Graduation Thesis

Design Study of a Flexible-Membrane Tsunami Barrier Concept

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Preface

This report is the result of my master thesis and is the final step to complete the Masters program Hydraulic Engineering with a specialisation in Hydraulic Structures at the Technical University of Delft. The aim of this research is to perform a feasibility assessment of the flexible-membrane barrier 'Tsunami Catcher' for the case study of Kamakura.

I would like to thank all the members of my graduation committee, consisting of prof. dr. ir. S.N. Jonkman, dr. ir. B. Hofland, prof. dr. ir. R. Marissen and ir. F. van der Ziel, for their input and support during the process. Furthermore I would like to thank the people of Deltares and Royal HaskoningDHV. Finally I would like to thank my family and friends for supporting me, conducting this research.

Bas Horsten

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Abstract

On March 11 2011, an earthquake occurred with a magnitude of 9.0 and triggered a tsunami just off the Pacific coast of Tohoku in Japan. The tsunami inundated over 400 km$^2$ of land and there were over 15,000 fatalities and over 5,000 people missing. The inundation height of the Tohoku tsunami exceeded the design levels of many coastal barriers and several structures collapsed.

After the tsunami, the government of Japan announced a new coastal defence system. Hundreds meter of seawall and breaker are (planned to) (re)built in the three worst-hit areas. Some critics have raised objections and the question arose if there are any alternative tsunami barriers.

One of the latest concepts in the field of tsunami barriers, is the so-called 'Tsunami Catcher'. This is a flexible-membrane barrier which will self-deploy when a tsunami occurs, due to the buoyancy of a floating element. The concept has been studied experimentally at Deltares and it was proven that the deploying principle of the barrier works. The complex behaviour of the structure however causes uncertainties in the design. So the aim of the research is to perform:

a feasibility assessment of the flexible-membrane barrier 'Tsunami Catcher' for the case study of Kamakura.

Kamakura is a coastal city located in the Sagami Bay, approximately 50 kilometers South-East of Tokyo. The city is characterised by beautiful beaches and the city has a large touristic value. A flexible-membrane tsunami barrier could be a proper tsunami counter measure, without reducing the touristic value.

It is assumed that the barrier will be part of a Multi Layer System and it will function as a primary barrier in the layer. The barrier is required to fully retain a Level 1 tsunami (return period = 100 years and the assumed height of the incoming wave is 8 meter) and must be able to withstand a Level 2 tsunami (return period = 1000 years and the assumed height of the incoming wave is 14 meter), where overflow of the wave is allowed.

A numerical one-dimensional SWASH model is composed to model the incoming tsunami wave for Kamakura. The model is validated for the reference case study of Sendai with the measured data during the Tohoku tsunami and with analytical models. It is concluded that the incoming tsunami can be approximated with the N-wave theory.

The flexible-membrane barrier consists out of 6 elements. The main retaining element is the membrane. This is held up by a floating element which is also connected to cables to keep the floater at its position. These cables are connected to an offshore foundation. The membrane is connected to a concrete bottom recess structure which is placed on a pile foundation [Figure 3]
During normal conditions the membrane, floater and cables are stored in the bottom recess structure at ground level. When a tsunami occurs, the barrier is inflated due to the buoyancy of the floater. When the incoming tsunami wave is reflected, the membrane is fully inflated and retains the raised water level.

In the Global Design, the relations between the critical elements are studied in function of the retaining height of the barrier, for the retaining phase of a Level 1 tsunami. These elements are the cable configuration, the membrane configuration and floater dimensions. The derived theory is applied to the case study of Kamakura.

It is concluded that to obtain an optimum configuration for Kamakura, the retaining height of the membrane must be decreased. This is achieved by increasing the height of the bottom recess structure with three meters. With this measurement, an optimum configuration for Kamakura is found with a membrane length of 42 meter, a cable length of 90 meters and a floater diameter of 5 meter.

The derived Global Design is further elaborated and integrated in the surroundings for the case study of Kamakura in the Site Specific Design. The different load conditions are analysed for both tsunami levels. The 6 elements are dimensioned and checked for different failure mechanisms. The costs of the barrier are estimated at 70,000 euro per running meter. The barrier is integrated in the surroundings of Kamakura. It is recommended to integrate the barrier in the existing dike structure of Kamakura. Several adjustments and optimisations are presented to improve the integration of the barrier.

It can be concluded, based on the presented, preliminary design for the case study of Kamakura is technical feasible, and therefore can be considered to be an alternative solution for the conventional sea walls. This research study of the tsunami barrier is a first step towards a total design of the flexible membrane barrier.

It is recommended to do further research. The wave analysis can be extended by applying a three dimensional wave model. The Global Design can be extended by study the dependencies between the remaining elements; the pile foundation, the drag embedded anchor and the bottom recess structure. For the Site Specific Design several recommendations are proposed to further design the flexible membrane barrier. Examples of the elements that can be further detailed are: the design of the connections, the analysis of other failure mechanisms and the behaviour of the barrier in the assumed coastal defence system.

Figure 3: Impression of the integrated tsunami barrier
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Introduction

1.1 Motivation of the Research

On March 11 2011, an earthquake with a magnitude 9.0 occurred and triggered a tsunami just off the Pacific coast of Tohoku in Japan. The tsunami reached the Japanese mainland after 20 minutes the initial earthquake. The tsunami wave affected a 2000 kilometer stretch of Japan’s Pacific coast. The tsunami inundated over 400 km$^2$ of land, there were over 15,000 fatalities and over 5,000 people missing [Mori et al., 2011].

There were several earthquake and tsunami disaster-countermeasures in the Tohoku district, such as offshore and onshore tsunami barriers, vertical evacuation buildings and periodic evacuation training. The area could be therefore characterised as a well prepared tsunami area. The inundation height of the Tohoku tsunami however, exceeded the design levels of the coastal barriers, and several structuers collapsed [Figure 1.1].

![Figure 1.1: Examples of structural failures of tsunami barriers after the Tohoku tsunami [Ewing, 2011];(a) Structural failure of a seawall in Noda;(b) Foundation failure of the seawall in Kojirahama after the 2011 Tsunami](image)

After the Tohoku tsunami of 2011, the government of Japan announced a new coastal defence system. Hundreds of sea walls and breakers are (planned to be) (re)built in the Tohoku district. There is much debate about the tsunami walls. Some critics say that the new coastal defence system could have a very negative impact on marine wildlife, while others have raised objections about the wall blocking the view out to sea [Stone, 2015]. So therefore it is suggested to consider alternative tsunami barrier.
One of the latest concepts in the field of tsunami barriers, is the so-called 'Tsunami Catcher'. This is a flexible-membrane barrier which is self-deploying when a tsunami occurs, due to the buoyancy of a floating element. This floater is connected to the membrane and cables which hold the floater on its position [Figure 1.2]. During normal conditions, the membrane is folded in a trench-type structure and does not form a visual obstruction of the coastline [Hofland et al., 2015]. Therefore the 'Tsunami Catcher' could be a good alternative for the sea walls.

![Figure 1.2: Concept design of the tsunami barrier [Hofland et al., 2015]](image1)

The concept is studied experimentally at Deltares [2015] and it is shown that the deploying principle of the barrier works [Figure 1.3]. Even during extreme tsunami conditions, the barrier remains erected. This makes the 'Tsunami Catcher' a potential tsunami barrier for communities which should remain connected to the sea but protected from devastating tsunamis [Hofland et al., 2015].

![Figure 1.3: Principle of the tsunami barrier, shown by video stills from the experiments [Hofland et al., 2015]](image2)

### 1.2 Problem Analysis

The uncertainties in the design of the flexible membrane barrier are considerable. The first tests indicated that the tsunami barrier would deploy under tsunami attack and further research is suggested to improve the understanding of the possibilities of the barrier concept.

The most relevant aspects influencing the design are the following: the behaviour of the incoming tsunami wave, the dimensions of the tsunami barrier including the foundation and connections between the elements, the integration of the tsunami barrier in the surroundings and the behaviour of the tsunami barrier.
1.3 Aim

The flexible-membrane tsunami barrier is an unconventional design in the field of hydraulic structures. An unconventionality is for example the inflation process of the flexible membrane barrier. The complex behaviour of the structure causes uncertainties in the design. Therefore a feasibility study must be performed to come up with a preliminary design. The feasibility study will be applied to a test case, namely ’Kamakura Beach’, nearby Tokyo in Japan. The aim of the research is to obtain a:

*A feasibility assessment of the flexible-membrane barrier ’Tsunami Catcher’ for the case study of Kamakura*

The feasibility assessment is divided in five research questions:

1. **What are the requirements and boundary conditions for the flexible tsunami barrier applied to the test case of Kamakura Beach?**
2. **How does the incoming tsunami develop at Kamakura?**
3. **What are the general characteristics of the ’Tsunami Catcher’?**
4. **What is the optimum design of the flexible tsunami barrier for Kamakura?**
5. **How is the flexible tsunami barrier integrated in the surroundings at Kamakura?**

1.4 Approach and Methodology

A study concerning the relevant background information [Chapter 2] is executed to determine the boundary conditions and the requirement for the tsunami barrier. Different wave theories are also analysed for the tsunami wave analysis.

A one dimensional, SWASH (Simulating WAves till SHore) [The SWASH Team (2010-2015), 2015], numerical model is prepared to determine the wave conditions for the case study of Kamakura [Chapter 3]. The reference case of Sendai is used to compose the numerical model. The model is validated by means of the available data of the Tohoku tsunami of 2011 and analytical models.

The elements, phases and load conditions of the tsunami barrier are analysed to determine the general characteristics of the ’Tsunami Catcher’ [Chapter 4].

In the Global Design [Chapter 5], the relation between the critical elements of the barrier are investigated. Goal of this analysis is to determine the possible configurations of these elements for the case study of Kamakura.

The outcome of the Global Design will be further elaborated in the Site Specific Design [Chapter 6] for Kamakura. The dimensions of the important elements are determined and checked for different failure-mechanisms. Subsequently, the barrier will be integrated in the surroundings of Kamakura.
Theoretical Background Information

The objective of this Chapter is to determine the requirements, assumptions, boundary conditions and the applied wave theories for the case study of Kamakura. The full analysis of the background information is given in Appendix A.

2.1 Summation of Relevant Analysed Subjects

Tsunami General Characteristics

The general characteristics of the tsunami wave are analysed in Section A.1. There are different wave theories that can serve as input for the numerical SWASH model [The SWASH Team (2010-2015), 2015]. Two typical wave shapes are often used to describe a tsunami, the solitary wave and the N-wave theory. It is therefore decided to conduct two SWASH models and a comparison is made which theory is best applicable.

A tsunami wave undergoes similar wave transformations as an ordinary wind wave in shallow water. Five different wave transformations can be distinguished: The generation and propagation in relatively deep water from the source region to a coastal region, the enhancement and deformation in shoaling water, up to breaking of the wave, dissipation of energy and finally the run-up onto land [Battjes and Labeur, 2014]. Each stage is analysed and if necessary, the proper analytical model is proposed.

In Figure 2.1 the wave definitions are clarified, which are used in this report. The initial wave height \( H_0 \) is the wave in deep water. The incoming wave height \( H \), measured at the shore, just before the dike without the effects of reflection from the elevated coastal protection wall and the maximum inundation height \( H_i \), is the measured wave height just after the dike [Okumura, 2016].

![Figure 2.1: Explanation of the wave terms [Okumura, 2016]](image)
Wave Loads

Different wave loads for tsunami waves are analysed in Section A.2. These are approached by existing theories for wave loading on a vertical wall. The analysed loads are wave loading, wave overflowing and the bore impact which can be approximated by the linear wave theory, the Miche-Rundgren theory and Ramsden or FEMA theory, respectively.

Tsunami Flood Protection

After the Tohoku tsunami of 2011, the flood risk management was improved, by implementing a Multi Layer Safety (MLS) System and adapting a new tsunami categorisation [Section A.3].

The flood risk management concept consists out of three safety layers. This system aims to optimise the safety of the whole system with a combination of 3 layers. The three layers consist of a prevention layer, a spatial solution layer and a emergency management layer. The prevention layer consists out of hard structures which must prevent inundation [Okumura, 2016]. It is assumed that the 'Tsunami Catcher' is an element of a MLS system, located in the first layer.

For the design of the new flood protection structures, two different tsunamis levels have been identified [Shibayama et al., 2013]. These levels are chosen based on political decisions, with some insight from coastal engineers from governmental institutes. A Level 1 tsunami event has a return period of several decades to 100+ years and must be able to retain this type of tsunami. The other category is the Level 2 tsunami. This event has a return period up to 1000 years and the coastal barrier must be able to withstand the tsunami, although overflowing is allowed [Okumura, 2016]

Case Study: Kamakura

The case study is applied to the city of Kamakura, a coastal city located in the Sagami Bay (population=173,500), approximately 50 kilometers South-East of Tokyo [Figure 2.2a]. The city is known for its historical monuments and beaches, and has therefore a large cultural and touristic value. A flexible-membrane tsunami barrier could be a proper tsunami counter measure, without reducing the touristic value. In Section A.5 different subjects of Kamakura are studied to determine the boundary conditions.

Figure 2.2: The (a)location [Google Maps, 2016] and (b)bathymetry of Kamakura [General Bathymetric Chart of the Ocean, 2016]
In the past centuries, several tsunamis occurred, which affected the city of Kamakura. In the Msc. Thesis of N. Okumura [2016] the available historical data is used to determine the return period for the Level 1- and Level 2 tsunami for the case study of Kamakura. These return periods are approximated with the method of regression. It was concluded that the Reverse-Weibull type of Generalised Extreme Value distribution line had a good statistical fit with the calculated, approximated return periods. The method can cause inaccuracies for extrapolating data however, the accuracy still holds within a range not far from the plotted values [Okumura, 2016].

From the method of regression analysis it is concluded that the maximum wave height of the incoming tsunami wave for a Level 1 event is assumed to be 8 meters and the wave height for a Level 2 tsunami event is assumed to be 14 meter [Okumura, 2016].

In the project location analysis, the bathymetry and the environment of Kamakura are evaluated. It is concluded that Kamakura can be seen as a small bay with shallow waters [5-10 meters] with a mild slope [1:100] [Figure 2.2b]. The location can be characterised as a narrow beach with an average width of 60 meters [Figure 2.3]. The beach is enclosed by a small dike [3 meters] whereupon a main road is located.

The soil conditions are based on a single Standard Penetration Test [SPT] [Kanagawa Prefecture, 2016]. From this test, it is assumed that the soil is homogeneous and consists out of fine sand.

Reference Projects

One of the first hydraulic membrane barriers was developed by J.K. Vrijling in the early 80's, the so called- Spinnaker Barrier, which functions as a storm surge barrier. This barrier was one of the first, where the vertical stability was ensured by the use of floating bodies [Figure 2.4a]. Unlike the 'Tsunami Catcher', the Spinnaker Barrier is not automatically deploying. When storm conditions are predicted, the fabric is pulled horizontally over the entire width of the water [Regeling, 1989].

The concept of the Spinnaker Barrier was the inspiration of the Kite Barrier. This was an open, moveable, flood water barrier, which can be seen as a parachute opened horizontal in the water way [Figure 2.4b]. In the initial design, the vertical stability was guaranteed by the use of floating bodies. It was concluded that a barrier where only floating bodies are used for the vertical load distribution, seems to be unpractical due to the needed size of these floating bodies [van der Ziel, 2009].
The first realised, inflatable flood barrier was constructed in 2012 in Rampsol. Despite that this barrier is not a tsunami barrier, this structure is an interesting reference project. Similar to the 'Tsunami Catcher', the Rampsol Barrier is a membrane-type structure. The membrane is a fibre-reinforced fabric, connected to a foundation and is manually inflated by creating an over-pressure in an enclosed space, by means of air and water [Figure 2.4c].

![Figure 2.4: Principle of the Spinnaker barrier(a), Kite Barrier(b) and the Rampsol Barrier(c)](image)

2.2 Assumptions, Requirements, Boundary Conditions and Wave Theories

From the analysis of the relevant background information, the following conclusions can be made. These are subdivided in requirements-, assumptions- and boundary conditions for the case study of Kamakura and the applied wave theories.
Requirements

1. The barrier is a part of a Multi-Layer Safety system, functioning as primary barrier in Layer 1 and must be able to:
   (a) Fully retain a Level 1 tsunami (return period = 100 years);
   (b) Withstand a Level 2 Tsunami (return period = 1000 years), where overflow is allowed and scour does not undermine the stability of the barrier.

2. The dimensions of the barrier must be minimised in order to integrate the barrier in the surroundings.

Assumptions

1. The density of the tsunami water \( \rho_w \) is assumed to be equal to 1100 kg/m\(^3\) [FEMA, 2012];
2. The possible storm surge of typhoons is inferior to the incoming design tsunami wave.
3. The height of the undisturbed incoming Level 1 Tsunami wave \( H_{Level\_1} \) is assumed to be equal to 8 meters [Okumura, 2016] [Section A.5.1];
4. The height of the undisturbed incoming Level 2 Tsunami wave \( H_{Level\_2} \) is assumed to be equal to 14 meters [Okumura, 2016] [Section A.5.1];
5. The soil is assumed to be homogeneous consisting out of fine sand with a volumetric weight of 18 kN/m\(^2\) and a wet volumetric weight of 20 kN/m, with no cohesion and an angle of internal friction of 30\(^\circ\) [Institute of Kanagawa Prefecture (2016)].

Boundary Conditions Kamakura

1. The schematised design cross section of the project location that is used to make the design is given in Figure 2.3;
2. The bathymetry of Kamakura is given in Figure 2.2b;
3. The maximum high water level is + 90 cm and the minimum low water is -100 cm below mean water level [Japan Oceanographic Data Center];

Wave Theories

1. The modelled, design tsunami wave [Chapter 3] will be approximated by a Solitary Wave Theory and a N-Wave Theory;
2. The shoaling effect of the design tsunami wave will be validated by the Green’s Law;
3. The type of breaking for the design tsunami wave will be validated by the slope parameter \([S_0]\), proposed by S.T. Grilli et al [1997];
4. Dissipation of the design tsunami wave will be validated by the dissipation model proposed by Battjes [1986];
5. The run-up of the design, tsunami wave will be validated by observational data;
6. The linear wave theory for wave loading on a vertical wall, will be applied to approximate the pressure distribution;
7. The Miche-Rundgren theory [1958] for non-breaking wave forces for vertical walls of low height will be applied to estimate the pressure distribution due to overflowing;
8. The bore impact will be approximated by the theory of Ramsden [1990] and the theory of FEMA [2012].
Tsunami Wave Analysis for Kamakura

This chapter is made in collaboration with N. Okumura

The design tsunami wave of Kamakura is approximated by an one dimensional numerical SWASH model. The first step is to evaluate the solitary wave- and the N-wave theory [Section 3.1]. The different transformations of the incoming tsunami wave are briefly clarified and if possible, analytical models are proposed [Section 3.2]. The numerical SWASH model is set-up based on the reference case study of Sendai, which was affected by the Tohoku tsunami of 2011. A significant amount of data was achieved after the Tohoku tsunami which is used to derive the initial time-series for the N-wave and the solitary wave [Section 3.3]. The SWASH model is validated by the obtained data and analytical models for both wave theories [Section 3.4]. Finally, the tsunami conditions for Kamakura are defined with the validated SWASH model [Section 3.5]. For the full derivation of the tsunami wave is referred to Appendix B and more information concerning the SWASH calculations is referred to Appendix C.

3.1 Evaluation of the Wave Theories

Tsunami waves can be approximated by different wave theories. For the numerical SWASH model an appropriate wave theory must be selected. In Section A.1.1 it is suggested to use the solitary wave theory and the N-wave theory.

Solitary waves [Figure 3.1a] are often used to model some of the important features of a tsunami approaching the shore. The solitary wave can be seen as an amount of water riding completely above the mean sea level. They have the advantage, although nonlinear, they can be described with just two parameters namely; the initial wave height \( H_0 \) and the initial water depth \( h_0 \) and propagate with constant form in constant depth [Holthuijsen, 2007]. The link to the geophysical scales of the earthquake is not well established and therefore conclusions drawn, based on the solitary wave model, should be made with great care [Madsen et al., 2008].

It is often observed that before the wave crest of the incoming tsunami wave arrives, the water level retreats. This trough in front of the wave can be modelled with a so-called N-wave [Figure 3.1b]. The physical characteristics of the N-wave fits better with the geophysical characteristics of the earthquake. It was found that this type of wave displayed very interesting and counter-intuitive behaviour [Tadepalli and Synolakis, 1994]. The disadvantage of this wave theory, is the limited amount of available studies.

\[ a) \textit{Solitary wave} \\
\text{b) } \textit{N - waves} \]

Figure 3.1: A Solitary wave and a N-wave [Yamao et al., 2015]
3.2 Evaluation of the Tsunami Wave Transformations

Tsunamis have similar wave transformations as ordinary wind waves in shallow waves. A tsunami wave can be characterised by the wavelength \( L_w \), a wave period \( \omega \) and a deep-water wave height \( H_0 \). There are five stages distinguished: The generation and propagation in relatively deep water from the source region to a coastal region, the enhancement and deformation in shoaling water, up to breaking of the wave, dissipation of energy and finally the run-up onto land [Figure 3.2][Battjes and Labeur, 2014]. Each wave transformation is briefly clarified and the suggested analytical wave model is formulated. For more information regarding the analytical wave models is referred to Appendix A.1.2

Figure 3.2: Overview of the different wave transformation stages

About 90 percent of earthquakes occur in subduction zones and these areas are the prime source for tsunamis. The tsunamis are caused by rupture along active fault lines, where two sections of the Earth’s crust are moving opposite each other. The greater the vertical displacement, the greater the amplitude of the deep water wave height [Bryant, 2008].

Shoaling is the effect of increasing wave height due to decreasing water depth. A decreasing water depth, yields to a decreasing wave speed. Wave energy will be concentrated, causing the wave to steepen and rise to many meters in height. The relation between the wave height and the water depth \( h \) is known as the Green’s Law [Equation 3.1] [Bosboom and Stive, 2015]. Where subscript 1 stands for location 1 and subscript 2 for location 2:

\[
\frac{H_2}{H_1} = \left( \frac{h_1}{h_2} \right)^{0.25}
\]  

(3.1)

The wave shoals until the wave becomes too steep or too high and starts to break. The type of breaking can be estimated by the breaking parameter, proposed in the report of S.T. Grilli et al [1997], and depends on the slope of the shore and the initial wave height.

The tsunami wave could eventually break into multiple solitons. Short waves split from the tsunami rest due to non-linearity and dispersion. Eventually an incoming bore is formed.

A tsunami wave does not lose all of their energy due to the breaking process, which can lead to high run-up heights and inundation depths. The dissipation of the tsunami wave can be estimated by the energy dissipation model for breaking solitary waves, derived in the report of Battjes [1986].

Eventually, the tsunami arrives at the shore and runs up to the land. Tsunamis are known for their significant run-up heights [Bryant, 2008], which is the distance the wave travels inland.
3.3 Reference Case Study of Sendai

The city of Sendai is chosen as a reference case study to validate the SWASH models, because of the significant amount of available data from the Tohoku tsunami of 2011. The bathymetric characteristics of Sendai are similar to the case study of Kamakura [Figure 3.3]. Both locations have a bay type of coast with a mild slope [1 : 100] [General Bathymetric Chart of the Ocean, 2016][USGS, 2016].

The generation of the Tohoku tsunami of 2011 was recorded by the ocean bottom pressure and GPS wave gauges, deployed in and around Japan [Saito et al., 2011]. Several buoy stations have measured the incoming tsunami wave [Kawai et al., 2013] and a survey along the coast was executed to estimate the inundation height [Mori et al., 2011]. There is also an amount of video and photo material available.

The assumed tsunami path for the numerical calculations does not go through the buoy stations [Figure 3.3]. The relative depth of the assumed tsunami path is within the same depth contours as the buoy stations. So the measurements of the buoy stations can be used to validate the model. The time series of buoy GB801 [Figure 3.4], located 60 kilometres off the coast of Sendai, is used to determine the length and period of the initial tsunami wave. Furthermore, the tsunami path is chosen such that it is not incorporated by the islands and shallow parts of the North.

![Figure 3.3: Plan view of chosen 1D cross-section of bathymetry, along with the PARI buoy stations GB801 and WG205 [General Bathymetric Chart of the Ocean, 2016]](image1)

![Figure 3.4: The observed time series for the tsunami wave heights for the 2011 event at buoy GB801 and WG205, 1 unit equals 1 meter wave height [Kawai et al., 2013]](image2)
3.4 Validation

The two numerical SWASH models are validated with the observational data of the Tohoku tsunami [Section B.3.1] and the proposed analytical models [Section A.1.2]. The validation is broken down for each wave transformation. In this Section the conclusion of the validation for each wave transformation is given. The full analysis with corresponding graphs, is given in Section B.3.3.

The shoaling of the tsunami wave is validated with the Green’s Law and with the measurements of buoy GB801 and WG205 [Figure 3.4]. It is shown that the amount of shoaling for both the N-wave as the soliton wave is less than that of the analytical model. The modelled maximum wave height of the N-wave however, approximates the measured wave height of the buoy station GB801.

The type of breaker is estimated with the theory proposed by S.T. Grilli. This result is compared with the available video and photo material. It is concluded that the type of breaker corresponds with the observed spilling breaker. The modelled tsunami wave breaks into multiple solitons. This phenomenon, known as ‘tsunami soliton fission’ is also observed during the Nihonkai-Chubu tsunami in 1983. Short waves split from the tsunami crest due to non-linearity and dispersion [Matsuyama et al., 2007]. These short waves will eventually disappear and the incoming tsunami will develop into a bore. The location of the wave breaking is however, not clarified.

A tsunami wave retains a significant amount of energy when it reaches the coastline. In fact, when the soliton fission occurs, an increase of the wave height can be noticed, thereafter the wave starts dissipating energy. This process can be approximated by the model of Battjes [1986]. It is concluded that the dissipation of the N-wave has approximately the same slope as the dissipation of the Battjes model between the point of breaking and the arrival at the coast. The total amount of dissipated energy of the N-wave does not coincide with the Battjes theory.

The final run-up of the tsunami is validated by the measurements obtained from the report of N. Mori and Yanagisawa [2011]. The results from the SWASH models is compared with the envelope of the run-up measurements. It is concluded that the run-up of the N-wave approximates with the measured run-up. The solitary wave however, overestimated the amount of run-up.

From the SWASH model results, it can be concluded that the N-wave has the best fit with the data of the Tohoku tsunami of 2011 and the analytical models [Table 3.1]. For the present aim of this thesis, the N-wave is taken as it captures the essential features as observed in 2011. Therefore, the N-wave will be used to model the tsunami for the case study of Kamakura.

<table>
<thead>
<tr>
<th>Table 3.1: Comparison of the SWASH models results with the measured data</th>
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<tbody>
<tr>
<td>Data [m]</td>
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<tr>
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<tr>
<td>Initial wave height</td>
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<tr>
<td>Wave height just off-shore</td>
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<tr>
<td>Maximum inundation depth</td>
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<tr>
<td>Run-up</td>
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</tbody>
</table>

The N-wave has some drawback which must be mentioned. First of all, the mathematical background of the N-wave is a simple model to capture the essential characteristics. The solitary wave has explicit input parameters, while the N-wave is manipulated until the initial conditions are met. The changes in steepness and amplitude of an N-wave could significantly change the results which are seen on the shore.

Several improvements could be made to improve the model. First of all, a 2- or 3-dimensional model for real sea-bed changes for different tsunamis can be simulated. New tsunami models could be used, such as to include the direct geophysical impact to the water [Smith et al., 2016], allowing the initial tsunami wave to have its unique wave form. Secondly, different sensitivity analyses can be conducted to check the influence of different parameters.
3.5 Results for Kamakura

The N-wave model is applied to the Kamakura case study with the same boundary conditions, used for the reference case of Sendai. It is assumed that the incoming tsunami is an N-wave with a leading crest. Figure 3.5 shows the bathymetry and the assumed tsunami path for the case of Kamakura. The worst-case scenario occurs when the incoming tsunami wave directly hits the beach of Kamakura, in order to simulate the dynamic impact.

![Bathymetry of Kamakura with the transact of the modelled wave](General Bathymetric Chart of the Ocean, 2016)

It is assumed that the height of the incoming Level 1 tsunami wave is approximately 8 meters [Okumura, 2016]. In the SWASH model, the initial wave conditions are varied until the undisturbed incoming wave height coincides with the height of a Level 1 tsunami wave. Figure 3.6 shows the wave profile [Blue Line] and velocity profile [Red Line] of the incoming tsunami wave, at the moment the bore strikes the barrier \(x = 0\).

![Profiles of the modelled, incoming bore of the Level 1 tsunami for Kamakura, visualised for different time steps](image)
The height of the incoming Level 2 tsunami wave is approximately 14 meters [Okumura, 2016]. Also the wave- and velocity profile of the Level 2 tsunami bore can be derived from the SWASH model [Figure 3.7].

Figure 3.7: Profiles of the modelled, incoming bore of the Level 2 tsunami for Kamakura, visualised for different time steps

The maximum velocity of the incoming Level 1 \( u_{\text{level,1}} = 17 \text{ m/s} \) and Level 2 \( u_{\text{level,2}} = 25 \text{ m/s} \) tsunami is significantly greater than the bore velocities used for the experiments of B. Hofland et all [2015]. It was concluded that for a 10 meter per second approach flow, initially some water was spilled over the barrier. It is therefore recommended to test the inflation process of the barrier for an incoming tsunami with a velocity of 25 meter per second.

The barrier must be able to withstand the tsunami, where overflowing of the wave is allowed. The amount of overflow can be derived from the SWASH model. The barrier is modelled as a rigid, dike-type structure which resembles with an already inflated membrane tsunami barrier [Figure 3.8a]. The initial overflow could be therefore somewhat conservative, but is a proper estimation for the first calculations.

The overflow is measured at the top of the dike [Dashed Line:Figure 3.8a]. The maximum measured flow depth on top of the barrier is approximately 9 meters. The first ten seconds however, an additional overflow was measured, with a maximum overflow of 11 meters. This additional overflow is modelled as a secondary wave [Section 6.1].

Figure 3.8: Overflow of the Level 2 tsunami for the case study of Kamakura (a) Point of measuring; (b) The measured flow depth on top of the barrier
Analysis of the Tsunami Barrier

In this Chapter, the flexible-membrane barrier is further analysed. First, the different elements of the barrier are studied [Section 4.1]. The next step is to evaluate the different phases of the structure [Section 4.2]. Also the different type of load conditions are considered [Section 4.3]. Lastly, the load derivation of the structure is studied [Section 4.4].

4.1 Analysis of the Elements

In this section the different elements of the barrier [Figure 4.1] are analysed. The function, material and design, and the requirements are discussed.

1. Bottom Recess Foundation

The function of the bottom recess foundation is to transfer the horizontal and vertical forces acting on the bottom recess, to the surrounding soil. In general, there are two foundation options; a shallow, steel foundation or a pile foundation. It is assumed that the bottom recess foundation consists of a pile foundation due to its favourable behaviour during seismic activity [Korff and Meijers, 2015] [Section 4.3]. The type and dimensions of the piles depends on the load- and soil conditions. It is assumed that the pile foundation will have battered piles to withstand the high horizontal loads.

2. Bottom Recess

The bottom recess will function as storage for the membrane and the floater during normal conditions. The bottom recess is a trench-type structure with a cavity. It is assumed that the bottom structure will be made out of concrete and that the slope of the trench is equal to 1:1 [Figure 4.1]. Furthermore, it is assumed that the bottom recess will be integrated in the soil and that the top of the bottom recess is located at ground level. The bottom recess must remain stable during the inflation- and retaining phase. The structure must also provide space to for the membrane to fully deflate. During normal conditions, the structure must be able to protect the floater and membrane for weathering.
3. Membrane

The membrane is the main retaining component of the barrier. The equilibrium of the membrane is defined by its shape. The membrane must be able to transfer high tension forces. It is assumed that the membrane will be made out of the Dyneema CF10 cloth. Dyneema is a polyethylene fibre with a very high tensile strength of 3400 MPa and a mass density comparable to water of \(975 \text{ kg/m}^3\). It has a large tear resistance and a very high axial modulus of about 120 GPa. The fracture strain is in the order 3 percent [Hofland et al., 2015]. The membrane must be able to inflate, when a tsunami occurs and retain the incoming wave. The lengthen of the membrane must be taken in to considering.

