement of the limit state involves large deformations or bodily movement.
[c] For wind-sensitive structures such as slender non-redundant structures that exhibit significant cross-wind response, a larger factor may be considered. Specialist literature should be consulted.
[d] Un-F = Un-favourable.
[e] F = Favourable.

K.4. Load combination factors Ψ

Table K.4: Load combination factors Ψ from SANS10160-1:2018.

1	2	3	4	5
Variable actions	**SANS 10160 Part**	**Category**	**Specific use**	**Combination factor** Ψ
Imposed loads for occupancy class category	2	A	Domestic and residential areas	0,3
		B	Public areas not susceptible to crowding	0,3
		C	Public areas where people may congregate	0,3
		D	Shopping areas	0,3
		E1	Light industrial use	0,5
		E2	Industrial use	0,6
		E3	Storage areas	0,8
		$FL_1 - FL_6$	Fork lifts	0,6
		F	Traffic and parking areas for vehicles $\leq$ 25 kN	0,8
		G	Traffic and parking areas for vehicles 25 kN to 160 kN	0,3
		H	Inaccessible roofs	0
		J	Accessible flat roofs, excluding occupancy categories A to D	0,3
		K	Accessible flat roofs with occupancies A to D	In accordance with categories A to D
		HCL1-HCL2	Helicopter load	0
Wind actions	3		Applied to accompanying action	0
			Applied to reversible and long-term serviceability actions	0,3
Geotechnical actions: Variable	5			$\psi_{geotechnical}$
			Groundwater	(1,0)
			Ground water (Fluids)	(1,0)
Actions due to cranes (horizontal and vertical)	6			ψ_{crane} [a]
Thermal actions	7			0,3
Other types of variable loads not considered above (for example, material loads) in the absence of more detailed information				ψ [b]

[a] Refer to SANS 10160-6 for the determination of an appropriate value of ψ_{crane}.

[b] Appropriate value, based on value of variable action with similar arbitrary-point-in-time properties.

K.5. Governing load combinations

Table K.5: Load combinations for the barrier.

ULS load combination: — **Location: at groundlevel** — **MOMENT**

permanent:		γg	g		γq	q		Mq		k*Mq	
1	Qwind:	1.35	G	+	1.0	qwind	=	0.79	kNm/m'	0.71	kNm/m'
2	Qwater:	1.35	G	+	1.0	qwater	=	0.21	kNm/m'	0.19	kNm/m'
3	Qimpact	1.35	G	+	1.0	Fimpact	=	0.80	kNm/m'	0.72	kNm/m'

MOMENT

variable:		γg	g		γq	q		ψ	γq	q		ψ	γq	q		Mq		k*Mq	
1.1	Qwind:	1.2	G	+	1.6	qwind	+	1.0	0.0	qwater	+	0.0	1.6	Fimpact	=	0.57	kNm/m'	0.51	kNm/m'
1.2	Qwind:	1.2	G	+	1.6	qwind	+	0.0	1.6	qwater	+	0.3	1.6	Fimpact	=	1.65	kNm/m'	1.49	kNm/m'
2	Qwater:	1.2	G	+	1.6	qwater	+	1.0	0.0	qwind	+	0.0	1.6	Fimpact	=	0.34	kNm/m'	0.30	kNm/m'
3	Qimpact	1.2	G	+	1.6	Fimpact	+	0.3	1.6	qwind	+	0.0	1.6	qwater	=	1.45	kNm/m'	1.31	kNm/m'

MOMENT — **SHEAR FORCE**

accidental:		γg	g		γA	A		ψ	γq	q		ψ	γq	q		ψ	γq	q		Mq+MA		k*(Mq+MA)		Vq+VA		k*(Vq+VA)	
1	qvehicle	1.0	G	+	1.0	qvehicle	+	0.3	1.6	Fimpact	+	0.3	1.6	qwind	+	0.0	1.6	qwater	=	11.76	kNm/m'	10.59	kNm/m'	21.43	kN/m'	19.29	kN/m'
2	qvehicle	1.0	G	+	1.0	qvehicle	+	0.0	1.6	Fimpact	+	0.3	1.6	qwind	+	0.3	1.6	qwater	=	11.27	kNm/m'	10.14	kNm/m'	20.56	kN/m'	18.50	kN/m'

SLS load combination: — **Location: at the top of the wall** — **DEFLECTION**

variable:		γg	g		γq	q		ψ	γq	q		ψ	γq	q		k*w tot		wmax =h/250	
1.1	Qwind:	1.1	G	+	0.6	qwind	+	1.0	1.0	qwater	+	0.0	1.0	Fimpact	=	0.01	<	3.20	mm
1.2	Qwind:	1.1	G	+	0.6	qwind	+	0.0	1.0	qwater	+	0.3	1.0	Fimpact	=	0.01	<	3.20	mm
2	Qwater:	1.1	G	+	1.0	qwater	+	1.0	0.6	qwind	+	0.3	1.0	Fimpact	=	0.01	<	3.20	mm
3	Qimpact	1.1	G	+	1.0	Fimpact	+	0.3	0.6	qwind	+	0.0	1.0	qwater	=	0.018	<	3.20	mm

Legend			
0.0	Not possible/irrelevant	A	Accidental load
0.0	Unfavourable	k	factor for RC1
	Maximum value	M	Moment
γ	Load factor	V	Shear force
ψ	Combination factor	q	Variable load
g	Permanent load	w	deflection

Input			Notes
h_wall	0.8	m	
t_wall	150	mm	assumed, conservative tickness
E_concr	31000	N/mm2	
I_wall	3E+08	mm4	
k	0.9	-	
qwind	2.48	kN/m/m'	
qwater	-		multiple input values
qvehicle	20	kN/m'	
qimpact	0.5	kN/m'	maximum consequences of qimpact or Fimpact
Fimpact	1	kN	maximum consequences of qimpact or Fimpact

Table K.6: Load combinations for the emergency barrier.