4. Floater

The main function of the floater is to keep the membrane up by its uplift force. The floater provide this uplift force by the buoyancy of the element. The shape of the floater is arbitrary, as long as its dimensions provide enough buoyancy to keep the membrane up. For the first design calculations it is assumed that the floater is a pipe with a certain diameter and thickness, made out of steel [S235]. Besides providing the necessary uplift force, the floater must also be able to withstand the stresses imposed by the membrane and the cables and potential wave loads.

5. Cables

The cables keep the floater into position. These cables transfer the membrane tension force to the offshore foundation. The cables are positioned in regular intervals along the floater. It is required to minimise the elongation of the cables, therefore it is recommended to choose a cable with a low fracture strain. For the first design calculations, it is assumed that the cables are made out of the Dyneema yarns. This fabric can withstand high tension loads and has a low fracture strain, in order of 2-3 percent [Royal Lankhorst Euronete, 2016]. Besides, the cable is floating which has a beneficial effect on the floater [Section E.1]. It is desirable to maximise the cable interval length to preserve the recreational value of the beach. During normal conditions, the cables must be stored in a gutter-type of structure which minimise the weathering effects. The length of the cables must be taken in to considering.

6. Offshore Foundation

The offshore foundation transfers the cable tension force to the surrounding soil. There are three main possibilities for the shallow offshore foundation; Dead weight, a pile foundation or a drag embedded anchor. Although multiple solutions can be applied, for the first design calculations, it is assumed that the drag embedded anchor will be the offshore foundation. The main requirement for the offshore design is that the anchor remains stable during a tsunami event. For the drag embedded anchor this also implies that the anchor will not drag after installation. The drag of the anchor can be minimised by properly pre-tension the anchor on site [Vryhof Anchor, 2015].

7. Secondary Cables

The barrier must be able to withstand a Level 2 tsunami with overflow of water. Te be able to allow this overflow, the floater must be prevented its uplift to raise too much. Otherwise there is a possibility that the floater pulls out the membrane. It is therefore assumed that there are secondary cables which keep the floater in position. The length of this cable is such that the floater stays at a retaining height of a Level 1 tsunami. Similar to the cable design, these secondary cables must have low fracture strain, in order of 2 percent.
4.2 Analysis of the Phases of the Barrier

For the flexible membrane barrier there are four phases distinguished: the inactive phase, the inflation, the retaining and the deflation phase. The latter phase is not included in this analysis.

Inactive phase of the barrier

During normal conditions, the structure is inactive and the elements are stored, in order to preserve the recreational value of the shore. The membrane and floater are retained in the bottom recess and the cables must be stored in a gutter type of structure. These measures provide protection for the elements against the environmental aspects, such as rain, sun and abrasion.

The Inflation of the Barrier

Prior to the inflation of the barrier, the incoming bore will create an impact load on the floater and the bottom recess structure [Figure 4.2b]. It must be noted that for the experimental set-up, the amplitude and the slope of the incoming bore profile is rather small [Figure 4.2a] and therefore the impact force is minimal. The modelled incoming bore for Kamakura however, has a more steep profile and a greater velocity [Figure 3.7]. So it can be concluded that the bore impact for the case study of Kamakura can be significant.

![Figure 4.2: Snap shots of (a) the incoming bore profile and (b) the tsunami bore impact of the experimental set-up][Hofland et al., 2015]

After the bore impact, the floater will rise with the reflected bore and the membrane will be 'filled'. The inflation of the barrier can be compared with the principle of a parachute, where the cables resembles the parachute lines and the membrane resembles with the canopy. First the cables will be activated [Figure 4.3a], preventing that the membrane and the floater will flush away. The floater will further rise with the increasing water level, until the membrane will be fully filled. [Figure 4.3b]. This process occurs in matter of seconds, and these dynamic loads can cause peak stresses in both the membrane as the cable.

![Figure 4.3: Snap shots of the inflation process of the experimental set-up, characterised by the dynamic (a) cable forces and (b) membrane force][Hofland et al., 2015]
Larger tsunamis, with a 33 meter reflected water depth and higher bore velocities, are also investigated in the paper of B. Hofland et al. [2015]. It was concluded that the barrier remained erected, although overflow evidently occurred [Figure 4.4]. This resembles with the situation of a Level 2 tsunami.

Figure 4.4: Snap shots of the (a) inflation and (b) retaining phase of a Level 2 tsunami of the experimental set-up [Hofland et al., 2015].

The retaining phase of the barrier

The retaining phase occurs when the incoming tsunami wave is reflected and that there is only a raised water level left. For the Level 1 tsunami wave this implies that there are no dynamic loads, and therefore this condition can be seen as a static situation [Figure 4.5a]. The retaining height of the barrier is equal to the reflected tsunami wave, which is equal to twice times the incoming bore [Equation 4.3].

During the retaining phase of Level 2 tsunami, the barrier retains a water level, equal to the retaining height of a Level 1 tsunami and the wave will overflow the barrier [Figure 4.5b]. Based on the experiments of B. Hofland et all (2015), it is assumed that the overflow of water can occur, without jeopardise the stability of the membrane. The floater is kept at the retaining water level by secondary cables.

Figure 4.5: Retaining phase of (a) Level 1 and (b) Level 2 tsunami [Hofland et al., 2015].
4.3 Analysis of the Load Conditions

In this section, the load conditions [Figure 4.6] are further analysed. For the full derivation is referred to Appendix D.

![Figure 4.6: Overview of the load conditions](image)

**Seismic Activity**

The majority of the tsunamis are generated by earthquakes [Bryant, 2008], which can cause dynamics loads onto the foundations. It is required that these foundations remain stable during the initial earthquake.

The behaviour of the foundation and soil during seismic activity is complex. The foundation is dynamically loaded during an earthquake and there is the probability that the soil loses its bearing capacity. This can cause weakening-, large deformations- or total softening of the soil. The bottom recess will have a pile foundation, because a pile foundation is less sensitive to earthquake loading than a shallow steel foundation [Korff and Meijers, 2015]. The effect of seismic loading on the pile foundation is not included in the first designs. But it is recommended to include an earthquake analysis with a finite element model such as Plaxis.

Also the drag embedded anchor can be dynamically loaded by the earthquake. Deformations in the soil can create additional drag of the anchor which can be unfavourable for the stability of the barrier. The effect of seismic loading on the offshore foundation is not included in the first designs. It is recommended to include an earthquake analysis with a finite element software such as Plaxis.
Inflation Phase

The inflation phase is analysed in Section 4.2 and two main loads are distinguished: the initial bore impact and the dynamic loads during the inflation. The initial bore impact will cause an impact load onto the floater and onto the bottom recess. This load can be calculated by two different methods.

In the report of Ramsden [1990] it was concluded that for large bores with a height greater than 8 meter, the impact load \( F_{s, \text{Ramsden}} \) on a vertical wall can be estimated by 7.5 times the hydrostatic force of the incoming bore height \( H_b \).

\[
F_{s, \text{Ramsden}} = 7.5 \cdot \frac{1}{2} \cdot g \cdot \rho_w \cdot H_b^2
\]  
(4.1)

The theory of FEMA [2012], states that the impact load depends on the maximum momentum flux \( (H_b u^2)_{\text{max}} \) of the incoming bore profile. Where \( H_b \) is the height of the bore and \( u \) the x-velocity of the bore.

\[
F_{s, \text{FEMA}} = 1.5 \left( \frac{1}{2} \cdot \rho_w \cdot C_d \cdot (H_b u^2)_{\text{max}} \right)
\]  
(4.2)

The inflation of the membrane can be compared with the inflation of a parachute. In both cases, peak stresses are distinguished during the inflation. In Figure 4.7a the force histogram of the opening of a parachute is given. Two peak stresses can be noticed: the snatch force in the cables and the opening shock in the membrane [Doherr, 2005]. These forces are also seen in the inflation of the barrier, Figure 4.3a and Figure 4.3b respectively. Due to the complexity of these phenomena, amplitude factors are used to include these peak stresses [Figure 4.7b]. It is assumed that the snatch force is equal in amplitude to the opening shock force and both forces depend on the parachute amplitude factor \( \zeta_p \). The magnitude of this amplitude factor is further elaborated in Section G.

Retaining Phase

In Section 4.2 it is stated that the retaining phases can be seen as a static situation. And that the retaining height of the barrier \( H_{\text{retaining}} \) is equal to the reflected height of the Level 1 tsunami \( H_{\text{Level 1}} \), so:

\[
H_{\text{retaining}} = 2 \cdot H_{\text{Level 1}}
\]  
(4.3)

The static load for a Level 1 tsunami is equal to the hydrostatic water pressure distribution [Figure 4.8a]. Where the maximum pressure \( p_{1, \text{level 1}} \) is equal to:

\[
p_{1, \text{level 1}} = \rho_w \cdot g \cdot H_{\text{retaining}}
\]  
(4.4)
The pressure distribution in the retaining phase of a Level 2 tsunami can be derived from the theory proposed by Miche-Rundgren [1958] [Section A.2.2] [Figure 4.8b]. During a Level 2 tsunami overflowing is allowed, and the floater is kept at the retaining height by the secondary cables. The amount of overflowing $H_{\text{overflow}}$ depends on the height of the incoming tsunami wave. The maximum pressure $p_{1,\text{level2}}$ and the pressure distribution at the floater $p_{2,\text{level2}}$ can also be calculated by the hydrostatic force:

\[
\begin{align*}
    p_{1,\text{level2}} &= \rho_w \cdot g \cdot H_{\text{overflow}} \\
    p_{2,\text{level2}} &= \rho_w \cdot g \cdot (H_{\text{overflow}} - H_{\text{retaining}})
\end{align*}
\]

Figure 4.8: Water pressure distribution for (a) the retaining phase of a Level 1 tsunami, (b) the retaining phase of a level 2 tsunami and (c) due to secondary waves

### Accidental Loads

There are three accidental loads distinguished: Debris loads, secondary wave loading and vandalism and weathering. The latter force will not be calculated but must be incorporated in the integration of the barrier in the surroundings [Section 6.3]

The incoming tsunami wave can be seen as a train of waves. So there is a possibility, that after the incoming bore is reflected, the barrier is loaded by secondary waves that will overflow the structure. This will change the pressure distribution [Figure 4.8c] for a certain period and lead to increase of the tension forces.

Debris could cause a significant impact on the barrier. These debris can vary from small items of the beach till large structures such as containers or vessels. The large structures can have a significant impact force on the membrane or the floater and even lead to instability of the barrier. For the case study of Kamakura it can be assumed that the possibility for large debris is negligible due to the lack of shipping activity.

Debris could cause a significant impact on the barrier. These debris can vary from small items of the beach till large structures such as containers or vessels. The large structures can have a significant impact force on the membrane or the floater and even lead to instability of the barrier. For the case study of Kamakura it can be assumed that the possibility for large debris is negligible due to the lack of shipping activity.

So the type of debris loading, strongly depends on the conditions of the project location. Besides an impact load, these debris could cause a tear in the membrane. From the first tear-resistance tests of the Dyneema membrane from the report of B. van Rodijnen [2017], it can be concluded that the membrane retains approximately 75 percent of its strength. Therefore a safe material factor $\gamma_m$ of 1.5 is used in the design calculations.
4.4 Load Derivation of the Barrier

The load derivation of the barrier is analysed by the static situation in the retaining phase. The shape of the membrane depends on the pressure distribution of the retaining water level. The pressure \( p \) will be balanced by the curvature of the tensioned membrane \( T_m \) [Segment 1; Figure 4.9].

The stability of the floater [Segment 3; Figure 4.9] is affected by the shape of the membrane, the cable configuration and the weight of the floater \( F_{g,\text{floater}} \). All these components have a downward vertical component which must be counteracted by the uplift force \( F_h \) due to the buoyancy of the floater [Segment 5; Figure 4.9].

The membrane tension load is transferred to the bottom recess and to the pile foundation. Besides this horizontal force, the pile foundation must also be able to withstand the vertical forces due to the water pressure and the weight of the bottom recess structure [Segment 4; Figure 4.9].

The membrane tension force will be transferred through the cables \( T_C \) [Segment 3; Figure 4.9] to the drag embedded anchor [Segment 5; Figure 4.9]. The cable tension force must be smaller than the ultimate holding capacity of the anchor, otherwise the anchor is pulled out of the soil. The holding capacity of the drag embedded anchor is determined by the weight of the anchor \( T_{A,W} \), the weight of the soil in the failure wedge of the soil \( T_{A,S} \), the friction of the soil \( T_{A,F} \) and the bearing capacity of the mooring line \( T_{A,B} \) [Vryhof Anchor, 2015].

The dynamic loads of the inflation phase and secondary waves can create peak stresses in the membrane and cables. These are incorporated in the design calculations by amplitude factors. These peak loads can affect the stability of the floater and there is a possibility that the floater will be pulled down and water will overflow the structure. It can be argued that this situation is not problematic as long as the floater will be stable after the dynamic load situation and the construction can derivative these peak loads to the foundations. For that reason the amplitude factors are not implemented in the force analysis of the floater.
5

Global Design

In the global design the relations between the cable configuration, membrane configuration and floater dimensions are analysed. Goal of this chapter is to determine an optimum configuration of these elements, based on simple cost functions.

This analysis is based on several simplifications. The optimum configuration is solely based on the retaining phase of the Level 1 tsunami wave and the dynamic loads are not incorporated in the calculations. The load derivation of the membrane tension force to the bottom recess and pile foundation, and the load derivation of the cable tension force [Segments 4 and 5 Figure 4.9] are omitted from this analysis and are assumed to be able to transfer the load to the soil. Also the connections between the elements are not further analysed and assumed to be able to transfer the loads between the elements.

The first step in the analysis, is to further study the force balance of the floater [Section 5.1]. In Section 5.2 the dependencies between the membrane configuration, cable configuration and floater dimensions are studied for a general design of the barrier. With the derived relations, a first global design can be presented for the case study of Kamakura [Section 5.3].

5.1 Force Balance of the Floater

In Section 4.4 the load derivation of the barrier is analysed, it is concluded that the floater is affected by the shape of the membrane, the cable configuration and the weight of the floater. For the first design calculations, it is assumed that the floater consists of a steel pipe with a certain diameter and thickness and can be seen as a stiff, rigid structure [Section 4.1].

In Figure 5.1a the three dimensional force situation for a segment of the floater is given. The membrane tension force $T_m$ can be seen as a distributed load over the entire length of the floater. The cable forces $T_c$ can be modelled as a point load, evenly distributed over the floater length, the cable interval distance $L_n$ [Figure 5.5]

Figure 5.1: Force analysis of the floater, (a) 3D-Schematization; (b) Cross-section of the floater
It is assumed that the cable forces are evenly distributed over the cable interval length because the floater can be seen as a stiff, rigid structure. So the force analysis of the floater can be analysed in a two dimensional cross section [Figure 5.1b]. From Figure 5.1b, the buoyancy requirement of the floater is derived [Equation 5.1], where \( T_{m,\text{ver}} \) is the vertical force component of the membrane, \( T_{qc,\text{ver}} \), the vertical, distributed, force component of the cable, \( F_{g,\text{floater}} \) the self-weight of the floater and \( F_b \) the uplift force due to the buoyancy of the floater:

\[
F_b > F_{g,\text{floater}} + T_{m,\text{ver}} + T_{qc,\text{ver}}
\] (5.1)

The uplift force and the self-weight of the floater depend on the dimensions of the floater. The vertical force components of the membrane and cable depend on the configuration of both elements. These dependencies will be further investigated in the next section.

The function of the cable is to transfer the membrane tension force to the offshore foundation, so the second requirement is:

\[
T_{m,\text{ver}} = T_{qc,\text{ver}}
\] (5.2)

The last requirement concerns the strength of the floater. The cables will induce moments in the floater, which generate bending stresses in the floating element. The floater must be able to withstand these stresses:

\[
f_y < \frac{M_{\text{max}}}{W_{\text{floater}}}
\] (5.3)

The yield stress \( f_y \) for steel with grade [S235] is equal to 235 \( \text{N/mm}^2 \). The maximum moment \( M_{\text{max}} \) in the floater depends on the cable interval length The section modulus \( W_{\text{floater}} \) depends on the floater dimensions.

The choice of a rigid, stiff floater could create secondary load effects, due to the moments in the floater. The element could deflect and can cause stress variation in the membrane. This increase in the membrane tension force can affect the vertical stability of the floater and therefore an amplitude factor \( \zeta_m \) is introduced. This value is estimated to be 2.0. In Section 6.1, the magnitude of this factor is further investigated.

### 5.2 Analysis of the Elements of the Barrier

**Membrane**

The vertical force component of the membrane influences the vertical stability of the floater [Equation 5.1]. The magnitude of this vertical force component depends on the membrane configuration. There are different membrane configuration possible. For the first calculations, it is assumed that a large membrane is required. As a result, it can be assumed that the membrane will lay horizontal on the bottom recess, where the attach point of the membrane is just next to the floater [Figure 5.2].

![Figure 5.2: Schematization of the membrane configuration](image-url)
The equilibrium shape of the membrane can be approximated with a second order numerical model [Appendix F.1], which is based on the proposed model of the weightless inextensible membrane of Parbery [1976].

For each membrane length \([L_m]\), a membrane tension force \([T_m]\) and curvature of the membrane at the floater \([\beta_m]\) [Figure 5.1b] can be computed. This latter parameter determines the decomposition of the membrane tension force in a horizontal \([T_{m,\text{hor}}]\) and vertical \([T_{v,\text{hor}}]\) force component. So the membrane length is in function of all three membrane force components at the floater, which is given in Figure 5.3.

Note that the dependency is made dimensionless, by dividing the membrane length by the retaining height \([H_{\text{retaining}}]\), which is equal to the reflected Level 1 tsunami wave. The membrane tension force is divided by the minimal tension force \([T_{\text{min}}]\). This membrane tension force corresponds with a configuration with a very large membrane length. The curvature of the membrane at the floater becomes zero, and the membrane will resemble with a half ellipse [Figure 5.4a]. The membrane tension load is then equal to half the resulting force of the hydrostatic water pressure [Figure 4.8], for a Level 1 tsunami this force is equal to:

\[
T_{\text{min}} = \frac{F_{p,\text{level 1}}}{2} = \frac{1}{2} \cdot H_{\text{retaining}} \cdot P_{t,\text{level 1}}
\]  

(5.4)

![Figure 5.3: Relation between the membrane length \([L_m]\) and the horizontal [Red], vertical [Yellow] and resulting membrane tension force [Blue], for the level 1 tsunami pressure distribution](image)

From Figure 5.3 it can be concluded that the variation of the total- and horizontal force component of the membrane is minimal, from a membrane length greater than two times the retaining water height. The vertical force component however, does have a great variation. It can be therefore concluded that the effect of the vertical force component can be significant in the force analysis of the floater.
Cable

The cable configuration can be described with two variables: the cable interval length \( L_n \) and the horizontal distance between the float and offshore foundation \( L_{c,\text{hor}} \) [Figure 5.5]. It is assumed that the cable forces are equally divided over the float, and so the analysed cable forces, are these distributed cable forces. The eventual cable force is determined by multiplying the distributed force by the cable interval length. Now, the cable forces are in function of the membrane length [Equation 5.2] and of the horizontal cable distance.

![Figure 5.5: Configuration of the cables](image)

The shape of the cable is approximated with a first order numerical model [Appendix F.2]. It is assumed that the cable is inextensible.

For each combination of a membrane length and a horizontal cable distance, the cable force \( T_{qc} \) and the curvature of the of the cable at the float \( \beta_c \) can be calculated. This latter parameter determines the decomposition of the cable force in a horizontal \( T_{qch,\text{hor}} \) and vertical \( T_{qch,\text{ver}} \) force component [Figure 5.1b].

The total- and horizontal cable force component are not normative for the cable configuration [Section E.3.2]. The relation between the vertical cable force component between the membrane length [x-axis] and the horizontal cable distance [y-axis] is given in the Figure 5.6:
A greater cable length results in a smaller vertical force component. This effect is however, not further beneficial after a length greater than twelve times the retaining water.

**Floater**

With the known dependencies of the membrane and the cable configuration, the floater dimensions can be calculated. The floater diameter and corresponding thickness can be calculated [Equation 5.1][Equation 5.3], for each combination of the membrane length and horizontal cable length, for a given cable interval length.

There are different combinations of the floater diameter and floater thickness possible, goal is to find the minimal required floater diameter. To achieve this an iteration process is presented in Appendix E.4.

The magnitude of the uplift force [Equation 5.1] is related to the underwater volume of the floater. If it is assumed that the floater will be entirely lay below the water surface, every small deviation in the forces can result in instability of the floater. For that reason, the uplift force is based on 90 percent of the underwater volume of the floater.
5.3 Global Design for Kamakura

An optimum configuration of the membrane-, cable- and floater dimensions is determined for the case study of Kamakura, using the derived dependencies in the previous sections.

The following input parameters are used:

1. The maximum crest height of the level 1 tsunami is equal to 8 meter, so the retaining water level \( H_{\text{Level1}} \) is equal to 16 meter [Section 3.5];
2. The horizontal cable length variates from 32 till 317 meters with interval steps of 5 meter;
3. The membrane length variates from 20 till 64 meters with different interval steps;
4. The fold factor \( \zeta_m \) is assumed to be equal to 2 [Section 5.1];
5. The cable interval lengths variates for 10, 30 and 50 meters;
6. The beach has a width of 60 meters [Figure 2.3]
7. The cost estimation is based for a membrane barrier with a length of 90 meters.

For the first calculations, it is assumed that the barrier will be placed onshore, just before the dike of Kamakura [Figure 2.3]. To ensure an optimum integration of the barrier in the surroundings, a maximum floater diameter is assumed to be 5 meter.

The cost estimation is based on a rough estimation of the required materials (membrane, cable and steel). To include the fabrication costs, the the material cost are multiplied by 2. The following cost estimations for the required materials are used, including the fabrication cost:

- The membrane cost are estimated on 180 euro per square meter with the required thickness;
- The cable cost are estimated on 70 euro per kilogram;
- The steel cost are estimated on 3 euro per kilogram.

**Global Design Results**

The diameter of the floater is represented in function of the membrane- and cable length for three different interval lengths [Section E.5].

From the results it can be concluded that the influence of the cable interval length on the floater diameter is not dominant. This relation in combination with the maximum set floater diameter of five meter, results in a very small design space for the Kamakura case study. There is no configuration of the elements possible for a cable interval length of 50 meter, and a very limited design space for the cable interval length of 30 meter. The optimum configuration is therefore, found for the small cable interval length of 10 meter, with a large membrane length of 60 meters and a horizontal cable distance of 160 meter. The cost is estimated on approximately 42,000 euro per running meter.

A sketch design of the computed configuration is given in Figure 5.7 for a total barrier length of 90 meter.

There is a large amount of material needed for the proposed configuration. The small cable interval length of 10 meter is also not optimal. It can be therefore concluded that the proposed configuration is not a desirable solution for the case study of Kamakura.
The objective is to improve the design by decreasing the amount of material. This can be achieved, by lowering the retaining height of the membrane, by elevating the bottom recess with three meters. The barrier will be placed on top of the shore instead of fully integrated in the soil.

The following assumptions are adjusted for the improved design:

1. The maximum crest height of the level 1 tsunami is equal to 8 meter, with a bottom recess height of 3 meter, the retaining water level is equal to 13 meter;
2. The cable length varies from 26 till 256 meters with a 5 meter interval;
3. The membrane length varies from 17 till 50 meters with different interval steps;

The floater diameter is once again analysed [Section E.5.3], which results in an enlarged design space based on these proposed measurements. Based on the same cost functions, an optimum configuration was chosen: a membrane length of 42 meters with a horizontal cable length of 86 meter with a corresponding cable interval length of 30 meters. The floater has a diameter of 5 meter with a thickness of 2.5 centimetre. The total cost is estimated on 21,200 Euro per running meter for the membrane, cable and floater. So the optimised design leads to a cost reduction of approximately 50 percent. A sketch design is given in Figure 5.8 for a total barrier length of 90 meter.

It can be concluded that the improved design [Figure 5.8] is preferred, compared with the previous design [Figure 5.7]. The improved barrier requires less space, which benefits the recreational value of the beach. There is a significant cost reduction of more than 50 percent. This improved, global design will therefore be further analysed in Chapter 6.
Site Specific Design

The derived global design, presented in the previous chapter, will be further elaborated and integrated in the surroundings for the case study of Kamakura. In order to come with a first, preliminary design of all distinguished elements.

The global design was solely based on the Level 1 tsunami wave, so the first step is to specify the remaining load conditions [Section 6.1]. The next step is to redefine the optimum membrane-, cable- and floater configuration based on both tsunami levels and with an elevated bottom recess. Also the bottom recess, pile foundation and anchor foundation are further elaborated [Section 6.2]. The final step is to optimise the design by presenting optimisations of the separate elements. [Section 6.3]

6.1 Analysis of the Load Conditions

In this section the following loads are analysed: the retaining height of the level 2 tsunami wave, the bore impact, the peak stresses due to the inflation phase, the peak stresses due to folds in the membrane and the peak stresses due to secondary waves. The three peak stresses are approximated by amplitude factors.

**Retaining phase: Level 2 tsunami**

In Section 4.3 the pressure distribution during a Level 2 tsunami was analysed. Formulas for the pressure at the top of the floater \( p_{2,\text{level}2} \) and at the bottom recess \( p_{2,\text{level}2} \) [Equation 4.5] were proposed, from which the resulting force can be calculated:

\[
F_{p,\text{level}2} = \frac{1}{2} \cdot (p_{1,\text{level}2} + p_{2,\text{level}2}) = 2174\, \text{kN}
\] (6.1)

The amount of pressures depends on the amount of overflow. From the SWASH analysis, this overflow was derived, which was estimated to be 9 meters on top of the barrier [Section 3.5].

With Equation 6.1 the resulting force due to retaining phase of the Level 2 tsunami is equal to 2174 kN. With Equation 5.4, the minimal tension force of the membrane is equal to 1087 kN [Section 5.2].

**Fold Factor**

It is assumed that the floater is a rigid stiff floater. The cable forces create moments in the floater, which could lead to deflections and therefore peak stresses in the membrane. In the Global Design, an amplitude factor of 2.0 was incorporated. This factor was independent of the cable interval length, which determines the magnitude of the moments [Section 5.1]. So, this amplitude factor seems to be conservative.
The interaction between the cable, floater and the membrane can be modelled as an elastic supported beam. Where the floater represents the beam, the membrane can be seen as the elastic support and the cable force as a point load. The stress variation in the membrane depends on the amount of deflection of the beam [Bouma, 2000].

An analytical model is composed to describe the deflection line for a mid-section of the floater. This analytical model is used to validate a structural model, created with the software program Matrixframe. With the Matrixframe software, the entire floater is modelled [Figure 6.1][Section G.1].

The peak stresses of the membrane is related to the maximum deflection \( w_{\text{max}} \). The amplitude factor due to the folds can be calculated by comparing these peak stresses with the tension membrane force in the retaining phase of the barrier [Equation 6.2].

The spring constant \( k \) depends on the membrane characteristics: the membrane length \( L_m \), the thickness \( t_m \) (\( \approx 1\text{cm} \)), the maximum tension strength [3400 MPa] and the fracture strain [3 percent].

\[
\zeta_m = \frac{k \cdot w_{\text{max}} + T_m}{T_m}
\]

\[
k = \frac{t_m}{L_m} \cdot \frac{\sigma_m}{\epsilon_m}
\]

The maximum deflection occurs at the outer ends of the floater, where the cable force is half the cable force in the mid-sections. This results in a fold factor of 1.15 for a Level 1 tsunami and a fold factor of 1.2 for a Level 2 tsunami [Figure 6.1]

![Figure 6.1: Schematic overview of the floater; Deflection line of the floater during a Level 1 tsunami; Deflection line of the floater during a Level 2 tsunami](image)

It can be concluded that the fold factor can be strongly reduced. Even so, the reduced fold factor of 1.2 seems to be conservative. It is assumed that the deflection of the floater creates an additional stress in the membrane. These deflections are rather small, and it can be assumed that the membrane can move along with these deflections. The differences in the membrane tension forces are then negligible.
Incoming Bore

The incoming bore causes an impact load on the vertical part of the bottom structure. This horizontal impact load can be estimated by the theory proposed by Ramsden (1990) [Section A.2.3 for vertical walls. It was concluded that for large bore, the impact load is estimated at 7.5 times the hydrostatic force of the hydrostatic water pressure [Equation 6.4]. However, due to the finite height of the bottom recess \([H_{\text{wall}} = 3\text{m}]\), an empirical reduction factor is proposed [Equation 6.5] [Thomas and Cox, 2012]. Where the factor \(\frac{x}{L}\) represents the location of the wall, relative to the shore, and is estimated at 0.875 [Section A.2.3]:

\[
F_{x,\text{max}} = \frac{7.5 \cdot \frac{1}{2} \cdot \rho_w \cdot g \cdot H_{\text{bore}}^2}{C_{\text{bore}}}
\]

\[
C_{\text{bore}} = -0.331\left(\frac{H_{\text{wall}}}{H_{\text{bore}}}\right) + 0.027\left(\frac{H_{\text{wall}}}{H_{\text{bore}}}\right)^2 + 0.341\left(\frac{x}{L}\right) - 0.076\left(\frac{x}{L}\right)^2 + 1.109
\]

The impact load for a Level 1 tsunami wave is equal to 2108 kN and for a Level 2 tsunami, equal to 6200 kN.

Dynamic Load of the Inflation Phase

The inflation process of the barrier can be characterised by two dynamic loads: the snatch force in the cables and the opening shock in the membrane [Section 4.3]. In Section G.3 the opening shock of the membrane was analysed to determine the amplitude factor of the peak stress in the membrane. It is assumed that this peak stress in the membrane is equal to the peak stress in the cables.

The membrane configuration is filled with a certain amount of water. It is assumed that the opening shock occurs when the membrane is fully 'filled', as the reflecting bore is formed. At that specific moment, the membrane must be able to withstand the resulting force \([F_{\text{opening}}]\) which is composed out of the opening shock \([F_x]\) and the static force of the stationary water inside the membrane \([F_{\text{static}}]\). To determine the amplitude factor, this force is compared with the membrane force in the retaining phase:

\[
\zeta_m = \frac{F_x + F_{\text{static}}}{2 \cdot T_m}
\]

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\[
\zeta_m = \frac{F_x + F_{\text{static}}}{2 \cdot T_m}
\]
The moment of the opening shock can be determined with the SWASH model. The incoming water profile is measured in function of the time, at the location of the barrier. At the moment the maximum volume of the membrane has crossed the barrier, the bore height \([H^*]\) and velocity \([u^*]\) are determined. The opening shock \([F_x]\) is assumed to be the bore impact force at a vertical wall and calculated with the theory of FEMA (2012):

\[
F_x = 1.5 \left( \frac{1}{2} \cdot \rho_w \cdot C_d \cdot (H^* \cdot (u^*)^2) \right)
\]  

(6.7)

The static force of the water is assumed to be the hydrostatic water pressure:

\[
F_{\text{static}} = \frac{1}{2} \cdot \rho_w \cdot g \cdot (H^*)^2
\]

(6.8)

This calculation is executed for both tsunami levels. At the moment of inflation during a Level 1 tsunami, the velocity of the bore is equal to 5.4 meter per second, with a corresponding bore height of 6.8 meter. This results in an amplitude factor which is smaller than 1.0 and the inflation load can be therefore considered as a not normative load.

At the moment of inflation during a Level 2 tsunami, the velocity of the bore is equal to 13 meter per second, with a corresponding bore height of 13 meter. This results in an amplitude factor of 1.6.