ULS load combinations: boards

permanent:	γg	g		γq	q		q		k*q	
1 Qwind:	1.35	G	+	1.0	qwind	=	0.50	kN/m'	0.45	kN/m'
2 Qwater:	1.35	G	+	1.0	qwater	=	0.46	kN/m'	0.41	kN/m'
3 qimpact:	1.35	G	+	1.0	Fimpact	=	0.67	kN/m'	0.60	kN/m'

variable:	γg	g		γq	q		ψ	γq	q		ψ	γq	q		MOMENT Mq		k*Mq			SHEAR FORCE V		k*V	
1 Qwind:	1.2	G	+	1.6	qwind	+	0.0	1.6	qwater	+	0.3	1.6	Fimpact	=	1.25	kNm	1.13	kNm	and with Fimpact = qimpact:	3.93	kN	3.54	kN
2 Qwater:	1.2	G	+	1.6	qwater	+	0.0	1.6	qwind	+	0.3	1.6	Fimpact	=	1.19	kNm	1.07	kNm	and with Fimpact = qimpact:	3.67	kN	3.30	kN
3.1 qimpact:	1.2	G	+	1.6	Fimpact	+	0.3	1.6	qwind	+	0.0	1.6	qwater	=	1.47	kNm	1.32	kNm	and with Fimpact = qimpact:	2.87	kN	2.58	kN
3.2 qimpact:	1.2	G	+	1.6	Fimpact	+	0.0	1.6	qwind	+	0.3	1.6	qwater	=	1.45	kNm	1.30	kNm	and with Fimpact = qimpact:	2.13	kN	1.92	kN

accidental:

SLS load combinations: boards

variable:	γg	g		γq	q		ψ	γq	q		ψ	γq	q		DEFLECTION k*w tot		wmax = L/250	
1 Qwind:	1.1	G	+	0.6	qwind	+	0.0	1.0	qwater	+	0.3	1.0	Fimpact	=	4.24	<	12	mm
2 Qwater:	1.1	G	+	1.0	qwater	+	0.0	0.6	qwind	+	0.3	1.0	Fimpact	=	5.75	<	12	mm
3.1 qimpact:	1.1	G	+	1.0	Fimpact	+	0.3	0.6	qwind	+	0.0	1.0	qwater	=	5.77	<	12	mm
3.2 qimpact:	1.1	G	+	1.0	Fimpact	+	0.0	0.6	qwind	+	0.3	1.0	qwater	=	6.22	<	12	mm

Legend			
0.0	Not possible/irrelevant	k	factor for RC1
	Maximum value	M	Moment
γ	Load factor	V	Shear force
ψ	Combination factor	q	Variable load
g	Permanent load	w	deflection

Input			Notes
L_board	3	m	
t_board	80	mm	assumed, conservative tickness
E_timber	12000	N/mm2	assumed, South African pine class 10
I_board	9E+06	mm4	
k	0.9	-	
qwind	2.48	kN/m/m'	
qwater	0.46	kN/m'	highest load by water originated from hydrostatic pressure.
qimpact	0.5	kN/m'	maximum consequences of qimpact or Fimpact
Fimpact	1	kN	maximum consequences of qimpact or Fimpact

L Calculations

In this appendix the calculations of the concrete barrier, timber boards and the foundation are shown.

L.1. Concrete barrier

L.1.1. SLS calculations

To design the section of the wall, reinforcement should be considered in the next stage, concluding in a thickness of $t = 150\ mm$.

The deflection is calculated via engineering formulas, to be found in appendix M. The governing load combination is shown in equation 4.6 and the deflection is calculated and checked to its requirement. To simplify the calculation, the deflection is calculated for any section of 1 meter.

However, at the edge of the concrete wall, where it meets with the emergency barrier, the forces acting on the timber barrier will be transferred to the edge of the concrete wall via an U-shape slot. Therefore, extra forces act on this part of the barrier, making the assumed 1 meter wall in theory invalid. However, when the emergency barrier is in use, it is assumed that the accidental load by vehicle impact is not governing, causing only the SLS requirement to be invalid.

The governing load combination in SLS:
$1.1 \cdot G_{k,j} + 1.0 \cdot F_{impact} + 0.6 \cdot 0.3 \cdot Q_{wind}$

$w_{Fimpact} = F_{impact} \cdot h^3/(3EI) = 0.018\ mm$
$w_{wind} = q_{wind} \cdot h^4/(8EI) = 0.0005\ mm$
$w_{tot} = k \cdot (1.1 \cdot G_{k,j} + 1.0 \cdot w_{Fimpact} + 0.6 \cdot 0.3 \cdot w_{wind}) = 0.018\ mm$

Where:
$F_{impact} = 1.0\ kN$
$q_{wind} = 2.48\ kN/m^2$
$E = 31\ GPa$
$I = 1/12 \cdot b \cdot t^3$
$b = 1000\ mm$, assumed section of 1 meter
$t = 150\ mm$
$h = 800\ mm$, and is determined in [39]

$w_{max} = h/250 = 3.20\ mm$
$UC = w_{tot}/w_{max} = 0.006 < 1.0$

It can be concluded that the deflection is not governing given the very low Unity Check (UC), $UC = 0.006$.

L.1.2. ULS calculations

Based on the governing load combinations introduced in equation 4.5, one could design the concrete sea wall. The maximum obtained moment and shear force in ULS are used in design of reinforcement of the wall. The governing situation in ULS is presented in figure L.1

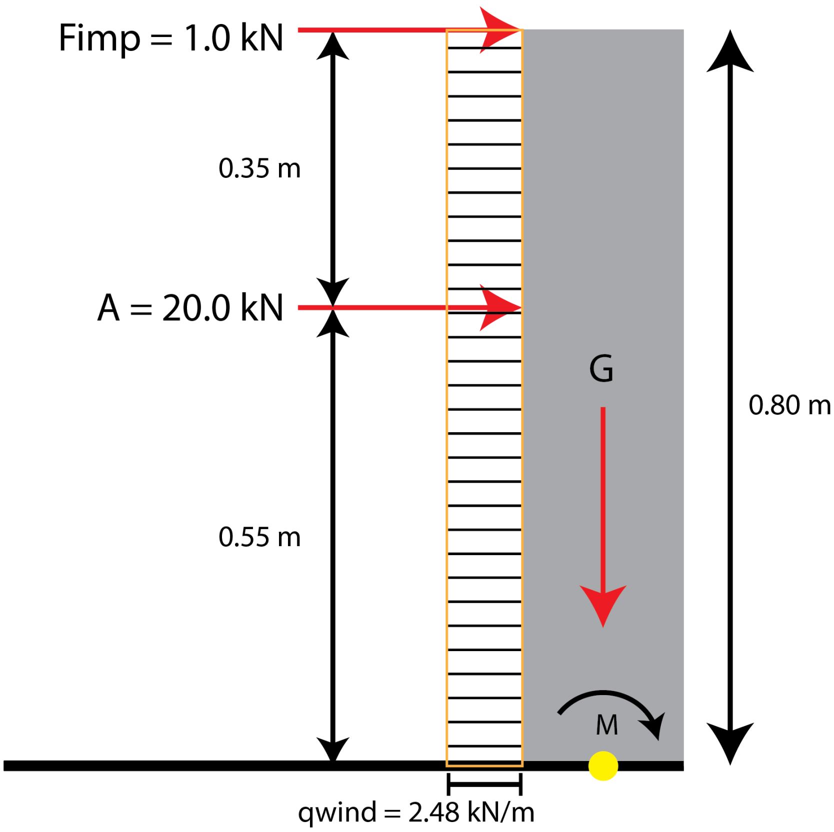


Figure L.1: Governing load combination for both the moment and shear force at ground level.

The governing load combination is presented in section 4.5.3 and appendix K.5 and is here further elaborated to give insight in the calculations.

M_d is defined by:
$M_d = k \cdot \Sigma M = 1.0 \cdot 20 \cdot 0.55 + 0.3 \cdot 1.6 \cdot 1.0 \cdot 0.8 + 0.3 \cdot 1.6 \cdot 2.48 \cdot 0.5 \cdot 0.8^2 = 0.9 \cdot 11.76 = 10.59\ kNm/m$

V_d is defined by:
$V_d = k \cdot \Sigma V = 1.0 \cdot 20 + 0.3 \cdot 1.6 \cdot 1.0 + 0.3 \cdot 1.6 \cdot 2.48 \cdot 0.8 = 0.9 \cdot 21.43 = 19.29\ kN/m$

Assumptions

The wall is simplified to a cantilever beam with a height of 0.8 m and a width of 1.0 m, see figure L.2 for an situation sketch. In this appendix it is shown how much reinforcement there is needed according the South-African standard (SABS 0100-1 edition 2.2, from the year 2000) if the thickness of the seawall is 0.15 m. These are the first calculations and the amount of steel is an indication and not the precise amount of steel which is needed.

Firstly the parameters, taken from the South-African standards are stated:

- Material factor concrete (ULS), $\gamma_{m,c} = 1.50$;
- Material factor reinforcement steel (ULS), $\gamma_{m,s} = 1.15$;
- Material factor steel and concrete (SLS), $\gamma_m = 1.0$;
- Concrete compression cube strength , $f_{cu} = 40\,Mpa$;
- Concrete Young's modulus, $E_c = 31\,Gpa$;
- Reinforcement steel characteristic strength, $f_y = 250\,Mpa$;
- Reinforcement steel young's modulus, $E_s = 200\,Gpa$;
- The concrete cover when concrete is in contact with sea water, $c = 65\,mm$;

In the South-African standards there are several assumptions made during the calculation of the amount of concrete, which are:

- Plain sections remain plain;
- The tensile strength of concrete is ignored;

The thickness of the seawall needs to assumed. The cover of 65 mm is on both sides, thus in total 130 mm. The thickness is chosen to be 150 mm, after some trial and error, and there is assumed there is only one bar needed in the middle of the beam and only tension reinforcement is needed. This bar can have a maximum diameter of 20 mm, assuming that the bar doesn't fall in the the concrete compression part. With this assumption, the d, distance from bar to the edge of the cross-section is equal to $\frac{h}{2} = 75\,mm$. The maximum moment at the support of the cantilever is (M_{ed} =)10.59 kNm and the maximum shear force (V_{ed} =)19.29 kN due to the normative load combinations, determined in section 4.5.2. A sketch of the situation is shown in figure L.2.