**Dynamic Load of Secondary Waves**

The final amplitude factor concerns the peak loading due to secondary waves [Section 4.3]. It is assumed that these secondary waves create an increase in the pressure distribution. Because the numerical SWASH model only represents the first, initial wave, an estimation is made for the height of these secondary waves. It is assumed that the secondary waves can be small in order to 1 meter, till considerable waves of 3 meters. The various pressure distributions are implemented in the numerical model of the membrane for both tsunami levels.

From the results, it can be concluded that the amplitude factor for secondary waves for a Level 1 tsunami is equal to 1.48 and for a Level 2 tsunami 1.19. These are peak stresses are generated in the membrane and transferred to the cables.

**Overview Amplitude Factors**

There are three amplitude factors that influence the peak stresses in the membrane and in the cable. These are estimated for both tsunami levels [Table 6.1]. The next step is to determine the normative load condition combination and determine the design peak load in the membrane \([T_{m,\text{peak}}]\) and cable \([T_{c,\text{peak}}]\).

<table>
<thead>
<tr>
<th></th>
<th>Level 1 Tsunami</th>
<th>Level 2 Tsunami</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fold Factor</td>
<td>1.15</td>
<td>1.20</td>
</tr>
<tr>
<td>Secondary Waves</td>
<td>1.48</td>
<td>1.19</td>
</tr>
<tr>
<td>Parachute</td>
<td>0.6</td>
<td>1.6</td>
</tr>
</tbody>
</table>

The normative load combination is during the inflation of the membrane and there is a deflection in the floater for a Level 2 tsunami. These loads are generated in the membrane and are transferred to the bottom recess foundation and through the cables to the offshore foundation.
The final step is to include a material factor to compute the design peak load. In Section 4.3 it was concluded that for the membrane \( \gamma_m \) this factor is 1.5 due to possibility of tears in the membrane. For cables \( \gamma_c \) this factor is assumed to be 1.2:

\[
T_{m, \text{peak}} = \gamma_m \cdot \zeta_m \cdot \zeta_p \cdot T_m = 2.88 \cdot T_m \\
T_{c, \text{peak}} = \gamma_c \cdot \zeta_m \cdot \zeta_p \cdot T_m = 2.30 \cdot T_C
\] (6.9)

### 6.2 Dimensioning of the Elements

The optimum membrane-, cable-, and floater configuration is re-calculated for a Level 2 tsunami for the case study of Kamakura. From the Global Design it was concluded that the bottom recess must be elevated by three meters in order to come up with a preferable configuration.

Also the load derivation to the pile foundation and the drag embedded anchor is analysed. Thereafter a first preliminary design of both foundations can be determined.

#### 6.2.1 Optimum Cable-, Membrane and Floater Configuration

In the Global Design an optimum configuration of the cable-, membrane- and floater was derived for the Kamakura case. However, this was solely based on a Level 1 tsunami wave. For the site specific design, the optimum configuration is re-calculated, with the optimised fold factor and for both tsunami wave levels.

The major difference between the Level 1 and Level 2 tsunami is the shape of the membrane. Therefore the dependencies between the membrane length \( L_m \) and membrane forces for a Level 2 tsunami [Figure 6.3] are compared to the dependencies for a Level 1 tsunami [Figure 5.3].

![Figure 6.3: Relation between the membrane length and the resulting membrane tension force component [Green], the horizontal force component [Red] and the vertical force component [Blue] during a Level 2 tsunami](image)

Figure 6.3: Relation between the membrane length and the resulting membrane tension force component [Green], the horizontal force component [Red] and the vertical force component [Blue] during a Level 2 tsunami
It can be concluded that for a Level 2 tsunami the membrane tension force is independent of the membrane length, when its length greater than 2 times the retaining height. This is due to the shape of the membrane, which will form an ellipse [Figure 6.4] and the vertical membrane tension force goes to zero. This implies that the floater diameter only depends on the cable configuration [Section 5.2].

The cable and floater configuration are determined, based on the Level 2 tsunami conditions. Thereafter, the membrane length is determined, based on the Level 1 tsunami condition and where the membrane length must be greater than two times the retaining height.

From the calculations it is concluded that an optimum membrane-, cable-, and floater configuration is found with a membrane length of 37 meters, a horizontal cable length of 106 meter and a floater diameter of 5 meter with a corresponding thickness of 4.6 centimeter.

Dimensions of the Membrane

The membrane configuration with a membrane length of 37 meter, during the retaining phase of both tsunami levels, is given in the Figure 6.4:

![Figure 6.4](image)

Figure 6.4: Deflection line of the membrane during a Level 1 [Blue Line] and a Level 2 [Red Line] tsunami for a membrane length of 37 meter

It is assumed that the attach point of the membrane is just next to the floater [Figure 5.2]. This implies that the membrane needs 15 meter space to fully inflate.

The peak tension force depends on the membrane tension force in the retaining phase, which can be calculated with Figure 6.3 and is equal to \( T_{min} = 1087 \text{ kN} \). This leads to a peak membrane tension load is equal to \( T_{m; peak} = 3131 \text{ kN} \).

Based on the peak membrane tension load, the thickness of the membrane can be derived. This depends on the maximum tensile strength of the CF10 fibre is equal to 3400 MPa. It can be assumed that the tensile strength of the Dyneema membrane will be reduced with a factor 4, this factor includes the rubber protection layer and the fact that half of the fabric fibres are in the other direction [R. Marissen, Personal communication, 27 Oktober 216]:

\[
t_m = \frac{T_{m; peak}}{4 \cdot 3400} \approx 4 \text{ mm}
\]  

(6.10)
Dimensions of the Cables

For the determination of the optimum cable configuration, the decreasing slope \( \approx 1:100 \) of the shore was not taken into account. Also the height of the cables was assumed to be 13 meters, so the cable needs to be extended. The eventual shape of the cable is 140 meters. Also, the hogging affect of the floating cable is negligible and the cable shape can be seen as a straight line between the floater and the offshore foundation.

The maximum peak load in the cable is determined by the tension load in the retaining phase. Figure E.9 is used to determine the distributed tension load in the cables \( T_{qc} \) and is equal to 1095 kN. The peak load in the cable is equal to:

\[
T_{C; peak} = T_{qc} \cdot Ln \cdot 2.3 = 75,555 \text{kN}
\]  

(6.11)

The required diameter can be derived from the tensile strength of the cable, which is equal to 700 MPa. The diameter of the cable is equal to:

\[
d_{cable} = \sqrt{\frac{T_{C; peak}}{0.25 \cdot \pi \cdot 700}} = 375 \text{mm}
\]  

(6.12)

6.2.2 Load Derivation to the Pile Foundation

In Section 4.4 it is stated that the membrane tension \( T_{m; peak} \) force is transferred to the bottom recess and its foundation. The bottom recess must also be able to withstand the impact load of the bore \( F_{bore} \). Besides the horizontal forces, the pile foundation also be able to withstand the vertical forces [Figure 6.5] due to the water pressure \( p \), the weight of the bottom structure \( F_{g; float} \) and the upward water pressure below the bottom recess \( F_{p; up} \). This latter force is caused by the hydraulic head over the bottom recess. During a Level 2 tsunami this hydraulic head is significant. It is assumed that the gradient of this pressure distribution is linear [Bezuyen et al., 2011].

![Figure 6.5: The vertical loads acting on the bottom recess during the Level 2 tsunami for the case study of Kamakura](image)

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The first step is to assume a pile plan for the bottom recess structure. For the first calculations, 8 piles are evenly distributed, with an interval distance of 3.25 meters and where the centre of gravity is located halfway the bottom recess. The next step is to determine the normative pile forces. These are decomposed in a vertical component and a horizontal component.

The vertical component is determined by the resulting vertical forces and the resulting moment acting on the bottom recess, and the horizontal force component depends on the maximum horizontal force acting on the structure. From the load analysis [Section 6.1] it can be concluded that the impact load of the bore during a Level 2 tsunami is greater than the peak membrane force.

The outcome of this analysis is that there are only compression piles. The horizontal force is normative for the design of the piles. In order to withstand the high horizontal force, the first 6 piles will be in a battered position.

**Pile Dimensions**

The pile dimensions are determined by the compression- and lateral load capacity. From the load analysis, it is concluded that the horizontal load will be normative. For the full derivation of the pile dimensions is referred to Appendix H.2.3. The calculations are based on the assumed soil conditions [Section 2.2], without taking into account the possible weakening of the soil due to the seismic activity.

The derived pile foundation is given in Figure 6.6, where the first 4 pile rows are battered with an angle of 20 degrees, the piles in row 5 are battered with an angle of 10 degrees and the piles in row 6 are battered with an angle of 5 degrees. The piles in row 7 and 8 are straight compression piles. These latter piles must have the greatest compression capacity. It is assumed that they are driven, square piles with a width of 0.5 meter and a length of 11 meters.

![Figure 6.6: Assumed pile foundation of the bottom recess](image)
Bottom Recess

In Section 4.1 it is assumed that the bottom recess will be made out of concrete. The assumed dimensions are given in the Figure 6.7. Note that the design of the bottom recess is without a storage of the membrane, this will be further elaborated in Section 6.3.

Figure 6.7: Assumed dimensions of the bottom recess structure

The bottom recess is a robust structure with significant dimensions. A large weight of the structure is needed to withstand the upward water pressure to prevent uplift of the bottom recess.

The hydraulic head not only causes a great upward water pressure, it could also cause another mechanism which could cause instability, namely piping. This is the flow of water through a pipe-like channel below the bottom structure, that has been created by internal erosion [Vrijling et al., 2011]. It is therefore recommended to install sheet walls to increase the seepage length. These sheet piles could also decrease the magnitude of the upward water pressure and therefore decreases the moment on the bottom recess.

6.2.3 Load Derivation to the Drag Embedded Anchor

The final load segment concerns the load derivation of the cable to the drag embedded anchor. The main requirement is that the peak load in the cable \( T_{C; \text{peak}} \) does not exceed the holding capacity of the anchor. If the holding capacity is exceeded, the anchor will be pulled out.

The drag embedded anchor must be able to withstand the high tension force of the cable \( T_C = 75,000 \text{kN} \). There is a wide range of anchors available. The anchor with the greatest holding capacity in sand is the MK6 anchor [Vryhof Anchor, 2015]. It is therefore assumed that a Stevpris MK6 anchor will be used.

Figure 6.8: Design of the Stevpris MK6 anchor [Vryhof Anchor, 2015]
The ultimate holding capacity is related to the size of the anchor. For the preliminary design calculations, design graphs are used. From the calculations [Section H.2.4] it is concluded that an anchor with a size of 120 ton is needed, with an assumed penetration depth up to 8-10 meters [Vryhof Anchor, 2015].

It must be noted that the maximum size available was up to 100 ton. It can be therefore questioned if the drag embedded anchor is the best suitable offshore foundation. However, the calculations are a rough estimation and solely based on the peak load of the cable. An option is to decrease the anchor size by increasing the penetration depth of the anchor. Overall it can be concluded that the offshore foundation needs further research, however the option for the drag embedded anchor seems feasible.

6.2.4 Cost Estimation

The dimensions of the different elements are estimated. Based on these calculations, a rough cost estimation can be executed. This estimation is based on simple cost functions, based on the required materials. To include the fabrication costs, the material costs are multiplied by 2. The following cost estimations are used:

- The membrane cost are estimated on 180 euro per square meter for the required thickness;
- The cable cost are estimated on 70 euro per kilogram;
- The steel cost are estimated on 3 euro per kilogram;
- The concrete costs are estimated on 70 euro per cubic meter;
- The pile costs are estimated on 100 euro per meter pile;

From Table 6.2 it can be concluded that the costs of the barrier, is estimated at 73,000 euro per meter barrier.

<table>
<thead>
<tr>
<th>Element</th>
<th>Dimensions [m]</th>
<th>Volume/m</th>
<th>Cost/(m;kg)</th>
<th>Total cost/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable</td>
<td>$L_c = 140$</td>
<td>$0.52 \text{ m}^3$</td>
<td>70 €/kg</td>
<td>25,256 €</td>
</tr>
<tr>
<td></td>
<td>$d_{cable} = 0.375$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$L_m = 30$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom Recess</td>
<td>Figure 6.7</td>
<td>$118.5 \text{ m}^2$/m</td>
<td>70 €/m$^3$</td>
<td>8,295 €</td>
</tr>
<tr>
<td>Floater</td>
<td>$d_{floater} = 5$</td>
<td>$2.19 \text{ m}^3$/m</td>
<td>3 €/kg</td>
<td>17,113 €</td>
</tr>
<tr>
<td></td>
<td>$t_{floater} = 0.047$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Membrane</td>
<td>$L_m = 37$</td>
<td>$37 \text{ m}$/m</td>
<td>180 €$/m^2$</td>
<td>6,660 €</td>
</tr>
<tr>
<td>Anchor</td>
<td>$V_{anchor} = 120$ ton</td>
<td>4000 kg</td>
<td>3 €/kg</td>
<td>12,000 €</td>
</tr>
<tr>
<td>Pile Foundation</td>
<td>Figure 6.6</td>
<td>$30 \text{ m}$/m</td>
<td>100 €$/m$</td>
<td>3,000 €</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>72,324 €</td>
</tr>
</tbody>
</table>
6.3 Optimisation and Integration of the Tsunami Barrier

The final step of the preliminary design of Kamakura is to integrate the barrier in the surroundings. Starting point is the derived design cross section of Kamakura [Figure 2.3]. It is assumed that the flexible barrier will be part of a total coastal defence system. An example of a possible coastal defence system is given in Figure 6.9.

The beach of Kamakura is divided in two parts. In the middle of the beach, there is a water outlet, therefore it is assumed that there will be a type of flood barrier. On each side of the flood barrier, a flexible membrane barrier will be placed. On the outer-edges of the barrier, there will be a type of abutment structure. The side of the abutment structure will be flat and vertical such that the membrane can keep a 2 dimensional shape, and a watertight connection can easily be obtained. The remaining part of the bay will be protected by tsunami walls.

![Figure 6.9: Assumed coastal defence plan for the case study of Kamakura](image)

The height of the bottom recess structure coincides with the height of the existing dike of Kamakura. Therefore, it is suggested to integrate the barrier in the existing structure.

For the integration of the barrier, the derived dimensions of the elements are analysed and if necessary adjustments or optimisations are proposed.

**Membrane**

The membrane length was estimated to be 37 meters with a thickness of 4 millimetres. For the calculations it was assumed that the membrane was inextensible, however, the membrane has a fracture strain of 3 percent [Section 4.1]. So the membrane length could extend to a length of 38 meters. This stretch of one meter is considered to be not problematic, and it will be assumed that the membrane will have a larger deflection, but the floater will be at the same position.

During normal conditions the membrane is stored in the bottom recess structure. This ensures that the membrane will be protected from weathering and vandalism. When a tsunami occurs the membrane must be able to fold out, to retain the water. During this process, the membrane may not be damaged. Therefore a type of storing structure must be developed. For example, the Ramspol Barrier uses rollers to improve the transport of the membrane [Breukelen, 2013].
Cables

The cable length was computed to be 140 meters with a diameter of 375 millimetres. For the calculations, it was assumed that the cable was inextensible. However, the cable has a assumed fracture strain of 3 percent [Section 4.1]. So the cable length could extend to a length of 144 meters. This elongation is significant and could lead to a displacement of the floater. The elongation can be reduced by applying used cables and applying a larger material factor and will lead to a fracture strain of 1.5 percent.

To obtain a diameter of 375 millimetres, the cable consists out of sub-ropes [Figure 6.10]. The cable must also be protected against external abrasion and ingress of abrasive particles. That is why a filter is applied.

![Cable composed of sub-ropes](image)

Figure 6.10: Impression of the cable, composed out of sub-ropes [Royal Lankhorst Euronete, 2016]

During normal conditions, the cable must be stored in a gutter type of structure [Figure 6.11b]. For the calculations, it was assumed the cable would be floating which could have a beneficial affect on the floater. It is concluded that this hogging effect was negligible. So for the integration, it is preferable to use a cable with a greater mass density. As a result, the cable will lay onto the seabed.

Floater

For the first calculations it is assumed that the floater is a steel pipe with a certain diameter [Section 4.1]. There are also other floater designs possible, for example a more ellipse type of floater [Figure 6.11a]. The dimensions are such that the upward water pressure is similar as in the calculations. Note that the ends of the floater are closed off, to prevent inflow of water.

For the integration of the floater in the surroundings, it is recommended to optimise the design, such that it can be used during normal conditions. For example as footpath or cycle path. This implies other load conditions, and it must be investigated if the floater must be strengthened.

![Floater optimisation](image)

Figure 6.11: Impression of the optimisations of the (a)floater and (b)bottom recess
Bottom Recess

The bottom recess must be re-designed in order to store the membrane and the optimised floater. An impression is given in Figure 6.11b.

For Kamakura case, it is assumed that the bottom recess is integrated in the existing dike, so that the membrane will inflate over the main road. This implies that when the tsunami alarms sounds, the main road must be closed off. Also existing structures, such as lampposts, fences and other objects must be replaced.

Pile Foundation

The pile dimensions are based on an assumed pile plan [Section H.2.3]. There are more pile configuration possible, which can result in an improved design.

From the analysis, it could be concluded that the lateral load capacity was normative for the pile dimensions. It could be an optimisation to apply an L-shaped retaining wall, which can be integrated in the bottom recess structure. This will lead to an increase in the lateral load capacity. This retaining wall will also increase the seepage length [Section H.2], and could lead to a decrease of the upward water pressure [Section 6.2.2]. Besides the retaining wall, battered piles are applied [Figure 6.12].

Figure 6.12: Impression of the suggested, optimised pile foundation
Connections

The connections between the elements are not further analysed and it was assumed that these connections were able to transfer the loads between the elements. In this section, the connections are briefly analysed. There are three major connections points, at the bottom recess, at the offshore foundation and at the floater.

The membrane is connected at the bottom recess. It is assumed that the membrane will lay horizontal onto the bottom recess. A possibility to connect the membrane at the bottom recess is by means of a wrapping the membrane around a steel bar, which is anchored in the concrete bottom recess [Figure 6.13]. This type of connection is able to derive a high tension load without stress concentrations. There is a possibility of unreeling of the membrane, therefore an oval shaped beam is recommended which will not rotate in the concrete structure [R. Marissen, Personal communications, 27 October 2016]

Figure 6.13: Impression of the wrapping connection of the membrane to the bottom recess [R. Marissen, 2016]

There are multiple solutions to connect the cable to the offshore foundation. For synthetic fibre ropes it is generally terminated with a special spool and shackle for connection to other components in the mooring system.

The critical connections are at the floater. There must be connections for the membrane, cables and secondary cables. The location, the type of connection and the load interaction between the connection and the floater must be further investigated.

An interesting possibility is to fold the membrane around a steel bar and connect both faces of the membrane by stitching. This steel bar can be connected to the steel floater. The strength of the stitched membrane is tested at the laboratory and it appears possible to create a connection with no strength loss [R. Marissen, Personal communication, 27 October 2016]. These connections must be further investigated.
6.4 Impression of the Integrated Barrier

An impression of the integrated, optimised, flexible-membrane barrier for Kamakura is given in Figure 6.14.

The dimensions of the integrated, optimised, flexible-membrane barrier are:

- The optimised floater is ellipse shaped with a length of 11 meter and a height of 2.5 meter
- A membrane length of 37 meter;
- A cable configuration which consists of a cable interval length of 30 meter and a cable length of 140 meter with a cable diameter of 0.375 meter;
- A drag embedded anchor of 120 ton;
- An optimised pile foundation [Figure 6.12];
- A bottom recess structure [Figure 6.7];
- The cost per running meter barrier is estimated on 73,000 Euro.
Conclusions and Recommendations

In this research, a feasibility study is executed for the conceptual design of a flexible-membrane barrier. Based on the presented, preliminary design for the case study of Kamakura, it can be concluded that the tsunami barrier is technical feasible. The flexible membrane barrier can be considered to be an alternative solution for the conventional sea walls. This research study of the tsunami barrier is a first step of a total design of the flexible membrane barrier.

All results are summarised, divided in to the wave analysis, global design and site specific design

Wave Analysis of the incoming tsunami wave

It can be concluded that a numerical one-dimensional SWASH model, based on the N-wave theory, can be used to approximate the essential features of an incoming tsunami wave. The SWASH model is validated for the reference case of Sendai, by analytical models and observational data from the Tohoku tsunami of 2011.

It is furthermore assumed that the height of the incoming Level 1 tsunami wave height is equal to 8 meters and the height of the incoming Level 2 tsunami wave is equal to 14 meter [Okumura, 2016].

The velocities of the incoming bore for a Level 1 and Level 2 tsunami are respectively 17 and 25 meter per second. These velocities are greater than the bore velocities of the experiments of B. Hofland et all. [2016].

Global Design

In the global design of the tsunami barrier, the relations between the membrane-, the cable-, and the floater configurations are studied in function of the retaining height of the barrier. In order to come up with an optimum design of these elements. The relations between the elements are based on the static load condition of the retaining phase of the Level 1 tsunami. The retaining height of the barrier is equal to two times the height of the incoming Level 1 tsunami bore.

The derived theory of the relations between the structural elements is applied to the case study of Kamakura. It is concluded that an optimum is found for a large membrane length of 42 meters, a horizontal cable length of 86 meters, a corresponding cable interval length of 30 meters and a floater diameter of 5 meters, where the bottom recess is elevated 3 meter above the soil.

- The membrane length, the cable length and the floater dimensions are determined by the force balance of the floater;
- For a large retaining height of the membrane [>16m] an optimum configuration with a relatively small floater, can only be obtained with a very large membrane length, a very large horizontal cable length and a minimal cable interval length;
- The tsunami barrier can be optimised by decreasing the retaining height of the membrane, by elevating the bottom recess with three meters. This results in a cost reduction of approximately 56 percent.
Site Specific Design
A preliminary design of the flexible-membrane barrier is presented for the case study of Kamakura, with the following dimensions:

- A membrane length of 37 meters with a thickness of 4 millimeters made out of Dyneema CF10 cloth;
- A cable configuration with a cable interval length of 30 meters, a cable length of 140 meters and a thickness of 37.5 centimeters made out of Dyneema yarn;
- A steel floater with a diameter of 5 meter and a thickness of 4.7 centimeters;
- A concrete bottom recess which is integrated in the existing dike of Kamakura;
- The bottom recess foundation consists out of 8 (battered) square piles with a width of 0.5 meter and a length of 11 meter;
- A drag embedded anchor with a mass of 120 ton.

Several optimisations are presented to integrate the barrier in the surroundings of Kamakura. The following sub-conclusions can be derived:

- The assumed normative load combination occurs in the inflation phase with deflections in the floater, during a Level 2 tsunami. With an approximated membrane peak load equal to 2.9 times the retaining membrane force and an approximated cable peak load equal to 2.3 times the retaining cable force;
- The deflections due to the cable induced moments is approximated by an analytical model, causes peak stresses equal to 1.1 times the membrane tension force in the retaining phase of a Level 2 tsunami;
- The membrane tension force is independent of the membrane length, from a membrane length greater than 2 times the retaining height for a Level 2 tsunami;
- The cable and floater configuration is based on the retaining phase of the Level 2 tsunami and the membrane length is determined by the retaining phase of the Level 1 tsunami;
- The barrier is integrated in the existing dike of Kamakura by placing the cavity of the bottom recess in front of the existing dike. During a tsunami event, the membrane will inflate over the existing road on top of the dike.
7.1 Recommendations

In this report, a preliminary design is presented of the flexible, membrane tsunami barrier for the case study of Kamakura. However, further research is recommended:

Wave Analysis
The wave conditions are based on the numerical one-dimensional SWASH model. However, it is recommended to improve the model by a two- or three-dimensional wave model.

- Study the effect of refraction and diffraction of the tsunami wave;
- Use the soil deformation caused by the earthquake as initial conditions for the tsunami wave to simulate the entire tsunami wave train;
- Preform sensitivity analyses to check the influences of different parameters;
- Couple the SWASH model to the inflation of the barrier and study the behaviour of the reflected wave and the water inside the barrier.

Global Design
In the Global Design, only the relation between the membrane, cables and floater in function of the retaining height of the barrier are investigated. However, it is recommended to extend this study with the remaining elements; the bottom recess structure, the bottom recess foundation and the offshore foundation. As a result, the design space of the tsunami barrier is obtained, in which an optimum configuration can be determined based on cost functions. Also, the following aspects are recommended to investigate:

- Investigate other membrane configurations, offshore foundations and bottom recess foundations;
- Further specify the cost functions.

Site Specific Design
The site specific design is based on several assumptions and simplifications which gives the design its limitations. Therefore the following aspects are recommended to improve in further research:

- The assumed normative dynamic loads are approximated with amplitude factors. These factors are based on simple analytical models and it is recommended to further extend these analyses;
- The dimensions of the pile- and anchor foundation are based on rough estimations, it is therefore recommended to improve these analyses;
There are still uncertainties in the design of the flexible membrane barrier, it is therefore recommended to study the following aspects

- Investigate other failure mechanisms, such as: behaviour of the foundation during seismic activity, behaviour of the barrier during overflowing, dynamic impact of debris, scour process, and the behaviour of the barrier if a cable breaks;
- The design of the connections of the tsunami barrier;
- The inflation process of the barrier under the high tsunami front velocities of 17 and 24 meter per second;
- The behaviour of the flexible membrane barrier in a total coastal defence system.

Lastly, it is recommended to study the applicability of the barrier:

- Determine for which type of tsunamis and project locations, the flexible-membrane barrier is a desirable alternative for the conventional sea walls;
- Other case study locations. For example a location with a very steep slope. This results in other tsunami conditions, where the tsunami can be modelled as a raising water level;
- The possibility to use the design of the flexible membrane barrier for a new type of extreme flood barrier.
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## Nomenclature

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<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<tr>
<td>$\beta_c$</td>
<td>Curvature of the cable at the floater</td>
<td>rad</td>
</tr>
<tr>
<td>$\beta_m$</td>
<td>Curvature of the membrane at the floater</td>
<td>rad</td>
</tr>
<tr>
<td>$\epsilon_m$</td>
<td>Strain of the membrane</td>
<td>3</td>
</tr>
<tr>
<td>$\eta$</td>
<td>Surface elevation of the water level</td>
<td>m</td>
</tr>
<tr>
<td>$\gamma_b$</td>
<td>Volumetric weight of concrete</td>
<td>$25,kN/m^3$</td>
</tr>
<tr>
<td>$\gamma_m$</td>
<td>Material factor of the membrane</td>
<td>1.5</td>
</tr>
<tr>
<td>$\omega$</td>
<td>Wave period</td>
<td>rad/s</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Curvature of the membrane</td>
<td>rad</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>Density of steel</td>
<td>$7800,kg/m^3$</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>Density of water</td>
<td>$1100,kg/m^3$</td>
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<tr>
<td>$\sigma_m$</td>
<td>Maximum tension strength of the membrane</td>
<td>$3400,MPa$</td>
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<tr>
<td>$\tau$</td>
<td>Turbulent stress</td>
<td>N/m</td>
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<tr>
<td>$\zeta_m$</td>
<td>Fold factor</td>
<td>–</td>
</tr>
<tr>
<td>$\zeta_p$</td>
<td>Parachute amplitude factor force</td>
<td>–</td>
</tr>
<tr>
<td>$\zeta_w$</td>
<td>Secondary wave factor</td>
<td>–</td>
</tr>
<tr>
<td>$a$</td>
<td>Distance between force and centre of gravity</td>
<td>m</td>
</tr>
<tr>
<td>$a_w$</td>
<td>Amplitude of the wave</td>
<td>m</td>
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<tr>
<td>$C_d$</td>
<td>Drag coefficient</td>
<td>2</td>
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<tr>
<td>$C_F$</td>
<td>Force coefficient</td>
<td>1</td>
</tr>
<tr>
<td>$c_f$</td>
<td>Bottom friction coefficient</td>
<td>–</td>
</tr>
<tr>
<td>$C_z$</td>
<td>Beddings constant</td>
<td>$N/m^3$</td>
</tr>
</tbody>
</table>
$C_{bore}$ Empirical reduction factor due to finite tsunami wall

$d_{cable}$ Diameter of the cable $mm$

$d_{floater}$ Diameter of the floater $m$

$E_m$ E-modulus of the membrane $N/m$

$E_s$ E-modulus of steel $N/m$

$F_b$ Buoyancy force of the floater $N$

$F_x$ Opening shock force $N$

$f_y$ yield stress of steel $235 N/mm^2$

$F_{g,bot}$ Weight of the bottom structure $N$

$F_{g,floater}$ Weight of the floater $N$

$F_{opening}$ Resulting force at the moment of inflation $N$

$F_{p,level1}$ Resulting force of pressure distribution in the retaining phase of a Level 1 tsunami $N$

$F_{p,level2}$ Resulting force of pressure distribution in the retaining phase of a Level 2 tsunami $N$

$F_{pile}$ Force in the pile $N$

$F_{s,FEMA}$ Bore impact force, calculated by the FEMA theory $N$

$F_{s,Ramsden}$ Bore impact force, calculated by the Ramsden theory $N$

$F_{static}$ Static force inside the membrane $N$

$F_{up}$ Upward water pressure force $N$

$F_{wave}$ Resulting force of pressure distribution during secondary waves $N$

$g$ Acceleration of gravity $9.81 m/s^2$

$H$ Incoming wave height $m$

$h$ Water depth $m$

$H^*$ Bore height at the moment of inflation of the barrier $m$

$H_0$ Deep water wave height $m$

$H_b$ Bore height $m$

$H_b$ Breaking wave height $m$

$h_b$ Water depth at breaking location $m$

$H_i$ Inundation height $m$

$H_{bore,Lev2}$ Height of the incoming Level 2 tsunami bore $m$

$H_{bore,Lev1}$ Height of the incoming Level 1 tsunami bore $m$

$H_{overflow}$ Amount of overflow at the top of the tsunami barrier $m$

$H_{retaining}$ Retaining height of the barrier $m$

$H_{wall}$ Height of the tsunami wall $m$

$H_{wave}$ Height of the wave which overtop the tsunami barrier $m$
Moment of inertia of the floater $I_{\text{floater}}$ $m^4$

Spring constant $k$ $N/m$

Dissipation factor $K'$ --

Wave number $k_w$ --

Cable length $L_c$ $m$

Membrane length $L_m$ $m$

Cable interval distance $L_n$ $m$

Wave length $L_w$ $m$

Horizontal cable distance, between the floater and the offshore foundation $L_{c,\text{hor}}$ $m$

Length of the floater $L_{\text{floater}}$ $m$

Maximum moment in the floater $M_{\text{max}}$ $Nm$

Maximum pressure at the bottom recess in the retaining phase of a Level 1 tsunami $p_{1;\text{level1}}$ $kPa$

Maximum pressure at the bottom recess in the retaining phase of a Level 2 tsunami $p_{1;\text{level2}}$ $kPa$

Pressure at the floater in the retaining phase of a Level 1 tsunami $p_{2;\text{level1}}$ $kPa$

Cone resistance $q_c$ $MPa$

Section of the membrane $S$ $m$

Slope parameter $S_0$ --

Force in the cable $T_C$ $N$

Tension force in the membrane $T_m$ $N$

Membrane thickness $t_m$ $m$

Bearing capacity of the mooring line $T_{A,B}$ $N$

Friction of the soil $T_{A,F}$ $N$

Failure wedge of the soil $T_{A,S}$ $N$

Weight of the soil $T_{A,W}$ $N$

Peak force in the cable $T_{C,\text{peak}}$ $N$

Thickness of the floater $t_{\text{floater}}$ $m$

Horizontal force component of the membrane tension force $T_{m,\text{hor}}$ $N$

Peak force in the membrane $T_{m,\text{peak}}$ $N$

Vertical force component of the membrane tension force $T_{m,\text{ver}}$ $N$

Minimal tension force in the membrane $T_{\text{min}}$ $N$

Horizontal force component of the distributed cable force at the floater $T_{q_c,\text{hor}}$ $N$

Vertical force component of the distributed cable force at the floater $T_{q_c,\text{ver}}$ $N$

Distributed cable force at the floater $T_{q_c}$ $N$

Thickness of the floater $t_{\text{floater}}$ $m$

Return period $t_{rp}$ $years$
$u^*$  Bore velocity at the moment of inflation of the barrier  \quad m/s

$u_{\text{level},1}$  Velocity of the incoming Level 1 tsunami  \quad m/s

$u_{\text{level},2}$  Velocity of the incoming Level 2 tsunami  \quad m/s

$V_{\text{anchor}}$  Anchor size  \quad \text{ton}

$V_{\text{membrane}}$  Volume of water inside the membrane  \quad m^2

$w$  Deflection  \quad m

$W_{\text{floater}}$  Section modulus of the floater  \quad m^3
## Nomenclature

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<th>Description</th>
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<td>SWASH</td>
<td>Simulating WAves till SHore</td>
</tr>
<tr>
<td>FEMA</td>
<td>Federal Emergency Management Agency</td>
</tr>
<tr>
<td>MLS</td>
<td>Multi Layer Safety</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard Penetration Test</td>
</tr>
<tr>
<td>GPS</td>
<td>Global Positioning System</td>
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Theoretical Background Information

In this Appendix, the essential background information are analysed. The objective of this study is to determine the requirements, assumptions and boundary conditions for the case study of Kamakura. Also the applied wave theories are studied.