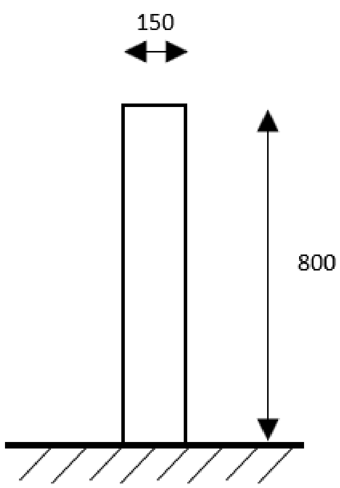


Figure L.2: Sketch of situation seawall, all measurements are in mm

Moment resistance
Firstly, the assumption that only tension reinforcement is needed is checked. If equation L.1 holds than only tension reinforcement is needed.

$$\frac{M_{ed}}{bd^2 f_{cu}} = \frac{10.59 \cdot 10^6}{1000 \cdot 75^2 \cdot 40} = 0.047 \leq 0.156 \qquad \text{(L.1)}$$

The check holds, thus only tension reinforcement is needed. Then the internal arm is calculated according to the equation L.2.

$$z = d\left(0.5 + \sqrt{0.25 - \frac{K}{0.9}}\right) = 75 \cdot \left(0.5 + \sqrt{0.25 - \frac{0.047}{0.9}}\right) = 70.85\ mm \tag{L.2}$$

The second assumption, that the bar isn't in the concrete compression zone can be checked. The length (from the side of the cross-section) of the concrete compression zone can calculated via equation L.3.

$$x = \frac{d - z}{0.45} = \frac{75 - 70.85}{0.45} = 9.23\ mm \tag{L.3}$$

The bar is at 75 mm, thus not in the concrete compression zone. In figure L.3 the forces, stresses and geometry is shown. The minimum amount of steel can be found with the internal lever arm, acting moment and characteristics of the steel, as shown in equation L.4.

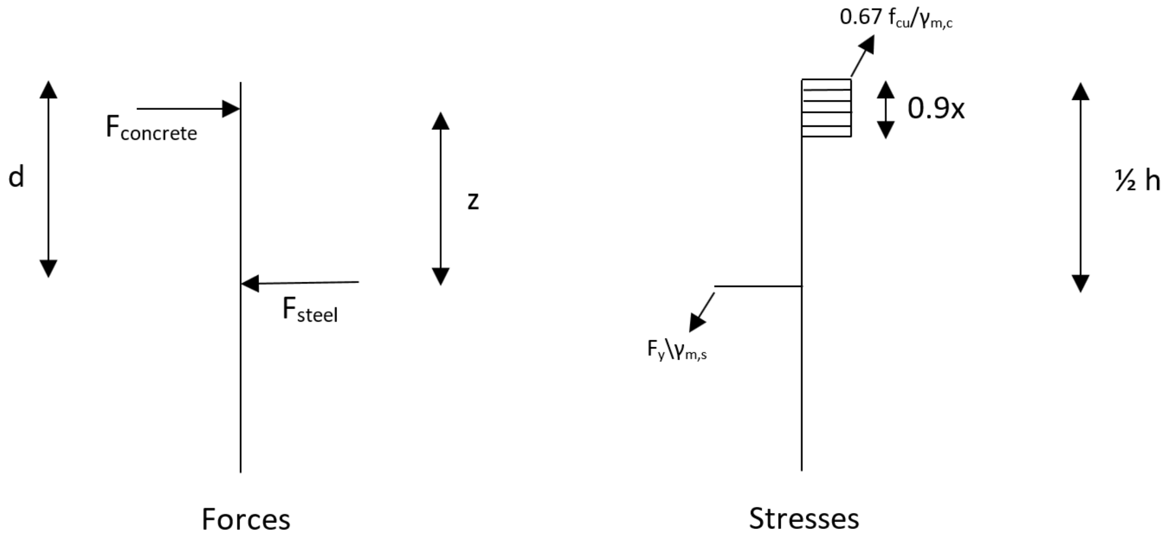


Figure L.3: Sketch of the forces and stresses in the cross-section

$$A_s = \frac{M_e d}{0.87 f_y z} = \frac{10.59 \cdot 10^6}{0.87 \cdot 250 \cdot 70.85} = 687.13\ mm^2 \tag{L.4}$$

According to the South-African design codes, there is a minimum and maximum amount of reinforcement, which respectively are shown in equation L.5 and L.6.

$$\frac{A_c \cdot 0.24}{100} = \frac{1000 \cdot 150 \cdot 0.24}{100} = 360\ mm^2 \tag{L.5}$$

$$A_c \cdot 0.04 = 1000 \cdot 150 \cdot 0.04 = 6000\ mm^2 \tag{L.6}$$

There is chosen for 8 bars of 11 mm in diameter and spaced each 125 mm. This gives you a total steel reinforcement area of 760.26 mm^2, which is bigger than the minimum amount and smaller than the maximum amount of reinforcement steel.

The total moment which the cross-section can resist is the force in the reinforcement steel multiplied with the internal arm:

$$M_{rd} = \frac{A_s f_y z}{\gamma m, s} = \frac{760.26 \cdot 250 \cdot 70.85}{1.15} = 11.71\ kNm \tag{L.7}$$

Thus the unity check, $\frac{M_{ed}}{M_{rd}}$, is equal to 0.90.

Shear resistance
The maximum stress which can occur through a shear force, according to the South-African standards (SABS 0100-1 edition 2.2, from the year 2000), is calculated in equation L.8.

$$min(0.75\sqrt{f_{cu}}; 4.75) = min(4.74; 4.75) = 4.74\ N/mm^2 \tag{L.8}$$

If the condition in equation L.9 holds then there is no need for shear reinforcement.

$$\frac{V_{ed}}{bd} < \frac{0.75}{\gamma_{m,c}}(\frac{f_{cu}}{25})^{\frac{1}{3}}(\frac{100A_s}{bd})^{\frac{1}{3}}(\frac{400}{d})^{0.25} <=> 0.26 < 0.78 \tag{L.9}$$

Thus there is no need for shear reinforcement. The total shear force which the cross-section can handle is the shear stress, calculated in equation L.9 multiplied with the cross section, shown in equation L.10.

$$V_{rd} = 0.78bd = 0.78 \cdot 1000 \cdot 75 = 58.75\ kN \tag{L.10}$$

The unity check, $\frac{V_{ed}}{V_{rd}}$, is equal to 0.33.

L.2. Timber boards

In this appendix, the design of the emergency barrier, as timber boards, is elaborated. After an iterative process of a trial dimension and the unity checks on bending moment capacity, shear capacity and deflection, the final dimensions of a single timber emergency board are $3300x80x200\ mm$ with a $L = 3\ m$ span. In the next sections unity check and weight are elaborated following SANS 10163-1:2003 standards, using these dimensions and a SA pine timber class 10, see table L.1.

Table L.1: Characteristic stresses for South African pine (SANS 10163-1:2003

N.1 Characteristic stresses for South African pine

Table N.1 — Characteristic stress[1)] for SA pine

Values in megapascals

1	2	3	4	5	6	7	8
Grade[2)]	Bending f_b	Tension parallel to grain f_t	Tension perpendicular to grain f_{tp}	Compression parallel to grain $L_e/b = 0$ f_c	Compression perpendicular to grain f_{cp}	Shear parallel to grain f_v	Modulus of elasticity E
5	11,5	6,7	0,36	18,0	4,7	1,6	7 800
7	15,8	10,0	0,51	22,8	6,7	2,0	9 600
10	23,3	13,3	0,73	26,2	9,1	2,9	12 000
14	32,4	19,1	1,04	35,3	12,9	4,0	16 000

1) These stresses apply to visually, mechanically or proof-graded timber that complies with SANS 1783-2 (grades 5, 7 or 10), SANS 1460, grades 5, 7, 10 and 14 as appropriate.

2) Grade stresses given here are for a range of grades beyond those covered in current SANS standards. The intention is to cater for any special purpose grade that could be introduced.

NOTE 1 Designers should check the availability of any grade they wish to specify.

NOTE 2 SANS 1460 refers to both hardwoods and softwoods. In the case of laminated timber, use the appropriate grade stress for the applicable grade of timber.

L.2.1. ULS calculations

Bending moment capacity
The bending moment due to the governing load combinations gives an maximum bending moment of $M_{Ed} = 1.32\ kNm$. For the bending moment capacity SANS 10163-1:2003 uses the following equation in ULS:

$$M_{Rd} = \phi * Z_e * \frac{f_b}{\gamma_1 * \gamma_2 * \gamma_3 * \gamma_4 * \gamma_5} = 1.63\ kNm \quad \text{(L.11)}$$

Where:

- Resistance factor: $\phi = 0.68$
- Section modulus: $Z_e = \frac{bh^2}{6} = 2.13 * 10^5\ mm^3$
- Characteristic bending stress: $f_b = 23.3\ N/mm$
- Partial material factor for load duration: $\gamma_1 = \frac{C_{fD}W_{DU}+C_{fI}W_{IU}+C_{fW}W_{WU}}{W_{DU}+W_{IU}+W_{WU}} = 1.72$
- Partial material factor for load sharing: $\gamma_2 = 0.87$
- Partial material factor for stressed volume: $\gamma_3 = 0.85 + 0.03L = 0.94$
- Partial material factor for moisture content: $\gamma_4 = 1.33$
- Partial material factor for pressure treatment: $\gamma_5 = 1.11$
- Factored self-weight load effect: $W_{DU} = 0$
- Factored imposed load effect: $W_{IU} = 0.667\ N/mm$
- Factored wind load effect: $W_{WU} = 2.48 * 0.2 = 0.496\ N/mm$
- Load coefficient for self-weight loads: S_{fD} = not applicable
- Load coefficient for imposed loads: $S_{fI} = 2.42$
- Load coefficient for wind loads: $C_{fW} = 0.77$

The factored load effect following from the governing load combinations in appendix K.5. The corresponding load coefficients are determined using table L.2, except from the self-weight coefficient as the timber board has no self-weight in the direction of the imposed load.