First the general characteristics of the tsunami wave are treated [Section A.1]. Several wave theories are analysed in order to estimate the different types of tsunami wave loading [Section A.2]. Thereafter, the new tsunami flood protection plans are analysed [Section A.3], to establish the location and function of the ‘Tsunami Catcher’. Also other types of membrane structures that function as reference case studies are investigated [Section A.4]. Finally the case study location of Kamakura is closely studied to determine the boundary conditions for the design of the barrier [Section A.5].

A.1 Tsunami’s General Characteristics

The term tsunami is derived from two Japanese words, namely: 'Tsu', meaning harbour, and 'nani', meaning wave. A tsunami is a wave, or series of waves in a wave train, generated by the sudden vertical displacement of a column of water [Bryant, 2008]. They can be generated by landslides, meteorite impact, volcanic activity or tectonic activity [Camfield, 1980]. The majority of these tsunamis are caused by earthquake, above 80 percent [Bryant, 2008]. The waves generated by tectonic uplifting may travel across an ocean basin, causing great destruction at location far from their source [Camfield, 1980]. They travel away from its origin in a pattern comparable with the patterns generated by the landing of a pebble in a pond [Bosboom and Stive, 2015].

A.1.1 Tsunami Wave Theories

Tsunami waves can be approximated by different theories, each with different characteristics. The wave theory will serve as input for the numerical SWASH model [Chapter 3]. Two wave theories are analysed and compared. The simplest way of representing a wave is by a sine or cosine function [Figure A.1a]. Its features can be characterised mathematically by linear, trigonometric functions known as the Airy wave theory. This theory can represent local tsunami propagation in water depth greater than 50 meter [Camfield, 1980].

When waves propagates towards the shore, the wave becomes more and more asymmetric. This is due to the shoaling process, which is characterised by an increase in wave height and by the gradual peaking of the wave crest and flattening of the trough. The separation between wave crest becomes so large that the trough disappears and only one peak remains, which resembles a solitary wave [Figure A.1b] [Bosboom and Stive, 2015].

![Figure A.1: The different types of wave theories](Yamao et al., 2015)
Many observations of a tsunami approaching the shore note that first water is drawn before the wave crest arrives. This is caused by non-linear effects that produce a trough in front of the wave. This can be modelled with a so-called N-wave [Figure A.1c] [Yamao et al., 2015].

**Solitary Wave Theory**

Since the early 1970’s, it has been frequently assumed that solitary waves can be used to model some of the important features of tsunamis approaching the beach and shoreline. The solitary wave can be seen as an amount of water riding completely above the mean sea level.

Solitary waves have the advantage, although nonlinear, that they can be described with just two parameters, namely the initial wave height \([H_0]\) and the initial water depth \([h_0]\) and they propagate with constant form in constant depth [Holthuijsen, 2007]. Also many studies have been executed, to discuss the hydrodynamics of solitary waves shoaling and breaking on a slope.

The main problem with the solitary wave theory, is that the link to geophysical scales is not well established. Therefore, conclusions in relation to tsunamis modelled as a soliton, should be made with great care [Madsen et al., 2008]. To generate a times series of the water elevation of a solitary wave, the following formula will be used:

\[
\eta = a_w \cdot sech \left( \sqrt{0.75 \cdot \frac{a_w}{h^3}} x \right)^2 \tag{A.1}
\]

Where \(\eta\) is the surface water level, \(a\) is the amplitude of the wave, \(h\) is the initial water depth and \(x\) is the distance from the initial location. The shape of the solitary wave does not change with constant depth. The propagation speed \(c\) of a solitary wave is computed by Equation A.2.

\[
c = \sqrt{gh} \left( 1 + \frac{\alpha}{2} \right) \tag{A.2}
\]

**N-Wave Theory**

There are two types of N-wave, determined by the geophysical characteristics of the earthquake; the leading-depression or the leading-elevation. In the paper of S. Tadepalli and C.E. Synolakis (1994) a type of N-shaped waves was found that displayed very interesting and counteractive behaviour of the tsunami wave.

The main advantage of a N-wave is that the physical characteristics of the wave fits better with the geophysical scales. It is suggested that the solitary wave may not be adequate for predicting an upper limit for the run-up of near-shore generated tsunamis [Tadepalli and Synolakis, 1994]. A N-wave however, has no mathematical connection. The wave is obtained by manipulating a soliton until it quantitatively changes to give the required shape with a leading or following trough. The amount of studies available is also limited compared with the solitary waves.

A N-wave can be obtained by multiplying the equation for the soliton [Equation A.1] with an equation of a linear line. A N-wave can have a leading elevation or a leading depression depending on the positive or negative slope of the linear line. The equation to generate a N-wave is derived from the report of S. Tadepalli and C.E. Synolakis [1994]:

\[
\eta(x, 0) = (\epsilon \cdot H)(x - X_2) \cdot sech \left( \Gamma_s(x - X_1) \right)^2 \tag{A.3}
\]

Where the factor \(\epsilon \cdot H\) is a scaled N-wave amplitude, the distance \([L]\) is defined as \([X_2 - X_1]\) and \(\gamma_s\) can be described with the following equation.

\[
\Gamma_s = \sqrt{0.75 \cdot \cot H} \tag{A.4}
\]
A.1.2 Tsunami Wave Transformations

Tsunami waves undergo similar wave transformations as ordinary wind waves in shallow water. A tsunami wave has a wavelength \( L_w \), a wave period \( \omega \) and a deep-water wave height \( H_0 \). The big difference relates to the fact that tsunami are very long waves (in order of kilometers). Five different stages of tsunami wave transformations can be distinguished [Figure A.2]: the generation and propagation in relatively deep water from the source region to a coastal region, the enhancement and deformation in shoaling water, up to breaking of the wave, dissipation of energy and finally the run-up onto land [Battjes and Labeur, 2014].

Figure A.2: Different stages of tsunami wave transformations

Tsunami Wave Generation

Tsunamigenic earthquakes are caused by rupture along active fault lines, where two sections of the Earth’s crust are moving opposite of each other. Three types of faults can generate an earthquake: a strike-slip earthquake on a vertical fault [Figure A.3a], a dip-slip earthquake on a vertical fault [Figure A.3b], and a thrusting earthquake on a dipping plane [Figure A.3c]. The greater the vertical displacement (or slip), the greater the amplitude of the tsunami [Bryant, 2008].

Figure A.3: Types of faults which causes tsunamigenic earthquakes [Bryant, 2008]

Tsunami Wave Shoaling

Shoaling is the effect of increasing wave height due to decreasing water depth. A decreasing water depth, yields to a decreasing wave speed. So when a tsunami wave travels into progressively shallower water, the wave energy will be concentrated, causing the wave to steepen and rise to many meters in height [Bosboom and Stive, 2015]. The relation between the wave height \( H \) and the water depth \( d \) is known as the Green’s law [Equation A.5] [Camfield, 1980]. Where subscript 1 stands for location 1 and subscript 2 for location 2.

\[
\frac{H_2}{H_1} = \left( \frac{d_1}{d_2} \right)^{0.25}
\]  

(A.5)
Tsunami Wave Breaking

The wave shoals until the wave becomes too steep or too high and starts to break. The wave steepness depends on the wave height \( H \) and the wave length \( L_w \). In the report of S.T. Grilli et al. [1997] the breaking characteristics for solitary waves on a slope are discussed. A Non-dimensional parameter \( S_0 \) is derived to predict whether waves will break or not, and which type of breaking will occur.

In the report, the slope parameter \( S_0 \) is used, which depends on the slope \( s \) and the initial wave height \( H_0 \).

\[
S_0 = 1.521 \cdot \frac{s}{\sqrt{H_0}}
\]  

(A.6)

The breaker type can be estimated in terms of values of the parameter \( S_0 \) as [Grilli et al., 1997]:

- Surging breaker: \( 0.30 < S_0 < 0.37 \)
- Plunging breaker: \( 0.025 < S_0 < 0.30 \)
- Spilling breaker: \( S_0 < 0.025 \)

The tsunami wave could eventually break into multiple solitons, called split waves. This phenomenon, known as ‘tsunami soliton fission’ is also observed during the Nihonkai-Chubu earthquake tsunami in 1983. Short waves split from the tsunami rest due to non-linearity and dispersion. The new leading wave increases and breaks. Eventually these split waves disappears and an incoming bore is formed [Figure A.5] [Matsuyama et al., 2007].

![Figure A.4: Different type of breakers [Bosboom and Stive, 2015]](image)

![Figure A.5: Time histories of water surface elevation for a case (t=20s, s=1:200)](image)[Tadepalli and Synolakis, 2010]
Tsunami Wave Dissipation

Normal wind waves lose most of their energy in the surf-zone during breaking [Equation A.7]. Tsunami waves on the other hand, retain a significantly amount of energy leading to high run-up heights and inundation depth.

\[ H(x) = \gamma h(x) = \text{const.} \]  \hspace{1cm} (A.7)

In the report of Battjes [1986] an energy dissipation model for breaking solitary waves is derived for gentle slopes \([s < 1 : 30]\), which corresponds with a spilling breaker [Equation A.6]. The dissipation rate of the spilling breaker is estimated from a bore with the same height of the spilling foam region. Observations of breaking waves in constant or increasing depth, suggest that the height of the foam region decreases more rapidly than the total wave height [Battjes, 1986].

A dissipation formula is derived from the energy balance and can be written in function of the breaker conditions. The formula [Equation A.8] resembles the dissipation formula for normal wind waves [Equation A.7], where the breaking index \(\gamma\) is replaced by the dissipation factor \(K'\) which depends on the bottom and spilling conditions.

\[ \tilde{H}^{-\frac{2}{3}} = (1 - \frac{1}{3}K')\tilde{h} + \frac{1}{3}K'\tilde{h}^{-\frac{2}{3}} \]  \hspace{1cm} (A.8)

Where the non-dimensional parameters \(\tilde{H}\) and \(\tilde{h}\) depends on the breaking wave height \(H_b\) and corresponding depth \(h_b\):

\[ \tilde{H} = \frac{H}{H_b} \quad \tilde{h} = \frac{h}{h_b} \]

The Battjes model is checked by comparing data for solitary waves breaking on a slope of 1:100, illustrated by the solid line in Figure A.6, from [Street and Camfield, 1966]. Choosing \(K' = 32.4\) gives a curve which in most of the decay region agrees remarkably well with the data. It must be noted that this Battjes model does not predict the fitted hyperbola of "observed" surviving wave height at the shoreline, from the data of the report of Street and Camfield [1966].

![Figure A.6: Decay of solitary waves near the shoreline [Battjes, 1986]]
The main advantage of the Battjes model is that the dissipation model depends only on the breaking wave height \( H_b \) and the breaking depth \( h_b \) with a dissipation factor \( K' \) equal to 32.4. However, the model does not predict the incoming wave height at the shoreline \( H \). Thus, the Battjes formula is only used to check if the modelled SWASH wave dissipates energy in the breaking zone.

**Tsunami Wave Run-up**

Tsunamis are known for their dramatic run-up heights [Bryant, 2008]. This is the distance the wave travels inland.

The run-up of a modelled tsunami wave on a linear slope can be quasi-analytically derived for both a solitary wave [Synolakis, 1987] and an N-wave [Tadepalli and Synolakis, 1994]. Several studies have been conducted to describe the relation between the offshore wave conditions and the amount of run-up on shore. However, these relations are for non-breaking waves and therefore no analytical model is proposed to approximate the run-up of a solitary- or N-wave.

**A.1.3 Definition of Wave Terms**

Many observations of the Tohoku tsunami of 2011 state that the tsunami wave was approximately 20 meters. But it is rather unclear where this wave is measured, offshore, onshore and with or without influence of protection measures. Therefore, the following wave definitions will be used for this research, based on Figure A.7.

![Figure A.7: Explanation of the wave terms] [Okumura, 2016]

The initial wave height \( H_0 \) is the wave before the shoaling or breaking process. This wave height is related to the soil deformation due to the earthquake.

The incoming wave height \( H \) is measured at the shore, just before the dike. So the wave height is not affected by existing protection measures along the coast.

The water levels which are influenced by the protection measures, will be referred as the maximum inundation height \( H_i \). The length of the inundated area, is the run-up distance of the wave.
A.2 Tsunami Wave Loads

A tsunami barrier must be able to withstand different type of wave loads. In this section three types of wave loads are analysed and wave theories are proposed which are based on wave loads for vertical walls.

A.2.1 Secondary Wave Loading

One of the characteristics of the tsunami wave is that the wave breaks into several solitions, known as ‘tsunami solition fission’ [Matsuyama et al., 2007] [Section A.1.1]. Therefore a tsunami barrier must be able to withstand secondary wave loading which occurs after the initial impact.

A preliminary method is taken from the linear wave theory [Vrijling et al., 2011] to calculate the loads on a vertical wall due to non-breaking waves. The pressure distribution \( p \), in case of full reflection, can be divided in two distributions [Figure A.8], from the toe of the structure \( z = h \) till the mean sea level \( z = 0 \) and from this latter point till the top of the incoming wave \( z = -H_{\text{wave}} \):

\[
p = \rho_w \cdot g \cdot H_{\text{wave}} \cdot \frac{\cosh(k_w(h+z))}{\cosh(k_w \cdot h)} \quad \text{for} \quad h < z < 0
\]

\[
p = \left(1 - \frac{z}{H_{\text{wave}}} \right) \rho_w \cdot g \cdot H_{\text{wave}} \quad \text{for} \quad 0 < z < -H_{\text{wave}}
\]

In which \( H_{\text{wave}} \) is the height of the secondary wave, \( k_w \) the wave number [Equation A.10] and \( L_w \) the length of the incoming secondary wave, and \( h \) is the depth in front of the tsunami barrier.

\[
k_w = \frac{2\pi}{L_w}
\]

Tsunamis can be characterised as very long waves \( L_w = >> \), which corresponds with a very small wave number \( k_w \approx 0 \). As a result, the hyperbolic cosine tends to 1.0 and the pressure distribution becomes constant:

\[
p_w = \rho_w \cdot g \cdot H_{\text{wave}}
\]

Figure A.8: Pressure distribution for secondary wave loading for non-breaking waves on a vertical wall[Vrijling et al., 2011]
A.2.2 Overflowing

A failure mechanism for the tsunami walls during the Tohoku tsunami of 2011 was wave overflowing [Figure 1.1]. The tsunami barrier must be able to withstand a Level 2 tsunami wave, where overflowing is allowed [Section A.3].

Miche-Rundgren [1958] derived formulae for non-breaking wave forces for vertical walls of low height [Coastal Engineering Research Center, 1984]. When the overflowing is not too severe, the majority of the incident wave will be reflected and the resulting pressure distribution is as shown in Figure A.9. This results in a truncated pressure distribution.

![Figure A.9: Pressure distribution for overflowing of the tsunami wave](image)

The pressure at the bottom is equal to \( p_0 + p_1 \), where \( p_0 \) is the hydrostatic pressure:

\[
p_0 = \rho_w \cdot g \cdot h \tag{A.12}
\]

And \( p_1 \) depends on the wave conditions and can be approximated by:

\[
p_1 = \left( \frac{1 + \chi}{2} \right) \cdot \rho_w \cdot g \cdot \frac{H_{\text{overflow}}}{\cosh(2\pi h/L_w)} \tag{A.13}
\]

Where \( \chi \) is the reflection coefficient, \( h \) the height of the retaining wall, \( H_{\text{overflow}} \) the height of the overflowing wave and \( L_w \) the length of the incoming wave. In case of full reflection \( \chi = 1 \) and for a tsunami wave \( L_w \gg \), the distribution becomes linear and \( p_1 \) becomes:

\[
p_1 = \rho_w \cdot g \cdot H_{\text{overflow}} \tag{A.14}
\]

The pressure on top of the wall \( p_2 \) can be calculated with simple geometric relation:

\[
p_2 = \rho_w \cdot g \cdot ((h + H_{\text{overflow}}) - h) \tag{A.15}
\]

So the pressure distribution becomes a truncated linear distribution.
A.2.3 Bore Impact

The incoming tsunami wave could develop into a bore and will give an impact force on the tsunami barrier. This bore impact can be calculated by two different methods, both for impact loads on a vertical wall. A first approximation can be obtained by the theory derived by Ramsden [1990]. This theory is solely based on the height on the incoming bore \( H_{\text{bore}} \). The second method is a calculation procedure based on the bore profile. This method is recommended by the Federal Emergency Management Agency (FEMA) [2012].

**Theory of Ramsden**

In the report of J.D. Ramsden [1990], the impact bore forces on a vertical wall for a broken solitary wave were measured in a laboratory. The measurements show a peak load just after the initial impact of the bore [Figure A.10b]. This is where the bore reflects [Figure A.10a] and the surge profile \( \eta \) is maximum.

![Figure A.10](image)

Figure A.10: Impact load by the incoming bore; (a) Expended view of incident bore and run-up, shown with solid and dashed lines, (b) Normalised Energy Spectrum of the impact force [Ramsden, 1990]

It is concluded that for large bores \( H_{\text{bore}} > 8 \text{ m} \) the expression proposed by Cross [1967] [Equation A.16] was in reasonable agreement with the maximum measured force \( F_T \) within five percent:

\[
F_T = \gamma_w \cdot \frac{g \cdot b \cdot H_{\text{bore}}^2}{2} \left( \frac{\eta}{H_{\text{bore}}} \right)^2 + C_F N_F^2 \left( \frac{\eta}{H_{\text{bore}}} \right)^2
\]  

(A.16)

Where \( b \) is the width of the wall and \( C_F \) is a force coefficient depending on the angle of the incoming bore profile \( \theta \).

It is shown experimentally that the non-dimensional bore celerity \( N_F \) can be considered constant and depends on the bore celerity \( c \) and the height of the bore \( H_{\text{bore}} \) [Equation A.17]. For the majority of the analysed bores, it was found that the non-dimensional bore celerity is approximately equal to a value of 1.8.

\[
N_F = \frac{c}{\sqrt{gH_{\text{bore}}}} \approx 1.8
\]  

(A.17)

Equation A.16 can be simplified by using \( \eta = H_{\text{bore}} \) and \( C_F = 1 \) together with the found value for \( N_F \) and the maximum force due to the bore impact can be estimated by 7.5 times the hydrostatic force of the incoming bore height.
Theory of FEMA

In the report of FEMA [2012], it is stated that for structural wall elements of significant width, it is recommended that the impulsive force \( F_s \) be taken 1.5 times the hydrodynamic force \( F_d \). Where the hydrodynamic force can be computed with the following equation:

\[
F_d = \frac{1}{2} \cdot \rho_w \cdot C_d \cdot b \cdot (Hu^2)_{\text{max}} \tag{A.18}
\]

Where \([C_d]\) is the drag coefficient and the combination \([Hu^2]\) represents the momentum flux of the incoming bore profile, per unit mass per unit width. The drag coefficient may be conservatively taken as \([C_d = 2.0]\). The parameter \([(Hu^2)_{\text{max}}]\) depends on the bore profile and the corresponding velocity profile. These can be derived from the numerical SWASH model [Chapter B].

Reduction of the Tsunami Inundation Force

In the paper of S. Thomas and D. Cox, predictive equations for the reduction of the tsunami inundation force were derived for finite-length seawalls. These equations were based on a hydraulic model experiments [Figure A.11]. An empirical formula is derived to predict the reduction factor for the maximum force due to the impact of the bore:

\[
C_{\text{bore}} = -0.331 \left( \frac{H_{\text{wall}}}{H_{\text{bore}}} \right) + 0.027 \left( \frac{H_{\text{wall}}}{H_{\text{bore}}} \right)^2 + 0.341 \left( \frac{x}{L} \right) - 0.076 \left( \frac{x}{L} \right)^2 + 1.109 \tag{A.19}
\]

Where the factor \([\frac{x}{L}]\) represents the location of the wall, relative to the shore [Figure A.11]. The parameters \([H_{\text{wall}}]\) and \([H_{\text{bore}}]\) represent the height of the wall and the bore respectively. It is stated that these equation is only valid for the data ranges \(0.31 < H_{\text{wall}}/H_{\text{bore}} < 3.1\) and \(0 < x/L < 0.875\).
A.3 Tsunami Flood Protection Plan

After the Tohoku tsunami of 2011, the flood risk management of Japan was improved. First of all, a new tsunami categorisation is proposed [Section A.3.1], which determines the design return period for the 'Tsunami Catcher'. Also a new flood risk management concept is introduced, the Multi Safety Layer (MLS) system [Section A.3.2], which show the location of the barrier in a total coastal defence system.

A.3.1 Tsunami Categorization

After the Tohoku tsunami of 2011, the flood risk management was improved, and two different tsunami levels have been identified [Shibayama et al., 2013]. These tsunami levels are based on political decisions, which some insight from coastal engineers from governmental institutes [Okumura, 2016].

Level 1 tsunami events have a return period of several decades to 100+ years. The coastal structures must protect human lives and property against Level 1 tsunami events. So the membrane barrier must fully retain a Level 1 tsunami.

Level 2 tsunami events are more rare, taking place at intervals between every few hundred, to a few thousand years apart. The idea that hard measures can always protect against the loss of life has been discarded. Instead, evacuation buildings and tsunami shelters will be designed. For the primary defences, no structural failure must occur during a Level 2 tsunami event, however overflowing and overflowing can be observed.

A.3.2 Multi Layer Safety System

The Tohoku tsunami of 2011 severely affected the city of Sendai. The flood protection in Sendai were mainly seawalls and coastal dikes. The retaining height of these structures was based on the most extreme event in the past which was at most 3 to 6 meter. However, the tsunami in 2011 had a maximum inundation height of 19.5 meter. To have a better protection for future tsunamis, a Multi-Layer Safety (MLS) system is currently begin implemented and constructed [Okumura, 2016].

The MLS is a flood risk management concept that introduces the integration of probability reducing and loss-mitigating measures in a flood protection system. The multilayer safety classifies measures into three safety layers [Jongejan et al., 2012]:

- Layer 1: Prevention; Prevents river or sea water from inundating areas which are usually dry. This is done by building a flood defence structure or by preventing the cause of inundation;
- Layer 2: Spatial solutions; Decreases the loss by spatial planning and adaptation of buildings when a flood occurs;
- Layer 3: Focuses on the organisational preparation for floods such as disaster plans, hazard maps, early-warning systems, evacuation, temporary physical measures.
A cross-sectional view of the MLS concept for Sendai City is given in Figure A.12. Layer 1 measures include the coastal embankment, coastal breakwater and coastal disaster-prevention forest. The layer 2 measures mainly focuses on building houses away from the disaster risk areas and restoring damaged residential land by heightening them. Layer 3 measures are concentrated on evacuation. This measures implies education on evacuation procedures, building of evacuation roads and creating more high ground evacuations centres.

Figure A.12: Cross-sectional view of the MLS concept for Sendai, Tohoku [Okumura, 2016][Sendai City Post Disaster Reconstruction division, 2011]

For the case study of Kamakura it is assumed that the 'Tsunami Catcher' is an element of a MLS system. Furthermore it is assumed that the barrier is part of the primary coastal defence system, so a Layer 1 measure. It must be able to retain a Level 1 tsunami wave and withstand a Level 2 tsunami wave, where overflowing can occur.
A.4 Development of Hydraulic Membrane Structures

One of the first hydraulic membrane barriers was developed by J.K. Vrijling in the early 80’s, the so-called Spinnaker Barrier [Section A.4.1], which functions as a storm surge barrier. This concept was the inspiration of the Kite barrier [Section A.4.2], developed by F. van der Ziel. Finally this idea is evolved into the design of the 'Tsunami Catcher’ [Section A.4.4]. Also another type of membrane barrier is discussed, the Ramspol barrier [Section A.4.3] which functions as a storm surge barrier. In this section, a short description of the system is given, even as the design challenges.

A.4.1 Spinnaker Barrier

The Spinnaker Barrier was one of the first barrier, where the vertical stability was ensured by the use of floating bodies [Figure A.13a]. Unlike the 'Tsunami Catcher’, the Spinnaker Barrier is not automatically deploying. When storm conditions are predicted, the fabric is pulled horizontally by cables over the entire width of the water [Regeling, 1989].

The concept of the barrier was tested experimentally in 1989 by H.J. Regeling. It was concluded that the principle of the Spinnaker Barrier works and functions as a stable barrier in the retaining phase of the operation. Problems arise during the transport and closure phases [Regeling, 1989].

A.4.2 Kite Barrier

The Msc. Thesis of F. van der Ziel [2009] considers the investigation of an innovative barrier design, namely that of an open fabric moveable flood water barrier. Which can be seen as a parachute opened horizontal in the water way [Figure A.13b]. In the initial design, the vertical stability was guaranteed by the use of floating bodies. But it was concluded that a barrier where only floating bodies are used for the vertical load distribution, seems to be unpractical due to the needed size of these floating bodies [van der Ziel, 2009].

![Figure A.13: Principle of the (a)Spinnaker, (b)Kite Barrier and the (c)Ramspol Barrier](image-url)
A.4.3 Ramspol Barrier

In 2012, the first inflatable flood barrier was realized in Ramspol. Despite that this barrier is not a tsunami barrier, this structure is an interesting reference project to study. Similar to the ‘Tsunami Catcher’, the Ramspol Barrier is a membrane-type structure. The membrane is a fiber-reinforced fabric connected to a foundation and is manually inflated by creating an over-pressure in an enclosed space, by means of air and water [Figure A.13c].

Also in the design phase of the Ramspol Barrier, there were a significant amount of uncertainties. Nowadays, safety factors of 9 are used in the design process. This is due to the highly deformable character of the structure. For example, there are folds in the membrane at the abutments, which could create peak stresses in the membrane [Horsten, 2015].

A.4.4 Tsunami Catcher

The flexible membrane tsunami barrier consists out of different components and phases. The main retaining component is the membrane [Element 3; Figure A.14], which is attached to a floater element [Element 4; Figure A.14], which keep the membrane up by the buoyancy of the floater element. The floater is kept in place by cables [Element 5; Figure A.14], which are placed offshore and are secured to an offsho foundation [Element 6; Figure A.14]. The membrane is attached to a bottom recess structure [Element 2; Figure A.14] which is founded on piles [Element 1; Figure A.14]

![Figure A.14: Overview of the different components and the connections](image)

During normal conditions, the membrane and the floater are stored in the bottom recess [Element 2; Figure A.14], and the barrier is inactive. When a tsunami hits the shore, the structure becomes active and the floater rises with the water level. Once the incoming wave is fully reflected, the ‘Tsunami Catcher’ is fully inflated. Finally, when the tsunami is passed, the barrier will deflate.

There must made an adjustment to allow overflowing of the barrier. This due to the buoyancy of the floater, which will keep rising until the membrane collapse. Therefore it is assumed that there is a cable-type structure which keep the floater at maximum crest height of a Level 1 tsunami (16m) [Figure A.14]
A.5 Case Study: Kamakura

The case study is applied to the city of Kamakura, a coastal city located in the Sagami Bay (population is 173,500), approximately 50 kilometers South-East of Tokyo [Figure A.15]. This bay is in direction connection with the Pacific Ocean and is located closely to the three convergent boundary plates which are known for its tectonic activity. Sagami bay has therefore a history of tsunamis.

The city is famous for its historical monuments and its beaches, and has therefore a large cultural and touristic value. A flexible-membrane tsunami barrier could be a proper tsunami counter measure, without reducing this touristic value.

![Figure A.15: Overview of Kamakura, Japan [Google Maps (2016)]](image)

In this section, the case study location is further analyzed. Goal is to distinguish the different boundary conditions. Therefore the following subjects are studied:

1. History of Tsunamis:
2. Environmental Analysis:
3. Soil Conditions:
4. Normal wave conditions:
A.5.1 History of Tsunamis

In the past centuries, several tsunamis occurred in the Sagami Bay which affected the city of Kamakura. In the Msc. Thesis of N. Okumura [2016], these data are used to determine the design tsunami wave for the Level 1 and Level 2 tsunami event for Kamakura.

The return period $[t_{rp}]$ is approximated with the method of regression, where the maximum incoming tsunami wave heights $[H]$ [Figure A.7] are derived from the governmental tsunami hazard maps [Kanagawa Prefecture, 2012]. This is done for seven historical tsunami events [Table A.1]. It was concluded that the Reverse-Weibull type of Generalised Extreme Value (GEV) distribution line had a good statistical fit with the calculated, approximated return periods [Figure A.16]. This method can cause inaccuracies for extrapolating data, however, the accuracy still holds within a range not far from the plotted values [Okumura, 2016].

Table A.1: Approximated return period $[t_{rp}]$ based on the expected maximum incoming tsunami wave height $[m]$ for the historical tsunami events [Okumura, 2016]

<table>
<thead>
<tr>
<th>Year</th>
<th>Name of Event</th>
<th>Time since occurrence</th>
<th>Expected $H$</th>
<th>Expected $t_{rp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1498</td>
<td>Meiou</td>
<td>518</td>
<td>12.9</td>
<td>260</td>
</tr>
<tr>
<td>1605</td>
<td>Keicho</td>
<td>411</td>
<td>14.5</td>
<td>519</td>
</tr>
<tr>
<td>1633</td>
<td>Sagami Bay</td>
<td>383</td>
<td>8</td>
<td>104</td>
</tr>
<tr>
<td>1703</td>
<td>Genroku</td>
<td>307</td>
<td>9.9</td>
<td>173</td>
</tr>
<tr>
<td>1782</td>
<td>Sagami Bay</td>
<td>234</td>
<td>7.3</td>
<td>87</td>
</tr>
<tr>
<td>1854</td>
<td>Ansei Tokai</td>
<td>162</td>
<td>3.8</td>
<td>74</td>
</tr>
<tr>
<td>1923</td>
<td>Kanto</td>
<td>93</td>
<td>9.2</td>
<td>130</td>
</tr>
</tbody>
</table>

Based on the distribution line of Figure A.16, the maximum wave height of the incoming tsunami wave for a Level 1 event $[t_{rp} = 100\ years]$ is equal to 8 meter and the wave height for a Level 2 event $[t_{rp} = 1000\ years]$ is equal to 14 meters.
A.5.2 Project Location Analysis

Bathymetry of Kamakura

The water depth in the middle of the Sagami Bay varies between 1000 and 1500 meters depth [Figure A.17a]. The East coast of the bay has very steep beaches. However, in the North, North-East of the bay, close to Kamakura, the bay can be characterised as a shallow water area. Where the waters around Kamakura Beach can be seen as a small bay with shallow waters, approximately 5-10 meters deep.

Figure A.17: Overview of Kamakura Beach (a)Bathymetry of Kamakura; (b)Environmental view of Kamakura Beach
Environmental Analysis

The beach is approximately 2000 meters long with an average width of 60 meters. The beach is enclosed by a main road [Red; Figure A.17b] which is located approximately three meters above mean water level [Figure A.17b]. Directly behind the main road, starts the city of Kamakura. On the beach itself, there is hardly any construction, only some small storage cabins for fishery and such [Green; Figure A.17b]. In the middle of the beach there is a small water outlet [Blue; Figure A.17b]. The beach is enclosed by a small dike [3 meters] whereupon a main road is located. An impression is given in Figure A.18.

Figure A.18: Design cross-section for Kamakura

A.5.3 Soil Conditions

The soil conditions are derived from a standard penetration test [SPT] close to the project location. The measurements are obtained from the Institution of Kanagawa Prefecture [2016]. In a SPT, a sampling tube is driven into a borehole in the ground using a standardised hammering weight. The actual test consists of measuring the number of blows needed to achieve a penetration of 300mm (1 foot) into the ground. This is denoted as $N$, the blow count, the numbers of blows per foot [Verruijt, 2012].

From Figure A.19, it can be assumed that from a depth of approximately 3.5 meters, the soil is homogeneous. To distinguish which type of soil corresponds with the N-value, the data of Terzaghi and Peck can be used. It is stated that a N-value greater than 50, corresponds with dense or very dense sand [Verruijt, 2012].

For the design calculations, it is assumed that the entire soil column consists of fine sand. However, if the top layers are weak, it will be replaced by sand. Furthermore it is assumed that the general soil profile applies for the entire project location, both onshore as offshore.
The SPT is not commonly used in the Netherlands. There the so-called Cone Penetration Test [CPT]. This latter method consists of pushing a steel rod into the soil and measure the force in function of the depth [Verruijt, 2012]. This force, the cone resistance \(q_p\) is used to determine the soil structure and serves as input for the design of the foundation. Therefore it is important to try to link the N-value of the SPT to the cone resistance of the CPT.

Many researches have tried to obtain a correlation between the SPT and the CPT but their results are not very consistent [Verruijt, 2012]. However, there are some commonly used ratios to approximate the cone resistance. For sand this ratio lays between the 0.4 and 0.5 [van Tol, 2003]. So:

\[
q_c \approx 0.4 \cdot 50 \approx 20[MPa]
\]  

Besides the cone resistance, some other soil characteristics can be estimated based on the results of the SPT. These soil properties are derived from the data of the NEN 6740 (1990). First of all the density \(\gamma\), for dry sand this is equal to 18 kN/m\(^3\) and for wet sand, equal to 20 kN/m\(^3\). Furthermore it is assumed that there is no cohesion \(c = 0\) and that the angle of internal friction \(\phi\) is estimated on 30°.

Note, that these assumptions were made, based on one SPT. Therefore any calculations or conclusions concerning the bearing capacity of the soil must be made with care. Therefore it is recommended to do a more thorough soil investigation.
A.5.4 Alternative Wave Conditions

The tide conditions for Kamkarua are derived from the data obtained from two buoy stations from the Japan Oceanographic Data Center [2016][Figure A.20a]. Two months of wave data are analyzed [Figure A.20b-A.20c] of station 1 and station 2. From the data, it can be concluded that the tide can be characterized as a mixed type, predominantly semi-diurnal [Bosboom and Stive, 2015]. Furthermore, based on the analyzed data, the maximum high water level is approximately +90 cm above mean water level and the minimum low water level is approximately −100 cm below mean water level.

![Image](image.png)

**Figure A.20**: Derivation of the normal tidal conditions for the Sagami Bay [Japan Oceanographic Data Center]; (a)Location of the buoy station, (b)Tidal data for the month January for Station 1, (c)Tidal data for the month May for station 2

In the past, Japan has encountered any devastation typhoons such as the Muroto Typhoon (1934) and the Ise Bay Typhoon (1959). In an extreme year, the number of typhoons which hit Japan can increase to ten typhoons [Tian, 2014]. These typhoons can cause large storm surges. However, it can be concluded that the height of the storm surge is rather pale, compared to the maximum crest height of the incoming tsunami wave.

A.6 Conclusion

From the analysis of the relevant background information, the following conclusions can be made. These are subdivided in assumptions, requirements for the tsunami barrier, boundary conditions for the case study of Kamakura and the wave theories which are used.
**Requirements**

1. The barrier is a part of a Multi-Layer Safety system, functioning as primary barrier in Layer 1 and must be able to:
   - (a) Fully retain a Level 1 tsunami (return period = 100 years);
   - (b) Withstand a Level 2 Tsunami (return period = 1000 years), where overflow is allowed and scour does not undermine the stability of the barrier.

2. The dimensions of the barrier must be minimised in order to integrate the barrier in the surroundings.

**Assumptions**

1. The density of the tsunami water \( \rho_w \) is assumed to be equal to 1100 \( kg/m^3 \) [FEMA, 2012];
2. The possible storm surge of typhoons is inferior to the incoming design tsunami wave.
3. The height of the undisturbed incoming Level 1 Tsunami wave \( H_{Level1} \) is assumed to be equal to 8 meters [Okumura, 2016] [Section A.5.1];
4. The height of the undisturbed incoming Level 2 Tsunami wave \( H_{Level2} \) is assumed to be equal to 14 meters [Okumura, 2016] [Section A.5.1];
5. The soil is assumed to be homogeneous consisting out of fine sand with a volumetric weight of 18 \( kN/m \) and a wet volumetric weight of 20 \( kN/m \), with no cohesion and an angle of internal friction of 30° [Institute of Kanagawa Prefecture (2016)].

**Boundary Conditions Kamakura**

1. The schematised design cross section of the project location that is used to make the design is given in Figure A.18;
2. The bathymetry of Kamakura is given in Figure A.17b;
3. The maximum high water level is + 90 cm and the minimum low water is -100 cm below mean water level [Japan Oceanographic Data Center];

**Wave Theories**

1. The modelled, design tsunami wave [Chapter 3] will be approximated by a Solitary Wave Theory and a N-Wave Theory;
2. The shoaling effect of the design tsunami wave will be validated by the Green’s Law;
3. The type of breaking for the design tsunami wave will be validated by the slope parameter \( S_0 \), proposed by S.T. Grilli et al [1997];
4. Dissipation of the design tsunami wave will be validated by the dissipation model proposed by Battjes [1986];
5. The run-up of the design, tsunami wave will be validated by observational data;
6. The linear wave theory for wave loading on a vertical wall, will be applied to approximate the pressure distribution;
7. The Miche-Rundgren theory [1958] for non-breaking wave forces for vertical walls of low height will be applied to estimate the pressure distribution due to overflowing;
8. The bore impact will be approximated by the theory of Ramsden [1990] and the theory of FEMA [2012].
Tsunami Wave Analysis for Kamakura

This Appendix is made in collaboration with N. Okumura

B.1 Selection of the Wave Theories

In Section A.1.1 a Solitary Wave theory a the N-Wave theory are analysed. Both theories have their positive and negative aspects. To determine which wave theory is best applicable for Kamakura, a numerical model will be drafted for the reference case of Sendai City. Then both waves are validated by observational data and analytical calculations. After analysing the results, a best fit is chosen for the case study of Kamakura.

The selected wave is intended to illustrate the characteristics of the first wave, or in other words, the wave at first impact. It is very difficult to portray the entire tsunami wave accurately, for example, prior to the main wave of the Tohoku tsunami of 2011, small bores were noticed. Hence, this modelled wave is generated for a first approximation, allowing for many improvements to be done for further research.

Another approach is to model the entire depression of the sea floor. This method is regarded the best method to model the tsunami, and can include the 3-dimensional effects [Smith et al., 2016]. However, for the preliminary design, a 1-dimensional wave model (SWASH) is used.

B.2 Analytical Description of the Wave Deformations

The SWASH model can be validated by studying the wave deformation for both the solitary as the N-wave. The deformation is characterised by five stages, namely [Figure A.2]:

- Generation of tsunami waves,
- Shoaling of tsunami waves,
- Breaking of tsunami waves,
- Dissipation of tsunami waves,
- Run-up of tsunami waves.

To validate the model, analytical models are proposed [Section A.1.2]. The shoaling of the wave can be approximated by the Green’s Law [Equation A.5], the type of breaker can be estimated by the theory of S.T. Grilli (1997) and the model of Battjes (1986) [Equation A.8] is used to validate the dissipation of the wave. The generation and run-up of the tsunami wave will be validated by observational data.
B.3 Reference Case Study: Sendai

The city of Sendai is chosen as a reference case study, because a lot of data and measurements are available after the Tohoku tsunami of 2011. Sendai has similar bathymetry characteristics as Kamakura, both have a bay type of coast and the average slope near shore is roughly 1 : 100 [Figure B.1]. The bathymetry data is obtained from the General Bathymetric Chart of the Ocean (GEBCO)[2016], and the topography data is obtained from USGS [2016].

The first step to calibrate the model, is to analyse and choose the right data from observational data and other sources, in order to derive the wave boundary conditions [Section B.3.1]. Both the N-wave and the Solitary wave are modelled [Section B.3.2], and validated with the observational data and the suggested analytical models [Section B.3.3]. Based on the validation, a conclusion can be made for which wave type is appropriate to be used to model the case study of Kamakura [Section B.3.4].
B.3.1 Boundary Conditions and Observation Data

The Tohoku tsunami of 2011 was recorded by the ocean bottom pressure and GPS wave gauges, deployed in and around Japan [Saito et al., 2011]. Several buoys measured the incoming tsunami wave [Kawai et al., 2013] and a survey along the coast estimated the inundation of the coast [Mori et al., 2011]. In this section this data is analysed.

The initial tsunami wave height is derived from the inversion analysis of the buoy and satellite data. Figure B.2 show the outcome of this inversion analysis. There is a large water level rise approximately 250 kilometers away from the coast of Sendai, and a slight water level drop 100 kilometers away from the coast of Sendai.

![Initial tsunami wave height distribution](image)

This whole rise and drop of the water level for the cross-section can be modelled in a time-series as one large wave. To model this initial tsunami as accurate as possible, a wave of approximately 120 kilometers in length and a period of approximately 30 minutes is generated. At the starting node of the boundary, the period of the modelled tsunami is close to that of the data, however, it is difficult to create a time series of a wave with a trough and crest far away from each other. Therefore the available time series, measured by buoys are used to determine the final length and period of the tsunami wave.

These buoys are deployed by the Port and Airport Research Institute (PARI), which were able to collect the time series of the 2011 tsunami [Kawai et al., 2013]. The location of buoys GB801 and WG205, also illustrated in Figure B.1a, are obtained in coordinates from PARI. The given coordinates for GB801 are 38° 13’ 57”, 141° 41’ 01”, and for WG205 are 38° 15’ 00”, 141° 03’ 58”, and are illustrated to scale. There is also data available for other buoy stations along the coastline of Tohoku, but for this thesis, only these two buoy stations are analysed.

The time series of the buoy GB801 [Figure B.3], located 60 kilometer off the coast of Sendai was used, to determine the final length and period of the initial tsunami wave.
The assumed tsunami path for the numerical calculations, does not go through the buoy stations [Figure B.1a]. The relative depth of the assumed tsunami path is within the same depth contours as the buoy stations. So the measurements of the buoy stations can be used to validate the model. Furthermore, the tsunami path is chosen such that it is not incorporated by the islands and shallow parts of the North.

The 2011 Tohoku Earthquake Tsunami Joint Survey Group has comprehensively recorded the inundation height and its impact along the affected coastal region. Observations were made on the maximum inundation height for different locations along the Tohoku coastline. Here, the definitions which were specified in Chapter A.1.3 [Figure A.7] are kept, and the inundation height refers to the tsunami height with influence from flood protection measures, such as dikes and seawalls. The undisturbed wave height, or in other words, the incoming wave height, could not be measured. Based on the inundation heights recorded, the incoming tsunami wave height on the coast is inferred to be in the range from 5 to 15 meters in the Sendai plain depending on the location [Yamao et al., 2015].

Finally, the measured run-up distance and inundation heights are observed. Figure B.4 illustrates the envelop of the run-up distance with its relative inundation heights in Sendai City, which is surveyed by [Mori et al., 2011]. The relative inundation heights in multiple locations illustrated as plot can be found in Figure B.4.
B.3.2 SWASH Computations

The two SWASH models are drafted based on the derived offshore wave conditions. For a detailed description of the model and the time series of the solitary wave and N-wave is given in Appendix C.

To model a tsunami in SWASH, specific boundary conditions must be chosen. These boundary conditions are based on the previous work on solitary wave and tsunami modelling [The SWASH Team (2010-2015), 2015]. The boundary condition on the left, or what is referred to as the west side in SWASH, includes the wave time series which was created from Equation A.1 and A.3, with a weak reflection as it is one large wave which acts as a shallow water wave and has minimal disturbances that follow. On the right side, or the east side in SWASH, the Sommerfeld or radiation boundary condition is chosen.

The computation grid length is 140 kilometer with 20,000 cells, resulting in a resolution of cells with 7 meter. For different scenarios, which were run for Sendai, the number of cells were sometimes changed to 10,000 in order to increase the speed of the computations. This reduction in the resolution of the cell to 14 meter had negligible effect on the tsunami wave modelled. The SWASH simulations for this thesis are only modelled with one vertical layer. Including multiple layers will improve the accuracy of the wave development and the flow velocity.

Run-up distance computed in the SWASH model is influenced from the slope roughness set by a Manning’s coefficient of 0.019, which illustrates a roughness of a smooth earthy surface. This roughness coefficient is lower than what is recommended in [Bricker et al., 2015] for tsunami modelling, where the coefficient ranges from 0.08 to 0.17 for urban high density areas. However, for the simulations, an uniform Manning’s coefficient was implemented instead of its being site specific. The first impact of the tsunami wave would be on the sandy coast, thus a Manning’s coefficient for sand is chosen as a representative value. For the case of this thesis, the effect of this will not be looked at in detail, and will be considered as one of the steps to be included to improve the accuracy of the results in this chapter.

Figure B.5 shows the different stages of the wave deformation for the solitary wave. Note that Figure B.5c has a different scale, and is zoomed at the coast for a more detailed view of the wave.

Figure B.5: Results of the SWASH model for the solitary wave: a) Shoaling effect b) Breaking of the wave c) Bore formation
Figure B.6 shows a crest-leading N-wave theory for the reference case of Sendai. Also Figure B.6c has a different scale, and is zoomed at the coast for a more detailed view of the wave run-up.

![Figure B.6](image)

Figure B.6: Results of the SWASH model for a N-wave: a) Shoaling effect b) Breaking of the wave c) Bore formation

### B.3.3 Validation of the SWASH Models

The validation of the SWASH models is broken down for the different wave transformations, where the generation of the tsunami wave is based on the inversion analysis of the buoy and satellite data, as explained in Section B.3.1.

**Shoaling**

Shoaling of the modelled tsunami wave is validated with the Green’s Law [Section A.1.2] and with the measurements of buoy GB801 and WG205 [Figure B.3].

The Green’s Law effect ensures an increase of the amplitude of the incoming wave, when it approaches the shore, due to the decreasing water depth. This relation [Black Line; Figure B.7] is compared with the results of the modelled solitary wave [Blue Line; Figure B.7] and a N-wave [Red Line; Figure B.7]. Note that the effect for both modelled wave is stopped when the wave breaks. The breaking of the tsunami wave is further elaborated in the following section.

![Figure B.7](image)

Figure B.7: Maximum wave height during the simulation for the non-broken wave, compared with the Green’s law and the measured wave height at station GB801
From Figure B.7 it can be concluded that both modelled waves shoals to a wave height of approximately 9.5 meters. It also shows that the amount of shoaling for both the N-wave as the soliton is less than that of the analytical model. However, at the buoy location of GB801, the modelled maximum wave height for the N-wave is equal to 8.5 meter and for the solitary wave equal to 8.9 meter, which approximates to the measured wave height of 8 meter.

Breaking

The breaking of the tsunami wave can be characterised by the breaking wave height \( H_b \) with the corresponding water depth \( h_b \) and the type of breaker. The latter phenomenon can be estimated by the theory of S.T. Grilli [1997] by calculating the slope parameter \( S_0 \), Equation A.6. For both SWASH models, this value is less than 0.025 which corresponds which implies that the breaking wave is expected to correspond to a spilling breaker. The spilling breaker will eventually transform to an incoming bore. Both SWASH models fulfil this requirement [Figure B.5 and Figure B.6]. This result can be validated with the available video and photo material of the Tohoku tsunami of 2011 [Figure B.8].

This phenomenon, known as ‘tsunami soliton fission’ is also observed during the Nihonkai-Chubu earthquake tsunami in 1983. Short waves split from the tsunami crest due to nonlinearity and dispersion. The new leading wave height increases and breaks [Matsuyama et al., 2007][Figures A.5].

The 'tsunami soliton fission' also occurs in both SWASH models [Figure B.9]. The breaking location is defined when the first wave splits from the incoming wave.
Figure B.9: Snap shots of the different stages of the breaking wave for both the solitary wave and the N-wave. For the non-broken wave a), the first wave split b) and when multiple solitons are formed c)
Dissipation

A tsunami wave retains a significant amount of energy when it reaches the coastline. The energy dissipation for a breaking solitary wave for gentle slopes is studied by Battjes [1989][Section A.1.2]. A dissipation formula is proposed [Equation A.8] in function of the breaking parameters. This formula is used to check if the modelled SWASH waves dissipate energy in the breaking zone [Figure B.10. The maximum wave height of the incoming wave is plotted for both the N-wave [Red Line] and the solitary wave [Blue Line]. The dissipation formula of Battjes is implemented [Green Line] for the N-wave breaking conditions.

![Figure B.10: Comparison of the maximum wave heights of the breaking waves of the SWASH models with the analytical model of Battjes (1986) and the measured wave data](image)

The breaking of the tsunami wave is characterised by split waves, which create an increase of the wave height [Matsuyama et al., 2007]. This explains the increase of the maximum wave height just after the initial wave break for the N-wave and the solitary wave.

From Figure B.10 it can be concluded that the solitary wave hardly dissipates any energy. The N-wave however, dissipates energy between the breaking point and the coast. If the average maximum wave height of the N-wave is compared with the dissipation theory of Battjes, it can be concluded that both distributions have approximately the same slope between 25 and 2.5 kilometers for the coast. The total amount of dissipation energy of the N-wave does not coincide with the Battjes theory.
Run-up

The Tohoku tsunami of 2011 had a significant amount of run-up inland. It is already mentioned that the maximum incoming wave height observed at the Sendai coast lays in the range between 5 and 15 meter [Yamao et al., 2015]. The maximum measured run-up and inundation heights is shown in Figure B.11, and the maximum run-up distance lays between the 6 and 7 kilometers inland.

From the SWASH model results, the modelled run-up can be derived for the N-wave [Blue; Figure B.11] and the solitary wave [Red; Figure B.11]. Both models lie in the range of the measured inundation depths. The maximum run-up of the N-wave approximates the measured run-up. The solitary wave however, overestimates the run-up.

![Figure B.11: Comparison of the run up and inundation depth of both the SWASH models with the measured envelope [Grilli et al., 1997]](image)

B.3.4 Conclusion

From both SWASH models it can be concluded that the N-Waves is the best fit with the observed data of the Tohoku tsunami and the analytical models [Table B.1] and is therefore assumed that the N-wave model gives a good approximation of the incoming tsunami wave.

<table>
<thead>
<tr>
<th>Data [m]</th>
<th>N-wave [m]</th>
<th>Solitary Wave [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial wave height</td>
<td>6</td>
<td>6.2</td>
</tr>
<tr>
<td>Wave height just off-shore</td>
<td>6.5</td>
<td>7</td>
</tr>
<tr>
<td>Maximum inundation depth</td>
<td>5-15</td>
<td>10</td>
</tr>
<tr>
<td>Run-up</td>
<td>6500</td>
<td>7000</td>
</tr>
</tbody>
</table>

The N-wave has some drawback which must be mentioned. First of all, the mathematical background of the N-wave is a simple model to capture the essential characteristics. The solitary wave has explicit input parameters, while the N-wave is manipulated until the initial conditions are met. Also, the point of breaking for an N-wave cannot be clarified with the proposed analytical models. The changes in steepness and amplitude of an N-wave could significantly change the results which are seen on the shore. Nevertheless, the N-wave shoals and breaks into multiple solitons, eventually transforming into an incoming bore.
Another missing phenomena in the N-wave simulation, is a retreat of the water along the coast, which was observed by the WG205 buoy [Figure B.3]. An explanation can be that the design tsunami path [Figure B.1a] is not chosen correctly. This retreat could also be influenced by different coastal boundary conditions, hence, this missing phenomenon is not looked at in detail.

For the present aim of this thesis, the N-wave is taken as it captures the essential features as observed in 2011. Therefore, the N-wave will be used to model the tsunami for the case study of Kamakura.

Several improvements could be made to improve the model. First of all, a 2- or 3-dimensional model for real sea-bed changes for different tsunamis can be simulated. New tsunami models could be used, such as to include the direct geophysical impact to the water [Smith et al., 2016], allowing the initial tsunami wave to have its unique wave form. Secondly, different sensitivity analyses can be conducted to check the influence of different parameters. For example, the influence of the Manning’s coefficient on the run-up could be checked, and also insight on the relationship between the steepness and amplitude of an N-wave could be obtained to improve the results.

B.4 Case Study of Kamakura

The validated SWASH model is applied to the Kamakura case study. The first step is to define the path of the design tsunami wave [Figure B.12]. The tsunami path was chosen such that there is a smooth slope, avoiding large bars and troughs. The worst-case scenario occurs when the tsunami wave directly hits Kamakura in order to simulate the dynamic impact of the wave. Refraction is not taken into account when choosing the tsunami path.

The computational grid in SWASH for Kamakura is 55 kilometers, with 5,000 cells, resulting in a one dimensional resolution of 11 meter per cell. The remaining boundary conditions in SWASH are kept the same as in the case study for Sendai.

![Bathymetry and chosen design slope for Kamakura case study](image)

Similar to the reference case of Sendai, it is assumed that the tsunami resulting from an earthquake is an N wave with a leading crest.
In Section A.3.1 it is stated that a primary defence must protect the city from a tsunami wave defined with a return period of one in a hundred years, in other words a Level 1 event tsunami. For Kamakura, this wave height equals 8 meters [Okumura, 2016][Section A.5.1]. In the SWASH model, the initial wave conditions are varied until the incoming wave height input nearly coincides with this expected incoming wave height on the coast. Figure B.13 shows the incoming Level 1 tsunami wave, holding a characteristic like a bore.

![Figure B.13: Profiles of the modelled, incoming bore of the Level 1 tsunami for Kamakura, visualised for different time steps](image)

The height of the incoming bore of the Level 2 tsunami is equal to 14 meters [Okumura, 2016] [Section A.5.1]. Also the profile of this bore can be derived from SWASH [Figure B.14].

![Figure B.14: Profiles of the modelled, incoming bore of the Level 2 tsunami for Kamakura, visualised for different time steps](image)
The maximum velocity of the incoming Level 1 \( u_{\text{level},1} = 17 \text{ m/s} \) and Level 2 \( u_{\text{level},2} = 25 \text{ m/s} \) tsunami is significantly greater than the bore velocities used for the experiments of B. Hofland et al [2015]. It was concluded that for a 10 meter per second approach flow, initially some water was spilled over the barrier. It is therefore recommended to test the inflation process of the barrier for an incoming tsunami with a velocity of 25 meter per second.

The barrier must be able to withstand the tsunami, although overflowing is allowed. The amount of overflow can also be derived from the SWASH model. The barrier is modelled as a rigid, dike-type structure which resembles with an already inflated flexible membrane tsunami barrier [Figure B.15a]. The initial overflow could be somewhat conservative, but is a proper estimation for the first calculations.

The overflow is measured at the top of the dike [Dashed Line:Figure B.15a]. The maximum measured overflow is approximately 9 meters. However, at the first ten seconds an additional overflow was measured, with a maximum of 11 meters. This additional overflow is modelled as a secondary wave [Section G.2].
SWASH Equations

This Appendix is made in collaboration with N. Okumura

C.1 Governing Equations

The governing equations of SWASH are the non-linear shallow water equations which are derived from the incompressible Navier-Stokes equations which describe the conservation of mass and momentum [Rijnsdorp et al., 2014]. The SWASH model considers the depth-averaged form of the equations in Cartesian notation. The governing equations are [Zijlema et al., 2011]:

\[
\frac{\partial \eta}{\partial t} + \frac{\partial hu}{\partial x} + \frac{\partial hv}{\partial y} = 0 \tag{C.1}
\]

\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + g \frac{\partial \eta}{\partial x} + \frac{1}{h} \int_{-d}^{\zeta} \frac{\partial q}{\partial x} dz + cf \frac{u\sqrt{u^2 + v^2}}{h} = \frac{1}{h} \left( \frac{\partial h \tau_{xx}}{\partial x} + \frac{\partial h \tau_{xy}}{\partial y} \right) \tag{C.2}
\]

\[
\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + g \frac{\partial \zeta}{\partial y} + \frac{1}{h} \int_{-d}^{\eta} \frac{\partial q}{\partial y} dz + cf \frac{v\sqrt{u^2 + v^2}}{h} = \frac{1}{h} \left( \frac{\partial h \tau_{yx}}{\partial x} + \frac{\partial h \tau_{yy}}{\partial y} \right) \tag{C.3}
\]

Here, \( t \) is time, \( x \) and \( y \) are located at the still water level and the \( z \)-axis is pointing upwards. \( \eta(x, y, t) \) is the surface elevation measured from the still water level, \( d(x, y) \) is the still water depth, or the bottom level measured downwards from the still water level, and \( h = d + \eta \) is the total depth. \( u(x, z, t) \) and \( w(x, z, t) \) are the depth-averaged flow velocities in the \( x \)- and \( y \)-directions respectively, \( q(x, y, z, t) \) is the non-hydrostatic pressure normalised by the density. \( g \) is the gravitational acceleration, \( cf \) is the dimensionless bottom friction coefficient, and \( \tau_{xx}, \tau_{xy}, \tau_{yx}, \tau_{yy} \) are the horizontal turbulent stress terms.

When waves are travelling for a very long distance, in the order of several kilometers, the influence of bottom friction increases. Moreover, the long waves close to the shoreline, such as infragravity waves and nearshore circulations, are affected. Friction can be expressed in many ways, but for the case of this research, friction will be expressed based on Manning’s roughness coefficient \( n \), as this provides a better representation of wave dynamics in the surf zone. The expression is:

\[
f_f = \frac{n^2 g}{h^{1/3}} \tag{C.4}
\]

Along with the governing equations of the SWASH model, appropriate boundary conditions must be imposed in order to have a complete system. At the offshore boundary, different waves can be specified by local velocity distributions.
C.2 Boundary Conditions

For the validation of the reference case study in Sendai, a bore was observed near the shore. Initially, this large wave breaking caused many wiggles to develop, and the motive was to remove them. To be able to do this, the u-momentum and its advection term were looked at. The u-momentum term will be conserved throughout the simulations as that is what physically happens. Secondly, the advection term changes its order of accuracy from higher harmonics in the deep water, to second or even first order accuracies as it is breaking. This allows the reduction of wiggles when simulating a breaking wave such as a bore. These boundary conditions specifically for the SWASH model will be kept the same throughout the thesis, for both simulations for Sendai and Kamakura.

C.3 Time Series of Tsunami Waves

The time series of the solitary wave is first created in MATLAB, based on the wave equations which are introduced in Section A.1.1[Equation A.1]. It is worthy to note that the initial wave front illustrated in Figure C.1 slightly differs from the time series which was made in MATLAB and put into SWASH, as the wave is already getting influenced from the bottom. Here, it can be concluded that the solitary wave theory does not fully agree with the measured wave data. However, the modelled solitary wave does undergo the different wave transformations: the wave shoals, breaks and eventually runs-up.

Figure C.1: Time-series of the simulated initial wave front of the solitary wave at the most offshore location in transect
From the time series of the N-wave [Figure C.2] it can be concluded that the modelled initial wave form matches the measured wave form at buoy GB801 [Figure B.3].

Figure C.2: Time-series of the simulated initial wave front of the N-wave at the most offshore location in transect
**D**

**Loads**

This Appendix covers the analysis of the different loads of the 'Tsunami Catcher'. The loads can be divided in four categories, based on the phases the loads occur [Figure D.1]: The loads during the initial earthquake [Section D.1], the dynamic loads of the inflation phase [Section D.2], the static loads in the retaining phase [Section D.3] and lastly, the accidental loads [Section D.4].

**Figure D.1: Overview of the load conditions**

### D.1 Seismic loads

In Section A.1 it is stated that the majority of tsunamis is generated by earthquakes [Bryant, 2008]. Therefore the tsunami barrier must be able to withstand the forces due to the incoming tsunami waves but also withstand the loading due to the seismic activity. It is required that the barrier remains stable after the earthquake and able to deploy when an incoming tsunami is generated.

When an earthquake occurs, seismic waves are produced which induce different types of loads on the tsunami barrier. These loads effects the pile foundation and the drag embedded anchor. Another phenomenon which could occur during an earthquake is liquefaction. This is the process whereby soil repeatedly is compressed by the pressure waves. The pore water pressure builds up to the point where it exceeds the contact pressure between the soil particles and the soil loses all of its strength and behaves as a liquid [Heemskerk et al., 2014].
The behaviour of the foundation and soil during seismic activity is complex. The foundation is dynamically loaded during an earthquake and there is the probability that the soil loses its bearing capacity. This can cause weakening or total softening of the soil or large deformations [Korff and Meijers, 2015].

The stability of the bottom recess is crucial for the barrier. Therefore it is chosen to apply a pile foundation for the Kamakura case study. Settlements can jeopardise the stability of the barrier. However, this also means that the pile foundation must be able to withstand, and where necessary adjusted for the seismic loading.

Also the drag embedded anchor can be dynamically loaded by the earthquake. Deformations in the soil can create additional drag of the anchor which can be unfavourable. The effect of seismic loading on the offshore foundation is not included in the first designs.

For the first calculations, it is assumed that there is no weakening of the soil and the barrier remains stable. However, in further research, the soil behaviour must be modelled with numerical program, such as Plaxis.

D.2 Loads During the Inflation of the Barrier

In Section 4.2, the inflation phase of the barrier is analysed. Two main loads are distinguished; the initial bore impact and the dynamic loads during the inflation.

D.2.1 Incoming Bore

The incoming modelled tsunami wave for Kamakura, will develop into a bore, with a height equal to 8 meter [Section 3.5][Figure 3.5] for a Level 1 tsunami, and a bore with a height of 14 meter for a Level 2 tsunami. The bore causes an impact load onto the floater and onto the bottom recess. In Section A.2.3 two methods are proposed to calculate the impact load due to the incoming bore. Both models will be used to calculate this load.

Ramsden Theory

In the report of Ramsden [1990] it was concluded that for large bores \( H_{bore} > 8 \), the impact load can be estimated by 7.5 times the hydrostatic force of the incoming bore height. So for Kamakura, this impact load for a Level 1 \( F_{s,Ramsden,level1} \) and Level 2 \( F_{s,Ramsden,level2} \) will respectively be:

\[
F_{s,Ramsden,level1} = 7.5 \cdot \frac{1}{2} \cdot \rho_w \cdot g \cdot H_{bore,level1}^2
= 2590kN
\]

\[
F_{s,Ramsden,level2} = 7.5 \cdot \frac{1}{2} \cdot \rho_w \cdot g \cdot H_{s,Ramsden,level2}^2
F_{bore,level2} = 7931.4kN
\] (D.1)
FEMA Theory

In the theory of FEMA [2012] it is stated that the impulsive force depends on the maximum momentum flux \([H\cdot u^2]_{\text{max}}\) of the incoming bore profile. The maximum momentum flux can be derived from the incoming bore profile, which is derived from the SWASH model [Section 3.5]. The bore characteristics are derived at the position of the barrier. The bore profile [Blue Line] and velocity [Red Line] for Level 1 and Level 2 tsunami are given in Figure D.2.