Table L.2: Load coefficients for different load combinations

1	2	3	4	5	6
Load combinations to SANS 10160	C_{fD}	C_{fI}			C_{fW}
		Short	Medium	Long	
Self-weight load only	1,00	–	–	–	–
Self-weight load plus imposed load	1,26	0,62	0,76	0,94	–
Self-weight load plus imposed load plus wind load[1)]	1,26	1,99	2,42	3,02	0,77
Self-weight load plus wind load[1)]	1,67	–	–	–	0,77
1) Factors to be applied if effect of loads is in the same direction. Where wind load predominates, use only C_{fW}.					

A side note, γ_4 has to be taken into account in cases where the moisture content could occasionally exceed 200 g/kg. For this construction it is assumed this is the case.

This leads to the following bending moment capacity unity check, which has been satisfied:

$$\frac{M_{Ed}}{M_{Rd}} = \frac{1.32}{1.63} = 0.81 \quad \text{(L.12)}$$

Shear capacity

The shear due to the governing load combinations gives an maximum shear force of $V_{Ed} = 3.54\ kN$. For the shear capacity SANS 10163-1:2003 uses the following equation in ULS:

$$V_{Rd} = 0.67 * \phi * A_v * \frac{f_v}{\gamma_1 * \gamma_2 * \gamma_3 * \gamma_4 * \gamma_5} = 10.18\ kNm \quad \text{(L.13)}$$

Where:

- Resistance factor: $\phi = 0.68$
- Shear area: $A_v = b * h = 1.6 * 10^4\ mm^2$
- Characteristic shear stress parallel to the grain: $f_v = 2.9\ N/mm$
- Partial material factor for load duration: $\gamma_1 = \frac{C_{fD}W_{DU}+C_{fI}W_{IU}+C_{fW}W_{WU}}{W_{DU}+W_{IU}+W_{WU}} = 1.72$
- Partial material factor for load sharing: $\gamma_2 = 0.87$
- Partial material factor for stressed volume: $\gamma_3 = 0.85 + 0.03L = 0.94$
- Partial material factor for moisture content: $\gamma_4 = 1.33$
- Partial material factor for pressure treatment: $\gamma_5 = 1.11$
- Factored self-weight load effect: $W_{DU} = 0$
- Factored imposed load effect: $W_{IU} = 0.667\ N/mm$
- Factored wind load effect: $W_{WU} = 2.48 * 0.2 = 0.496\ N/mm$
- Load coefficient for self-weight loads: S_{fD} = not applicable
- Load coefficient for imposed loads: $S_{fI} = 2.42$
- Load coefficient for wind loads: $C_{fW} = 0.77$

All factors and coefficients are already explained above. This leads to the following shear capacity unity check, which has been satisfied:

$$\frac{V_{Ed}}{V_{Rd}} = \frac{3.54}{10.18} = 0.35 \tag{L.14}$$

L.2.2. SLS calculations

The deflection due to the governing load combination gives a calculated deflection of $w_{calculated} = 6.22\ mm$ using the elastic theory in SLS. For the governing design deflection SANS 10163-1:2003 uses the following equation:

$$w_{design} = w_{calculated} * d_1 * d_2 = 10.57\ mm \tag{L.15}$$

Where:

- Factor for load duration: $d_1 = \frac{W_{DS}+W_{IS}+W_{WS}}{C_dD*W_{DS}+C_{dI}*W_{IS}+C_{dW}*W_{WS}} = 1.0$
- Factor for moisture content: $d_2 = 1.7$
- Factored self-weight load effect: $W_{DS} = 0$
- Factored imposed load effect: $W_{IS} = 0.667 + 0.46\ N/mm$
- Factored wind load effect: $W_{WS} = 0$
- Load coefficient for self-weight loads: S_{dD} = not applicable
- Load coefficient for imposed loads: $S_{dI} = 1.0$
- Load coefficient for wind loads: C_{dW} = not applicable

The factored load effect following from the governing load combinations in appendix K.5. The corresponding load coefficients are determined using clause 12.1.2 from SANS 10163-1:2003, except from the self-weight coefficient and wind coefficient as the timber board has no self-weight in the direction of the imposed load and wind load is not in governing in SLS.

For the deflection criteria SANS 10160-1:2018 uses table L.3 for reversible and long-term serviceability state. In this case the terminal deviation of vertical members is applicable, which gives the following equation:

$$w_{max} = \frac{L}{250} = \frac{3000}{250} = 12\ mm \tag{L.16}$$

This leads to the following deflection unity check, which has been satisfied:

$$\frac{w_{design}}{w_{max}} = \frac{10.57}{12} = 0.8 \tag{L.17}$$

1	2	3	4	5	6	7	8	9	10	11	12
			Actions and deflections[a]								
Deformation	Effect	Criterion	Construction deviation and camber	Differential settlement	Structural self-weight	Non-structural self-weight	Pre-stressing	Imposed load	Wind	Clause	Conditions and comments
Terminal deviation of non-cantilever horizontal members	Use (slope)	Span/100	Yes	E[b] C[c]						C.9.1	
Terminal deviation of vertical members	Appearance	Storey height/250	Yes	E[b] C[c]	E[b] C[c]	E[b] C[c]		E[b] C[c]		C.8.2	Where self-weight, imposed and wind loads act eccentrically
Oscillations of members	Resonance	–						E[b]	E[b]	C.9.3	
	Use	–						E[b]	E[b]		
Oscillations of building as a whole	Use	–						E[b]	E[b]		
Horizontal terminal deflection of high-rise buildings		Building height/500									

[a] Columns 4 to10 indicate which actions and displacements are to be considered when calculating compliance of the structure with the given criterion.
[b] Elastic effect.
[c] Creep effect.

Table L.3: Summary of recommended criteria for the reversible and long-term serviceability limit state

Weight
For practical reasons during installation of the emergency barrier the weight is an important parameter. Given the dimension of a single timber emergency board of length x width x height is $3300 \cdot 80 \cdot 200\ mm$ and a density of $420\ kg/m^3$ the total weight of the board is: $420 \cdot 3.3 \cdot 0.08 \cdot 0.2 = 22.2\ kg$. For labour conditions for installation the emergency barriers by two people this can be assumed as reasonable.

L.3. Foundation

L.3.1. Horizontal

Blum's method is described in [39] and summarized in this appendix. Blum's method is used to find the embedded depth of the sea wall. With Blum's method, it is assumed that the wall will rotate around point D, as shown in figure L.4.

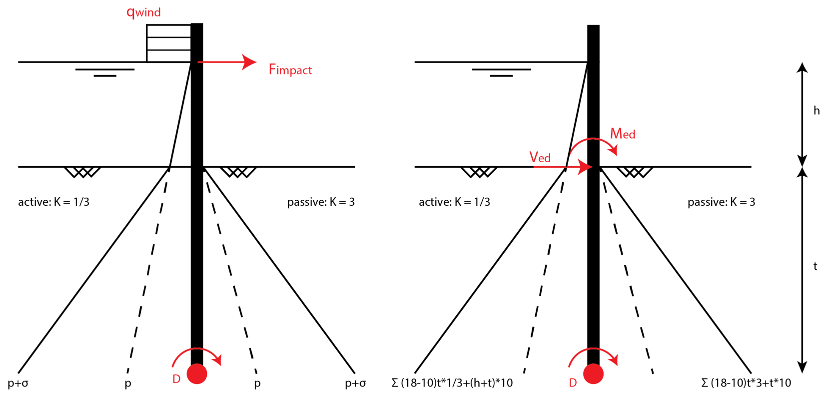


Figure L.4: Pressure diagrams as used for Blum's method for an infinitely stiff sea wall.

The theoretical embedded depth can be calculated considering an equilibrium of forces around point D in the governing load case. The governing load case in SLS is water dominated and combined with wind (above sea level) and an impact load. The combination should be multiplied by factor $k = 0.9$ as described in section 4.5.1

$$w : 1.1G_{k,j} + 1.0q_{water} + 0.3 \cdot 1.0q_{wind} + 0.3 \cdot 1.0F_{impact} \tag{L.18}$$

Where:
M_{ed} is the design moment caused by wave induced forces, wind and impact load
V_{ed} is the design shear force caused by wave induced forces, wind and impact load
t is the embedded depth
h is the water level
K is the coefficient considering active or passive ground pressure.

$M_{ed} = M_{wind} + M_{impact} + M_{water} = 0.55\ kNm/m$
$V_{ed} = V_{wind} + V_{impact} + V_{water} = 0.89\ kN/m$
$\Sigma M = M_{ed} + V_{ed}t + \left(\frac{8}{3}t + 10t\right) \cdot \frac{1}{3}t \cdot \frac{1}{2}t + h \cdot 10 \cdot \frac{1}{2}t \cdot t - 8t \cdot 3 \cdot \frac{1}{2}t \cdot \frac{1}{3}t - t \cdot 10 \cdot \frac{1}{2} \cdot \frac{1}{3}t = 0$

By solving this, the embedded depth (t) can be obtained, which becomes $t = 1.72\ m$ According to Blum's method, the passive pressure is underestimated. Therefore an extra of 20% of the embedded length is added [39]. This results in a total embedded depth of $t = 2.06\ m$.

L.3.2. Vertical

In this section the vertical pressure on the ground, underneath the seawall is calculated. The total height of the construction is $0.8 + 2.06 = 2.86\ m$. The thickness is $0.15\ m$ and for this calculation there is looked at a section of $1.0\ m$ in width. Which results in a total volume of $0.15 \cdot 1.0 \cdot 2.86 = 0.429\ m^3$.