![Bore height vs. Time for Level 1 tsunami](image1)

(a) Figure D.2: The height [Blue Line], velocity [Red Line] and maximum flux [Dashed Line] of the incoming bore for (a) Level 1 tsunami (b) Level 2 tsunami

The incoming bore characteristics of the Level 1 tsunami are given in Figure D.2a. The maximum flux occurs after 6 seconds. At that moment, the height of the incoming bore \([H]\) is equal to 4.4 meter with a corresponding velocity \([u]\) of 14.7 meter per second. These parameters are filled in Equation 4.2:

\[
F_{S,FEMA,level1} = 1.5 \cdot C_d \cdot \left( \frac{1}{2} \cdot \rho_w \cdot (H\cdot u^2)_{\text{max}} \right)
\]

\[
F_{S,FEMA,level1} = 1568.8kN
\]  

(D.2)
The incoming bore characteristics of the Level 2 tsunami are given in Figure D.2b. The maximum flux occurs after 6 seconds. At that moment, the height of the incoming bore \(H\) is equal to 8.5 meter with a corresponding velocity \([u]\) of 20.0 meter per second. These parameters give the following impact force:

\[
F_{S,FEMA,level2} = 1.5 \cdot C_d \cdot \left( \frac{1}{2} \cdot \rho_w \cdot (Hu_{max}^2) \right)
\]

\[
F_{S,FEMA,level2} = 2904kN
\]

There is a significant difference in magnitude for the bore impact load calculated by the Ramsden theory and the bore impact load calculated by the FEMA theory. The method based on the FEMA theory depends on the characteristics of the bore. An explanation of the force reduction could be due to the mild slope of the bore profile. A steeper profile leads to a greater impact load. The Ramsden theory, on the other hand, does not incorporate the bore velocity and only depends on the bore height. In the report of Ramsden (1992) there is no maximum bore height given for which the impact load estimation is valid. It could be therefore that the Ramsden theory overestimates the impact load.

For the design calculations, the bore impact load is calculated by the Ramsden theory. It is recommended to do further research to determine the maximum bore impact load.
D.2.2 Parachute Loading

In Section 4.2 the inflation process of the barrier was analysed. It was concluded that this process resembles with the principle of a parachute, and that movement can cause peak stresses in the cables and membrane.

The structural loads of a parachute during inflation, are analysed in several studies with supported experiments. A force history diagram is given in Figure D.3a. Two peak loads can be distinguished, the Snatch Force \(F_s\) in the cables, and the Opening Shock Force \(F_x\) [Doherr, 2005] which is related to the membrane. The Snatch Force and the Opening Shock are also found in the experimental set-up, Figure 4.2a and Figure 4.2b, respectively.

The Snatch Force depends on the characteristics of the cable and the potential energy stored in these cables. Where the latter parameter can be related to the incoming wave characteristics. The Opening Shock Force is related to the drag area of the membrane, time of opening and the velocity of the incoming wave. It is directly coupled to the Snatch Force [Doherr, 2005].

Due to the complexity of these phenomena, amplitude factors are used to include these peak loads into the design of the ‘Tsunami Catcher’. It is furthermore assumed that the snatch force is equal in amplitude to the opening shock force, and both depend on the parachute amplitude factor \(\zeta_p\) [Figure D.3b]. The peak loads in the membrane or cable \(T_p\) can be computed by multiplying the tension load found in the static situation \(T\) by the amplitude factor [Figure D.3b]:

\[
T_p = T \cdot \zeta_p
\] (D.4)

These peak loads may cause instability (lowering) of the floater during inflation. However, this is of a short duration, and eventually the barrier will inflate and the instability can cause some overflow of water which is acceptable. Therefore these amplitude factors will not be included in the buoyancy model.
D.3 Retaining Phase

The retaining phase of the barrier can be seen as a static situation [Section 4.2] for both tsunami levels.

D.3.1 Retaining Height of a Level 1 Tsunami

The tsunami barrier must be able to fully withstand a Level 1 tsunami. The retaining height of the barrier \( H_{retaining} \) is equal to the reflected height of the incoming Level 1 tsunami bore. For the Kamakura case study, the height of the incoming tsunami bore is equal to 8 meter [Section B.4]. So the design retaining level is equal to 16 meter.

It is assumed that the retaining phase can be seen as a static situation and that the barrier is loaded by the hydrostatic water pressure due to the raised water level [Figure D.4], where the pressure at the position of the floater is equal to zero kPa. The maximum water pressure at the foot of the barrier is equal to:

\[
p_{1;\text{level}1} = \rho_w \cdot g \cdot H_{retaining} = 172.7 \text{kPa}
\]  

(D.5)

The resulting force of the pressure distribution is equal to:

\[
F_{p;\text{level}1} = \frac{1}{2} \cdot p_{1;\text{level}1} \cdot H_{retaining} = 1392 \text{kN}
\]  

(D.6)

Figure D.4: Pressure distribution due to a Level 1 tsunami
D.3.2 Retaining Height of a Level 2 Tsunami

The other type of tsunami wave is the so-called Level 2 tsunami wave which has a return period of 1000 years [Section A.3]. This level allows overflowing of the barrier and an adjustment (cable) is proposed [Figure D.5a] which keep the floater at the design retaining level.

The Level 2 tsunami will give a different type of pressure distribution in comparison with the Level 1 tsunami wave. The pressure distribution is approximated with the derived formulas for non-breaking waves forces for vertical walls of low height from Miche-Rundgren [1958] [Section A.2.2] [Coastal Engineering Research Center, 1984].

The amount of overflow can be determined by the one numerical SWASH model [Chapter 3]. For the case study of Kamakura, the amount of overflow \( H_{\text{overflow}} \) was determined to be 9 meters.

The maximum pressure \( p_{1,\text{level}2} \) at the foot of the barrier, will be equal to the hydrostatic water pressure of the maximum crest height of the overflowing wave:

\[
p_{1,\text{level}2} = \rho_w \cdot g \cdot (H_{\text{retaining}} + H_{\text{overflow}})
\]

\[
p_{1,\text{level}2} = 269.8kPa
\]

(D.7)

In Section A.2.2 it was derived that the distribution can be assumed linear. So the magnitude of the pressure at the floater can be calculated with simple geometric:

\[
p_{2,\text{level}2} = \rho_w \cdot g \cdot H_{\text{overflow}}
\]

\[
p_{2,\text{level}2} = 97.1kPa
\]

(D.8)

The resulting force of the pressure is equal to:

\[
F_{p,\text{level}2} = \frac{1}{2} \cdot (p_{1,\text{level}2} + p_{2,\text{level}2}) \cdot H_{\text{retaining}}
\]

\[
F_{p,\text{level}2} = 2935.2kN
\]

(D.9)
D.4 Accidental Loads

There are three accidental loads distinguished: Debris loads, secondary wave loading and vandalism and weathering. The latter force will not be calculated but must be incorporated in the integration of the barrier in the surroundings [Section 6.3]

**Debris**

Debris could cause a significant impact on the barrier. These debris can variate from small items of the beach till large structures such as containers or vessels. The large structures can have a significant impact force on the membrane or the floater and even lead to instability of the barrier. For the case study of Kamakura it can be assumed that the possibility for large debris is negligible due to the lack of shipping activity.

So the type of debris loading, strongly depends on the conditions of the project location. Besides an impact load, these debris could cause a tear in the membrane. From the first tear-resistance tests of the Dyneema membrane from the report of B. van Rodijnen (2017), it can be concluded that the membrane retains approximately 75 percent of its strength. Therefore a material factor $\gamma_m$ of 1.5 is used in the design calculations.

$$\gamma_m = \frac{1}{0.7} = 1.45$$

**Additional Wave Loading**

In Section A.1.1 it is stated that the incoming tsunami wave can be seen as a train of waves. Therefore there is a possibility that after the first bore is reflected, secondary waves can strike the tsunami barrier and create a rise of the floater due to the buoyancy. Similar as for the Level 2 tsunami, it is assumed that the floater is kept in place by an additional cable and is restrained to retaining height of a Level 1 tsunami.

The pressure distribution due to the additional wave loading for vertical wall is calculated with the linear wave theory [Section A.2.1][Equation A.9]. Because the floater is restricted to a Level 1 tsunami wave height, a part of the wave will overflow and only the constant part of pressure distribution will be taken into account. This pressure distribution will be combined with the pressure distribution for a Level 1 tsunami and results in a truncated pressure distribution.

Wave loads will cause peak stresses in the membrane which is transferred through the cables. These peak stresses can be expressed in an amplitude factor $\zeta_w$, by comparing the membrane tension load due to wave loading, with the membrane tension load due to the Level 1 pressure distribution:

$$\zeta_w = \frac{T_{\text{wave}}}{T_{\text{Level 1}}}$$

(D.11)
Global Design

In the global design, the relations between the cable configuration, membrane configuration and floater dimensions are further studied. Goal is to determine an optimum configuration of these elements, based on simple cost functions.

The first step is to further analyse the force balance on the floater [Section E.1]. Thereafter the membrane configuration [Section E.2], the cable configuration [Section E.3] and the floater dimensions [Section E.4] are studied. Finally the derived method is applied to the case study of Kamakura, and a first global design can be presented [Section E.5].

The global design is based on several simplifications. The configuration is solely based on the Level 1 tsunami load condition [Section 4.3]. Furthermore the connections are not further analysed and it is assumed to be able to transfer the loads between the elements. Also the bottom recess and mooring system will be omitted from the global designed and assumed to be stable and able to transfer the loads to the surrounding soil.

E.1 Analysis of the Force Balance of the Floater

In Section 4.4 the load derivation of the barrier is analysed, it is concluded that the floater is affected by the shape of the membrane, the cable configuration and the weight of the floater. For the first design calculations, it is assumed that the floater consists of a steel pipe with a certain diameter and thickness and can be seen as a stiff, rigid structure [Section 4.1].

The floater must fulfil two requirements. First of all, the floater must have sufficient buoyancy to be able to float and ensure the vertical stability [Equation E.3]. Secondly, the floater must be able to withstand the tension forces, induced by both the membrane and the cables [Equation E.6].

The first step is to analyse the forces working on the floater. Figure E.1a shows the three-dimensional force situation. The membrane tension force \([T_m]\) can be seen as a distributed load over the length of the floater. The cable force however \([T_C]\), is a point load. Because the floater can be seen as a stiff, rigid structure, it is assumed that the cable force is evenly distributed over the entire interval length between the cables \([L_n]\). So the distributed cable force is equal to:

\[
T_{qc} = \frac{T_C}{L_n} \tag{E.1}
\]

The membrane- and the cable tension forces can be decomposed in horizontal and vertical force components [Figure E.1b]. The distribution between the horizontal and vertical force component depends on the configuration of the membrane and the cables. These dependencies will be further elaborated in Section E.2 and Section E.3 respectively.

The function of the cable is to transfer the membrane tension load to the offshore foundation. From the force balance [Figure E.1b], this function becomes the following requirement:

\[
T_{m:hor} = T_{q:hor} \tag{E.2}
\]

\[110\]
Buoyancy of the Floater

The buoyancy of the floater \( F_b \) is affected by the vertical force component of the membrane \( T_{m,\text{ver}} \), the vertical force component of the cable \( T_{qc,\text{ver}} \), and the self-weight of the floater \( F_{g,\text{floater}} \). The buoyancy requirement can be derived from Figure E.1b:

\[
F_b > F_{g,\text{floater}} + T_{m,\text{ver}} + T_{qc,\text{ver}} \tag{E.3}
\]

The maximum buoyancy of the floater can be calculated with the law of Archimedes. The force depends on the entire volume under water, which can be calculated with the diameter of the floater and the density of the water \( \rho_w \):

\[
F_b = \rho_w \cdot g \cdot \frac{\pi \cdot d_{\text{floater}}^2}{4} \tag{E.4}
\]

The self-weight of the floater is determined by the dimensions of the floater:

\[
F_{g,\text{floater}} = \rho_s \cdot g \cdot \left( \frac{\pi \cdot d_{\text{floater}}^2}{4} - \frac{\pi \cdot (d_{\text{floater}} - 2 \cdot t_{\text{floater}})^2}{4} \right) \tag{E.5}
\]

Strength of the Floater

The floater is assumed to be a rigid, stiff structure and can be modelled as beam-type structure. The floater is subjected by moments, induced by the cable forces [Figure E.1a]. These moments generate bending stresses and the floater must be able to withstand these stresses.

The induced bending stresses \( \sigma \) can be calculated with Equation E.6, where \( M_{\text{max}} \) is the maximum moment and \( W_{\text{floater}} \) the section modulus of the floater. These stresses must not exceed the yield stress \( f_y \) of the floater. For steel S235, this stress is equal to 235 \( \text{N/mm}^2 \):

\[
\sigma = \frac{M_{\text{max}}}{W_{\text{floater}}} < f_y \tag{E.6}
\]

The maximum moment in the floater is further elaborated in Section E.4. The section modulus depends on the dimension of the floater:

\[
W_{\text{floater}} = \frac{\pi \cdot d_{\text{floater}}^3}{32} - \frac{\pi \cdot (d_{\text{floater}} - 2 \cdot t_{\text{floater}})^3}{32} \tag{E.7}
\]
Secondary effect: Folds

The choice of a rigid, stiff floater could create secondary load effects. Due to the moments in the floater, the element could deflect and cause stress variation in the membrane. An increase in the membrane tension force can affect the vertical stability of the floater. To incorporate these secondary load effects in the design calculations, an amplitude factor $[\zeta_m]$ is introduced. So Equation E.3 becomes:

$$F_b > F_{g,\text{floater}} + \zeta_m \cdot T_{\text{m,ver}} + T_{qc,ver}$$ (E.8)

For the first calculations, an amplitude factor of 2.0 is assumed. In Section G.1, the magnitude of this factor is further investigated.

Conclusion

The minimum floater dimensions can be derived from the buoyancy and strength requirement. These dimensions depend on the forces acting on the floater. These forces are related to the membrane- and cable configuration. The dependencies between the forces and the membrane- and cable dimensions are therefore further analysed.

E.2 Analysis of the Membrane

The vertical force component of the membrane tension force $[T_{m,\text{ver}}]$ affects the buoyancy of the floater. The dependency between the membrane length $[L_m]$, the membrane tension force $[T_m]$ and angle between the membrane and floater $[\beta_m]$ are here further analysed. In order to determine these parameter it is required to study the shape of the membrane. A numerical model is composed to approximate the shape.

The eventual membrane shape must be in equilibrium with the resulting force of the imposed load distribution $[F_{p,\text{level1}}]$. This requirement can be described with Equation E.9, where $\beta_{m,0}$ and $\beta_{m,1}$ are the angles of the membrane at the bottom recess and the floater respectively [Figure E.2].

$$F_{p,\text{level1}} = T_m \cdot \cos \beta_{m,0} + T_m \cdot \cos \beta_{m,1}$$ (E.9)

Where the resulting force of the Level 1 pressure distribution can be calculated with Equation E.10. The parameter $[H_{\text{retaining}}]$ is the retaining height of the barrier, and is equal to twice the height of the incoming bore of the Level 1 tsunami.

$$F_{p,\text{level1}} = \frac{1}{2} \cdot \rho_w \cdot g \cdot H_{\text{retaining}}^2$$ (E.10)
There are different membrane configurations possible, the minimum membrane tension force $T_{min}$ is obtained with a very large membrane. As a result, both membrane angles become zero and the membrane tension forces becomes:

$$T_{min} = \frac{F_{p,level1}}{2} \quad (E.11)$$

For the preliminary designs it is assumed that a large membrane length is required, in order to minimise the floater dimensions. As a result, it can be assumed that the initial curvature of the membrane $[\beta_{m,0}]$ is zero and will lay horizontal on the bottom recess. Furthermore it is assumed that the attach point of the membrane is just next to the floater [Figure E.3].

\[ \text{Figure E.3: Schematization of the membrane configuration} \]

### E.2.1 Numerical Model

It is already mentioned that the shape of the membrane depends on the load condition. The pressure $[p]$ is balanced by the curvature $[\phi]$ of the tensioned membrane $[T]$. From Figure E.4 the differential equation can be derived, for a small section of the membrane $[dS]$:

\[ \frac{d\phi}{ds} = \frac{T}{p} \quad (E.12) \]

Equation E.12 can be approximated with a second order model, based on the proposed model of the weightless inextensible membrane, in the report of Parbery [1976]. This numerical model is made for an air inflatable dam [Section 2.1], which implies that inside the barrier there is a constant pressure. This in contrary to the tsunami barrier, where there is a linear increasing pressure due to the hydrostatic pressure. The proposed second order model is based on the predictor-corrector model and it is shown that this finite element model method may be regarded as a second-order approximation [Parbery, 1976]. For the full derivation is referred to Appendix F.
### E.2.2 Dependencies of the Membrane

Every membrane configuration gives a membrane tension force $[T_m]$, a membrane length $[L_m]$ and the curvature of the membrane at the floater $[\beta_m]$. This latter parameter determines the decomposition of membrane tension force in a horizontal and vertical component. The membrane length and the vertical force component are of interest.

Firstly, the relation between the membrane force $[T_m]$ and the membrane length $[L_m]$ is analysed [Figure E.5. It can be concluded that for the assumed membrane configuration, there is a minimal variation in the membrane tension force. A membrane length, greater than 2.5 times the $[H_{retaining}]$ results in an almost constant tension force.

![Figure E.5: Relation between the membrane length $[L_m]$ and the membrane force $[T_m]$, for the level 1 tsunami pressure distribution](image)

The angle of the membrane at the connection to the floater $[\beta_m]$ determines the division between the horizontal $[T_{m,hor}]$ and vertical component $[T_{m,ver}]$ of the membrane tension force [Equation E.13]. Similar to the membrane tension force, the curvature depends on the membrane length. This relation for both force components is given in Figure E.6. The horizontal force component [Blue line] of the membrane tension is transferred through the cables [Equation E.2]. The vertical force component $[T_{m,ver}]$ [Red Line] is a variable in the buoyancy model of the floater [Equation E.3].

$$T_{m,ver} = T_m \cdot \sin \beta$$
$$T_{m,hor} = T_m \cdot \cos \beta$$  \hspace{1cm} (E.13)

From Figure E.6 it can be concluded that the variation of the horizontal- and total force component of the membrane tension force is minimal. The vertical force component, on the other hand, does have a great variation. This can be explained by examining the membrane configurations. Figure E.7 displays three different membrane configurations, based on the Kamakura Level 1 design tsunami wave $[H_{retaining} = 16]$, with a membrane length of 24 meter [Blue line], a membrane with a length of 40 meter [Red line] and a membrane with a length of 56 meter [Green line]. The shape of the membrane tend to form a half ellipse, with an increasing length.

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In can be concluded that the effect of the vertical force component can be significant in the force analysis of the floater. For retaining a tsunami wave, a large membrane length is required. The horizontal force component and the resulting force component is not normative for the membrane.
E.3 Analysis of the Cable

Cable structures are widely used in the offshore technology. When a cable sags under the action of its own weight, under the influence of gravity, its shape can be described by a hyperbolic cosine curve. Where the vertical load on the cable is uniform distributed, with respect to the arc length [Simone, 2006]. It is assumed that a light-weight cables are used [Section 4.1], these cables tend to float, causing a hogging cable. Therefore the hyperbolic cosine curve is used to model the cable force.

The cable configuration can be described by two variables; the interval cable length \( L_n \) and the horizontal distance between the floater and offshore foundation \( L_{c;hor} \) [Figure E.8]. This latter parameter determines the cable length \( L_c \) in combination with the retaining height of the tsunami \( H_{retaining} \). Note that for the first calculations, it is assumed that the slope of the shore is constant. So eventually, the cable length and the horizontal distance between floater and offshore foundation will be larger, with the increasing depth.

\[
L_c = \sqrt{(L_{c;hor})^2 + (H_{retaining})^2}
\]  
(E.14)

Figure E.8: Configuration of the cables

In Section E.1 it is assumed that the cable forces are equally distributed over the floater, due to the assumption of a stiff, rigid floater. The eventually cable forces depend on the cable interval length [Equation E.1]. So the cable forces analysed in this section, are the distributed cable force \( T_{qc} \) and the eventual cable force \( T_C \) is determined by multiplying the distributed force by the cable interval length.

E.3.1 Numerical Model of the Cable Shape

The cable configuration is approximated by a first order numerical model [Appendix F]. The curvature of the cable depends on the force balance of the cable. For the cable, this are the self-weight of the cable, the buoyancy force of the water and the tension force of the cable. The input parameters for the numerical model are the horizontal distance between the floater and the offshore foundation \( L_{c;hor} \) and the horizontal tension force of the membrane \( T_{m;hor} \). Since the horizontal component of the distributed cable force \( T_{qc;hor} \) must be equal to the horizontal component of the membrane tension force [Equation E.2][Figure E.1b]
E.3.2 Dependencies of the Cable

The function of the cable is to transfer the horizontal membrane load to the offshore foundation. This is described in Equation E.1. In the previous section, the dependency between the membrane force \([T_m]\) and the membrane length \([L_m]\) is discussed. The angle between the cable and the floater \([\beta_m]\) depends on the horizontal distance between the floater and the offshore foundation \([L_{c;hor}]\).

The two variables, membrane- and horizontal cable length, gives a matrix type of solution for the cable forces. This relation can be presented in a contour plot, where for each combination of the membrane length [y-axis] and horizontal cable length [x-axis] gives the cable force. Figure E.9 shows the distributed, total cable force \([T_{qc}]\):

![Contour plot showing the relationship between membrane length and cable force](image)

Figure E.9: Relation between membrane length \([L_m]\), cable length \([L_c]\) and the cable tension force \([T_{qc}]\)

Based on Figure E.9, it can be concluded that the majority of the cables, has a approximately constant cable force, which is equal to the minimal tension force \([T_{min}]\). It can be concluded that the the cable force is not governing for the cables.

Also the vertical force component of the cable tension force \([T_{qc;ver}]\) depends on the angle between the floater and the cable \([\beta_c]\):

\[
T_{qc;ver} = T_{qc;ver} \cdot \cos \beta_c 
\]  

(E.15)

Figure E.10 shows the distributed vertical cable force component in function of the horizontal distance between the floater and the offshore foundation [x-axis] and the membrane length [y-axis]. A greater horizontal cable distance, results in a smaller force component, however, it can be assumed that this effect is not further beneficial after a length greater than twelve times the retaining water \([H_{retaining}]\). Therefore, these cable lengths are not further included.
Figure E.10: Relation between membrane length \([L_m]\), horizontal cable length \([L_{c\,hor}]\) and the vertical cable force \([T_{qc\,ver}]\)
E.4 Analysis of the Floater

In Section E.1 the two requirements for the floater are derived. The floater must have sufficient buoyancy to be able to float [Equation E.3], and the strength of the floater must be able to withstand the induced stresses due to the moment caused by the cables [Equation E.6]. First the strength requirement is further analysed, thereafter an iteration process is derived to compute the optimum floater dimensions.

Strength Requirement

The positioning of the cables is crucial for determining the maximum moment in the floater. For the first calculations, a cable configuration is assumed [Figure E.11]. The floater length \( L_{\text{floater}} \) can be divided into an \( n \)-amount of cable interval lengths. The amount of cables is equal to \((n + 1)\) number of cables. The point loads \( F \) represent the cable force, and the distributed load \( q \) represents the tension load due to the membrane.

![Figure E.11: Assumed cable lay-out](image)

For this assumed cable configuration, the maximum moment occurs in the middle of each interval. Because the cable force \( F \) is equal to the distributed membrane tension load \( q \) times the cable interval length \( L_n \), the moment is only in function of the distributed load.

\[
M_{\text{max}} = \frac{F L_n}{4} - \frac{q L_n^2}{8} = \frac{q L_n^2}{8} \quad (E.16)
\]

In Section E.1, the different load acting on the floater are analysed, and reduced to vertical and horizontal force components [Figure E.12b]. Also the moments can be decomposed in a vertical \([x\text{-plane}; \text{Figure } E.1a]\) and horizontal \([y\text{-plane}; \text{Figure } E.1a]\) direction.

In the x-plane there are the buoyancy force \( F_b \), the self-weight of the floater \( F_{g,\text{floater}} \), the vertical downward force of the membrane \( T_{m,\text{ver}} \) and the vertical downward force of the cable \( F_{c,\text{ver}} \). Where the latter force is modelled as a point load, and the other loads can be seen as uniformly distributed loads. The resulting force of the distributed loads is equal to the cable force \( T_{q,\text{ver}} \) divided by the interval length of the cable \( L_n \).
The maximum moment occurs at the midpoint of the interval length and is equal to:

$$M_{\text{max},y} = \frac{1}{8} \cdot T_{\text{q,cver}} \cdot L_n^2 \quad (E.17)$$

In the y-plane, there are the horizontal tension force of the membrane \( T_{m,\text{hor}} \) and the horizontal tension force of the cable \( T_{C,\text{hor}} \) [Figure E.12b]. Similar to the y-direction, the horizontal point load of the cable equals to:

$$T_{C,\text{hor}} = T_{q,\text{hor}} \cdot L_n$$

The maximum moment in the y-plane is equal to:

$$M_{\text{max},x} = \frac{T_{q,\text{hor}} L_n^2}{8} \quad (E.18)$$

Analysing the forces on the floater, it can be concluded that the maximum moment in the x-direction will always be greater than the moment in the y-direction, due to the assumption of a large membrane [Figure E.6], the horizontal force components will be normative. The moment in x-direction will be normative. Hence, the moment \( M_{\text{max},x} \) will be used for the strength requirement [Equation E.6].

**Iteration Process**

For each given membrane length, horizontal cable length and cable interval length, a floater diameter and floater thickness can be calculated. There are different combinations of \( d_{\text{floater}} \) and \( t_{\text{floater}} \) possible, goal is to find the minimal required floater diameter. To achieve this, an iteration process is presented.

The amount of buoyancy is related to the volume under water. Equation E.4 states that the entire volume of the floater is used for the buoyancy. Every small deviation in the forces however, can result in instability of the floater. The buoyancy is therefore calculated for 90 percent of the volume under water.

The first step is to describe the floater dimensions in function of the floater thickness \( t_{\text{floater}} \) and floater diameter \( d_{\text{floater}} \). The buoyancy is affected by the buoyancy force \( F_b \) and the self-weight \( F_{g,\text{floater}} \) of the floater. The strength can be derived from the section modulus \( W_{\text{floater}} \). All three variables depend on the dimensions of the floater:

$$F_b = \rho_w \cdot g \cdot \frac{\pi \cdot d_{\text{floater}}^2}{4}$$

$$F_{g,\text{floater}} = \rho_s \cdot g \cdot \left( \frac{\pi \cdot d_{\text{floater}}^2}{4} - \frac{\pi \cdot (d_{\text{floater}} - 2 \cdot t_{\text{floater}})^2}{4} \right)$$

$$W_{\text{floater}} = \frac{\pi \cdot d_{\text{floater}}^3}{32} - \frac{\pi \cdot (d_{\text{floater}} - 2 \cdot t_{\text{floater}})^3}{32}$$
The minimum diameter is first calculated \([d_{0; \text{floater}}]\), based on solely the downward vertical force due to the membrane \([T_{m; \text{ver}}]\) and the downward force of the cable \([T_{c; \text{ver}}]\). The self-weight is assumed to be 10 percent of the total downward vertical force. The first calculations show that this is a proper estimation.

\[
(0.9 - 0.1) \cdot F_b = T_{m; \text{ver}} + T_{c; \text{ver}} \tag{E.19}
\]

Based on the minimum diameter \([d_{0; \text{floater}}]\), the required thickness \([t_{1; \text{floater}}]\) can be computed. The needed section modulus depends on the maximum moment in the floater [Equation E.18], which is related to the cable interval length \([L_n]\). Now, the thickness of the floater, can be related to the membrane length, the cable length and the cable interval length.

Next step is to compute again the diameter of the floater \([d_{1; \text{floater}}]\), but now including the self-weight of the floater. This implies that the diameter of the floater also depends on the cable interval length, and the solution is also given in different matrices, one matrix solution for each cable interval length.

Now one iteration step is completed. If the new calculated diameter \([d_{1; \text{floater}}]\) in combination with the corresponding thickness \([t_{1; \text{floater}}]\) has a sufficient section modulus to withstand the imposed moment, the iteration is completed. Else, a new thickness \([t_{2; \text{floater}}]\) is computed, now based on the calculated diameter \([d_{1; \text{floater}}]\). Now, the buoyancy requirement is again checked. If this is not met, a new diameter \([d_{2; \text{floater}}]\) is calculated. This process is repeated, until both requirements are met.

The iteration process is summarised in the following bullet scheme:

1. Calculate the minimum diameter \([d_{0; \text{floater}}]\) where the self-weight is estimated [Equation E.19]
2. Calculate the required thickness \([t_{1; \text{floater}}]\), based on the minimum diameter [Equation E.7 and E.6]
3. Calculate the new diameter \([d_{1; \text{floater}}]\), with thickness \([t_{1; \text{floater}}]\) [Equation E.8]
4. Check if the combination of \([d_{1; \text{floater}}]\) and \([t_{1; \text{floater}}]\) fulfils Equation E.6, if not, calculate the \([t_{2; \text{floater}}]\);
5. Check if the combination of \([d_{1; \text{floater}}]\) and \([t_{2; \text{floater}}]\) fulfils Equation E.8, if not, calculate the \([d_{2; \text{floater}}]\);
6. Go back to step 4 until a combination is found which fulfils both requirements
E.5 Global Design for Kamakura

In the previous sections, the dependency between the floater, cables and membrane are analysed. Based on a certain membrane length $[L_m]$, horizontal distance between floater and offshore foundation $[L_{c,hor}]$ and cable interval length $[L_n]$, the floater dimensions can be calculated. Next step is to find an optimum configuration of these elements. This will be based on simple cost functions.

The theory will be applied for the case study of Kamakura. The goal of the this Global Design is to investigate if a flexible membrane barrier is suitable to stop a Level 1 tsunami wave.

E.5.1 Assumptions

The theory, explained in the previous sections, is now applied to the case study of Kamakura, where an optimum configuration is chosen based on basic cost functions. The following assumptions are applied:

1. The maximum crest height of the level 1 tsunami is equal to 8 meter, so the retaining water level $[H_{Level1}]$ is equal to 16 meter [Section B.4];
2. The horizontal cable length variates from 32 till 317 meters with interval steps of 5 meter;
3. The membrane length variates from 20 till 64 meters with different interval steps;
4. The fold factor $[\zeta_m]$ is assumed to be equal to 2 [Section E.1][Equation E.8];
5. The cable interval lengths are 10, 30 and 50 meters;
6. To ensure an optimum integration of the barrier, the maximum floater diameter is set on 5 meters;
7. The beach has a width of 60 meters [Section A.5.2]

The cost estimation is based on a rough estimation of the required materials (membrane, cable and steel). To include the fabrication costs, the the material cost are multiplied by 2. The following cost estimations are used, including the fabrication cost:

- The membrane cost are estimated to be 180 euro per square meter;
- The cable cost are estimated to be 70 euro per kilogram;
- The steel cost are estimated to be 3 euro per kilogram.
E.5.2 Results

In the following contour plots, presented on the next page [Figure E.14a - Figure E.14c], the diameter is represented in function of the membrane- and cable length for the three different cable interval lengths.

The plots show the influence of the cable interval length regarding the floater diameter. A smaller cable interval length, results in a smaller bending moment in the floater [Equation E.18], which leads to a thinner floater [Equation E.6]. The self-weight of the floater will decrease, and the required diameter of the floater will be smaller. The influence of the cable interval length on the floater diameter is however not that dominant.

This relation in combination with the maximum floater diameter of five meter, result in a very small design space for the Kamakura case study. Figure E.14c shows that for a cable interval length of 50 meter, there is no combination possible to obtain a floater diameter of 5 meter. The possibilities for a cable interval length of 30 meters is very limited, and are only feasible with a very large cable- and membrane length [Figure E.14b]. So the economical optimum configuration will be with a very small cable interval length of 10 meter [Figure E.14a], and still a relatively large membrane length of 60 meters and a horizontal cable distance of 160 meters. The cost is estimated on approximately 39,100 Euro per running meter. Note that this are only the cost for the membrane, cables and floater.

A sketch design is given in Figure E.13 for a tsunami barrier with a total length of 90 meters during a Level 1 tsunami.

Figure E.13: Impression of the global design of the tsunami barrier
E.5.3 Improved Global Design

It can be concluded that the proposed configuration of the elements is not ideal for Kamakura. A great amount of material is needed which result in a high cost. So the objective is to improve the design to decrease the amount of material. This can be done, by lowering of the retaining height of the barrier. This is achieved by elevate the bottom recess by three meters and place the barrier on top of the shore instead of integrate in the soil.

The following assumptions are adjusted for the improved global design:

1. The maximum crest height of the level 1 tsunami is equal to 8 meter, with a bottom recess height of 3 meter, the retaining water level is equal to 13 meter;
2. The cable length variates from 26 till 256 meters with interval steps of 5 meter;
3. The membrane length variates from 17 till 50 meters with different interval steps;

The floater diameter is once again analysed. The contour plots are given on the next page [Figure E.15a - Figure E.15c]. These plots show an increase in the design space. Still, a cable interval length of 50 meter in combination with a floater diameter of 5 meter is not feasible. However, Figure E.15b show a considerable increase in the design space for a cable interval length of 30 meters. This cable interval length requires still a large membrane, between the 40 and 50 meters, but the horizontal cable distance of 180 can be reduced. Also, a larger cable interval length, results in an increase of the recreational value of the beach.
Figure E.15: Floater diameter in function of the membrane length $[L_m]$ and the horizontal cable distance $[L_{c,h}]$ for a cable interval length of (a) 10 meter, (b) 30 meter and (c) 50 meter, for a retaining height of 13 meters.

Based on this design space, an optimum configuration is found, based on the cost functions. The results are shown in Figure E.16. Note that only the possible configurations of the design space are analysed. These are the configuration with a floater diameter smaller than 5 meter [Figure E.15b]. Also the price of the configuration is expressed in the cost per running meter and only the membrane, cables and floater are considered.

Figure E.16: Cost estimation based on the membrane length, cable length and diameter for a cable interval length of 30 meters.
There are multiple optimum configurations, all in the area between the 20,000 and 22,000 Euro per running meter [Figure E.16]. The following solution is chosen:

- A membrane length of 42 meters;
- A cable length of 86 meters, with a corresponding cable interval length of 30 meters;
- A floater diameter of 5 meters;
- A floater thickness of 2.5 centimetres.
- Cost of 21,500 Euro per running meter

An sketch design is given in Figure E.17 for a tsunami barrier with a total length of 90 meters during a Level 1 tsunami.

![Figure E.17: Impression of the improved, global design of the tsunami barrier](image)

E.5.4 Conclusion

It can be concluded that the improved, global design [Figure E.17 is a preferred design compared with the global design of Figure E.13. The improved barrier requires less space, which benefits the recreational value of the beach. Besides, there is a significant cost reduction, almost 50 percent. Therefore this improved, global design will be further analysed in Chapter 6.
Models for the Static Situation

In this Appendix the following static models are derived:

- Numerical model for the membrane;
- Numerical model for the cable;

F.1 Numerical Model for the Membrane

The numerical model presented in the following paragraph is based on the proposed model of the weightless inextensible membrane in the report of Parbery (1976). The numerical model is made for an air inflatable dam, which implies that inside the barrier there is a constant pressure. This in contrast with the tsunami barrier, where there is a linear increasing pressure due to the hydrostatic water pressure.

The model is based on the predictor-corrector model and it is shown that the finite element method may be regarded as a second-order approximation to the equations of equilibrium [Parbery, 1976].

Initial Conditions

The model begins where the membrane starts to curve. Note that this location is rather unclear but can be calibrated with the position of the known floater.

The initial angle of inclination $\theta_0$ depends on the wave height $H$ and the tension force $T$ [Figure F.1]. It is assumed that the membrane will lay horizontal on the bottom recess. The model starts where the membrane starts to curve, note that the angle of inclination is measured from the vertical axis:

\[ \sin \phi_0 \cdot Fp_i = \cos \phi_0 \cdot T \]  

(F.1)
Where the pressure force $F_p_0$ can be calculated by the hydrostatic water pressure:

$$F_p_0 = p_i \cdot n + (p_0 - p_i) \cdot \frac{1}{2} h \quad (F.2)$$

$$p_0 = H \cdot \rho_w \cdot g \quad (F.3)$$

$$p_i = p_0 - \sin \phi_0 \cdot \rho_w \cdot g \cdot n \quad (F.4)$$

**Predictor**

Next step is to calculate the predictor angles by assuming that the loads acting at node $i + 1$ are assumed to be the horizontal and vertical loads on element $i$.

$$\cos \phi_i+1 = \cos \phi_i - \sin \phi_i \cdot \frac{F_{p_0}}{T} \quad (F.5)$$

$$\sin \phi_i+1 = \sin \phi_i - \cos \phi_i \cdot \frac{F_{p_0}}{T} \quad (F.6)$$

**Corrector**

A second approximation is that the loads at node $i + 1$ are assumed to be the average loads over element $i$ and $i + 1$:

$$\cos \phi_i+1 = \cos \phi_i - \frac{F_H}{T} \quad (F.7)$$

$$\sin \phi_i+1 = \sin \phi_i - \frac{F_V}{T} \quad (F.8)$$

Where:

$$F_H = \frac{1}{2} (F_p_i \cdot \sin \phi_i + F_{p_{i+1}} \cdot \phi_{i+1}) \quad (F.9)$$

$$F_V = \frac{1}{2} (F_p_i \cdot \cos \phi_i + F_{p_{i+1}} \cdot \cos \phi_{i+1}) \quad (F.10)$$

Finally the angle $\phi_{i+1}$ can be calculated:

$$\phi_{i+1} = \arctan \frac{\sin \phi}{\cos \phi} \quad (F.11)$$

And the new coordinates $x_{i+1}$ and $y_{i+1}$ can be computed.

**Conclusion**

The model is stopped when the water level $H_{retaining}$ is reached. The total membrane length $L_m$ is defined by the total amount of steps $n$ and the distance between the first curvature and the final position of the floater.

Another important output of the model, is the angle of inclination at the floater $[\beta_m]$. This angle determines the horizontal $[F_m; hor]$ and vertical force $[F_m; ver]$ at the floater.
F.2 Numerical Model for the Cable

A numerical model for the cable can be derived in the same manner as for the membrane. In this case, the weight of the cable is included. This, because if the weight is sufficient enough, the cable can have a greater curvature, which causes additional vertical force at the floater. The force equilibrium for a finite small element is derived:

Goal of the model is to determine the vertical force at the floater \[F_{q,vex}\] and the total tension force in the cable \[T_{qc}\].

Initial Conditions

The initial conditions for the numerical model of the cable depends on the model for the membrane and the cable configuration. The horizontal tension force of the membrane [Section F.1] follows from the lay-out of the membrane and the distance between the cables. The other important cable parameter is the cable length \[L_c\].

From these parameter the initial angle of inclination \(\phi_0\) can be estimated:

\[
\phi_0 = \arctan \frac{H_{retaining}}{L_{c,hor}} \quad (F.12)
\]

The final input parameter is the weight of the cable, which depends on the total cable force. This implies that how larger the distance between the cables, how heavier the cable.

Boundary Conditions

The boundary conditions for the model can be derived from the cable configuration. If the origin is set on the position of the floater. The coordinates of the end point of the cable are equal to:

\[
B.c : (x_{end}; y_{end}) = (L_{c,hor}; -H_{retaining}) \quad (F.13)
\]

Numerical Equations

The following derived numerical scheme is a first order model. Therefore, the steps size must be minimized in order to control the error.

It is already mentioned that the methodology is used as for the numerical model for the membrane [Figure F.2]. Based on the force equilibrium of a small element, the next angle of inclination \(\phi_{i+1}\) can be derived:

\[
\tan \phi_{i+1} = \frac{\sin \phi_i - \cos \phi_i \cdot F}{\cos \phi_i + \sin \phi_i \cdot T} \quad (F.14)
\]
Outcome

The model runs with the estimated angle of inclination $\phi_0$. If the outcome does not agree with the boundary condition, the angle of inclination becomes:

$$\phi'_0 = \phi_0 + d\phi$$  \hspace{1cm} (F.15)

Where $d\phi_0$ is a small modification of the angle. Once the boundary condition is met, the angle and therefore the force at the floater is calculated.
Analysis of the Amplitude Factors

The dynamic loads are approximated with amplitude factors. In this appendix the magnitude of these amplitude factors are further investigated. There are three amplitude factors analysed: the fold factor [Section G.1], secondary wave factor [Section G.2] and the parachute factor [Section G.3]

G.1 Fold Factor

In Section E.1 of the Global Design, it is stated that the choice of a rigid, stiff floater can create secondary load effects. The moments in the floater, caused by the cable forces, could create deflection and therefore stress variation in the membrane. An amplitude factor was introduced to incorporate these stress increases in the force analysis of the floater.

In the Global Design, this amplitude factor was estimated to be 2.0, regardless the interval length $[L_n]$. This can be seen as rather conservative. The fold factor is therefore further analysed.

The interaction between the cable, floater and the membrane can be seen as an elastic supported beam. Where the floater is modelled as a beam, the membrane the elastic support and the cable force as a point load. The stress variation depends on the amount of the deflection of the beam.

The deflection line can be approximated by two methods, an analytical model or with the structural software Matrixframe. First of all, the general differential equations will be derived. Then, a section of the floater is analytical analysed, whereafter this section is implemented in the Matrixframe software. If the Matrixframe model is validated, the entire floater is modelled with software program and a final fold factor can be computed for both tsunami levels.

General Differential Equations

In the book of A.L. Bouma (2000), the differential equation for elastic supported beams is derived [Equation G.1], based on the assumption of Winkler [1867]. Which implies that the upward reaction force of the support in a point, is proportional to the deflection of the supporting material [Bouma, 2000].

\[
EI_{\text{beam}} \frac{d^4w}{dx^4} + kw = q \tag{G.1}
\]

Where the $EI_{\text{beam}}$ value depends on the beam characteristics, $q$ the distributed load acting on the beam, and $kw$ represents the distributed vertical reaction force of the support [Figure G.1]. With $w$ the deflection and $k$ the spring constant, that depends on the characteristics of the elastic support.

\[
k = \frac{b}{h} E_{\text{support}} \tag{G.2}
\]

Where $b$ and $h$ are the dimensions and $E_{\text{support}}$ is the E-modulus of the support. In this case, $[b]$ represents the thickness of the membrane $[t_m]$ and $h$ the length of the membrane $[L_m]$. 

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Figure G.1: Equilibrium for a small section for the elastic supported beam [Bouma, 2000]

The general solution of the differential equation can be written as:

\[ w = e^{\beta x} (C_1 \cos \beta x + C_2 \sin \beta x) + e^{-\beta x} (C_3 \cos \beta x + C_2 \sin \beta x) \]  

(G.3)

With:

\[ 4\beta^4 = \frac{k}{EI_{\text{beam}}} \]  

(G.4)

And where the constants \( C_1, C_2, C_3 \) and \( C_4 \) can be found with the boundary conditions.

**Define the Parameters of the Floater**

In this paragraph, the parameters of the floater are determined. The analytical calculations will be based on the dimensions of the derived tsunami barrier of the Global Design [Section E.5.3].

The barrier has a membrane length \( [L_m] \) of 42 meters with an approximated thickness \( [t_m] \) of 1 centimetre. The cables are placed with an interval length \( [L_n] \) of 30 meters across the floater. This floating element has a diameter \( [d_{\text{floater}}] \) of 5 meter with a corresponding thickness \( [t_{\text{floater}}] \) of 2.5 millimetres.

The E-modulus of steel with a grade of S235 is approximately 200 GPa. The tensile strength \( [\sigma_{\text{max,m}}] \) of the membrane is 3400 MPa with a fracture strain \( [\epsilon_m] \) of 3 percent. The E-modulus of the membrane can then be calculated with the law of Hook [Equation G.5].

\[ E_m = \frac{\sigma_{\text{max,m}}}{\epsilon_m} = 1133 \cdot 10^6 [N/m] \]  

(G.5)

The distributed force in the membrane with a length of 42 meter for a retaining height of 13 meters is equal to 458 kN [Figure 5.3].

The spring constant \( [k] \) of the membrane is equal to:

\[ k = 2.7 \cdot 10^8 [N/m] \]  

(G.6)

The moment of inertia of the floater \( [I_{\text{floater}}] \) can be calculated with the following expression:

\[ I_{\text{floater}} = \frac{\pi d_{\text{floater}}^4}{64} - \frac{\pi (d_{\text{floater}}^2 - 2t_{\text{floater}})^4}{64} = 1.21 [m^4] \]  

(G.7)
The factor $\beta$ can be calculated with Equation G.4:

$$\beta = \left( \frac{k}{EI_{beam}} \right)^4$$

$$= 2.3 \cdot 10^{-2} \quad \text{(G.8)}$$

The final parameter is the point load $P$, which is equal to half the cable load:

$$P = \frac{1}{2} \cdot 700 \cdot 10^3 \cdot L_n$$

$$= 1.75 \cdot 10^7 N \quad \text{(G.9)}$$

**Analytical Model**

The cables are distributed over the floater by regular intervals. Where the cables at the end of the floater only takes half the cable force [Figure G.2]. For the analytical model, a mid section of the floater is chosen.

![Figure G.2: Schematic overview of the cable configuration](image)

The first step is to check if the interface conditions of the section is influenced by the cable force. If this is not the case, the section can be modelled as a point load on an infinite long beam, and there is no deflection at the interface condition.

![Figure G.3: Schematization of an infinite long beam](image)

The general solution for a point load on an infinite long beam is derived in the book of A.L. Bouma [2000]. Here, it is concluded that the deflection can be described with the following equation:

$$w = \frac{P\beta}{k} e^{-\beta x} \cdot (\cos \beta x + \sin \beta x)$$

$$\quad \text{(G.10)}$$

All the parameters of Equation G.10 are derived in the previous paragraph and the deflection line is given in Figure G.4. The cable force is at location $x = 0$ and the edges of the graph are at $-\frac{L_n}{2}$ and $\frac{L_n}{2}$.  

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The additional stress due to this deflection can be calculated by multiplying the deflection with the spring constant $k$. So the amplitude factor for this analytical model becomes:

$$
\zeta_m = \frac{(k \cdot w_{max}) + T_m}{T_m} = 1.5
$$

(G.11)

From the deflection line it can be concluded that the assumption of a point load an infinite beam is not valid. At the interface conditions of the modelled section, the deflection is not zero and the entire modelled section is of the influence of the cable force.

Figure G.4: Deflection of the floater due to the cable load from $-\frac{L_n}{2}$ till $\frac{L_n}{2}$ for a cable interval length of 30 meters

Another possibility is to model the section as a double clamped situation [Figure G.5]. As a result, the angle of rotation is zero $[\phi]$ at the interface condition which can be seen as a continuous system. Due to symmetry, only the solution for one side has to be determined. This is not a general load situation and the solution must be derived from the interface conditions.

Figure G.5: Schematization of a double clamped, elastic supported beam

To find a solution for Equation G.3, 4 boundary conditions must be determined: at $x = 0$ and $x = \frac{L}{2}$

$$
\phi(x = 0) = \frac{dw}{dx}(x = 0) = 0 \quad \phi(x = \frac{L}{2}) = \frac{dw}{dx}(x = 0) = 0
$$

$$
V(x = 0) = \frac{d^3w}{dx^3}(x = 0) = \frac{P}{EI} \quad w(x = \frac{L}{2}) = 0
$$

(G.12)

Where $[\phi]$ is the angle of rotation and $[V]$ is the shear force of the beam
\[ C_1 = \frac{P \cdot e^{-\lambda}(2e^\lambda(\cos \lambda)^2 - 2e^\lambda \cos \lambda \sin \lambda - 3e^\lambda + e^{-\lambda})}{4 \cdot EI \cdot \beta^3 \left( 4 \cos \lambda \cdot \sin \lambda + e^{2\lambda} - e^{-2\lambda} \right)} \]

\[ C_2 = \frac{P \cdot e^{-\lambda}(2e^\lambda(\cos \lambda)^2 + 2e^\lambda \cos \lambda \sin \lambda - e^\lambda - e^{-\lambda})}{4 \cdot EI \cdot \beta^3 \left( 4 \cos \lambda \cdot \sin \lambda + e^{2\lambda} - e^{-2\lambda} \right)} \]

\[ C_3 = \frac{P \cdot e^{-\lambda}(2e^{-\lambda}(\cos \lambda)^2 + 2e^{-\lambda} \cos \lambda \sin \lambda - 3e^{-\lambda} + e^\lambda)}{4 \cdot EI \cdot \beta^3 \left( 4 \cos \lambda \cdot \sin \lambda + e^{2\lambda} - e^{-2\lambda} \right)} \]

\[ C_4 = \frac{P \cdot e^\lambda(-2e^{-\lambda}(\cos \lambda)^2 + 2e^{-\lambda} \cos \lambda \sin \lambda + e^\lambda + e^{-\lambda})}{4 \cdot EI \cdot \beta^3 \left( 4 \cos \lambda \cdot \sin \lambda + e^{2\lambda} - e^{-2\lambda} \right)} \]  

(G.13)

With:

\[ \lambda = \frac{\beta \cdot L}{2} \]  

(G.14)

The deflection line for this case is given in Figure G.6. The maximum deflection occurs at the location of the cable \([x = 0]\), and Equation G.3 is reduced to:

\[ w_{max} = C_1 + C_3 \]  

(G.15)

The deflection line still does not represent the actual situation of the floater. The deflections at the interface conditions of the model are equal to zero, where in reality, the membrane can have a deflection at the interface conditions.

So the next step should be to change the clamped beam into a rolling, clamped beam. This kind of support could have a vertical deflection, but does require an angle of rotation equal to zero. In the book of A.L. Bouma [2000] a method is proposed to find the solution for this type of situations. For the validation of the Matrixframe model however, the double clamped beam will be used.
Matrixframe Model

In the structural software program Matrixframe, elastic supported beams can be modelled. Instead of the spring constant \([k]\), the software uses a type of beddings constant \([C_z]\), which depends on the width of the beam:

\[
C_z = \frac{k}{b} = \frac{k}{d_{\text{floater}}} = 54 \cdot 10^3 \text{N/m}^3
\]  

(G.17)

In Matrixframe, the double clamped, elastic supported beam is modelled. The deflection line is given in Figure G.7.

\[
W
\]

Figure G.7: Deflection of the floater for a mid-section by the software program Matrix-frame

It can be concluded that the deflection line of the Matrixframe model approximates the deflection line of the analytical model. As a result, the total floater beam is implemented in the software program.

The cable configuration of Figure G.2 is implemented in the software program. There are no other supports then the elastic support. Note that until now, only the cable forces were considered, however there is also a membrane force pulling at the elastic support.

The deflection line from the Matrixmodel shows that the maximum deflection occurs at the ends of the floater. There were the cable force is only half. The maximum deflection is equal to 0.233 meter. And with Equation G.11, the amplitude factor is equal to 1.15 for the Level 1 tsunami.

The final step for the analysis of the fold factor is to incorporate the forces during a Level 2 tsunami. From Figure 6.3 it can be derived that the membrane tension force is equal to 1087 kN for a membrane length of 42 meter. The cable force is then equal to 32,610 kN. The other parameters are assumed to be equal to the Level 1 tsunami.

The deflection line of this load situation is given in Figure G.8. Also here, the maximum deflection occurs at the ends of the floater but is increased till 0.342, which gives an amplitude factor of 1.2 for the Level 2 tsunami

\[
\text{Level 1 Tsunami}
\]

\[
\text{Level 2 Tsunami}
\]

Figure G.8: Deflection line of the floater during a Level 1 tsunami; Deflection line of the floater during a Level 2 tsunami
G.2 Secondary Wave Loading

The incoming tsunami wave can be seen as a train of waves. So there is a possibility that after the first bore is reflected, secondary waves can strike the tsunami barrier and create an increase in the pressure distribution. This secondary wave load can be calculated by the linear wave theory [Section D.4].

The numerical SWASH model only represents the first, initial wave and not the entire wave train. The height of the secondary waves are therefore unknown and an assumption must be made. It is assumed that the secondary waves can be small in order of 1 meter, till considerable waves of 3 meter. The various pressure distributions are implemented in the numerical model of the membrane [Section F.1] for both the Level 1 tsunami as the Level 2 tsunami.

![Figure G.9: Dependency between the membrane tension force and the membrane length for different secondary waves for a Level 1 tsunami](image)

Figure G.9 shows the dependency between the membrane tension force $T_m$ and membrane length $L_m$ for three secondary waves with varied height between 1 and 3 meter, for a Level 1 tsunami. These dependencies are compared with the static, retaining situation for a Level 1 tsunami [Black Dashed Line G.9]. It can be concluded that for a membrane length of 42 meter, the amplitude factor $\zeta_w$ is equal to 1.48. The shape of the membrane resembles with the membrane shape during a Level 2 tsunami, and will form an ellipse type of shape.
For the Level 2 tsunami it was concluded that the amount of overflow was approximately 9 meters on top of the barrier [Figure G.10]. Due to secondary waves, this overtopping can reach up to 12 meters.

![Figure G.10](image)

Figure G.10: Dependency between the membrane tension force and the membrane length for different secondary waves for a Level 2 tsunami

Figure G.10 shows the dependency between the membrane tension force and membrane length for three secondary waves with varied height between 1 and 3 meter, for a Level 2 tsunami [Black Line G.10]. From the figure it can be concluded that for a membrane length of 42 meter, the amplitude factor \( \zeta_w \) is equal to 1.19.
In Section 4.3 the inflation process of the flexible tsunami barrier is analysed. It is concluded that this process can be characterised with two dynamic loads. The snatch force $F_s$ in the cables and the opening shock $F_x$ in the membrane. This analysis emphasises on the determination of this latter dynamic force. It is assumed that the amplitude factor $\zeta_p$ for the membrane due to the opening shock is equal for the cables due to the snatch force.

The opening shock of the membrane depends on the membrane configuration and on the incoming wave characteristics. A straightforward model is proposed which relates this dynamic force in the membrane to the incoming bore characteristics. This is done for both the Level 1 tsunami wave as for the Level 2 tsunami wave.

The membrane configuration can store a certain amount of water $V_{membrane}$. For a membrane length of 42 meter, this is approximately 200 $m^2$. It is assumed that the opening shock occurs when the membrane is fully 'filled', and the reflecting bore is formed [Figure G.11b]. At that specific moment, the membrane must be able to withstand a resulting force $F_{opening}$, which is composed out of the opening shock force $F_x$ and the static force of the stationary water inside the membrane $F_{static}$.

To determine the opening shock force, the incoming water profile is measured in function of the time, at the location of the barrier [Figure G.11a]. At the moment, the maximum volume $V_{membrane}$ of the membrane has crossed the observation point, the bore characteristics are derived. Note that the height of the bottom recess is incorporated in the SWASH model and will affect the shape of the incoming bore profile.

With the bore characteristics, the velocity $u^*$ and height $H^*$, the static- and dynamic impact force can be calculated at that specific moment. The static force is equal to the hydrostatic force of the water level:

$$F_{static} = \frac{1}{2} \cdot \rho_w \cdot g \cdot (H^*)^2$$  \hspace{1cm} (G.18)

The theory of FEMA (2012) [Section A.2.3] is used to calculate the impact force of the bore $F_s$ and is assumed to be equal to the opening shock of the membrane $F_x$:

$$F_x = F_s = 1.5 \left( \frac{1}{2} \cdot \rho_w \cdot C_d \cdot (H^* \cdot (u^*)^2) \right)$$  \hspace{1cm} (G.19)

Where the constant $C_d$ is assumed to be equal to 2.0
This analysis is done for both tsunami levels. Figure G.12 show the results of both SWASH computations for the Level 1 and Level 2 tsunami.

![Figure G.12: Measurements of the water level Blue Line and the velocity Red Line at the barrier for (a) the Level 1 tsunami and (b) the Level 2 tsunami](a)  

For the Level 1 tsunami, the barrier is inflated after 34 seconds [Figure G.12a]. The corresponding height $H^*$ is approximately 6.8 meter with a corresponding velocity $u^*$ is 5.4 meter per second. With Equation G.18, the static force at that moment is equal to 250 kN. With Equation G.19 the dynamic force is estimated to be 327 kN. The total force at the moment of inflation is equal to 577 kN. The membrane tension force is equal to half the total force:

$$T_{m, bore_1} = \frac{577}{2} = 288 kN$$  \hspace{1cm} (G.20)

This force is smaller than the static membrane tension force in the retaining phase. Therefore the opening shock for a Level 1 tsunami is not normative for the membrane. This low dynamic impact force is mainly a result of the elevated bottom recess. This dike stops a significant amount of the incoming bore.

For the Level 2 tsunami, the barrier is inflated after 20 seconds [Figure G.12b]. At that moment the bore height is approximately 13 meters with a corresponding velocity of 13 meters per second. This gives a static force of 912 kN and a dynamic impact force of 2596 kN. The tension membrane force becomes:

$$T_{m, bore_2} = \frac{912 + 2596}{2} = 1754 kN$$  \hspace{1cm} (G.21)

The tension membrane force at the retaining phase of a Level 2 tsunami is equal to 1087 kN. So the amplitude factor $\zeta_p$ is equal to

$$\zeta_p = \frac{1754}{1087} = 1.6$$  \hspace{1cm} (G.22)
G.4 Conclusion

There are three amplitude factors that influence the peak stresses in the membrane and in the cable. These are estimated for both tsunami levels [Table G.1]. The next step is to determine the normative load condition combination and determine the design peak load in the membrane \([T_{m,\text{peak}}]\) and cable \([T_{c,\text{peak}}]\).

<table>
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<tr>
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</tr>
</tbody>
</table>

The normative load combination is during the inflation of the membrane and there is a deflection in the floater for a Level 2 tsunami. These loads are generated in the membrane and are transferred to the bottom recess foundation and through the cables to the offshore foundation.
Site Specific Design for Kamakura

The derived global design, presented in the Chapter 5, will be further elaborated and integrated in the surroundings for the case study of Kamakura, to come with a first, preliminary design of all distinguished elements.

H.1 Specifying of the Load Conditions

In this Section the following loads are analysed: the retaining height of the level 2 tsunami wave, the bore impact, the peak stresses due to the inflation phase, the peak stresses due to folds in the membrane and the peak stresses due to secondary waves. These three loads are approximated by amplitude factors.

Retaining phase: Level 2 tsunami

In Section 4.3 the pressure distribution during a Level 2 tsunami was analysed. Formulas for the pressure at the top of the floater \([p_{2,\text{level}2}]\) and at the bottom recess \([p_{2,\text{level}2}^2]\)[Equation 4.5] were proposed, from which the resulting force can be calculated:

\[
F_{p,\text{level}2} = \frac{1}{2} \cdot (p_{1,\text{level}2} + p_{2,\text{level}2}^2) = 2174 \text{kN} \tag{H.1}
\]

The amount of pressures depends on the amount of overflow. From the SWASH analysis, this overflow was derived, which was estimated to be 9 meters on top of the barrier [Section 3.5].

With Equation H.1 the resulting force due to retaining phase of the Level 2 tsunami is equal to 2174 kN. With Equation 5.4, the minimal tension force of the membrane is equal to 1087 kN.

Fold Factor

For the first calculations a rigid stiff floater is chosen. The cable forces causes moments in the floater, which could create a deflection and therefore peak stresses in the membrane. In the Global Design, an amplitude factor of 2.0 was incorporated. This factor was independent of the cable interval length, which determines the magnitude of the moments [Section 5.1]. So, this amplitude factor seems to be conservative.
The interaction between the cable, floater and the membrane can be modelled as an elastic supported beam. Where the floater represents the beam, the membrane can be seen as the elastic support and the cable force as a point. The stress variation in the membrane depends on the amount of deflection of the beam [Bouma, 2000].

An analytical model is composed to describe the deflection line for a mid-section of the floater. This analytical model is used to validate a structural model created with the software program Matrixframe. With this program, the entire floater is modelled [Figure H.1][Section G.1].

The peak stresses of the membrane is related to the maximum deflection \( w_{\text{max}} \). The amplitude factor due to the folds can be calculated by comparing these peak stresses with the tension membrane force in the retaining phase of the barrier.

The spring constant \( k \) depends on the membrane characteristics: the membrane length \( L_m \), the thickness \( t_m \) (≈ 1), the maximum tension strength \( 3400 \text{ MPa} \) and the fracture strain \( 3 \).

\[
\zeta_m = \frac{(k \cdot w_{\text{max}}) + T_m}{T_m}
\]

\[
k = \frac{t_m \cdot \sigma_m}{L_m \cdot \epsilon_m}
\]

The maximum deflection occurs at the outer ends of the floater, where the cable force is half the cable force in the mid-sections. This results in a fold factor of 1.15 for a Level 1 tsunami and a fold factor of 1.2 for a Level 2 tsunami [Figure H.1].

![Figure H.1: Schematic overview of the floater; Deflection line of the floater during a Level 1 tsunami; Deflection line of the floater during a Level 2 tsunami](image)

The incoming bore causes an impact load on the vertical part of the bottom structure. This horizontal impact load can be estimated by the theory proposed by Ramsden [1990][Section A.2.3 for vertical walls. It was concluded that for large bore, the impact load is estimated at 7.5 times the hydrostatic force of the hydrostatic water pressure [Equation H.4]. However, due to the finite height of the bottom recess \( H_{\text{wall}} = 3 \text{ m} \), an empirical reduction factor is proposed [Equation H.5] [Thomas and Cox, 2012]. Where the factor \( \frac{x}{L} \) represents the location of the wall, relative to the shore, and is estimated at 0.875 [Section A.2.3]:

**Incoming Bore**

The incoming bore causes an impact load on the vertical part of the bottom structure. This horizontal impact load can be estimated by the theory proposed by Ramsden [1990][Section A.2.3 for vertical walls. It was concluded that for large bore, the impact load is estimated at 7.5 times the hydrostatic force of the hydrostatic water pressure [Equation H.4]. However, due to the finite height of the bottom recess \( H_{\text{wall}} = 3 \text{ m} \), an empirical reduction factor is proposed [Equation H.5] [Thomas and Cox, 2012]. Where the factor \( \frac{x}{L} \) represents the location of the wall, relative to the shore, and is estimated at 0.875 [Section A.2.3]:
\[ F_{\text{bore, max}} = \frac{7.5 \cdot \frac{1}{2} \cdot \rho_w \cdot g \cdot H_{\text{bore}}^2}{C_{\text{bore}}} \]  \hspace{1cm} (H.4)

\[ C_{\text{bore}} = -0.331\left(\frac{H_{\text{wall}}}{H_{\text{bore}}}\right) + 0.027\left(\frac{H_{\text{wall}}}{H_{\text{bore}}}\right)^2 + 0.341\left(\frac{x}{L}\right) - 0.076\left(\frac{x}{L}\right)^2 + 1.109 \]  \hspace{1cm} (H.5)

The impact load for a Level 1 tsunami wave is equal to 2108 kN and for a Level 2 tsunami, equal to 6200 kN.

**Dynamic Load of the Inflation Phase**

The inflation process of the barrier can be characterised by two dynamic loads: the snatch force in the cables and the opening shock in the membrane [Section 4.3]. In Section G.3 the opening shock of the membrane was analysed to determine the amplitude factor of the peak stress in the membrane. It is assumed that this peak stress in the membrane is equal to the peak stress in the cables.

The membrane configuration is filled with a certain amount of water. It is assumed that the opening shock occurs when the membrane is fully 'filled', and the reflecting bore is formed [Figure H.2]. At that specific moment, the membrane must be able to withstand the resulting force \[ F_{\text{opening}} \] which is composed out of the opening shock \[ F_z \] and the static force of the stationary water inside the membrane \[ F_{\text{static}} \]. To determine the amplitude factor, this force is compared with the membrane force in the retaining phase:

\[ \zeta_m = \frac{F_x + F_{\text{static}}}{2 \cdot T_m} \]  \hspace{1cm} (H.6)

The moment of the opening shock can be determined with the SWASH model. The incoming water profile is measured in function of the time, at the location of the barrier. At the moment the maximum volume of the membrane has crossed the barrier, the bore height \[ H^* \] and velocity \[ u^* \] are determined. The opening shock \[ F_z \] is assumed to be the bore impact force at a vertical wall and calculated with the theory of FEMA (2012):

\[ F_z = \frac{3}{2} \left(\frac{1}{2} \cdot \rho_w \cdot C_d \cdot (H^* \cdot (u^*))^2\right) \]  \hspace{1cm} (H.7)

The static force of the water is assumed to be the hydrostatic water pressure:

\[ F_{\text{static}} = \frac{1}{2} \cdot \rho_w \cdot (H^*)^2 \]  \hspace{1cm} (H.8)
This calculation is executed for both tsunami levels for barrier. It can be concluded that for a barrier with a elevated bottom recess, the amplitude factor during a Level 1 tsunami is smaller than 1 and can be considered as not a normative load. The amplitude during a Level 2 tsunami, however, gives an amplitude factor of 1.6.