Assuming that the density of the concrete is $2400\ kg/m^3$, the total mass of the 1 meter width wall is $2400 \cdot 0.429 = 1029.6\ kg$. Next to that there are only variables loads through visitors of the beach which use the seawall as a bench, which would give a vertical load. Assumed is that in the 1 meter width a maximum of 5 people (each $100\ kg$) could sit, which gives an extra mass of $500\ kg$. The two loads need to combined as presented in 4.5.2:

$1.2 \cdot 1029.6 \cdot 9.81 + 1.6 \cdot 500 \cdot 9.81 = 19.97\ kN$
The resistance by the soil is calculated via Brinch Hansen's method. Given undrained conditions indicating that the angle of friction $\phi = 0$. Therefore, no bearing capacity is obtained from the weight of the soil. Hansen's equation is therefore simplified to:

$p_{undrained} = c_u N_c s_c i_c + \sigma'_q$
Now with the knowledge that the undrained shear strength (c_u) for loose sand equals zero, the only bearing resistance comes from the effective soil pressure. Which is:
$\sigma'_q = q + \gamma' d$

Where:
$p_{undrained}$ is the bearing capacity q is the load at ground level, here $q = 0$;
γ' is the effective soil pressure, here in loose sand $\gamma' = 7\ kN/m^2$;
d is depth assumed, here $d = 2.06\ m$;

The result is a bearing capacity of $p_{undrained} = 14.42\ kN/m2$. Given the load of $19.97\ kN/m$, a footing with a width of $1.4\ m$ is needed to spread the load.

M

Engineering formulas

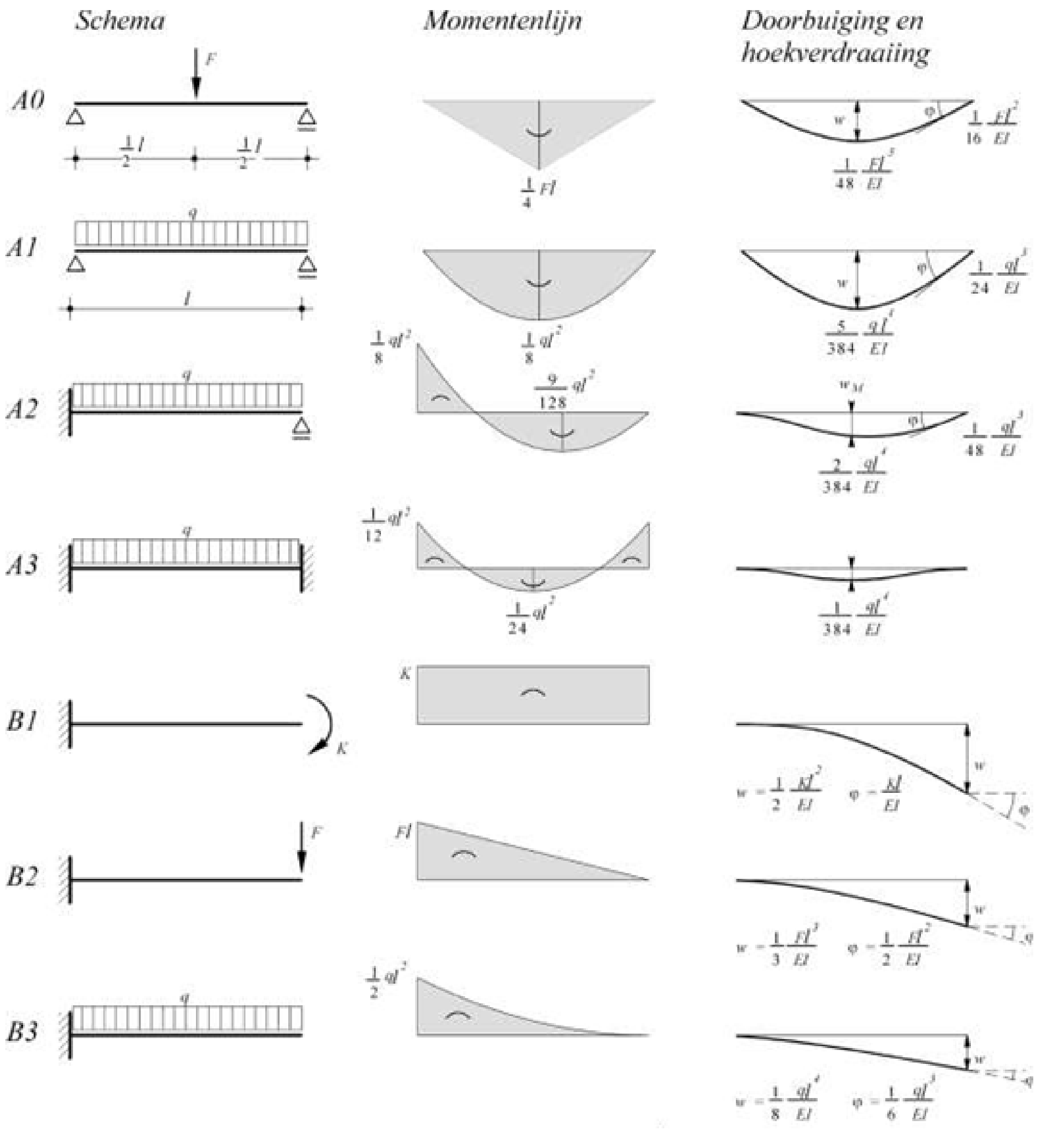


Figure M.1: Engineering formulas for deflection and rotation of a structural element.

N

Impressions of the integrated design

In this appendix, impressions of the integrated design are added from different perspectives and cross-sections.