Dynamic Load of Secondary Waves

The final amplitude factor concerns the peak loading due to secondary waves [Section 4.3]. It is assumed that these secondary waves create an increase in the pressure distribution. Because the numerical SWASH model only represents the first, initial wave, an estimation is made for the height of these secondary waves. It is assumed that the secondary waves can be small in order to 1 meter, till considerable waves of 3 meters. The various pressure distributions are implemented in the numerical model of the membrane for both tsunami levels.

From the results, it can be concluded that the amplitude factor for secondary waves for a Level 1 tsunami is equal to 1.48 [Figure] and for a Level 2 tsunami 1.19 [Figure ]. These are peak stresses are generated in the membrane and transferred to the cables.

Overview Amplitude Factors

There are three amplitude factors that influence the peak stresses in the membrane and in the cable. These are estimated for both tsunami levels [Table H.1]. The next step is to determine the normative load condition combination and determine the design peak load in the membrane \( T_{\text{m; peak}} \) and cable \( T_{\text{c; peak}} \).

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The normative load combination is during the inflation of the membrane and there is a deflection in the floater for a Level 2 tsunami. These loads are generated in the membrane and are transferred to the bottom recess foundation and through the cables to the offshore foundation.

The final step is to include a material factor to compute the design peak load. In Section 4.3 it was concluded that for the membrane \( \gamma_m \) this factor is 1.5 due to possibility of tears in the membrane. For cables \( \gamma_c \) this factor is assumed to be 1.2:

\[
T_{\text{m; peak}} = \gamma_m \cdot \zeta_f \cdot \zeta_p \cdot T_m = 2.88 \cdot T_m \\
T_{\text{c; peak}} = \gamma_c \cdot \zeta_f \cdot \zeta_p \cdot T_m = 2.30 \cdot T_c
\]  

(H.9)
H.2 Determine the Dimensions of the Tsunami Barrier

The optimum membrane-, cable- and floater configuration is determined for Level 2 tsunami. Whereafter the load derivation to the pile- and drag embedded anchor is calculated.

H.2.1 Optimum Cable-, Membrane and Floater Configuration

In the Global Design an optimum configuration of the cable- membrane- and floater was derived for the Kamakura case. This was however solely based on the load condition of the retaining phase of a Level 1 tsunami wave. The computations were also executed with a fold factor equal to 2.0. So for the site specific design, the optimum configuration is re-calculated with the optimised fold factor and for the retaining phase of both tsunami wave levels.

The difference between the retaining phase of the Level 1 and Level 2 tsunami is the shape of the membrane. Therefore the dependencies between the membrane length and membrane forces are compared for both tsunami levels [Figure H.3].

Figure H.3: The dependency between the membrane length and the resulting membrane tension force [Green Line], vertical component [Blue Line] and horizontal component [Red Line] for (a) Level 1 tsunami and (b) Level 2 tsunami
From Figure H.3 it can be concluded that for a Level 2 tsunami, the membrane tension force is independent of the membrane length, from a membrane length greater than 2 times the retaining height. This is due to the shape of the membrane, which will form an ellipse. So the membrane length is not normative for a Level 2 tsunami if the membrane length is greater than two times the retaining height. In Figure H.7 the differences between the membrane shape for a Level 1 and Level 2 tsunami are shown.

With this conclusion, the cable configuration and floater dimensions can be calculated for a Level 2 tsunami. Where the dependency between the cable forces and the cable configuration for a Level 2 tsunami is similar to a Level 1 tsunami. So the floater diameter only depends on the cable configuration [Section 5.2] and is given in Figure H.4.

![Figure H.4: Dependency between the floater diameter and the cable length for a Level 2 tsunami](image)

From Figure H.4 it can be concluded that for the case study of Kamakura, a cable length of 106 meters results in a floater diameter of 5 meter, with a corresponding thickness of 3.7 centimetre.

Next step is to determine the membrane length, which depends on load condition of the retaining phase of the Level 1 tsunami. The dependency between the membrane length, horizontal cable length and the floater is given in Figure H.5. The minimum, possible membrane length for a cable length of 106 meter and a floater diameter of 5 meter is equal to 34 meter.

So summarising, the optimum cable configuration is a membrane length equal to 34 meter, a horizontal cable length of 106 meter, a cable interval length of 30 meter and a floater with a diameter of 5 meter and thickness of 4.6 centimetre.
H.2.2 Force Analysis of the Floater

The first check concerns the buoyancy requirement [Section 5.1] of the floater for both tsunami levels.

Figure H.6a shows the force balance of the floater during a Level 1 tsunami. The membrane forces at the floater can be derived from Figure 5.3 and the cable forces from Figure E.9 and Figure E.10. The buoyancy force can be calculated with Equation E.4 and the self weight force with Equation E.5.

It must be noted that with the calculated optimum configuration from the previous section, the buoyancy requirement for the Level 1 tsunami is not met. This because the required thickness of the floater during a Level 2 tsunami is greater than the required thickness of a Level 1 tsunami. To solve this issue, a membrane length of 37 meter is chosen. The force balance for the Level 1 tsunami is given in Figure H.6a

A greater membrane length does not influence the force balance for a Level 2 tsunami. The calculated forces are shown in Figure H.6b
Equilibrium Shape of the Membrane

In Figure H.7 the different membrane configurations are given for the Level 1 [Blue Line] and Level 2 [Red Line] tsunami. From the figure it can be concluded that the connection point of the membrane can be at -4. So the needed membrane length is equal to 29 meters.

![Figure H.7: Membrane configuration for a Level 1 tsunami [Blue line] and Level 2 tsunami [Red line]](image)

The maximum peak load in the membrane is equal to the maximum load in the retaining phase \(T_{m;\text{level2}} = 1087kN\) times the amplitude factor of 2.9 [Appendix G]:

\[
T_{m;\text{peak}} = 3153 \text{ kN/m}
\]  (H.10)

The required thickness can be derived from the tensile strength of the membrane \(\sigma_{\text{membrane}}\), which is equal to 3400 MPa. The membrane force is per running meter, so:

\[
d_{\text{membrane}} = \frac{T_{m;\text{peak}}}{\sigma_{\text{membrane}}} = 1 \text{ mm}
\]  (H.11)

So the required thickness of the membrane is not normative.

Equilibrium shape of the cable

For the cable configuration, the hogging affect of the floating cable is negligible, and the cable shape can be seen as a straight line between the floater and the offshore foundation. Note that the eventual cable length will be longer. This because in the calculations, a retaining height of 13 was assumed, however, the actual retaining height is 16 meters plus 1 meter due to the slope of the shore. The eventual length of the cable is approximately 140 meters.

The maximum peak load in the cable is equal to the maximum load in the retaining phase \(T_{C;\text{level2}} = 1095kN\) [Figure H.6] times the cable interval length \(L_n = 30m\), times the amplitude factor of 2.3.

\[
T_{C;\text{peak}} = 75555 \text{ kN}
\]  (H.12)

The required thickness can be derived from the tensile strength of the cable \(\sigma_{\text{cable}}\), which is equal to 700 MPa. The diameter of the cable is equal to:

\[
d = \sqrt{\frac{T_{C;\text{peak}}}{0.25 \cdot \pi \cdot \sigma_{\text{cable}}}} = 375\text{ mm}
\]  (H.13)
H.2.3 Load Derivation to the Pile Foundation

In Section 6.2.2 it is stated that the membrane tension force is transferred to the bottom recess and its foundation. In this section the load derivation is further analysed and a design of a possible pile foundation is proposed.

Load Analysis of the Bottom Recess Structure

The first step is to analyse the loads acting on the bottom recess. In Figure H.8 the load situation of the bottom recess is given for the vertical loads during the retaining phase of a Level 2 tsunami.

The self weight of the bottom recess \( F_{g\text{;floater}} \) depends on the dimensions of this element [Figure H.11]. It is assumed that the bottom recess is made out of concrete, with a volumetric weight \( \gamma_b \) of 25 kN/m\(^3\) [Vrijling et al., 2011]. The total weight of the bottom recess is equal to 2790.0 kN [Equation H.14] and the point of action is the centre of gravity of the bottom structure \( a_{g\text{;floater}} \) is equal to 14.7 meter from the seaside.

\[
F_{g\text{;floater}} = A_{bot} \cdot \gamma_b = 2790.0 \text{kN}
\]  

(H.14)
The next step is to determine the downward water pressure force. This force depends on the pressure distribution on the bottom recess. It is assumed that the pressure in front of the floater and membrane is greater than the pressure inside the membrane, due to the overflowing of the Level 2 tsunami wave. Pressure \( p_1 \) and \( p_2 \) depend on the amount of overflow, the pressure inside the membrane depends on the retaining height of the barrier:

\[
\begin{align*}
  p_1 &= \rho_w \cdot g \cdot (H_{\text{overflow}} + H_{\text{retaining}}) = 237.4 \text{kPa} \\
  p_2 &= \rho_w \cdot g \cdot (H_{\text{overflow}} + H_{\text{retaining}} + d_{\text{floater}}) = 291.4 \text{kPa} \\
  p_3 &= \rho_w \cdot g \cdot (H_{\text{retaining}}) = 140.3 \text{kPa}
\end{align*}
\] (H.15)

The force due to the downward water pressure distribution can be determined by dividing the distribution in different sections. For each section, the pressure force \( F_p; i \) with corresponding distance from the pressure force to edge of the bottom recess \( a_{p;i} \) is determined and given in the table H.2.

The upward water pressure force \( F_{up} \) is caused by the hydraulic head over the bottom recess. During a Level 2 tsunami this hydraulic head is significant. It is assumed that the gradient of this pressure force distribution is linear [Bezuyen et al., 2011]. The maximum pressure occurs at the seaside of the bottom recess and is equal to:

\[
  p_4 = \rho_w \cdot g \cdot (H_{\text{overflow}} + H_{\text{retaining}} + d_{\text{floater}} + 1) = 302.1 \text{kPa}
\] (H.16)

The upward pressure force is equal to:

\[
  F_{up} = \frac{1}{2} \cdot 26 \cdot p_4 = 3927.9 \text{kN}
\] (H.17)

During a Level 2 tsunami, there are also horizontal pressures acting on the sides of the bottom recess structure, due to the water pressure and the soil pressure. This horizontal force however, is inferior to the horizontal force due to the initial bore impact during a Level 2 tsunami \( F_{bore;max} = 6200 \text{kN} \) [Section H.1]. It is therefore assumed that this dynamic load is normative for the horizontal force capacity of the pile foundation.
Pile Loads

To determine the pile loads \(F_{\text{pile,}i}\), the first step is to assume a pile plan for bottom recess structure. For the first calculations, it is assumed that there are 8 piles evenly distributed, with an interval distance of 3.25 meters and where the centre of gravity is located half way the bottom recess [Figure H.9]. It is furthermore assumed that the pile foundation will consists out of battered piles. The amount of battered piles and the inclination angle will be estimated in the next section.

The next step is to determine the horizontal and vertical force components of the pile. The normative horizontal load is due to the bore impact load and it is assumed that this force acts in the centre of gravity of the bottom recess and will therefore not cause a moment on the piles. The horizontal force for each pile is than equal to:

\[
F_{h,\text{pile}} = \frac{6200 \cdot 3.25}{8} = 2518.8\, \text{kN} \quad (H.18)
\]

The vertical forces acting on the pile foundation will cause a moment. In Table H.2 the distinguished vertical force with the corresponding distance to centre of gravity of the piles is given.

<table>
<thead>
<tr>
<th>Section</th>
<th>( F ) [kN]</th>
<th>( a ) [m]</th>
<th>( M ) [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_{g,\text{floater}} )</td>
<td>2637.5</td>
<td>14.7</td>
<td>-4483.8</td>
</tr>
<tr>
<td>( F_{p1} )</td>
<td>237.4</td>
<td>0.5</td>
<td>2967.5</td>
</tr>
<tr>
<td>( F_{p2} )</td>
<td>1187.5</td>
<td>3.5</td>
<td>11276.6</td>
</tr>
<tr>
<td>( F_{p3} )</td>
<td>134.9</td>
<td>4.3</td>
<td>1169.0</td>
</tr>
<tr>
<td>( F_{p4} )</td>
<td>1456.8</td>
<td>8.5</td>
<td>6555.5</td>
</tr>
<tr>
<td>( F_{p5} )</td>
<td>1543.1</td>
<td>16.5</td>
<td>-5400.9</td>
</tr>
<tr>
<td>( F_{up} )</td>
<td>-3927.9</td>
<td>8.67</td>
<td>-17021.0</td>
</tr>
</tbody>
</table>

The normative vertical load is equal to:

\[
F_{v,\text{pile}} = \frac{3269 \cdot 3.25}{8} = 1328.0\, \text{kN} \quad (H.19)
\]
The bending moment will create additional stresses on the piles. The stress developed on the piles due to bending can be calculated with the following equation:

\[ \frac{M}{I_{pile}} = \frac{\sigma_{pile}}{y_{pile}} \]  

(H.20)

Where \([\sigma_{pile}]\) is the bending stress, \([I_{pile}]\) the moment of area of the pile, and \([y_{pile}]\) the distance from the pile to the centre of gravity of the piles [Rajapakse, 2016]. The moment of area of the pile can be calculated with the following equation:

\[ I_{pile,i} = A_{pile} \cdot y_{pile,i}^2 \]

\[ I_{pile} = \sum I_{pile,i} \]  

(H.21)

The bending load per pile \([F_{pile,m}]\) is equal to:

\[ F_{pile,m} = \frac{\sigma_{pile}}{A_{pile}} \cdot A_{pile} \]  

(H.22)

The total vertical load per pile row is equal to:

\[ F_{pile,v;i} = F_{v,pile} + F_{pile,m} \]  

(H.23)

The results are summarised per pile row in Table H.3:

<table>
<thead>
<tr>
<th>Pile Row</th>
<th>(I_{pile,i})</th>
<th>(\sigma_i)</th>
<th>(F_{pile,v;i})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>129.4</td>
<td>-411.4</td>
<td>916.5</td>
</tr>
<tr>
<td>2</td>
<td>66.0</td>
<td>-293.9</td>
<td>1034.1</td>
</tr>
<tr>
<td>3</td>
<td>23.8</td>
<td>-176.3</td>
<td>1151.6</td>
</tr>
<tr>
<td>4</td>
<td>2.6</td>
<td>-58.8</td>
<td>1269.2</td>
</tr>
<tr>
<td>5</td>
<td>2.6</td>
<td>58.8</td>
<td>1386.7</td>
</tr>
<tr>
<td>6</td>
<td>23.8</td>
<td>176.3</td>
<td>1504.3</td>
</tr>
<tr>
<td>7</td>
<td>66.0</td>
<td>293.9</td>
<td>1621.8</td>
</tr>
<tr>
<td>8</td>
<td>129.4</td>
<td>411.4</td>
<td>1739.4</td>
</tr>
</tbody>
</table>

From Table H.3 it can be concluded that there are only compression piles needed and the normative vertical pressure force is equal to 1739.4 kN.
Pile Dimensions

The pile dimensions can be determined based on the compression and lateral load capacity. The compression and lateral load capacity must be greater than the normative vertical and horizontal force, respectively. It is assumed that the piles will be squared driven piles and the calculations are based on the assumed soil conditions [Section 2.2], without taking into account the possible weakening of the soil due to the seismic activity.

Compression Capacity of the Piles

The ultimate bearing capacity \( F_{bc;\text{max}} \) of a compression pile is determined by both the tip bearing capacity \( F_{bc;\text{max;tip}} \) and the shaft bearing capacity \( F_{bc;\text{max;shaft}} \) [Vrijling et al., 2011]:

\[
F_{bc;\text{max}} = F_{bc;\text{max;tip}} + F_{bc;\text{max;shaft}} \quad (H.24)
\]

Where:

\[
F_{bc;\text{max;tip}} = A_{\text{pile}} \cdot \rho_{bc;\text{max;tip}} \quad (H.25)
\]

And:

\[
F_{bc;\text{max;shaft}} = O_{p;\text{avg}} \cdot L_{\text{pile}} \cdot \rho_{bc;\text{max;shaft}} \quad (H.26)
\]

Where \( A_{\text{pile}} \) is the pile tip surface, \( O_{p;\text{avg}} \) the pile circumference and \( L_{\text{pile}} \) the pile length. The maximum pile resistance \( \rho_{r;\text{max;tip}} \) and the maximum pile shaft friction \( \rho_{r;\text{max;shaft}} \) can be derived from the SPT sounding [Section A.5.3]. There it is assumed that the soil can be characterised as fine sand with an average cone resistance \( q_c \) is equal to 20 MPa.

The maximum tip resistance can be calculated with the following equation:

\[
F_{bc;\text{max;tip}} = \frac{1}{2} \cdot \alpha_p \cdot \beta_p \cdot s \cdot \left( \frac{q_{c;I;\text{avg}} + q_{c;II;\text{avg}}}{2} + q_{c;III;\text{avg}} \right) \quad (H.27)
\]

So, Equation H.27 is reduced to:

\[
\rho_{bc;\text{max;tip}} = q_c \quad (H.28)
\]

The maximum pile shaft friction can be calculated with the following equation:

\[
\rho_{bc;\text{max;shaft}} = \alpha_s \cdot q_c \quad (H.29)
\]

Where the factor \( [\alpha_s] \) is determined by the method of installation. For driven piles with a little ground displacement is equal to 0.0075. For the shaft friction, the maximum cone resistance is limited to 15 MPa [Vrijling et al., 2011].

With the maximum tip resistance and the maximum shaft friction known, the ultimate bearing capacity can be calculated:

\[
F_{br;\text{max}} = A_{\text{pile}} \cdot \rho_{hr;\text{max;tip}} + O_{p;\text{avg}} \cdot L \cdot \rho_{hr;\text{max;shaft}} \quad (H.30)
\]
Lateral Force Capacity of the Piles

The foundation must be able to withstand the horizontal load due to the bore impact. A part of the horizontal force is adopted by the lateral force capacity of the piles.

Blum [1932] derived formulas for the maximum absorbable load and accompanying deformations of a pile in homogeneous ground [Vrijling et al., 2011].

The maximum absorbable horizontal force can be calculated with the following equation:

\[ F_{h,\text{max}} = \gamma_s' \cdot K_p \cdot \frac{L_0^3}{24} \cdot \frac{L_0 + 4 \cdot b}{L_{\text{pile},0} + h} \]  

Where \( \gamma_s' \) is the effective volumetric weight of the soil under water \([= 10 \, kN/m^2]\), \( b \) the width of the pile and \( K_p \) the passive soil pressure coefficient and depends on the soil conditions. For fine sand, the passive soil pressure coefficient is equal to 3.0. The parameter \( L_{\text{pile},0} \) is the depth where the moment of the ideal load is zero and is assumed to be equal to:

\[ L_{0,\text{pile}} = \frac{L_{\text{pile}}}{1.2} \]  

Pile dimensions

The pile dimensions consist out of two parameters, namely the pile length and the width of the pile. It is furthermore assumed that the piles can be placed in a battered position. This will increase the lateral load capacity. The determination of the pile dimensions is an iterative process, where there are multiple solutions possible. A possible pile configuration is presented in the next paragraph.

The derived pile foundation is given in Figure H.10, where the first 4 pile rows are battered with an angle of 20 degrees, the piles in row 5 are battered with an angle of 10 degrees and the piles in row 6 are battered with an angle of 5 degrees. The piles in row 7 and 8 are straight compression piles. These latter piles must have the greatest compression capacity. It is assumed that they are driven, square piles with a width of 0.5 meter and a length of 11 meters.
Bottom Recess Dimensions

In Section 4.1 it is assumed that the bottom recess will be made out of concrete. The assumed dimensions are given in the Figure H.11. Note that the design of the bottom recess is without a storage of the membrane, this will be further elaborated in Section H.3.

![Figure H.11: Assumed dimensions of the bottom recess structure](image)

The bottom recess is a robust structure with significant dimensions. A large weight of the structure is needed to withstand the upward water pressure to prevent uplift of the bottom recess.

The hydraulic head not only causes a great upward water pressure, it could also cause another mechanism which could cause instability, namely piping. This is the flow of water through a pipe-like channel below the bottom structure, that has been created by internal erosion [Vrijling et al., 2011]. Therefore it is recommended to install sheet walls to increase the seepage length. These sheet piles could also decrease the magnitude of the upward water pressure and therefore decreases the moment on the bottom recess.
H.2.4 Load Derivation to the Offshore Foundation

The final load segments concern the load derivation of the cable to the drag embedded anchor. The main requirement is that the peak load in the cable \( T_{C,\text{peak}} \) does not exceed the holding capacity of the anchor. The holding capacity of the drag embedded anchor is generated by the resistance of the soil in front of the anchor. If the holding capacity is exceeded, the drag anchor will be pulled out of the soil.

First step is to determine the type of drag embedded anchor. The basic anchor consists out of four elements [Figure H.12a], the anchor shackle [1], the shank [2], the fluke [3] and stabilisers [4]. There is a wide range of anchors available. The anchor with the greatest holding capacity in sand is the MK6 anchor [Vryhof Anchor, 2015]. It is therefore assumed that a Stevpris MK6 anchor will be used.

To calculate the ultimate holding capacity of an anchor form the commonly known soil mechanics formula’s is rather complex. The holding capacity of the anchor can be described as a combination of the following parameters:

\[
\begin{align*}
A & \text{ The weight of the anchor;} \\
B & \text{ The weight of the soil in the failure wedge;} \\
C & \text{ The friction of the soil in the failure wedge along fracture lines;} \\
D & \text{ Friction between fluke surface and soil (fluke area);} \\
E & \text{ The bearing capacity of shank and mooring line;} \\
F & \text{ The friction of the mooring line a and on the soil;} \\
\end{align*}
\]

Design graphs are used for the preliminary design calculations of the anchor. There is a relation between the size of the anchor \( V_{\text{anchor}} \), expressed in tons, and the ultimate holding capacity of the anchor, expressed in metric tons, for specific soil conditions. For the case study of Kamakura, it is assumed that there is a homogeneous fine sand layer. For the Stevpris Mk6 anchor in combination with a fine sand layer, a design graph is found [Figure H.13] [Vryhof Anchor, 2015]:

![Design of the Stevpris MK6 anchor](image1)

![Force balance of a drag embedded anchor](image2)

Figure H.12: (a) Design of the Stevpris MK6 anchor; (b) force balance of a drag embedded anchor [Vryhof Anchor, 2015]
The relation between the ultimate holding capacity and the anchor size can be described with a log-log relation. The maximum size of the anchor is equal to 100 ton which corresponds to an ultimate anchor capacity of 1000 metric tons or approximately 65000 kN. However, the peak load of the cable is equal to 75000 kN, and if the log-log plot is extended, this corresponds with an anchor size of 120 ton.

![Figure H.13: Design graph for the Stevpris MK6 anchor in fine sand [Vryhof Anchor, 2015]](image)

It can be concluded that a large anchor is needed to transfer the peak load of the cable to the soil. It can be questioned if the drag embedded anchor is the best suitable offshore foundation. However, the calculations are a rough estimation and solely based on the peak load of the cable. An option to decrease the anchor size is to enlarge the penetration depth. Overall, it can be concluded that the offshore foundation needs further research.
**H.2.5 Cost Estimation**

The dimensions of the different elements are estimated. Based on these calculations, a rough cost estimation can be executed. This estimation is based on simple cost functions, based on the required materials. To include the fabrication costs, the material costs are multiplied by 2. The following cost estimations are used:

- The membrane cost are estimated on 100 euro per square meter for the required thickness;
- The cable cost are estimated on 70 euro per kilogram;
- The steel cost are estimated on 3 euro per kilogram;
- The concrete costs are estimated on 70 euro per cubic meter;
- The pile costs are estimated on 100 euro per meter pile;

From Table H.4 it can be concluded that the costs of the barrier, is estimated at 70,000 euro per meter barrier.

<table>
<thead>
<tr>
<th>Element</th>
<th>Dimensions [m]</th>
<th>Volume/m</th>
<th>Cost/(m;kg)</th>
<th>Total cost/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable</td>
<td>$L_c = 140$</td>
<td>$d_c = 0.375$</td>
<td>$L_n = 30$</td>
<td>$0.52 m^3$</td>
</tr>
<tr>
<td>Bottom Recess</td>
<td>$L_n = 30$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floater</td>
<td>$d_{floater} = 5$</td>
<td>$t_{floater} = 0.047$</td>
<td>$d_{floater} = 5$</td>
<td>$2.19 m^3/m$</td>
</tr>
<tr>
<td>Membrane</td>
<td>$L_m = 37$</td>
<td>$37 m/m$</td>
<td>$180 €/m^2$</td>
<td>$6,660 €$</td>
</tr>
<tr>
<td>Anchor</td>
<td>$V_{anchor} = 120$ ton</td>
<td>$4000$ kg</td>
<td>$3 €/kg$</td>
<td>$12,000 €$</td>
</tr>
<tr>
<td>Pile Foundation</td>
<td>Figure H.10</td>
<td>$30 m/m$</td>
<td>$100 €/m$</td>
<td>$3,000 €$</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>$72,324 €$</td>
</tr>
</tbody>
</table>
H.3 Optimisation and Integration of the Tsunami Barrier

The final step of the preliminary design of Kamakura is to integrate the barrier in the surroundings. Starting point is the derived design cross section of Kamakura [Figure 2.3]. It is assumed that the flexible barrier will be part of a total coastal defence system. An example of a coastal defence system is given in Figure H.14.

The beach of Kamakura is divided in two parts. In the middle of the beach, there is a water outlet, therefore it is assumed that there will be a type of flood barrier. On each side of the flood barrier, a flexible membrane barrier will be placed. On the outer-edges of the barrier, there will be a type of abutment structure. The remaining part of the bay will be protected by tsunami walls.

The height of the bottom recess structure coincides with the height of the existing dike of Kamakura. Therefore, it is suggested to integrate the barrier in the existing structure.

For the integration of the barrier, the derived dimensions of the elements are analysed and if necessary adjustments or optimisations are proposed.

Membrane

The membrane length was estimated to be 37 meters with a thickness of 1 millimetres. For the calculations it was assumed that the membrane was inextensible, however, the membrane has a fracture strain of 3 percent [Section 4.1]. So the membrane length could extend to a length of 38 meters. This stretch of one meter is considered to be not problematic, and it will be assumed that the membrane will have a larger deflection, but the floater will be at the same position.

During normal conditions the membrane is stored in the bottom recess structure. This ensures that the membrane will be protected from weathering and vandalism. When a tsunami occurs the membrane must be able to fold out, to retain the water. During this process, the membrane may not be damaged. Therefore a type of storing structure must be developed. For example, the Ramspol Barrier uses rollers to improve the transport of the membrane [Breukelen, 2013].

Figure H.14: Assumed coastal defence plan for the case study of Kamakura
Cables

The cable length was computed to be 140 meters with a diameter of 375 millimetres. For the calculations, it was assumed that the cable was inextensible. However, the cable has a assumed fracture strain of 3 percent [Section 4.1]. So the cable length could extend to a length of 144 meters. This elongation is significant and could lead to a displacement of the floater. The elongation can be reduced by applying used cables and applying a larger material factor and will lead to a fracture strain of 1.5 percent.

To obtain a diameter of 375 millimetres, the cable consists out of sub-ropes [Figure H.15]. The cable must also be protected against external abrasion and ingress of abrasive particles. That is why a filter is applied.

![Figure H.15: Impression of the cable, composed out of sub-ropes [Royal Lankhorst Euronete, 2016]](image)

During normal conditions, the cable must be stored in a gutter type of structure [Figure H.16b]. For the calculations, it was assumed the cable would be floating which could have a beneficial affect on the floater. It is concluded that this hogging effect was negligible. So for the integration, it is preferable to use a cable with a greater mass density. As a result, the cable will lay onto the seabed.

Floater

For the first calculations it is assumed that the floater is a steel pipe with a certain diameter [Section 4.1]. There are also other floater designs possible, for example a more ellipse type of floater [Figure H.16a]. The dimensions are such that the upward water pressure is similar as in the calculations. Note that the ends of the floater are closed off, to prevent inflow of water.

For the integration of the floater in the surroundings, it is recommended to optimise the design, such that it can be used during normal conditions. For example as footpath or cycle path. This implies other load conditions, and it must be investigated if the floater must be strengthened.

![Figure H.16: Impression of the optimisations of the (a) floater and (b) bottom recess](image)
Bottom Recess
The bottom recess must be re-designed in order to store the membrane and the optimised floater. An impression is given in Figure H.16b.

For Kamakura case, it is assumed that the bottom recess is integrated in the existing dike, so that the membrane will inflate over the main road. This implies that when the tsunami alarms sounds, the main road must be closed off. Also existing structures, such as lampposts, fences and other objects must be replaced.

Pile Foundation
The pile dimensions are based on an assumed pile plan [Section H.2.3]. There are more pile configuration possible, which can result in an improved design.

From the analysis, it could be concluded that the lateral load capacity was normative for the pile dimensions. It could be an optimisation to apply a L-shaped retaining wall, which can be integrated in the bottom recess structure. This will lead to an increase in the lateral load capacity. This retaining wall will also increase the seepage length, and could lead to a decrease of the upward water pressure [Section 6.2.2]. Besides the retaining wall, battered piles are applied [Figure H.17].
Connections

The connections between the elements are not further analysed and it was assumed that these connections were able to transfer the loads between the elements. In this section, the connections are briefly analysed. There are three major connections points, at the bottom recess, at the offshore foundation and at the floater.

The membrane is connected at the bottom recess. It is assumed that the membrane will lay horizontal onto the bottom recess. A possibility to connect the membrane at the bottom recess is by means of a wrapping the membrane around a steel bar, which is anchored in the concrete bottom recess [Figure H.18]. This type of connection is able to derive a high tension load without stress concentrations. There is a possibility of unreeling of the membrane, therefore an oval shaped beam is recommended which will not rotate in the concrete structure [R. Marissen, Personal communications, 27 October 2016]

![Figure H.18: Impression of the wrapping connection of the membrane to the bottom recess [R. Marissen, 2016]](image)

There are multiple solutions to connect the cable to the offshore foundation. For synthetic fibre ropes it is generally terminated with a special spool and shackle for connection to other components in the mooring system.

The critical connections are at the floater. There must be connections for the membrane, cables and secondary cables. The location, the type of connection and the load interaction between the connection and the floater must be further investigated.

An interesting possibility is to fold the membrane around a steel bar and connect both faces of the membrane by stitching. This steel bar can be connected to the steel floater. The strength of the stitched membrane is tested at the laboratory and it appears possible to create a connection with no strength loss [R. Marissen, Personal communication, 27 October 2016]. These connections must be further investigated.
H.4 Impression of the Integrated Barrier

An impression of the integrated, optimised, flexible-membrane barrier for Kamakura is given in Figure H.19.

![Figure H.19: Impression of the integrated tsunami barrier](image)

The dimensions of the integrated, optimised, flexible-membrane barrier are:

- The optimised floater is ellipse shaped with a length of 11 meter and a height of 2.5 meter;
- A membrane length of 37 meter;
- A cable configuration which consists of a cable interval length of 30 meter and a cable length of 140 meter with a cable diameter of 0.375 meter;
- A drag embedded anchor of 120 ton;
- An optimised pile foundation [Figure H.17];
- A bottom recess structure [Figure H.11];
- A cost estimation of 72,324 Euro per running meter barrier.